Beam Calculation

ALLCOTT ASSOCIATES

Relating to:

XXX

XXX

structural engineers & building surveyors

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Project Preface

Client name:	xxx
Client address:	xxx xxx xxx

Prepared at:	Allcott Associates LLP 2 Victoria Works Vittoria Street Birmingham B1 3PE
Document prepared by:	**James Bodicoat** MEng CEng MICE MIstructE For and on behalf of Allcott Associates LLP

Date of Inspection:	xx/xx/xxxx

Job reference:	xxxxxx

1 Notes

1. The Building Contractors will need to, and must check all dimensions of openings on site prior to ordering steel, as we will not be held responsible for any incorrect lengths that are ordered.

2. Beams are designed to be the most economical size for the loading imposed in most cases, not for the thickness of wall above. It is therefore the contractor's responsibility to ensure that the load is correctly supported above the beam with a plate or other method.

3. The design is for the support Beam only and we are not responsible for ensuring the adequacy of any foundations beneath the supporting wall. This will generally the responsibility of the Building Contractor to open up areas for the Building Inspector to verify. If we have to confirm foundation adequacy which is over and above the design an extra charge will be levied.

4. Should the length of the beams be revised by the client or the Building Contractor after we have completed our design which results in a redesign we reserve the right to charge again for all redesign works.

5. All dimensions used for design are clear openings only. Add a minimum 150mm bearing length to each end of beam unless otherwise stated for actual beam length when ordering beams

6. The designs are based on domestic loads only unless otherwise stated.

7. Changes to all drawings are at client's instructions. All drawings should be checked for accuracy and should not be scaled from. Any discrepancy should be immediately informed to Allcott Associates.

8. The design is for the steel beams only, not the foundation or additional width support on the beam for the wall or other above the beam.

9. This design does not mean that any other part of the building or than the beam itself complies with current building regulations and this should be checked with the local authority.

10. Unless otherwise stated pad stones should be proprietary precast C35 concrete 450mm long by 225 deep and 100mm wide.

11. Designs are based on client's instructions. If these are given incorrectly by the client and the design beam is not correct for the opening, this will not the responsibility of this company. We reserve the right to charge again for any re-inspection and subsequent re-design.

12. The design will be based on the information including any drawings provided. Any opening up of areas will be the responsibility of the client. Any areas that are not opened up and are subsequently found not be as informed to us by the Client and requires either a re-visit or re-calculation will be charged again.

13. The scope and any limitations to miscellaneous services will be agreed with you before commencing.

14. Should the configuration of the building change after the design of the beams which subsequently affect Part A of the Building regulations, which requires re-calculation and a revisit then this will be charged again.

15. It is the responsibility of the Building Contractors to ensure that all walls for the support of the beams are adequate for the correct support of the beams.

16. These calculations are for the sole use of the person instructing the design and cannot be passed to a third party without the consent of Allcott Associates as the content will not be guaranteed to be correct as to when the report was transferred.

17. These calculations are only valid for 6 months from the date of the survey as codes of practice change. If the practical works are commenced within that period you should check that the calculations are still valid.

18. All beams to be finished with red oxide paint and correctly fire protected.

2 Calculations Sheet

Referance:-......23 000......

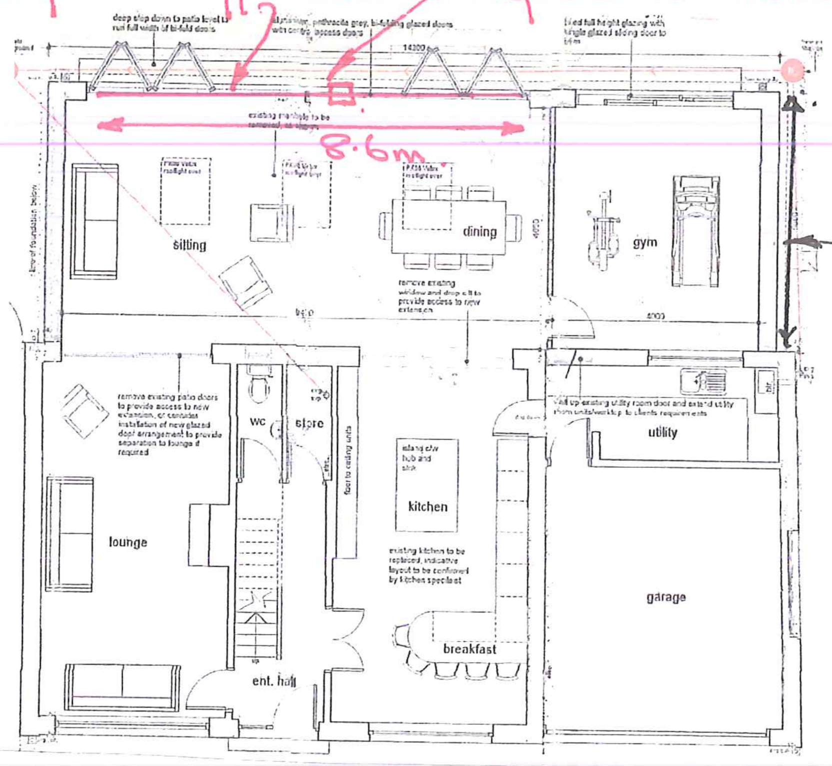
Date:- ......August 2019......

Prepared By:- Eur.Ing. **David J Allcott** BSc(hons),CEng,CBuildE,MICE,MCABE,MIWEM,AssocRICS

Sheet No:- ......1...... of ......4......

To provide support for Roof above Bifold doors in new Extension.

Proposed Support Beam Proposed Steel Column.

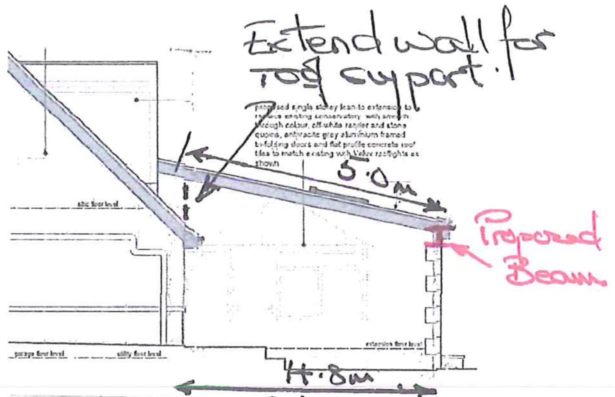


Plan (NTS)

NOTE

Please Read The Notes Section
Ref:- Measurements

Extend wall for Roof support.



