C. J. B. Structures
39 Georges Lane
Calverton
Nottingham
NG14 6JS

Tel 0115 8458086
e-mail c.bragg3@ntlworld.com

Consulting Engineers

Project No. 13/132
Structural Calculations
For the Proposed Alterations/Loft Conversion

St Quinten
Hallmoor Road
Darley Hillside
Matlock
DE4 2HF

Client

Ms E. Morton

Consultant

Rob Shephard

Jul-13

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Nottingham
NG14 6JS

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Calc. Sheet 1
Job Title St Quintens
Hallmoor Road
Matlock
Project No 13/132
Calcs by cjb
Checked by

Ref	Calculations

Introduction

The proposal on this dwelling is a loft conversion. The existing roof is to be removed and re-placed with a new roof. New first floor joists are provided adjacent to the existing ceiling joists. The new floor will be designed to support the roof structure via the loadbearing stud wall. The existing lintels will be checked for the additional load but it is anticipated that new lintels will be provided. The existing gables will be built up over the existing to the new ridge line. The existing foundations have been exposed and are suitable for the additional loading for the proposals. The sub-strata was a silty clay.

This report designs and details the necessary structural members required for the proposals.

Loadings

Roof		First floor	
Tiles	0.65	T & G	0.25
Felt & batts	0.1	Joists	0.15
Joists	0.15	Ceiling	0.2
Ceiling	0.4		
Dead Load	**1.40 Kn/sq. m**	Dead Load	**0.6 Kn/sq. m**
Live Load	**0.60 Kn/sq. m**	Live Load	**1.5 Kn/sq. m**
Brickwork	**2.1 Kn/sq. m**		
Blockwork	**1.5 Kn/sq. m**		

Tedds cjb Structures 39 Georges Lane Calverton Nottingham	Project St Quinten Hallmoor Road Matlock				Job Ref. 13/132	
	Section Velux trimmers ref ' c '				Sheet no./rev. 2	
	Calc. by cjb	Date 24/07/2013	Chk'd by	Date	App'd by	Date

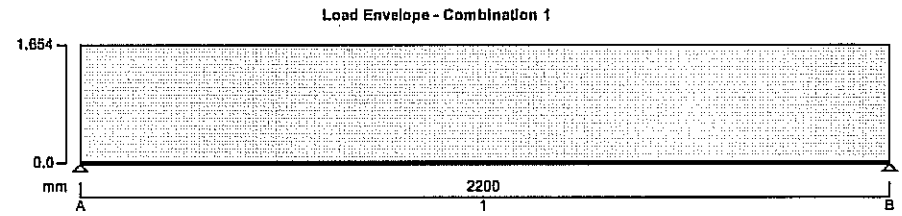
Trimmers adjacent velux roof lights.

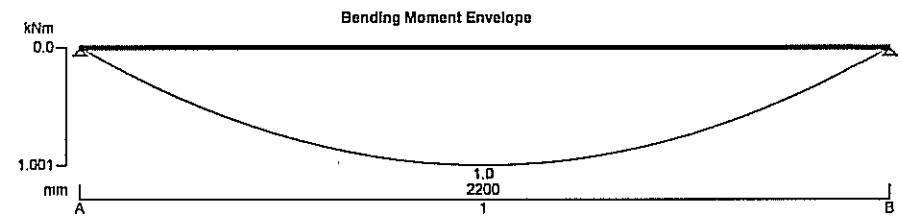
Load to trimmer Roof Gk 1.2 x 1.8/2 = 1.1Kn/mr
Qk 0.6 x 1.8/2 = 0.5Kn/mr

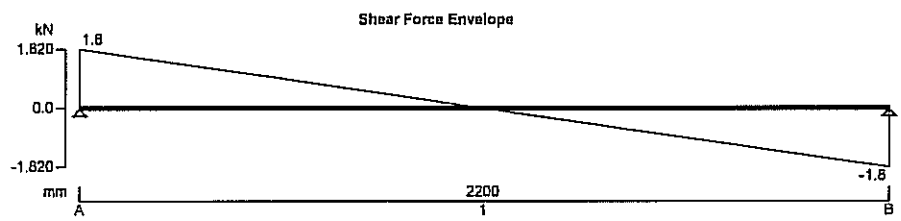
Span 2.2m

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.5.07







Applied loading

Beam loads

Dead self weight of beam × 1

Span 1 loads

roof	Dead UDL 1.100 kN/m from 0 mm to 2200 mm	
roof	Imposed UDL 0.500 kN/m from 0 mm to 2200 mm	

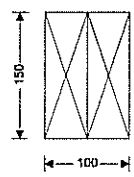
Load combinations

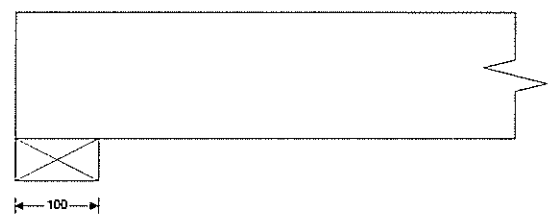
Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00

		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Maximum moment	M_{max} = **1.001** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **1.001** kNm	
Maximum shear	F_{max} = **1.820** kN	F_{min} = **-1.820** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **1.820** kN	
Total load on beam	W_{tot} = **3.640** kN	
Reactions at support A	R_{A_max} = **1.820** kN	R_{A_min} = **1.820** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **1.270** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **0.550** kN	
Reactions at support B	R_{B_max} = **1.820** kN	R_{B_min} = **1.820** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **1.270** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **0.550** kN	





Timber section details

Breadth of sections	b = **50** mm
Depth of sections	h = **150** mm
Number of sections in member	N = **2**
Overall breadth of member	b_b = N × b = **100** mm
Timber strength class	**C16**

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	A = N × b × h = **15000** mm^2
Section modulus	Z_x = N × b × h^2 / 6 = **375000** mm^3
	Z_y = h × (N × b)^2 / 6 = **250000** mm^3
Second moment of area	I_x = N × b × h^3 / 12 = **28125000** mm^4
	I_y = h × (N × b)^3 / 12 = **12500000** mm^4
Radius of gyration	i_x = √(I_x / A) = **43.3** mm
	i_y = √(I_y / A) = **28.9** 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 mm / h)$^{0.11}$ = **1.08**
Load sharing - cl.2.10.11	K_8 = **1.10**
Minimum modulus of elasticity - Table 20	K_9 = **1.14**

Lateral support - cl.2.10.8

No lateral support

Permissible depth-to-breadth ratio - Table 19	**2.00**
Actual depth-to-breadth ratio	$h / (N \times b) = 1.50$

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane)	$\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 = 2.420$ N/mm^2
Applied bearing stress	$\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b) = 0.182$ N/mm^2
	$\sigma_{c\_a} / \sigma_{c\_adm} = 0.075$

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

Bending parallel to grain

Permissible bending stress	$\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 6.292$ N/mm^2
Applied bending stress	$\sigma_{m\_a} = M / Z_x = 2.669$ N/mm^2
	$\sigma_{m\_a} / \sigma_{m\_adm} = 0.424$

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress	$\tau_{adm} = \tau \times K_3 \times K_8 = 0.737$ N/mm^2
Applied shear stress	$\tau_a = 3 \times F / (2 \times A) = 0.182$ N/mm^2
	$\tau_a / \tau_{adm} = 0.247$

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection	$E = E_{min} \times K_9 = 6612$ N/mm^2
Permissible deflection	$\delta_{adm} = \min(14 \text{ mm}, 0.003 \times L_{s1}) = 6.600$ mm
Bending deflection	$\delta_{b\_s1} = 2.714$ mm
Shear deflection	$\delta_{v\_s1} = 0.194$ mm
Total deflection	$\delta_a = \delta_{b\_s1} + \delta_{v\_s1} = 2.907$ mm
	$\delta_a / \delta_{adm} = 0.441$

PASS - Total deflection is less than permissible deflection

Provide 2No 50 x 150mm C16 joists bolted

Tedds cjb Structures 39 Georges Lane Calverton Nottingham	Project St Quinten Hallmoor Road Matlock				Job Ref. 13/132	
	Section Dormer roof/cheek support ref ' e '				Sheet no./rev. 5	
	Calc. by cjb	Date 24/07/2013	Chk'd by	Date	App'd by	Date

Dormer cheek/roof supports.