Extension Section (NTS)

FOR DESIGN

Factor of Safety Steel and Concrete (To BS5628)	1.4 (Dead Load) + 1.6 (Imposed Load) + 1.0 (Other)
Factor of Safety Timber (To BS5628)	1.0 (Dead Load) + 1.0 (Imposed Load) + 1.0 (Other)
Allowable Deflection	δ = Span / 360

Consider all Beams Un-Restrained

All Loadings to EN 1991 1 -4

Please note

Beams are designed for Clear openings. Add the bearing length either side for the actual beam length

Referance:- 23000

Date:- August 2019

Prepared By:- Eur.Ing. **David J Allcott** BSc(hons),CEng,CBuildE,MICE,MCABE,MIWEM,AssocRICS

Sheet No:- 2 of 4

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Beam Design.

NOTE

Please Read The Notes Section
Ref:- Measurements

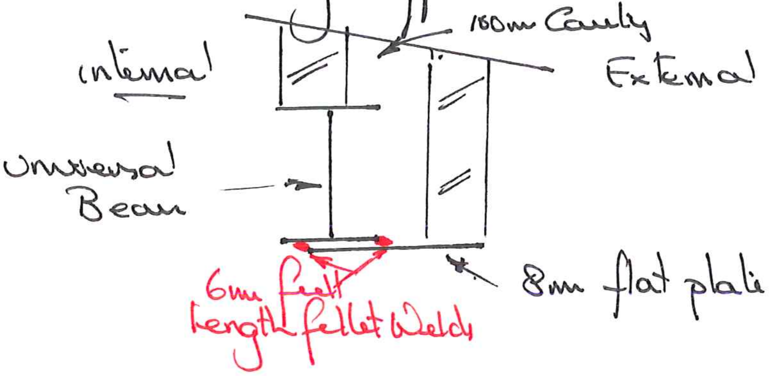
Loads used
dead - Roof tiles - 1.17 kN/m^2
imposed Snow - 0.75 kN/m^2

Load from Roof :- /m run of beam.

dead $\frac{5.0}{2} \times 1.17 = 2.93\ kN/m\ run$

imposed $\frac{5.0}{2} \times 0.75 = 1.88\ kN/m\ run$

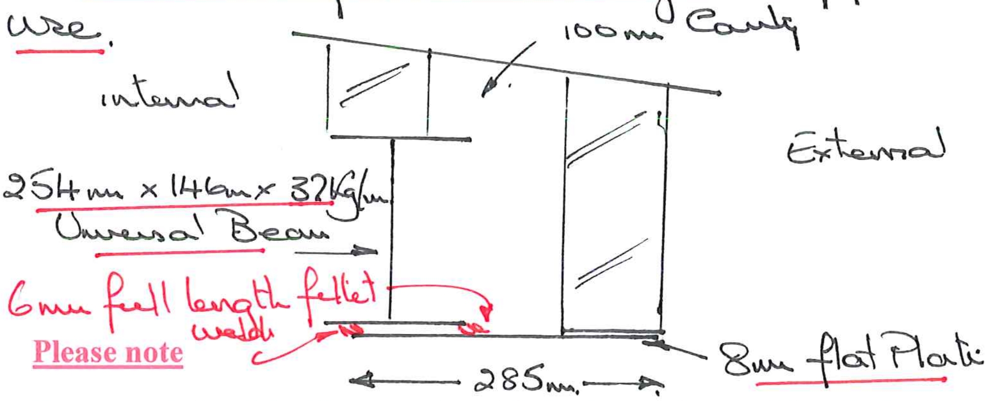
Consider Masonry Support Beam



Consider 2 Beams each 4.3m Clear Opening

∴ From Computer Analysis Appendix B1

use.



Please note

Beams are designed for Clear openings. Add the bearing length either side for the actual beam length

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NOTE

Please Read The Notes Section
Ref:- Measurements

Column Design

From Previous Calculation
BM = 21.7 KNm.
Shear = 20.2 KN.
Axial Force = 13.5 KN.

∴ As 2 Beams Support on Column
Use BM = 43.4 KNm.
Shear = 40.4 KN.
Axial force = 27 KN.

∴ From Computer Analysis Appendix B2
Use 120mm × 120mm × 10mm. (SHS)
(Square Hollow Section)

Please note

Beams are designed for Clear openings. Add the bearing length either side for the actual beam length

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NOTE

Please Read The Notes Section
Ref:- Measurements

Rafter Design.

Loads used

dead = 1.17 KN/m^2.

imposed = 0.75 KN/m^2.

Rafter Spacing 400mm.

Roof Slope – 11.5 degrees.

Length of Rafter on Slope 5.0m.

∴ From Computer Analysis Appendix B3.

Rafter will need to be.

250mm × 47mm C16. @ 400c.

Please note

Beams are designed for Clear openings. Add the bearing length either side for the actual beam length

3 Rights of Originator

We reserve the right to refuse copies of the report to any third party (other than those named above). We also reserve the right to amend our opinions in the event of additional information being made available at some future date. The Contracts (Rights of Third Parties) Act 1999 shall not apply to this agreement.

END OF REPORT

James Bodicoat MEng CEng MICE MIstructE
For and on behalf of **Allcott Associates LLP**

Appendix A

Dead Loads

225mm Solid Wall	9"Brickwork	4.73 KN/m²
	Plaster	0.20 KN/m²
		4.93 KN/m²
250mm Cavity Wall	102 Brick	2.10 KN/m²
	100 Block	1.35 KN/m²
	Plaster	0.20 KN/m²
		3.65 KN/m²
100mm Solid Wall	4"Brick	2.10 KN/m²
	Plaster	0.20 KN/m²
		2.30 KN/m²
Tiled Roof	Tiling/Battens/Felt	0.67 KN/m²
	Timber	0.30 KN/m²
	Ceiling/Insulation	0.20 KN/m²
		1.17 KN/m²
1st Floor	Floor joists, boarding	0.305 KN/m²
	Finishes	0.050 KN/m²
	Ceiling	0.20 KN/m²
	Miscellaneous	0.15 KN/m²
		0.705 KN/m²
Flat roof	Timbers / Felt Etc	0.42 KN/m²
Ceiling	Plasterboard / Plaster	0.21 KN/m²
Slate Roof	Slates/ Bath/ felt	0.28 KN/m²
	Timber	0.30 KN/m²
	Ceiling / Insulation	0.20 KN/m²
		0.78 KN/m²

Imposed Loads

	1st Floor	1.50 KN/m²
	Flat Roof (no access)	0.75 KN/m²
	Pitched Roof	0.75 KN/m²

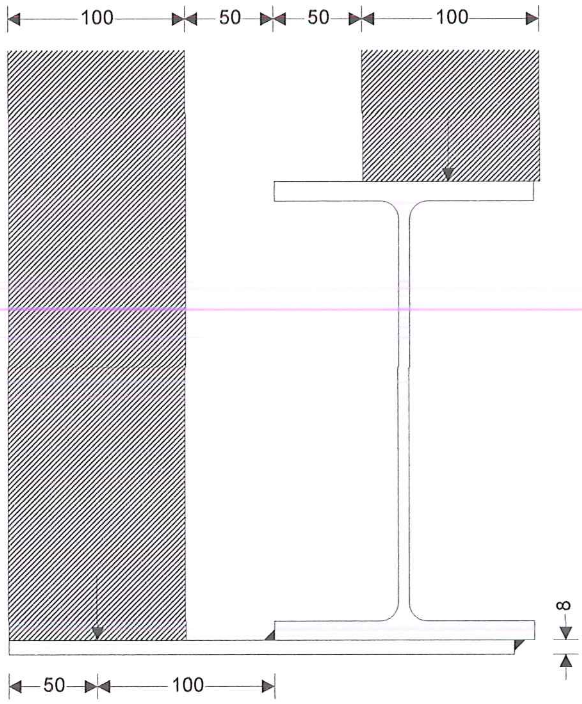
Appendix B

Appendix B1

STEEL MASONRY SUPPORT

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

Tedds calculation version 1.0.04



Steel member details

Torsion beam	UB 254x146x37
Masonry support plate	User
Steel grade of support plate	S275
Design strength of support plate	p_{ysb} = **275** N/mm^2
Modulus of elasticity	E = 205000 N/mm^2
Constant	$\varepsilon = \sqrt{(275N/mm^2 / p_{ysb})}$ = **1.000**
Length of plate beyond beam	l_h = **150** mm
Total length of plate	l_{plate} = **285 mm**
Thickness of plate	t_{sb} = **8 mm**
Width of main beam	B_{mb} = **146** mm
Area of plate	$A_{sbu} = t_{sb} \times l_{plate}$ = **2280.0** mm^2
Distance from weld position to CoG	$c_{yysb} = l_h / 2 - (l_{plate} - l_h) / 2$ = **7** mm

Supported materials detail

Density of masonry on main beam	$\rho_{m,mb}$ = **20.0** kN/m^3
Width of masonry on main beam	b_{mmb} = **100** mm
Height of masonry on main beam	h_{mmb} = **134** mm
Eccentricity of main beam material	e_{mb} = **50** mm
Add dead force main beam (not from masonry)	P_{Gaddmb} = **2.9** kN/m
Add live force main beam (not from masonry)	P_{Qaddmb} = **1.9** kN/m
Density of masonry on support beam	$\rho_{m,sb}$ = **20.0** kN/m^3
Width of masonry on support beam	b_{msb} = **100** mm
Height of masonry on support beam	h_{msb} = **400** mm

Tekla Tedds Allcott Associates LLP Unit 3, The Fosse Fosse Way, Radford Semele Warwickshire, CV31 1XN	Project Holmfield, Bill lane, Woodale, Holmfirth, HD9 1XX				Job no. 23000	
	Calcs for Masonry Support Beam				Start page no./Revision 2	
	Calcs by DJA	Calcs date 08/08/2019	Checked by	Checked date	Approved by	Approved date

Add dead force support beam (not from masonry) P_{Gaddsb} = **0.0** kN/m

Add live force support beam (not from masonry) P_{Qaddsb} = **0.0** kN/m

Geometry

Cavity width c = **100** mm

Supported width of masonry $d_m = l_h + e_{mb} - c$ = **100** mm

Biaxial stress effects in the plate (SCI-P-110)

Maximum overall bending moment M_x = **21.7** kNm

Dist to NA combined section (CoG torsion beam) $y_{e,all} = (D_{mb} + t_{sb}) \times A_{sbu} / (2 \times (A_{mb} + A_{sbu}))$ = **43** mm

Second moment of area of combined section $I_{xx,all} = (I_{xxmb} + A_{mb} \times y_{e,all}^2) + A_{sbu} \times (D_{mb} / 2 + t_{sb} / 2 - y_{e,all})^2$ = **8215** cm^4

Elastic section modulus of combined section $Z_{xx,all} = I_{xx,all} / (D_{mb} / 2 + t_{sb} - y_{e,all})$ = **883.47** cm^3

Section modulus of plate $Z_{xx,plate} = 1m \times t_{sb}^2 / (6 \times 1m)$ = **10.67** cm^3/m

Eccentricity of support beam masonry e_1 = **100** mm

Force of masonry on support plate $P_1 = (b_{msb} \times h_{msb} \times \rho_{m,sb} + P_{Gaddsb}) \times \gamma_{fG} + P_{Qaddsb} \times \gamma_{fQ}$ = **1.1** kN/m

Bending at heel $M_{x,plate} = P_1 \times e_1$ = **0.1** kNm/m

Moment capacity of plate $M_c = 1.2 \times Z_{x,plate} \times p_{ysb}$ = **3.5** kNm/m

PASS - Design strength exceeds stress at heel

Longitudinal stress due to overall bending $\sigma_1 = M_x / Z_{xx,all}$ = **24.5** N/mm^2

Constant relating to Von Mises curve $c_{fp} = (4 \times p_{ysb}^2 - 3 \times \sigma_1^2)^{0.5}$ = **548.4** N/mm^2

Transverse bending stress ratio limit $\alpha_{ts} = (c_{fp}^2 - \sigma_1^2) / (2 \times c_{fp} \times p_{ysb})$ = **0.995**

Transverse bending stress ratio $\alpha_{ls} = M_{x,plate} / M_c$ = **0.032**

PASS - Transverse bending stress ratio less than allowable limit

Deflection at toe

Unfactored force on support angle $P_{1SLS} = b_{msb} \times h_{msb} \times \rho_{m,sb} + P_{Gaddsb} + P_{Qaddsb}$ = **0.8** kN/m

Distance from weld to load position $a_m = e_1$ = **100** mm

Length of load resultant to edge of plate $b_m = l_h - e_1$ = **50** mm

Dist from weld to load position as ratio of length $a_l = a_m / (a_m + b_m)$ = **0.667**

Effective second moment of inertia $I_{eff\_def} = t_{sb}^3 / 12$ = **42667** mm^4/m

Deflection at toe $\delta = (a_l^2 \times (3 - a_l) / 6) \times (P_{1SLS} \times (a_m + b_m)^3) / (E_{S5950} \times I_{eff\_def})$ = **0.05** mm

Deflection limit δ_{lim} = **2.00** mm

PASS - Deflection is within specified criteria

Weld details - assume a full length weld and that the plate acts as a propped cantilever with the prop at the weld position and the fixed end at the centre of the torsion beam