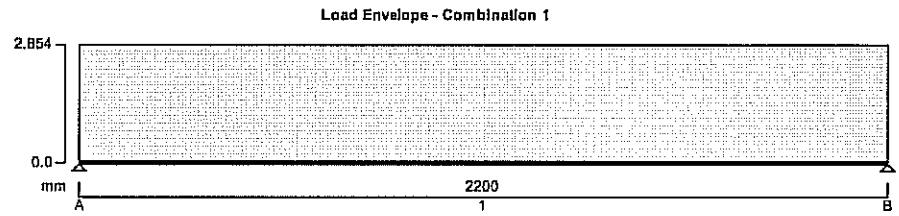
Load to trimmer Roof Gk 1.2 x 2.5/2 = 1.5Kn/mr
Qk 0.6 x 2.5/2 = 0.8Kn/mr

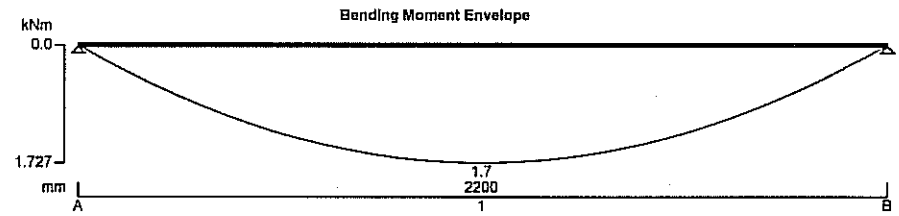
Cheek Gk 1.0 x 0.5 = 0.5Kn/mr

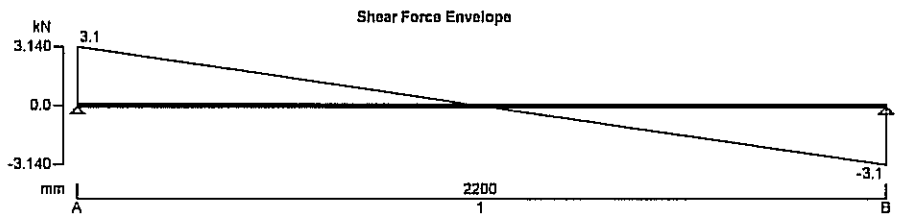
Span 2.2m

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.5.07







Applied loading

Beam loads

Dead self weight of beam x 1

Span 1 loads

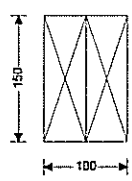
roof	Dead UDL 1.500 kN/m from 0 mm to 2200 mm
roof	Imposed UDL 0.800 kN/m from 0 mm to 2200 mm
cheek	Dead UDL 0.500 kN/m from 0 mm to 2200 mm

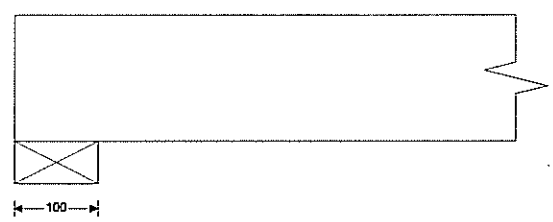
Load combinations

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

Analysis results

Maximum moment	M_{max} = **1.727** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **1.727** kNm	
Maximum shear	F_{max} = **3.140** kN	F_{min} = **-3.140** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **3.140** kN	
Total load on beam	W_{tot} = **6.280** kN	
Reactions at support A	R_{A_max} = **3.140** kN	R_{A_min} = **3.140** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **2.260** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **0.880** kN	
Reactions at support B	R_{B_max} = **3.140** kN	R_{B_min} = **3.140** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **2.260** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **0.880** kN	





Timber section details

Breadth of sections	b = **50** mm
Depth of sections	h = **150** mm
Number of sections in member	N = **2**
Overall breadth of member	b_b = N × b = **100** mm
Timber strength class	**C16**

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	A = N × b × h = **15000** mm^2
Section modulus	Z_x = N × b × h^2 / 6 = **375000** mm^3
	Z_y = h × (N × b)^2 / 6 = **250000** mm^3
Second moment of area	I_x = N × b × h^3 / 12 = **28125000** mm^4
	I_y = h × (N × b)^3 / 12 = **12500000** mm^4
Radius of gyration	i_x = √(I_x / A) = **43.3** mm
	i_y = √(I_y / A) = **28.9** 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	$K_7 = (300\ mm / h)^{0.11}$ = **1.08**
Load sharing - cl.2.10.11	K_8 = **1.10**
Minimum modulus of elasticity - Table 20	K_9 = **1.14**

Lateral support - cl.2.10.8

No lateral support

Permissible depth-to-breadth ratio - Table 19	**2.00**
Actual depth-to-breadth ratio	$h / (N \times b)$ = **1.50**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane)	$\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **2.420** N/mm^2
Applied bearing stress	$\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b)$ = **0.314** N/mm^2
	$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.130**

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

Bending parallel to grain

Permissible bending stress	$\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **6.292** N/mm^2
Applied bending stress	$\sigma_{m\_a} = M / Z_x$ = **4.605** N/mm^2
	$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.732**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress	$\tau_{adm} = \tau \times K_3 \times K_8$ = **0.737** N/mm^2
Applied shear stress	$\tau_a = 3 \times F / (2 \times A)$ = **0.314** N/mm^2
	τ_a / τ_{adm} = **0.426**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection	$E = E_{min} \times K_9$ = **6612** N/mm^2
Permissible deflection	$\delta_{adm} = \min(14\ mm, 0.003 \times L_{s1})$ = **6.600** mm
Bending deflection	δ_{b_s1} = **4.682** mm
Shear deflection	δ_{v_s1} = **0.334** mm
Total deflection	$\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **5.016** mm
	δ_a / δ_{adm} = **0.760**

PASS - Total deflection is less than permissible deflection

Provide 2No 50 x 150mm C16 joists bolted

Tedds cjb Structures 39 Georges Lane Calverton Nottingham	Project St Quinten Hallmoor Road Matlock				Job Ref. 13/132	
	Section Velux trimmer over staircase ref ' f '				Sheet no./rev. 8	
	Calc. by cjb	Date 24/07/2013	Chk'd by	Date	App'd by	Date

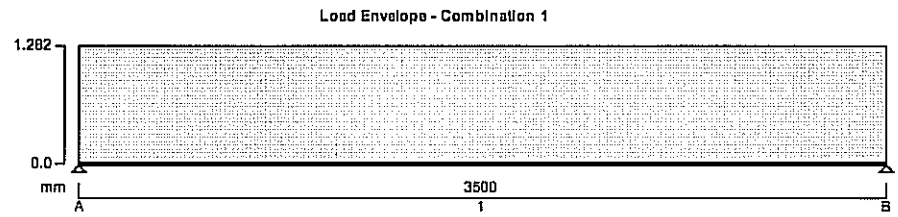
Trimmer to velux over staircase.

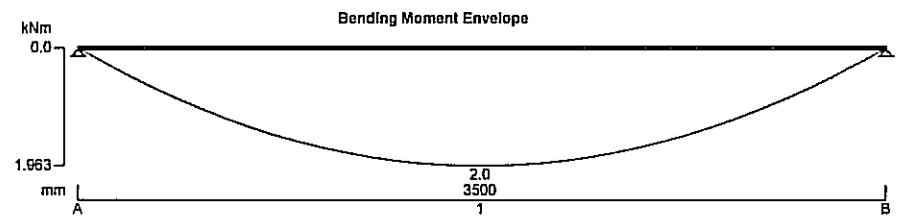
Load to trimmer Roof Gk 1.2 x 1.3/2 = 0.8Kn/mr
Qk 0.6 x 1.3/2 = 0.4Kn/mr

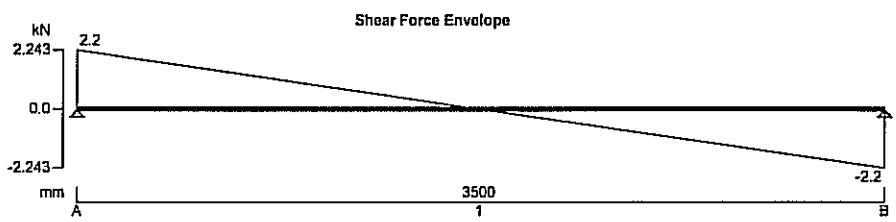
Span 3.5m

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.5.07







Applied loading

Beam loads

Dead self weight of beam × 1

Span 1 loads

roof — Dead UDL 0.800 kN/m from 0 mm to 3500 mm

roof — Imposed UDL 0.400 kN/m from 0 mm to 3500 mm

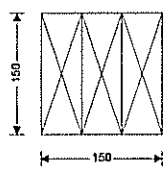
Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00