Leg length of weld s_{weld} = **6** mm

Throat size of weld $a_{weld} = 1/\sqrt{(2)} \times s_{weld}$ = **4.2** mm

Shear force at weld position $R_A = P_1 \times \max((1 + (3 \times e_1) / (2 \times B_{mb} / 2)), 1.4)$ = **3.4** kN/m

Maximum possible force in plate $R_p = (l_h + B_{mb}) \times t_{sb} \times p_{ysb}$ = **652.1** kN

Longitudinal shear between beam and plate $R_l = 2 \times R_p / L$ = **303.3** kN/m

Horizontal shear between beam and plate $R_h = P_1 \times e_1 / (s_{weld} / 2 + t_{sb} / 2)$ = **16.0** kN/m

Resultant weld force $R_{weld} = (R_A^2 + R_l^2 + R_h^2)^{0.5}$ = **0.304** kN/mm

Strength of weld (Table 37) p_{weld} = **220.0** N/mm^2

Capacity of full length weld $p_{c,weld} = a_{weld} \times p_{weld}$ = **0.933** kN/mm

PASS - Capacity of weld exceeds resultant force on weld

Torsional loading ULS

Loading of support beam masonry $w_{1ULS} = (h_{msb} \times b_{msb} \times \rho_{m,sb} + P_{Gaddsb}) \times \gamma_{fG} + P_{Qaddsb} \times \gamma_{fQ}$ = **1.12** kN/m

Loading of main beam masonry $w_{2ULS} = (h_{mmb} \times b_{mmb} \times \rho_{m,mb} + P_{Gaddmb}) \times \gamma_{fG} + P_{Qaddmb} \times \gamma_{fQ}$ = **7.49** kN/m

Self weight of support beam $w_{3ULS} = A_{sbu} \times \rho_{sb} \times \gamma_{fG} = \mathbf{0.25}$ kN/m

Torsional loading SLS

Loading of support beam masonry $w_{1SLS} = h_{msb} \times b_{msb} \times \rho_{m,sb} + P_{Gaddsb} + P_{Qaddsb} = \mathbf{0.80}$ kN/m

Loading of main beam masonry $w_{2SLS} = h_{mmb} \times b_{mmb} \times \rho_{m,mb} + P_{Gaddmb} + P_{Qaddmb} = \mathbf{5.08}$ kN/m

Self weight of support beam $w_{3SLS} = A_{sbu} \times \rho_{sb} = \mathbf{0.18}$ kN/m

Eccentricities

Distance to shear centre of main beam $e_{0mb} = \mathbf{0}$ mm

Eccentricity of support beam masonry $e_{1mb} = (B_{mb} + b_{msb}) / 2 + c - e_{mb} = \mathbf{173}$ mm

Eccentricity of main beam masonry $e_{2mb} = (B_{mb} - b_{mmb}) / 2 - e_{mb} = \mathbf{-27}$ mm

Eccentricity of support beam $e_{3mb} = B_{mb} / 2 + c_{yysb} = \mathbf{81}$ mm

Torsional effects

Applied torque (ULS) $T_{qULS} = abs(w_{1ULS} \times e_{1mb} + w_{2ULS} \times e_{2mb} + w_{3ULS} \times e_{3mb}) = \mathbf{0.01}$ kNm/m

Total torque (ULS) $T_q = T_{qULS} \times L = \mathbf{0.06}$ kNm

Applied torque (SLS) $T_{qSLS} = abs(w_{1SLS} \times e_{1mb} + w_{2SLS} \times e_{2mb} + w_{3SLS} \times e_{3mb}) = \mathbf{0.02}$ kNm/m

Total torque (SLS) $T_{qu} = T_{qSLS} \times L = \mathbf{0.07}$ kNm

STEEL BEAM TORSION DESIGN

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

Tedds calculation version 2.0.02

Section details

Section type UB 254x146x37

Steel grade S275

Design stength $p_{yw} = p_y = \mathbf{275}$ N/mm^2

Constant $\varepsilon = \sqrt{(275 \text{ N/mm}^2 / p_y)} = \mathbf{1.000}$

Geometry - Beam unrestrained against lateral-torsional buckling between supports.

Effective span $L = \mathbf{4300}$ mm

Length of segment for LT buckling $L_{LT} = \mathbf{5000}$ mm

Compression flanges laterally restrained

Both flanges free to rotate on plan

Effective length for LT buckling $L_{E\_LT} = L_{LT} \times 1.0 = \mathbf{5000}$ mm

Loading - Torsional loading comprises only full-length uniformly distributed load(s)

Internal forces & moments on member under factored loading for uls design

Applied shear force $F_{vy} = \mathbf{20.2}$ kN

Maximum bending moment $M_{LT} = M_x = \mathbf{21.67}$ kNm

Applied torque $T_q = \mathbf{0.06}$ kNm

Minor axis bending moment $M_y = 0$ kNm

Compression force $F_c = 0$ kN

Equivalent uniform moment factors

EUM factor (Cl. 4.3.6.6 and T18) $m_{LT} = \mathbf{1.000}$

Torsional deflection parameters

Beam is torsion fixed and warping free at each end. (as defined in SCI-P-057 section 2.1.6) - Appendix B case 4

Dist along the beam for first derivative of twist $z_1 = 0$ mm

Dist along the beam for second derivative of twist $z_2 = L / 2 = \mathbf{2150}$ mm

First derivative of angle of twist — $\phi'_1 = T_q / (G \times J) \times a / L \times [L^2 / (2 \times a) \times (1 / L - 2 \times z_1 / L^2) + \sinh(z_1 / a) - \tanh(L / (2 \times a)) \times \cosh(z_1 / a)] \times 1$ rads = **1.14×10^{-3}** rads/m

Third derivative of angle of twist — $\phi'''_1 = T_q / (G \times J \times a^2) \times a/L \times [\sinh(z_1 / a) - \tanh(L / (2 \times a)) \times \cosh(z_1 / a)] \times 1$ rads = **-8.82×10^{-4}** rads/m³

Angle of twist — $\phi_2 = T_q \times a / (G \times J) \times a / L \times [L^2 / (2 \times a^2) \times (z_2 / L - z_2^2 / L^2) + \cosh(z_2 / a) - \tanh(L / (2 \times a)) \times \sinh(z_2 / a) - 1] \times 1$ rads = **0.002** rads

Second derivative of angle of twist — $\phi''_2 = T_q / (G \times J \times a) \times a / L \times [\cosh(z_2 / a) - \tanh(L / (2 \times a)) \times \sinh(z_2 / a) - 1] \times 1$ rads = **-7.58×10^{-4}** rads/m²