	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Maximum moment	M_{max} = **1.963** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **1.963** kNm	
Maximum shear	F_{max} = **2.243** kN	F_{min} = **-2.243** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **2.243** kN	
Total load on beam	W_{tot} = **4.486** kN	
Reactions at support A	R_{A_max} = **2.243** kN	R_{A_min} = **2.243** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **1.543** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **0.700** kN	
Reactions at support B	R_{B_max} = **2.243** kN	R_{B_min} = **2.243** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **1.543** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **0.700** kN	



Timber section details

Breadth of sections	b = **50** mm
Depth of sections	h = **150** mm
Number of sections in member	N = **3**
Overall breadth of member	b_b = N × b = **150** mm
Timber strength class	**C16**

Member details

Service class of timber	1
Load duration	**Long term**
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	A = N × b × h = **22500** mm^2
Section modulus	Z_x = N × b × h^2 / 6 = **562500** mm^3
	Z_y = h × (N × b)^2 / 6 = **562500** mm^3
Second moment of area	I_x = N × b × h^3 / 12 = **42187500** mm^4
	I_y = h × (N × b)^3 / 12 = **42187500** mm^4
Radius of gyration	i_x = √(I_x / A) = **43.3** mm
	i_y = √(I_y / A) = **43.3** 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 mm / h)$^{0.11}$ = **1.08**
Load sharing - cl.2.10.11	K_8 = **1.10**

Minimum modulus of elasticity - Table 20 K_9 = **1.21**

Lateral support - cl.2.10.8

No lateral support

Permissible depth-to-breadth ratio - Table 19 **2.00**

Actual depth-to-breadth ratio $h / (N \times b)$ = **1.00**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **2.420** N/mm^2

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b)$ = **0.150** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.062**

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

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **6.292** N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x$ = **3.489** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.555**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

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

Applied shear stress $\tau_a = 3 \times F / (2 \times A)$ = **0.150** N/mm^2

τ_a / τ_{adm} = **0.203**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min} \times K_9$ = **7018** N/mm^2

Permissible deflection $\delta_{adm} = \min(14\ mm, 0.003 \times L_{s1})$ = **10.500** mm

Bending deflection δ_{b_s1} = **8.458** mm

Shear deflection δ_{v_s1} = **0.239** mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **8.697** mm

δ_a / δ_{adm} = **0.828**

PASS - Total deflection is less than permissible deflection

Provide 3No 50 x 150mm C16 joists bolted

Tedds cjb Structures 39 Georges Lane Calverton Nottingham	Project St Quinten Hallmoor Road Matlock				Job Ref. 13/132	
	Section Valley beam to dormers ref 'g'				Sheet no./rev. 11	
	Calc. by cjb	Date 24/07/2013	Chk'd by	Date	App'd by	Date

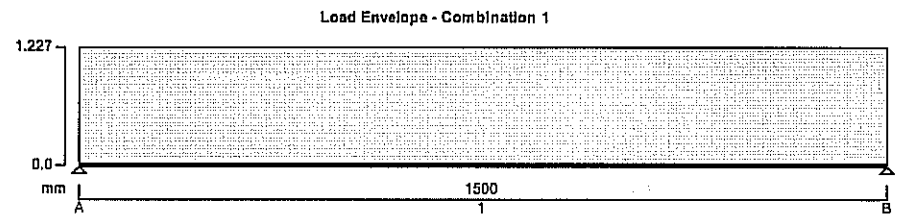
Valley beam to dormers.

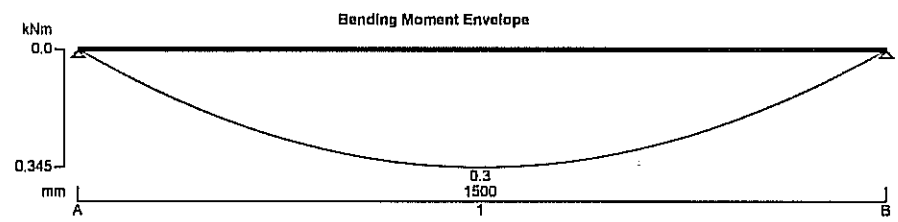
Load to trimmer Roof Gk 1.2 x 0.7 = 0.8Kn/mr
Qk 0.6 x 0.7 = 0.4Kn/mr

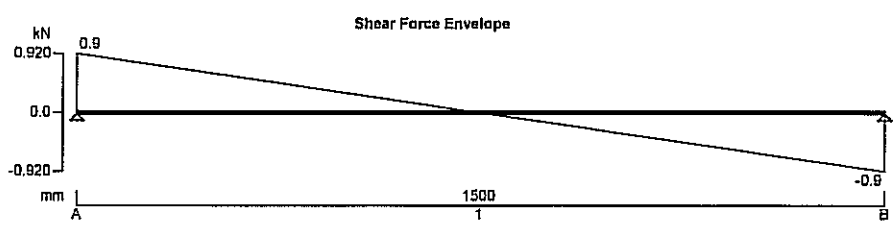
Span 1.5m

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.5.07







Applied loading

Beam loads

Dead self weight of beam × 1

Span 1 loads

roof — Dead UDL 0.800 kN/m from 0 mm to 1500 mm

roof — Imposed UDL 0.400 kN/m from 0 mm to 1500 mm

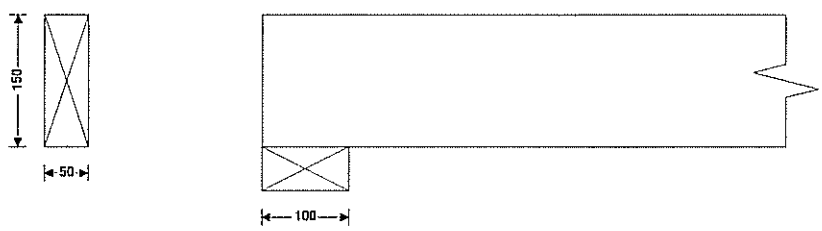
Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00

	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Maximum moment	M_{max} = **0.345** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **0.345** kNm	
Maximum shear	F_{max} = **0.920** kN	F_{min} = **-0.920** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **0.920** kN	
Total load on beam	W_{tot} = **1.841** kN	
Reactions at support A	R_{A_max} = **0.920** kN	R_{A_min} = **0.920** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **0.620** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **0.300** kN	
Reactions at support B	R_{B_max} = **0.920** kN	R_{B_min} = **0.920** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **0.620** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **0.300** kN	



Timber section details

Breadth of sections	b = **50** mm
Depth of sections	h = **150** mm
Number of sections in member	N = **1**
Overall breadth of member	b_b = N × b = **50** mm
Timber strength class	**C16**

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	A = N × b × h = **7500** mm^2
Section modulus	Z_x = N × b × h^2 / 6 = **187500** mm^3
	Z_y = h × (N × b)^2 / 6 = **62500** mm^3
Second moment of area	I_x = N × b × h^3 / 12 = **14062500** mm^4
	I_y = h × (N × b)^3 / 12 = **1562500** mm^4
Radius of gyration	i_x = √(I_x / A) = **43.3** mm
	i_y = √(I_y / A) = **14.4** 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 mm / h)$^{0.11}$ = **1.08**

Load sharing - cl.2.9	K_8 = **1.00**

Lateral support - cl.2.10.8

Ends held in position	
Permissible depth-to-breadth ratio - Table 19	**3.00**
Actual depth-to-breadth ratio	$h / (N \times b)$ = **3.00**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane)	$\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **2.200** N/mm^2
Applied bearing stress	$\sigma_{c\_a} = R_{B\_max} / (N \times b \times L_b)$ = **0.184** N/mm^2
	$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.084**

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

Bending parallel to grain

Permissible bending stress	$\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **5.720** N/mm^2
Applied bending stress	$\sigma_{m\_a} = M / Z_x$ = **1.841** N/mm^2
	$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.322**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress	$\tau_{adm} = \tau \times K_3 \times K_8$ = **0.670** N/mm^2
Applied shear stress	$\tau_a = 3 \times F / (2 \times A)$ = **0.184** N/mm^2
	τ_a / τ_{adm} = **0.275**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection	$E = E_{min}$ = **5800** N/mm^2
Permissible deflection	$\delta_{adm} = \min(14\text{ mm}, 0.003 \times L_{s1})$ = **4.500** mm
Bending deflection	δ_{b_s1} = **0.992** mm
Shear deflection	δ_{v_s1} = **0.152** mm
Total deflection	$\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **1.144** mm
	δ_a / δ_{adm} = **0.254**

PASS - Total deflection is less than permissible deflection

Provide 3No 50 x 150mm C16 joists bolted

The first floor joists support the main roof via the loadbearing stud wall. The joists are at 450mm c/c to match the existing ceiling joists. So below the floor joist is designed as a beam with the loads x 0.45 as a point load.