Design parameters

Total angle of twist	$\phi = abs(\phi_2)$ = **0.002** rads
First derivative of ϕ	$\phi' = abs(\phi'_1)$ = **1.14×10^{-3}** rads/m
Second derivative of ϕ	$\phi'' = abs(\phi''_2)$ = **7.58×10^{-4}** rads/m²
Third derivative of ϕ	$\phi''' = abs(\phi'''_1)$ = **8.82×10^{-4}** rads/m³

Section classification

b / T = **6.7**

d / t = **34.8**

$r_{1s} = \min(1.0, \max(-1.0, F_c / (d \times t \times p_{yw})))$ = **0.000**

$r_{2s} = F_c / (A_g \times p_{yw})$ = **0.000**

Section classification is plastic

Shear capacity (parallel to y-axis)

Design shear force	F_{vy} = **20.2** kN
Design shear resistance (Cl. 4.2.3)	$P_{vy} = 0.6 \times p_y \times A_{vy}$ = **266.1** kN

Pass - Shear

Moment capacity (x-axis)

Design bending moment	M_x = **21.7** kNm
Moment capacity	$M_{cxu} = p_y \times S_x$ = **132.9** kNm
Moment capacity low shear (Cl. 4.2.5.1)	$M_{cx} = \min(p_y \times S_x, 1.2 \times p_y \times Z_x)$ = **132.9** kNm

Pass - Moment capacity exceeds design bending moment

Lateral torsional buckling

Effective length for lateral torsional buckling	L_{E_LT} = **5000** mm
Slenderness ratio	$\lambda = L_{E\_LT} / r_y$ =**144**
Buckling parameter	u = **0.890**
Flange ratio	$\eta = 0.5$
Torsional index	x = **24.3**
Slenderness factor	$v = 1 / (1 + 0.05 \times (\lambda / x)^2)^{0.25}$ = **0.78**
Ratio - cl 4.3.6.9	$\beta_w = 1.0$ = **1.000**
Equvalent slenderness – cl 4.3.6.7	$\lambda_{LT} = u \times v \times \lambda \times \sqrt{(\beta_w)}$ = **99**
Limiting slendernes – Annex B2.2	$\lambda_{L0} = 0.4 \times \sqrt{(\pi^2 \times E_{S5950} / p_y)}$ = **34**
Euler stress	$p_E = \pi^2 \times E_{S5950} / \lambda_{LT}^2$ = **205** N/mm²
Perry factor	$\eta_{LT} = \max(7.0 \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.455**
	$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2$ = **286606886.137**
Bending strength	$p_b = p_E \times p_y / (\phi_{LT} + \sqrt{(\phi_{LT}^2 - p_E \times p_y)})$ = **126** N/mm²
Buckling resistance moment	$M_b = p_b \times S_x$ = **60.9** kNm

Max moment governing buckling resistance M_{LT} = **21.7** kNm

Equiv uniform moment factor for LTB m_{LT} = **1.00**

M_b / m_{LT} = **60.9** kNm

Pass - lat. tors. buckling

Buckling under combined bending & torsion -SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Span factor L / a = **3.57**

Angle of twist ϕ = **0.002** rads

Second derivative of ϕ ϕ'' = **758.×10^-6** rads/m²

Induced minor axis moment $M_{yt} = M_x \times \phi$ / 1 rad = **0.03** kNm

Normal stress at flange tip due to M_{yt} $\sigma_{byt} = M_{yt} / Z_y$ = **0** N/mm²

Normal stress at flange tip due to warping $\sigma_w = E_{S5950} \times W_{n0} \times \phi''$ / 1 rad = **1** N/mm²

Interaction index $i_b = M_x \times m_{LT} / M_b + (\sigma_{byt} + \sigma_w) / p_y \times (1 + 0.5 \times M_x \times m_{LT} / M_b)$ = **0.36**

Pass - Combined bending and torsion check satisfied

Local capacity under combined bending & torsion

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Max. direct stress due to M_x $\sigma_{bx} = M_x / Z_x$ = **50** N/mm²

Combined stress - eqn 2.22 $\sigma_{bx} + \sigma_{byt} + \sigma_w$ = **52** N/mm²

Design strength p_y = **275** N/mm²

Pass - Local capacity

Combined shear stresses - SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum shear stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Max shear stresses due to bending in web $\tau_{bw} = F_{vy} \times Q_w / (I_x \times t)$ = **14** N/mm²

Max shear stresses due to bending in flange $\tau_{bf} = F_{vy} \times Q_f / (I_x \times T)$ = **3** N/mm²

Max shear stresses due to torsion in web $\tau_{tw} = abs(G \times t \times \phi'$ / 1rad) = **1** N/mm²

Max shear stresses due to torsion in flange $\tau_{tf} = abs(G \times T \times \phi'$ / 1 rad) = **1** N/mm²

Max shear stresses due to warping in flange $\tau_{wf} = abs(-E_{S5950} \times S_{w1} \times \phi'''$ / 1 rad / T) = **0** N/mm²

Amp shear stress torsion & warping in web $\tau_{vtw} = \tau_{tw} \times (1 + 0.5 \times M_x \times m_{LT} / M_b)$ = **1** N/mm²

Amp shear stress torsion & warping in flange $\tau_{vtf} = (\tau_{tf} + \tau_{wf}) \times (1 + 0.5 \times M_x \times m_{LT} / M_b)$ = **1** N/mm²

Combined shear stresses due to bending, torsion & warping:

Combined shear stresses in web $\tau_w = \tau_{bw} + \tau_{vtw}$ = **15** N/mm²

Combined shear stresses in flange $\tau_f = \tau_{bf} + \tau_{vtf}$ = **4** N/mm²

Shear strength $p_v = 0.6 \times p_y$ = **165** N/mm²

Pass - Combined shear stresses

Twist check

Total applied torque (unfactored) T_{qu} = **0.07** kNm

Maximum twist under sls loading $\phi_{sls} = \phi \times T_{qu} / T_q$ = **0.11** deg