Load to trimmer Roof Gk 1.2 x 3.5/2 = 2.1 x 0.45 = 1.0Kn at 2.2m
Qk 0.6 x 3.5/2 = 1.0 x 0.45 = 0.5Kn

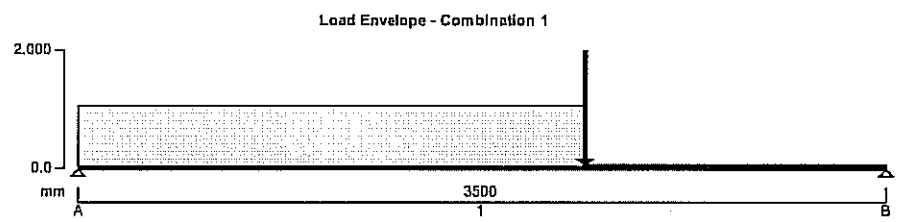
Stud Gk 1.0 x 1.0 = 1.0 x 0.45 = 0.5Kn

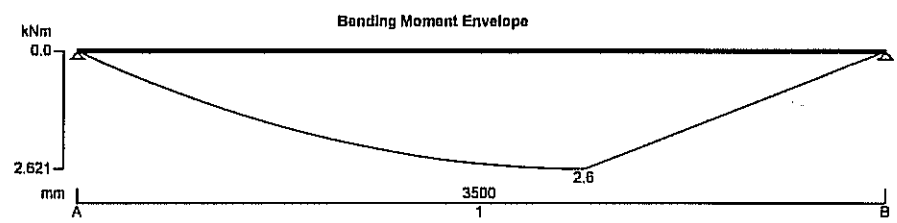
Floor Gk 0.6 x 0.45 = 0.3Kn/mr
Qk 1.5 x 0.45 = 0.7Kn/mr

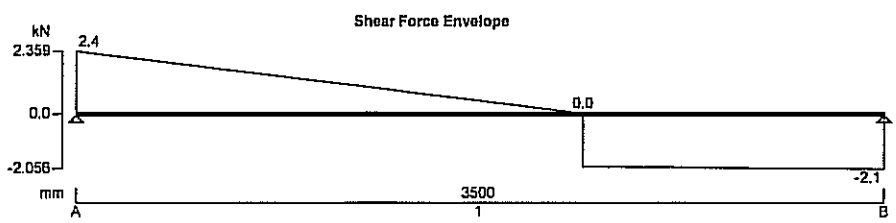
Span 3.5m

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.5.07







Applied loading

Beam loads

Dead self weight of beam x 1

Span 1 loads

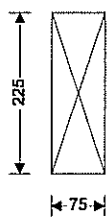
roof	Dead point load 1.000 kN at 2200 mm
roof	Imposed point load 0.500 kN at 2200 mm
stud	Dead point load 0.500 kN at 2200 mm
floor	Dead UDL 0.300 kN/m from 0 mm to 2200 mm
floor	Imposed UDL 0.700 kN/m from 0 mm to 2200 mm

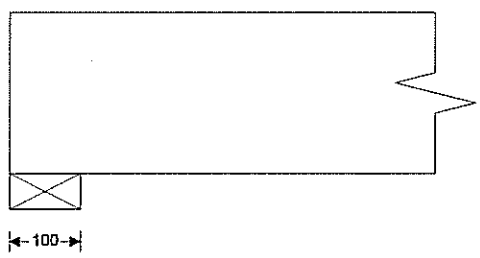
Load combinations

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

Analysis results

Maximum moment	M_{max} = **2.621** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **2.621** kNm	
Maximum shear	F_{max} = **2.359** kN	F_{min} = **-2.056** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **2.359** kN	
Total load on beam	W_{tot} = **4.414** kN	
Reactions at support A	R_{A_max} = **2.359** kN	R_{A_min} = **2.359** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **1.117** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **1.242** kN	
Reactions at support B	R_{B_max} = **2.056** kN	R_{B_min} = **2.056** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **1.257** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **0.798** kN	





Timber section details

Breadth of sections	b = **75** mm
Depth of sections	h = **225** mm
Number of sections in member	N = **1**
Overall breadth of member	b_b = N × b = **75** mm
Timber strength class	**C16**

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	A = N × b × h = **16875** mm^2

Section modulus	$Z_x = N \times b \times h^2 / 6 =$ **632813** mm^3
	$Z_y = h \times (N \times b)^2 / 6 =$ **210937** mm^3
Second moment of area	$I_x = N \times b \times h^3 / 12 =$ **71191406** mm^4
	$I_y = h \times (N \times b)^3 / 12 =$ **7910156** mm^4
Radius of gyration	$i_x = \sqrt{(I_x / A)} =$ **65.0** mm
	$i_y = \sqrt{(I_y / A)} =$ **21.7** 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	$K_7 = (300 \text{ mm} / h)^{0.11} =$ **1.03**
Load sharing - cl.2.9	$K_8 =$ **1.00**

Lateral support - cl.2.10.8

Ends held in position	
Permissible depth-to-breadth ratio - Table 19	**3.00**
Actual depth-to-breadth ratio	$h / (N \times b) =$ **3.00**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane)	$\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 =$ **2.200** N/mm^2
Applied bearing stress	$\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b) =$ **0.314** N/mm^2
	$\sigma_{c\_a} / \sigma_{c\_adm} =$ **0.143**

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

Bending parallel to grain

Permissible bending stress	$\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 =$ **5.470** N/mm^2
Applied bending stress	$\sigma_{m\_a} = M / Z_x =$ **4.141** N/mm^2
	$\sigma_{m\_a} / \sigma_{m\_adm} =$ **0.757**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress	$\tau_{adm} = \tau \times K_3 \times K_8 =$ **0.670** N/mm^2
Applied shear stress	$\tau_a = 3 \times F / (2 \times A) =$ **0.210** N/mm^2
	$\tau_a / \tau_{adm} =$ **0.313**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection	$E = E_{mln} =$ **5800** N/mm^2
Permissible deflection	$\delta_{adm} = \min(14 \text{ mm}, 0.003 \times L_{s1}) =$ **10.500** mm
Bending deflection	$\delta_{b\_s1} =$ **7.536** mm
Shear deflection	$\delta_{v\_s1} =$ **0.514** mm
Total deflection	$\delta_a = \delta_{b\_s1} + \delta_{v\_s1} =$ **8.051** mm
	$\delta_a / \delta_{adm} =$ **0.767**

PASS - Total deflection is less than permissible deflection

Provide 75 x 225mm C16 joists at 450mm c/c

Tedds cjb Structures 39 Georges Lane Calverton Nottingham	Project St Quinten Hallmoor Road Matlock				Job Ref. 13/132	
	Section First floor joists ref ' j ' supporting dormer				Sheet no./rev. 17	
	Calc. by cjb	Date 24/07/2013	Chk'd by	Date	App'd by	Date

The first floor joists support the main roof via the loadbearing stud wall. The joists are at 450mm c/c to match the existing ceiling joists. So below the floor joist is designed as a beam with the loads x 0.45 as a point load.

Load to trimmer Roof Gk 1.2 x 3.5/2 = 2.1 x 0.45 = 1.0Kn at 2.2m
Qk 0.6 x 3.5/2 = 1.0 x 0.45 = 0.5Kn

Stud Gk 1.0 x 1.0 = 1.0 x 0.45 = 0.5Kn

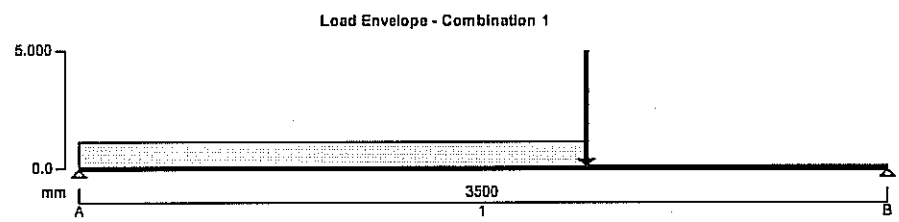
Floor Gk 0.6 x 0.45 = 0.3Kn/mr
Qk 1.5 x 0.45 = 0.7Kn/mr

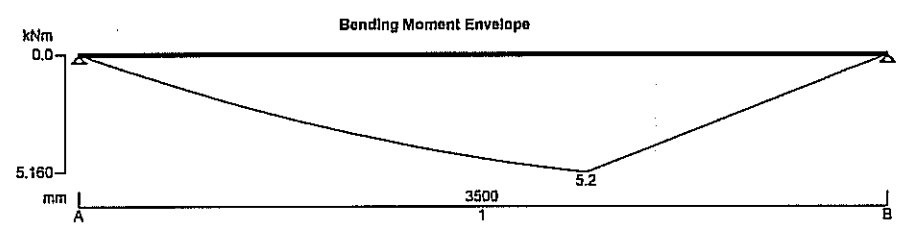
Point load from beam ' e ' Gk 2Kn & Qk 1Kn at 2.2m

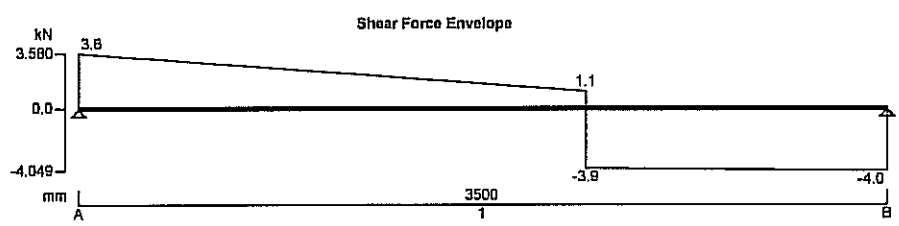
Span 3.5m

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.5.07







Applied loading

Beam loads

Dead self weight of beam × 1

Span 1 loads

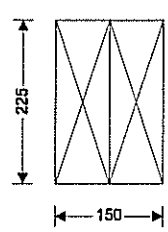
roof	Dead point load 1.000 kN at 2200 mm
roof	Imposed point load 0.500 kN at 2200 mm
stud	Dead point load 0.500 kN at 2200 mm
floor	Dead UDL 0.300 kN/m from 0 mm to 2200 mm
floor	Imposed UDL 0.700 kN/m from 0 mm to 2200 mm
bm e	Dead point load 2.000 kN at 2200 mm
bm e	Imposed point load 1.000 kN at 2200 mm

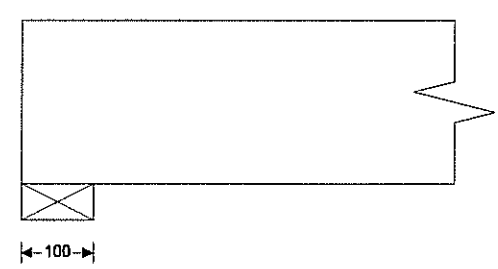
Load combinations

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

Analysis results

Maximum moment	M_{max} = **5.160** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **5.160** kNm	
Maximum shear	F_{max} = **3.580** kN	F_{min} = **-4.049** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **4.049** kN	
Total load on beam	W_{tot} = **7.629** kN	
Reactions at support A	R_{A_max} = **3.580** kN	R_{A_min} = **3.580** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **1.967** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **1.613** kN	
Reactions at support B	R_{B_max} = **4.049** kN	R_{B_min} = **4.049** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **2.622** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **1.427** kN	





Timber section details

Breadth of sections	b = **75** mm
Depth of sections	h = **225** mm
Number of sections in member	N = **2**
Overall breadth of member	b_b = N × b = **150** mm
Timber strength class	**C16**

Tedds	Project				Job Ref.	
	St Quinten Hallmoor Road Matlock				13/132	
cjb Structures 39 Georges Lane Calverton Nottingham	Section				Sheet no./rev.	
	First floor joists ref ' j ' supporting dormer				19	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	cjb	24/07/2013				

Member details

Service class of timber	1
Load duration	**Long term**
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	$A = N \times b \times h$ = **33750** mm^2
Section modulus	$Z_x = N \times b \times h^2 / 6$ = **1265625** mm^3
	$Z_y = h \times (N \times b)^2 / 6$ = **843750** mm^3
Second moment of area	$I_x = N \times b \times h^3 / 12$ = **142382813** mm^4
	$I_y = h \times (N \times b)^3 / 12$ = **63281250** mm^4
Radius of gyration	$i_x = \sqrt{(I_x / A)}$ = **65.0** mm
	$i_y = \sqrt{(I_y / A)}$ = **43.3** 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	$K_7 = (300 \text{ mm} / h)^{0.11}$ = **1.03**
Load sharing - cl.2.10.11	K_8 = **1.10**
Minimum modulus of elasticity - Table 20	K_9 = **1.14**

Lateral support - cl.2.10.8

Ends held in position	
Permissible depth-to-breadth ratio - Table 19	**3.00**
Actual depth-to-breadth ratio	$h / (N \times b)$ = **1.50**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane)	$\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **2.420** N/mm^2
Applied bearing stress	$\sigma_{c\_a} = R_{B\_max} / (N \times b \times L_b)$ = **0.270** N/mm^2
	$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.112**

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

Bending parallel to grain

Permissible bending stress	$\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **6.017** N/mm^2
Applied bending stress	$\sigma_{m\_a} = M / Z_x$ = **4.077** N/mm^2
	$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.677**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress	$\tau_{adm} = \tau \times K_3 \times K_8$ = **0.737** N/mm^2
Applied shear stress	$\tau_a = 3 \times F / (2 \times A)$ = **0.180** N/mm^2
	τ_a / τ_{adm} = **0.244**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection	$E = E_{min} \times K_9$ = **6612** N/mm^2
Permissible deflection	$\delta_{adm} = \min(14 \text{ mm}, 0.003 \times L_{s1})$ = **10.500** mm
Bending deflection	δ_{b_s1} = **6.032** mm
Shear deflection	δ_{v_s1} = **0.444** mm
Total deflection	$\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **6.476** mm
	δ_a / δ_{adm} = **0.617**

PASS - Total deflection is less than permissible deflection

Provide 2No 75 x 225mm C16 joists bolted

Tedds cjb Structures 39 Georges Lane Calverton Nottingham	Project St Quinten Hallmoor Road Matlock				Job Ref. 13/132	
	Section First floor joists ref ' I ' supporting dormer fascia				Sheet no./rev. 21	
	Calc. by cjb	Date 24/07/2013	Chk'd by	Date	App'd by	Date

The first floor joists support the dormer fascia. The joists are at 450mm c/c to match the existing ceiling joists. So below the floor joist is designed as a beam with the loads x 0.45 as a point load.

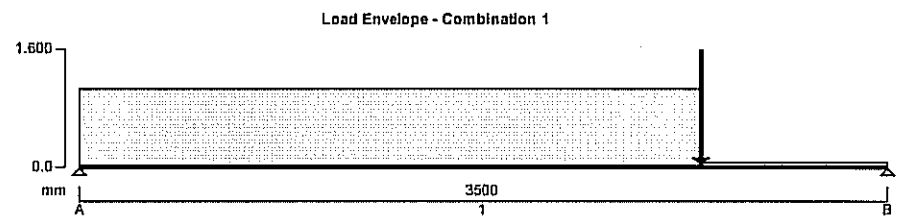
Load to floor Fascia Gk 1.0 x 3.5 = 3.5 x 0.45 = 1.6Kn at 2.7m

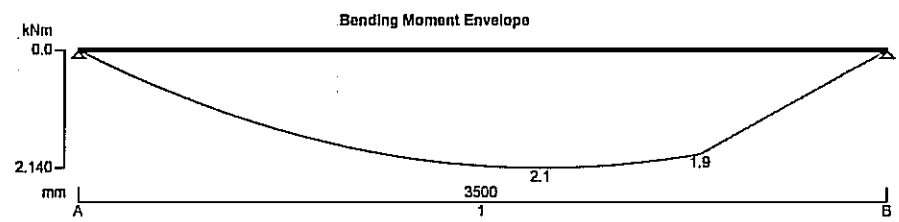
Floor Gk 0.6 x 0.45 = 0.3Kn/mr 0 – 2.7m
Qk 1.5 x 0.45 = 0.7Kn/mr