Twist limit ϕ_{lim} = **1.00** deg

Pass - Twist

Deflection

Maximum y-axis deflection δ_{y_max} = **2.5** mm

Deflection limit - cl. 2.5.2 δ_{lim} = min(L/ k_δ, δ_{lim_abs}) = **3.0** mm

Pass - Deflection within specified limit

Appendix B2

STEEL COLUMN DESIGN

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 1.1.04

Design summary

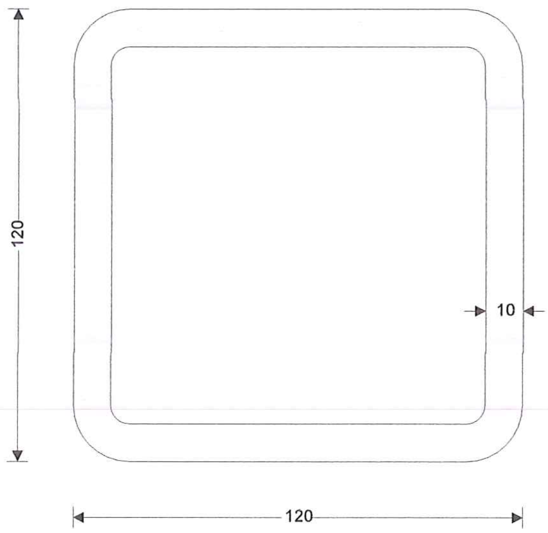
Description	Unit	Provided	Required	Utilisation	Result
Shear resistance (y-y)	kN	341	40	0.119	PASS
Axial compression	kN	1180	27	0.023	PASS
Bending resistance (y-y)	kNm	48	43	0.901	PASS
Combined bending & axial	kNm	48	43	0.901	PASS
Buckling in compression	kN	1029	27	0.026	PASS
Buckling in bending	kNm	48	43	0.901	PASS
Combined buckling				0.938	PASS

Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1**

Resistance of members to instability γ_{M1} = **1**

Resistance of cross-sections in tension to fracture γ_{M2} = **1.1**



SHS 120x120x10.0 (Tata Steel Celsius)
Section depth, h, 120 mm
Section breadth, b, 120 mm
Mass of section, Mass, 33.7 kg/m
Section thickness, t, 10 mm
Area of section, A, 4293 mm^2
Radius of gyration about y-axis, i_y, 44.555 mm
Radius of gyration about z-axis, i_z, 44.555 mm
Elastic section modulus about y-axis, $W_{el.y}$, 142025 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 142025 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 175159 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 175159 mm^3
Second moment of area about y-axis, I_y, 8521473 mm^4
Second moment of area about z-axis, I_z, 8521473 mm^4

Column details

Column section **SHS 120x120x10.0**

Steel grade **S275**

Yield strength f_y = **275** N/mm^2

Ultimate strength f_u = **410** N/mm^2

Modulus of elasticity E = **210** kN/mm^2

Poisson's ratio ν = **0.3**

Shear modulus $G = E / [2 \times (1 + \nu)]$ = **80.8** kN/mm^2

Column geometry

System length for buckling - Major axis L_y = **2500** mm

System length for buckling - Minor axis L_z = **2500** mm

The column is not part of a sway frame in the direction of the minor axis

The column is not part of a sway frame in the direction of the major axis

Column loading

Axial load	N_{Ed} = **27** kN (Compression)
Major axis moment at end 1 - Bottom	$M_{y,Ed1}$ = **43.4** kNm
Major axis moment at end 2 - Top	$M_{y,Ed2}$ = **43.4** kNm
	Major axis bending is single curvature
Minor axis moment at end 1 - Bottom	$M_{z,Ed1}$ = **0.0** kNm
Minor axis moment at end 2 - Top	$M_{z,Ed2}$ = **0.0** kNm
Major axis shear force	$V_{y,Ed}$ = **40** kN
Minor axis shear force	$V_{z,Ed}$ = **0** kN

Buckling length for flexural buckling - Major axis

End restraint factor	K_y = **1.000**
Buckling length	$L_{cr\_y} = L_y \times K_y$ = **2500** mm

Buckling length for flexural buckling - Minor axis

End restraint factor	K_z = **1.000**
Buckling length	$L_{cr\_z} = L_z \times K_z$ = **2500** mm

Web section classification (Table 5.2)

Coefficient depending on f_y	$\varepsilon = \sqrt{(235 \text{ N/mm}^2 / f_y)}$ = **0.924**
Depth between fillets	$c_w = h - 3 \times t$ = **90.0** mm
Ratio of c/t	$ratio_w = c_w / t$ = **9.00**
Length of web taken by axial load	$l_w = \min(N_{Ed} / (2 \times f_y \times t), c_w)$ =**4.9** mm
For class 1 & 2 proportion in compression	$\alpha = (c_w/2 + l_w/2) / c_w$ = **0.527**
Limit for class 1 web	$Limit_{1w} = (396 \times \varepsilon) / (13 \times \alpha - 1)$ = **62.53**

The web is class 1

Flange section classification (Table 5.2)

Depth between fillets	$c_f = b - 3 \times t$ = **90.0** mm
Ratio of c/t	$ratio_f = c_f / t$ = **9.00**
Limit for class 1 flange	$Limit_{1f} = 33 \times \varepsilon$ = **30.51**
Limit for class 2 flange	$Limit_{2f} = 38 \times \varepsilon$ = **35.13**
Limit for class 3 flange	$Limit_{3f} = 42 \times \varepsilon$ = **38.83**

The flange is class 1

Overall section classification

The section is class 1

Resistance of cross section (cl. 6.2)

Shear - Major axis (cl. 6.2.6)

Design shear force	$V_{y,Ed}$ = **40.4** kN
Shear area	$A_{vy} = A \times h / (b + h)$ = **2146** mm^2
Plastic shear resistance	$V_{pl,y,Rd} = A_{vy} \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **340.8** kN
	$V_{y,Ed} / V_{pl,y,Rd}$ = **0.119**

PASS - Shear resistance exceeds the design shear force

$V_{y,Ed} \le 0.5 \times V_{pl,y,Rd}$ - No reduction in f_y required for bending/axial force

Compression (cl. 6.2.4)

Design force	N_{Ed} = **27** kN

Design resistance $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} =$ **1180 kN**

$N_{Ed} / N_{c,Rd} =$ **0.023**

PASS - The compression design resistance exceeds the design force

Bending - Major axis (cl. 6.2.5)

Design bending moment $M_{y,Ed} = \max(\text{abs}(M_{y,Ed1}), \text{abs}(M_{y,Ed2})) =$ **43.4 kNm**

Section modulus $W_y = W_{pl.y} =$ **175.2** cm^3

Design resistance $M_{c,y,Rd} = W_y \times f_y / \gamma_{M0} =$ **48.2 kNm**

$M_{y,Ed} / M_{c,y,Rd} =$ **0.901**

PASS - The bending design resistance exceeds the design moment

Combined bending and axial force (cl. 6.2.9)

Ratio design axial to design plastic resistance $n = \text{abs}(N_{Ed}) / N_{pl,Rd} =$ **0.023**

Ratio web area to gross area $a_w = \min(0.5, (A - 2 \times b \times t) / A) =$ **0.441**

Ratio flange area to gross area $a_f = \min(0.5, (A - 2 \times h \times t) / A) =$ **0.441**