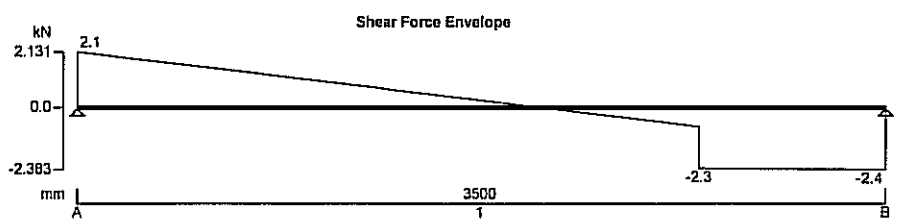
Span 3.5m

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.5.07







Applied loading

Beam loads

Dead self weight of beam × 1

Span 1 loads

fascia — Dead point load 1.600 kN at 2700 mm

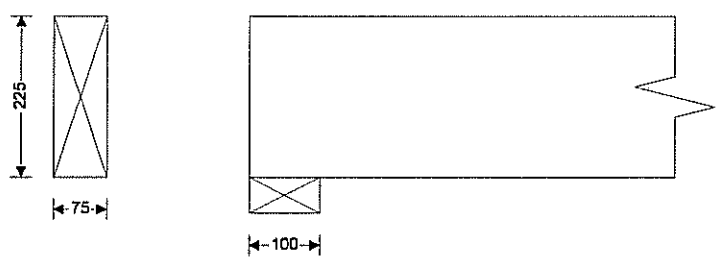
floor	Dead UDL 0.300 kN/m from 0 mm to 2700 mm	
floor	Imposed UDL 0.700 kN/m from 0 mm to 2700 mm	

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
		Imposed × 1.00

Analysis results

Maximum moment	M_{max} = **2.140** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **2.140** kNm	
Maximum shear	F_{max} = **2.131** kN	F_{min} = **-2.383** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **2.383** kN	
Total load on beam	W_{tot} = **4.514** kN	
Reactions at support A	R_{A_max} = **2.131** kN	R_{A_min} = **2.131** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **0.970** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **1.161** kN	
Reactions at support B	R_{B_max} = **2.383** kN	R_{B_min} = **2.383** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **1.654** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **0.729** kN	



Timber section details

Breadth of sections	b = **75** mm
Depth of sections	h = **225** mm
Number of sections in member	N = **1**
Overall breadth of member	b_b = N × b = **75** mm
Timber strength class	**C16**

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	A = N × b × h = **16875** mm^2
Section modulus	Z_x = N × b × h^2 / 6 = **632813** mm^3
	Z_y = h × (N × b)^2 / 6 = **210937** mm^3
Second moment of area	I_x = N × b × h^3 / 12 = **71191406** mm^4
	I_y = h × (N × b)^3 / 12 = **7910156** mm^4

Radius of gyration	$i_x = \sqrt{(I_x / A)} = \mathbf{65.0}$ mm
	$i_y = \sqrt{(I_y / A)} = \mathbf{21.7}$ mm

Modification factors

Duration of loading - Table 17	$K_3 = \mathbf{1.00}$
Bearing stress - Table 18	$K_4 = \mathbf{1.00}$
Total depth of member - cl.2.10.6	$K_7 = (300 \text{ mm} / h)^{0.11} = \mathbf{1.03}$
Load sharing - cl.2.9	$K_8 = \mathbf{1.00}$

Lateral support - cl.2.10.8

Ends held in position	
Permissible depth-to-breadth ratio - Table 19	**3.00**
Actual depth-to-breadth ratio	$h / (N \times b) = \mathbf{3.00}$

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane)	$\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 = \mathbf{2.200}$ N/mm^2
Applied bearing stress	$\sigma_{c\_a} = R_{B\_max} / (N \times b \times L_b) = \mathbf{0.318}$ N/mm^2
	$\sigma_{c\_a} / \sigma_{c\_adm} = \mathbf{0.144}$

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

Bending parallel to grain

Permissible bending stress	$\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{5.470}$ N/mm^2
Applied bending stress	$\sigma_{m\_a} = M / Z_x = \mathbf{3.382}$ N/mm^2
	$\sigma_{m\_a} / \sigma_{m\_adm} = \mathbf{0.618}$

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress	$\tau_{adm} = \tau \times K_3 \times K_8 = \mathbf{0.670}$ N/mm^2
Applied shear stress	$\tau_a = 3 \times F / (2 \times A) = \mathbf{0.212}$ N/mm^2
	$\tau_a / \tau_{adm} = \mathbf{0.316}$

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection	$E = E_{min} = \mathbf{5800}$ N/mm^2
Permissible deflection	$\delta_{adm} = \min(14 \text{ mm}, 0.003 \times L_{s1}) = \mathbf{10.500}$ mm
Bending deflection	$\delta_{b\_s1} = \mathbf{6.664}$ mm
Shear deflection	$\delta_{v\_s1} = \mathbf{0.420}$ mm
Total deflection	$\delta_a = \delta_{b\_s1} + \delta_{v\_s1} = \mathbf{7.084}$ mm
	$\delta_a / \delta_{adm} = \mathbf{0.675}$

PASS - Total deflection is less than permissible deflection

Provide 75 x 225mm C16 joists at 450mm c/c

Tedds cjb Structures 39 Georges Lane Calverton Nottingham	Project St Quinten Hallmoor Road Matlock				Job Ref. 13/132	
	Section Typical lintel ref ' m '				Sheet no./rev. 24	
	Calc. by cjb	Date 24/07/2013	Chk'd by	Date	App'd by	Date

Position the lintel over the existing in the same zone as the floor joists and support the floor joists off joist hangers.

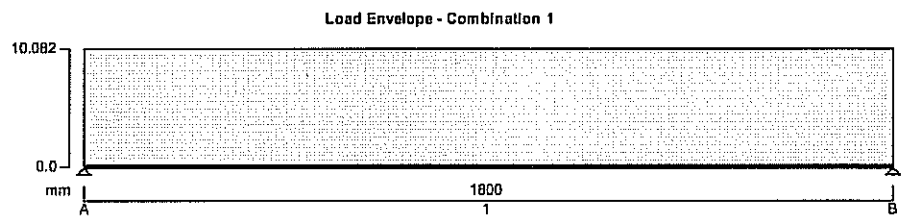
Load to lintel Roof Gk 1.2 x 7/2 = 4.2Kn/mr
Qk 0.6 x 7/2 = 2.1Kn/mr

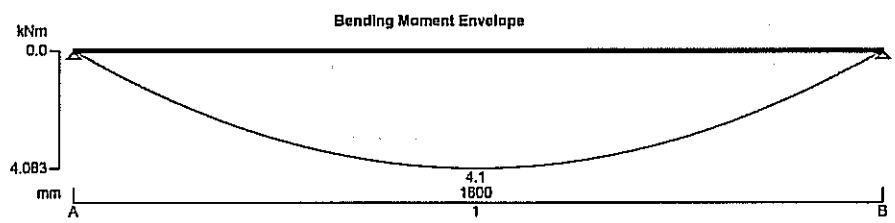
Floor Gk 0.6 x 3.5/2 = 1.1Kn/mr
Qk 1.5 x 3.5/2 = 2.6Kn/mr

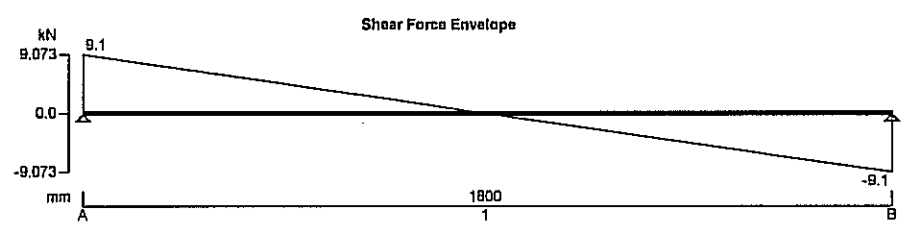
Span 1.8m

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.5.07







Applied loading

Beam loads

Dead self weight of beam x 1

Span 1 loads

roof

Dead UDL 4.200 kN/m from 0 mm to 1800 mm

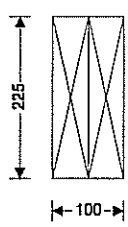
roof	Imposed UDL 2.100 kN/m from 0 mm to 1800 mm	
floor	Dead UDL 1.100 kN/m from 0 mm to 1800 mm	
floor	Imposed UDL 2.600 kN/m from 0 mm to 1800 mm	

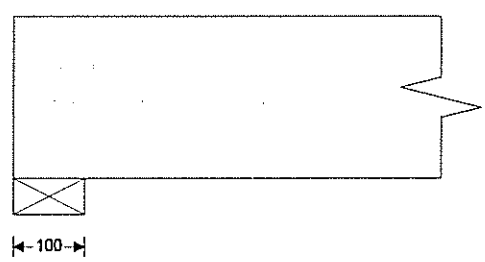
Load combinations

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

Analysis results

Maximum moment	M_{max} = **4.083** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **4.083** kNm	
Maximum shear	F_{max} = **9.073** kN	F_{min} = **-9.073** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **9.073** kN	
Total load on beam	W_{tot} = **18.147** kN	
Reactions at support A	R_{A_max} = **9.073** kN	R_{A_min} = **9.073** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **4.843** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **4.230** kN	
Reactions at support B	R_{B_max} = **9.073** kN	R_{B_min} = **9.073** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **4.843** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **4.230** kN	





Timber section details

Breadth of sections	b = **50** mm
Depth of sections	h = **225** mm
Number of sections in member	N = **2**
Overall breadth of member	b_b = N × b = **100** mm
Timber strength class	**C16**

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	A = N × b × h = **22500** mm^2
Section modulus	Z_x = N × b × h^2 / 6 = **843750** mm^3
	Z_y = h × (N × b)^2 / 6 = **375000** mm^3
Second moment of area	I_x = N × b × h^3 / 12 = **94921875** mm^4

	$I_y = h \times (N \times b)^3 / 12 =$ **18750000** mm^4
Radius of gyration	$i_x = \sqrt{(I_x / A)} =$ **65.0** mm
	$i_y = \sqrt{(I_y / A)} =$ **28.9** 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	$K_7 = (300 \text{ mm} / h)^{0.11} =$ **1.03**
Load sharing - cl.2.10.11	$K_8 =$ **1.10**
Minimum modulus of elasticity - Table 20	$K_9 =$ **1.14**

Lateral support - cl.2.10.8

Ends held in position	
Permissible depth-to-breadth ratio - Table 19	**3.00**
Actual depth-to-breadth ratio	$h / (N \times b) =$ **2.25**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane)	$\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 =$ **2.420** N/mm^2
Applied bearing stress	$\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b) =$ **0.907** N/mm^2
	$\sigma_{c\_a} / \sigma_{c\_adm} =$ **0.375**

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

Bending parallel to grain

Permissible bending stress	$\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 =$ **6.017** N/mm^2
Applied bending stress	$\sigma_{m\_a} = M / Z_x =$ **4.839** N/mm^2
	$\sigma_{m\_a} / \sigma_{m\_adm} =$ **0.804**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress	$\tau_{adm} = \tau \times K_3 \times K_8 =$ **0.737** N/mm^2
Applied shear stress	$\tau_a = 3 \times F / (2 \times A) =$ **0.605** N/mm^2
	$\tau_a / \tau_{adm} =$ **0.821**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection	$E = E_{min} \times K_9 =$ **6612** N/mm^2
Permissible deflection	$\delta_{adm} = \min(14 \text{ mm}, 0.003 \times L_{s1}) =$ **5.400** mm
Bending deflection	$\delta_{b\_s1} =$ **2.196** mm
Shear deflection	$\delta_{v\_s1} =$ **0.527** mm
Total deflection	$\delta_a = \delta_{b\_s1} + \delta_{v\_s1} =$ **2.723** mm
	$\delta_a / \delta_{adm} =$ **0.504**

PASS - Total deflection is less than permissible deflection

Provide 2No 50 x 225mm C16 joists bolted

Tedds cjb Structures 39 Georges Lane Calverton Nottingham	Project St Quinten Hallmoor Road Matlock				Job Ref. 13/132	
	Section Lintel over bi-fold doors ref 'm'				Sheet no./rev. 27	
	Calc. by cjb	Date 24/07/2013	Chk'd by	Date	App'd by	Date

Lintel over bi-fold doors. Beam fully restrained.

Load to beam Roof Gk 1.2 x 7/2 = 4.2Kn/mr
Qk 0.6 x 7/2 = 2.1Kn/mr

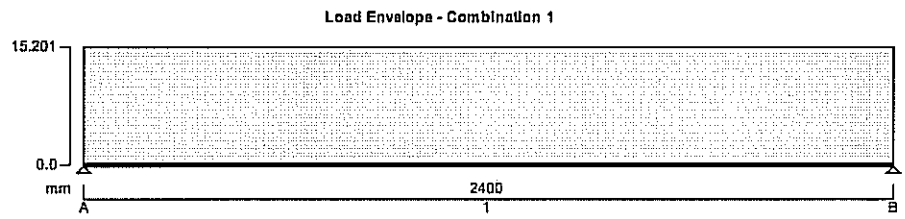
Floor Gk 0.6 x 3.5/2 = 1.1Kn/mr
Qk 1.5 x 3.5/2 = 2.6Kn/mr

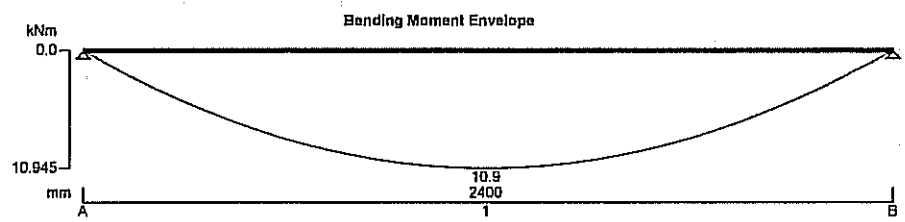
Span 2.4m

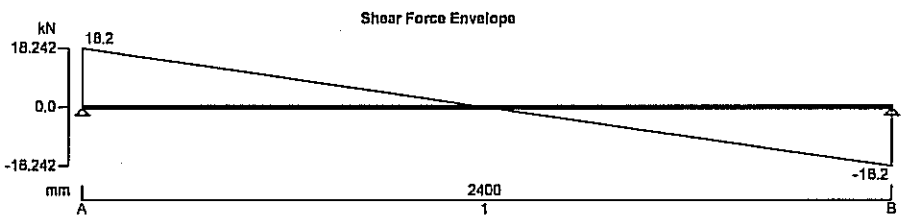
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.04







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
Span 1 loads	roof - Dead UDL 4.2 kN/m from 0 mm to 2400 mm
	roof - Imposed UDL 2.1 kN/m from 0 mm to 2400 mm
	floor - Dead UDL 1.1 kN/m from 0 mm to 2400 mm
	floor - Imposed UDL 2.6 kN/m from 0 mm to 2400 mm

Load combinations

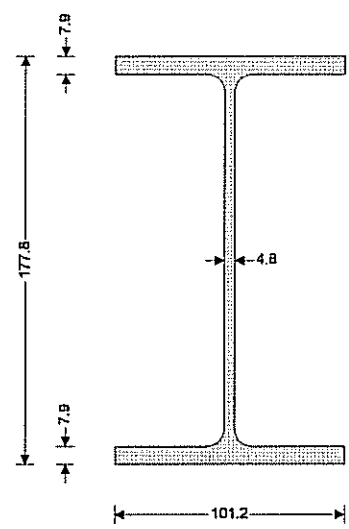
Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Span 1	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **10.9** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **18.2** kN	V_{min} = **-18.2** kN
Deflection	δ_{max} = **1.6** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **18.2** kN	R_{A_min} = **18.2** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **6.6** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **5.6** kN	
Maximum reaction at support B	R_{B_max} = **18.2** kN	R_{B_min} = **18.2** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **6.6** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **5.6** kN	

Section details

Section type	**UB 178x102x19 (BS4-1)**
Steel grade	**S275**
From table 9: Design strength p_y	
Thickness of element	max(T, t) = **7.9** mm
Design strength	p_y = **275** N/mm^2
Modulus of elasticity	E = **205000** 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 × D

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section d = **146.8** mm

d / t = 30.6 × ε <= 80 × ε Class 1 plastic

Outstand flanges - Table 11

Width of section b = B / 2 = **50.6** mm

b / T = 6.4 × ε <= 9 × ε Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force $F_v = max(abs(V_{max}), abs(V_{min}))$ = **18.2** kN

d / t < 70 × ε

Web does not need to be checked for shear buckling

Shear area $A_v = t \times D$ = **853** mm^2

Design shear resistance $P_v = 0.6 \times p_y \times A_v$ = **140.8** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = max(abs(M_{s1\_max}), abs(M_{s1\_min}))$ = **10.9** kNm

Moment capacity low shear - cl.4.2.5.2 $M_c = min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx})$ = **47.1** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection $\delta_{lim} = L_{s1} / 360$ = **6.667** mm

Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min}))$ = **1.583** mm

PASS - Maximum deflection does not exceed deflection limit

Provide a 178 x 102 x 19Kg UB

The extent of the glazing to this rear elevation exceeds the allowable thus a wind post is introduced in the inner block leaf.