Bending - Major axis (cl. 6.2.9.1)

Design bending moment $M_{y,Ed} = \max(\text{abs}(M_{y,Ed1}), \text{abs}(M_{y,Ed2})) =$ **43.4 kNm**

Plastic design resistance $M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0} =$ **48.2 kNm**

Modified design resistance $M_{N,y,Rd} = M_{pl,y,Rd} \times \min(1, (1 - n) / (1 - 0.5 \times a_w)) =$ **48.2 kNm**

$M_{y,Ed} / M_{N,y,Rd} =$ **0.901**

PASS - Bending resistance in presence of axial load exceeds design moment

Buckling resistance (cl. 6.3)

Yield strength for buckling resistance $f_y =$ **275** N/mm^2

Flexural buckling - Major axis

Elastic critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr\_y}^2 =$ **2826 kN**

Non-dimensional slenderness $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.646**

Buckling curve (Table 6.2) **a**

Imperfection factor (Table 6.1) $\alpha_y =$ **0.21**

Parameter Φ $\Phi_y = 0.5 \times [1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2] =$ **0.756**

Reduction factor $\chi_y = \min(1.0, 1 / [\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}]) =$ **0.872**

Design buckling resistance $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **1028.8 kN**

$N_{Ed} / N_{b,y,Rd} =$ **0.026**

PASS - The flexural buckling resistance exceeds the design axial load

Flexural buckling - Minor axis

Elastic critical buckling force $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr\_z}^2 =$ **2826 kN**

Non-dimensional slenderness $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} =$ **0.646**

Buckling curve (Table 6.2) **a**

Imperfection factor (Table 6.1) $\alpha_z =$ **0.21**

Parameter Φ $\Phi_z = 0.5 \times [1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2] =$ **0.756**

Reduction factor $\chi_z = \min(1.0, 1 / [\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}]) =$ **0.872**

Design buckling resistance $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} =$ **1028.8 kN**

$N_{Ed} / N_{b,z,Rd} =$ **0.026**

PASS - The flexural buckling resistance exceeds the design axial load

Minimum buckling resistance

Minimum buckling resistance $N_{b,Rd} = \min(N_{b,y,Rd}, N_{b,z,Rd}) =$ **1028.8 kN**

$N_{Ed} / N_{b,Rd} =$ **0.026**

PASS - The axial load buckling resistance exceeds the design axial load

Buckling resistance moment (cl.6.3.2.1)

Square hollow section not subject to lateral torsional buckling therefore:-

Reduction factor χ_{LT} = **1.0**

Design buckling resistance moment $M_{b,Rd} = \chi_{LT} \times W_y \times f_y / \gamma_{M1}$ = **48.2** kNm

Design bending moment $M_{y,Ed} = max(abs(M_{y,Ed1}), abs(M_{y,Ed2}))$ = **43.4** kNm

$M_{y,Ed} / M_{b,Rd}$ = **0.901**

PASS - The design buckling resistance moment exceeds the maximum design moment

Combined bending and axial compression (cl. 6.3.3)

Characteristic resistance to normal force $N_{Rk} = A \times f_y$ = **1180** kN

Characteristic moment resistance - Major axis $M_{y,Rk} = W_{pl.y} \times f_y$ = **48.2** kNm

Characteristic moment resistance - Minor axis $M_{z,Rk} = W_{pl.z} \times f_y$ = **48.2** kNm

ψ_y = if(abs($M_{y,Ed1}$)<=abs($M_{y,Ed2}$), $M_{y,Ed1}$ / if($M_{y,Ed2}$>=0 kNm,max($M_{y,Ed2}$,0.0001 kNm),$M_{y,Ed2}$), $M_{y,Ed2}$ / if($M_{y,Ed1}$>=0 kNm,max($M_{y,Ed1}$,0.0001 kNm),$M_{y,Ed1}$)) = **1.000**

Moment distribution factor - Major axis $\psi_y = M_{y,Ed1} / M_{y,Ed2}$ = **1.000**

Moment factor - Major axis $C_{my} = max(0.4, 0.6 + 0.4 \times \psi_y)$ = **1.000**

Moment distribution factor - Minor axis $\psi_z = M_{z,Ed1} / M_{z,Ed2}$ = **0.000**

Moment factor - Minor axis $C_{mz} = max(0.4, 0.6 + 0.4 \times \psi_z)$ = **0.600**

Moment distribution factor for LTB $\psi_{LT} = M_{y,Ed1} / M_{y,Ed2}$ = **1.000**

Moment factor for LTB $C_{mLT} = max(0.4, 0.6 + 0.4 \times \psi_{LT})$ = **1.000**

Interaction factor k_{yy} $k_{yy} = C_{my} \times [1 + min(0.8, \bar{\lambda}_y - 0.2) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})]$ = **1.012**

Interaction factor k_{zy} $k_{zy} = 0.6 \times k_{yy}$ = **0.607**

Interaction factor k_{zz} $k_{zz} = C_{mz} \times [1 + min(0.8, \bar{\lambda}_z - 0.2) \times N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1})]$ = **0.607**

Interaction factor k_{yz} $k_{yz} = 0.6 \times k_{zz}$ = **0.364**

Section utilisation $UR_{B\_1} = N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) + k_{yz} \times M_{z,Ed} / (M_{z,Rk} / \gamma_{M1})$

UR_{B_1} = **0.938**

$UR_{B\_2} = N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) + k_{zz} \times M_{z,Ed} / (M_{z,Rk} / \gamma_{M1})$

UR_{B_2} = **0.573**

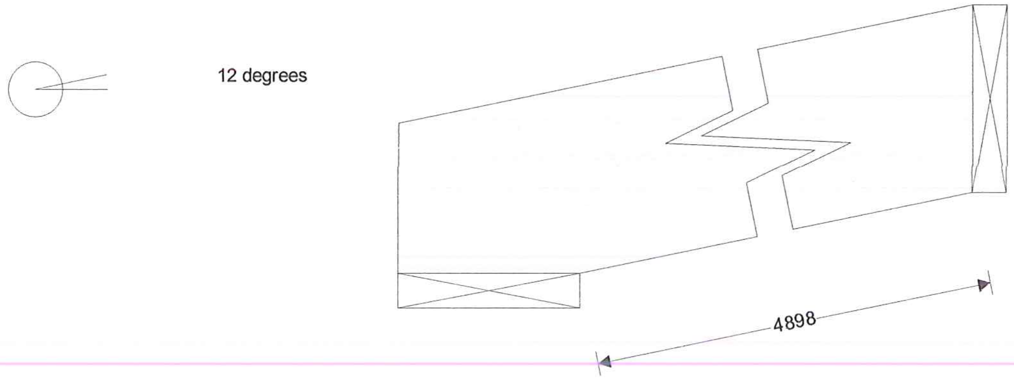
PASS - The buckling resistance is adequate