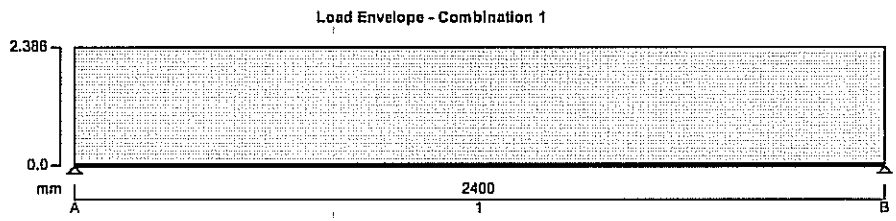
Load to beam Wind Wk 0.5 x 5.5/2 = 1.4Kn/mr

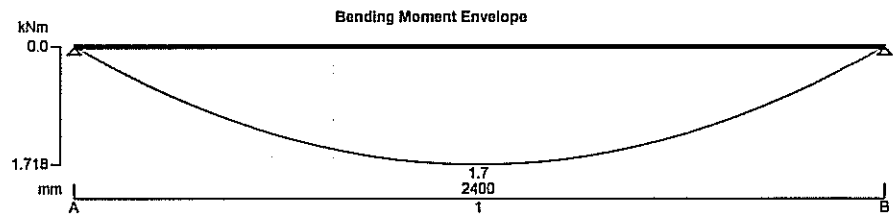
Span 2.4m

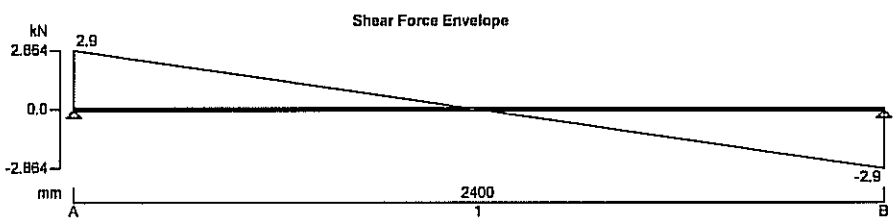
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.04







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
Span 1 loads	wind - Imposed UDL 1.4 kN/m from 0 mm to 2400 mm

Load combinations

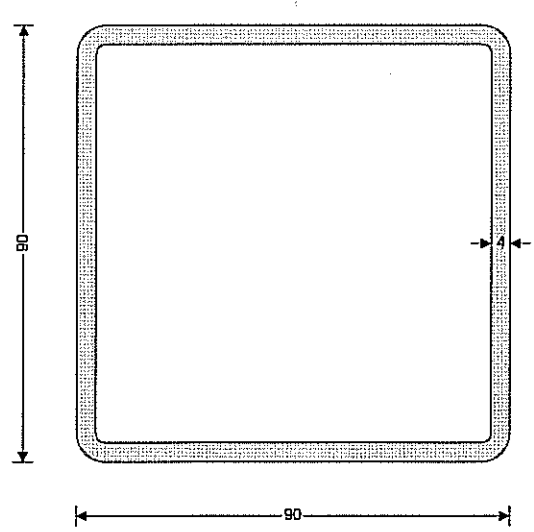
Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Span 1	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **1.7** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **2.9** kN	V_{min} = **-2.9** kN
Deflection	δ_{max} = **1.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **2.9** kN	R_{A_min} = **2.9** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **0.1** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **1.7** kN	
Maximum reaction at support B	R_{B_max} = **2.9** kN	R_{B_min} = **2.9** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **0.1** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **1.7** kN	

Section details

Section type	**SHS 90x90x4.0 (Corus Celsius)**
Steel grade	**S275**
From table 9: Design strength p_y	
Thickness of element	t = **4.0** mm
Design strength	p_y = **275** N/mm^2
Modulus of elasticity	E = **205000** 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 × D

Classification of cross sections - Section 3.5

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

Web - major axis - Table 12

Depth of section $d = D - 3 \times t = \mathbf{78}$ mm

$d / t = 19.5 \times \varepsilon <= 64 \times \varepsilon$ Class 1 plastic

Flange - major axis - Table 12

Width of section $b = B - 3 \times t = \mathbf{78}$ mm

$b / t = 19.5 \times \varepsilon <= \min(28 \times \varepsilon, 80 \times \varepsilon - d / t)$ Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{2.9}$ kN

$(D - 3 \times t) / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area $A_v = A \times D / (D + B) = \mathbf{679}\ mm^2$

Design shear resistance $P_v = 0.6 \times p_y \times A_v = \mathbf{112.1}$ kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{1.7}$ kNm

Moment capacity low shear - cl.4.2.5.2 $M_c = \min(p_y \times S, 1.2 \times p_y \times Z) = \mathbf{12}$ kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 = \mathbf{6.667}$ mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{1.907}$ mm

PASS - Maximum deflection does not exceed deflection limit

Provide a 90 x 90 x 4.0SHS

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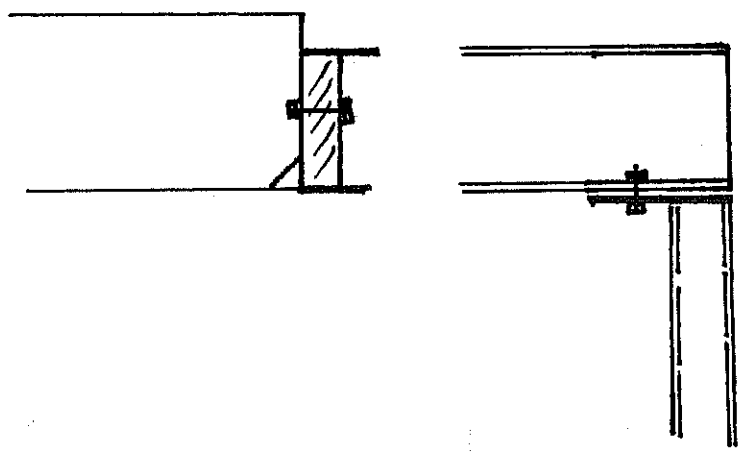
39 Georges Lane
Calverton
Nottingham
NG14 6JS

Consulting Engineers

Calc. Sheet 33
Job Title St Quintens
Hallmoor Road
Matlock
Job No. 13/132
Calcs by cjb
Checked by

Ref	Calculations

Detail Beam ' n '



1 Provide a 178 x 102 x 19Kg UB with infill timbers bolted at 600mm c/c with M12 dia bolts at 600mm c/c

2 Support the floor joists off joist hangers.

3 Beam has 150mm bearing onto a 215 x 100 x 150mm dp blue brick padstone.

4 Provide wind post as other bearing end with 200 x 100 x 10mm cap plate and bolted together with 2No M16 dia bolts.

5 Timber wall plate bolted to top flange M12 dia bolts at 600mm c/c

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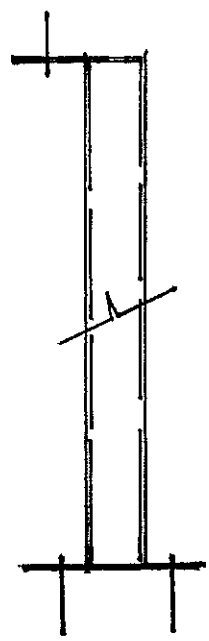
Calverton
Nottingham
NG14 6JS

Consulting Engineers

Calc. Sheet 34
Job Title St Quintens
Hallmoor Road
Matlock
Job No. 13/132
Calcs by cjb
Checked by

Ref	Calculations

Detail wind post ' wp '



1 Provide a wind post 90 x 90 x 4.0SHS with a 200 x 100 x cap plate and a 300 x 100 x 10mm base plate.

2 Post bolted down to a 450 x 100 x 215mm dp concrete padstone with 2No M12 dia resin anchors at approx ground floor level.

3 Post tied to adjacent blockwork with Ancon SPB ties 125mm long at 225mm vertical c/c shot fired to post at 225mm vertical c/c.