Tekla Tedds Allcott Associates LLP Unit 3, The Fosse, Radford Semele, Leamington Spa CV31 1XN	Project Holmfield, Bill lane, Woodale, Holmfirth, HD9 1XX				Job no. 23000	
	Calcs for Rafter Design				Start page no./Revision 1	
	Calcs by DJA	Calcs date 08/08/2019	Checked by	Checked date	Approved by	Approved date

Appendix B3

TIMBER RAFTER DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.03



Rafter details

Breadth of timber sections	$b = \mathbf{47}$ mm
Depth of timber sections	$h = \mathbf{250}$ mm
Rafter spacing	$s = \mathbf{400}$ mm
Rafter slope	$\alpha = \mathbf{11.5}$ deg
Clear span of rafter on horizontal	$L_{clh} = \mathbf{4800}$ mm
Clear span of rafter on slope	$L_{cl} = L_{clh} / \cos(\alpha) = \mathbf{4898}$ mm
Rafter span	**Single span**
Timber strength class	**C16**

Section properties

Cross sectional area of rafter	$A = b \times h = \mathbf{11750}$ mm^2
Section modulus	$Z = b \times h^2 / 6 = \mathbf{489583}$ mm^3
Second moment of area	$I = b \times h^3 / 12 = \mathbf{61197917}$ mm^4
Radius of gyration	$r = \sqrt{(I / A)} = \mathbf{72.2}$ mm

Loading details

Rafter self weight	$F_j = b \times h \times \rho_{char} \times g_{acc} = \mathbf{0.04}$ kN/m
Dead load on slope	$F_d = \mathbf{1.15}$ kN/m^2
Imposed load on plan	$F_u = \mathbf{0.75}$ kN/m^2
Imposed point load	$F_p = \mathbf{0.90}$ kN

Modification factors

Section depth factor	$K_7 = (300 \text{ mm} / h)^{0.11} = \mathbf{1.02}$
Load sharing factor	$K_8 = \mathbf{1.10}$

Consider long term load condition

Load duration factor	$K_3 = \mathbf{1.00}$
Total UDL perpendicular to rafter	$F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = \mathbf{0.486}$ kN/m
Notional bearing length	$L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = \mathbf{11}$ mm
Effective span	$L_{eff} = L_{cl} + L_b = \mathbf{4909}$ mm

Check bending stress

Bending stress parallel to grain	$\sigma_m = \mathbf{5.300}$ N/mm^2
Permissible bending stress	$\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{5.948}$ N/mm^2
Applied bending stress	$\sigma_{m\_max} = F \times L_{eff}^2 / (8 \times Z) = \mathbf{2.989}$ 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} = **0.60**
Permissible compressive stress	$\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **4.476** N/mm^2
Applied compressive stress	$\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A)$ = **0.561** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress	$\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **12.373** N/mm^2
Euler coefficient	$K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e)$ = **0.959**
Combined axial compression and bending check	$\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm}$ = **0.649** **< 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	$\tau_{max} = 3 \times F \times L_{eff} / (4 \times A)$ = **0.152** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection	$\delta_{adm} = 0.003 \times L_{eff}$ = **14.727** mm
Bending deflection	$\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I)$ = **6.820** mm
Shear deflection	$\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A)$ = **0.272** mm
Total deflection	$\delta_{max} = \delta_b + \delta_s$ = **7.091** mm

PASS - Total deflection within permissible limits

Consider medium term load condition

Load duration factor	K_3 = **1.25**
Total UDL perpendicular to rafter	$F = [F_u \times \cos(\alpha)^2 + F_d \times \cos(\alpha)] \times s + F_j \times \cos(\alpha)$ = **0.774** kN/m
Notional bearing length	$L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)]$ = **17** mm
Effective span	$L_{eff} = L_{cl} + L_b$ = **4915** mm

Check bending stress

Bending stress parallel to grain	σ_m = **5.300** N/mm^2
Permissible bending stress	$\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **7.435** N/mm^2
Applied bending stress	$\sigma_{m\_max} = F \times L_{eff}^2 / (8 \times Z)$ = **4.773** 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	$\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **5.196** N/mm^2
Applied compressive stress	$\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A)$ = **0.894** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress	$\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **12.341** N/mm^2
Euler coefficient	$K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e)$ = **0.940**

Combined axial compression and bending check $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm}$ = **0.855** < 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 $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A)$ = **0.243** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = 0.003 \times L_{eff}$ = **14.745** mm

Bending deflection $\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I)$ = **10.920** mm

Shear deflection $\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A)$ = **0.434** mm

Total deflection $\delta_{max} = \delta_b + \delta_s$ = **11.354** mm

PASS - Total deflection within permissible limits

Consider short term load condition

Load duration factor K_3 = **1.50**

Total UDL perpendicular to rafter $F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha)$ = **0.486** kN/m

Notional bearing length $L_b = [F \times L_{cl} + F_p \times \cos(\alpha)] / [2 \times (b \times \sigma_{cp1} \times K_8 - F)]$ = **14** mm

Effective span $L_{eff} = L_{cl} + L_b$ = **4913** mm

Check bending stress

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

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **8.922** N/mm^2

Applied bending stress $\sigma_{m\_max} = F \times L_{eff}^2/(8 \times Z) + F_p \times \cos(\alpha) \times L_{eff}/(4 \times Z)$ = **5.206** 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} = **0.52**

Permissible compressive stress $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **5.781** N/mm^2

Applied compressive stress $\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha))/(2 \times A) + F_p \times \sin(\alpha)/A$ = **0.576** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress $\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **12.353** N/mm^2

Euler coefficient $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e)$ = **0.964**

Combined axial compression and bending check $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm}$ = **0.705** < 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 $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) + 3 \times F_p \times \cos(\alpha) / (2 \times A)$ = **0.265** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = 0.003 \times L_{eff}$ = **14.738** mm

Bending deflection $\delta_b = L_{eff}^3 \times (5 \times F \times L_{eff} / 384 + F_p \times \cos(\alpha)/ 48) / (E_{mean} \times I)$ = **10.887** mm

Shear deflection $\delta_s = 12 \times L_{eff} \times (F \times L_{eff} + 2 \times F_p \times \cos(\alpha))/(5 \times E_{mean} \times A)$ = **0.473** mm

Total deflection $\delta_{max} = \delta_b + \delta_s = \mathbf{11.360}$ mm

PASS - Total deflection within permissible limits

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