HJ Structural Engineers Ltd

STRUCTURAL CALCULATIONS

Project Name:	**Woodlands Farm**
	Roof Truss & Purlin Design
Date:	June 2023
Job Number:	MAS 1543
Revision:	

HJ Structural Engineers Ltd

07482 194381 / 01206 675110
Hadleigh, Suffolk

Contract:	Woodlands Farm
Job Ref:-	MAS 1543
Date:-	June 2023

Designer: M.A.Shutt

HJSE brief is to undertake the truss design & support only.

All Timber to be C24

Please note:- After installation some deflection may occur. This is perfectly normal and is not a structural defect. Some re-decoration may be required

Client / Architect to ensure that all members derived fit within required allowable structural zones prior to construction

Notes

1. The structural calculation herein relates only to the truss design as described above.

2. The Design is based on the information received from Whymark & Moulton Drg's + HJSE's Site Visit

3. The works MUST BE carried out by a competent Building Contractor in accordance with the Building Regulations, relevant British Standards and to generally accepted standards and methods of building construction.

4. No allowance has been made for inspecting the works on site. However, should any detail differ from the calculations or sketches the engineer must be informed.

5. All drawings shall be read in conjunction with all relevant Civil / Structural Engineers drawings, the project specification and Architects, Services Engineers & Landscape Architects drawings.

6. The Contractor shall verify all site and setting out dimensions before putting work in hand. Where dimensions are shown on the Engineers drawings, any discrepancies shall be reported to him. All dimensions are in millimeters unless otherwise noted. Dimensions must not be scaled from the Engineers drawings.

7. Inspections made by the Local Authority, NHBC or other Statutory bodies, shall be arranged by the Contractor to suit his programme. Any costs arising out of failing to carry out the work to the satisfaction of the Checking Authority will be the sole responsibility of the Contractor.

8. All information provided by others is taken in good faith as being accurate, but HJ Structural Engineers cannot, and does not, accept any liability for the detailed accuracy, errors or omissions in such information.

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GENERAL NOTES

1. The works MUST BE carried out by a competent Building Contractor in accordance with the Building Regulations, relevant British Standards and to generally accepted standards and methods of building construction.

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7. The drawings, design and all information contained therein are the Copyright of HJ Structural Engineers Ltd and reproduction in any form is forbidden unless permission is obtained in writing.

8. No holes, chases, cut-outs or the like may be formed in any beam, column, or load bearing wall unless written permission is obtained from the Engineer.

9. For size and location of all services refer to the Service Engineer’s and Architect’s drawings.

10. Non-structural fixings are generally not shown on the Engineer’s drawings and if any such detail is indicated it must be confirmed by cross-reference to other specialists before construction.

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ROOFING CONSTRUCTION NOTES

1. Timber wall plates will not be less than 50x100mm in cross section unless otherwise noted. These should be fixed inline with the latest building regulation specifications or as stated on HJSE drawings.

2. Wall plates should be strapped down to masonry walls at not more 1.25m centres with galvanised 30x2.5mm straps having a size of not less than 100x900mm.

3. All members supported on proprietary hangers must be accurately cut to provide a full contact with the base of the hanger and shall be fixed in accordance with the hanger manufacturer's instructions.

4. All loose timber rafters, ceiling joists, prefabricated trussed rafters and the like shall be fixed to timber wall plates, purlins etc. with suitable proprietary galvanised truss clips.

   Strutting between joists is required at:-

   - Joist span of 2.5m to 4.5m: - one row at centre of span.
   - Joist span over 4.5m: - two rows equally spaced

5. Gable walls, Party Walls and internal partitions extending into the roof space shall be restrained at the top of ceiling joists and underside of rafter level at no more than 2.0m centres with galvanised 30x5.0mm straps having a size of not less than 100x900mm. Noggins not less than 75mm deep and timber blocking adjacent to walls shall be fixed between members at all strap locations. Straps shall be fixed between members at all strap locations. Straps shall be fixed to members/noggins with not less than 4No. 32x3.5mm galvanised or sherardised square twisted nails.

6. Timber members must not penetrate fire stop walls in roofs. Provide suitable galvanised metal hangers to support trusses, rafters etc., as required to avoid such penetrations.

7. Prefabricated trussed rafters shall be Contractor designed to the approval of the Engineer. Calculations, truss profiles and layout drawings shall be submitted for comment and no fabrication work shall commence until comments are received from the Engineer. (Allow 3 working days for comments). Please note the truss supplier shall specify all loose timber/fixings unless otherwise specified by the structural engineer. The truss designer/supplier shall design all roof trusses in line with all current building standards.

8. The supplier shall be solely responsible for the design of the trussed rafters **including all temporary and permanent bracing timbers** except where otherwise detailed on the drawings. The main Contractor is responsible and liable for ensuring the overall stability of the works

9. **Note that the Engineer is not responsible for setting out information other than where shown on the drawings.**

10. All roof timbers to be preservative treated in accordance with: BS:5268:pt3 and BS:5268:pt5.

11. Trusses shall be stored on site in accordance with the manufacturer's recommendations.

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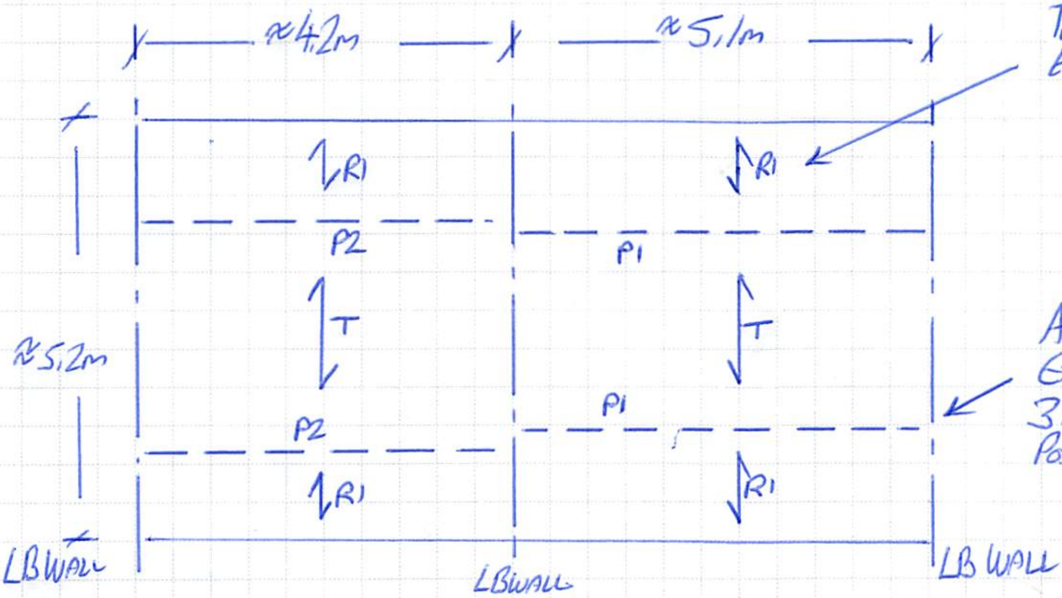
Project: Woodlands Farm | **Date:** June 2023

Element: Structural Design | **Sheet:**

Structural Loadings

Roof :- D.L = 1,50 kN/m²
L.L = 0,85 kN/m²



Treble R1's to Edge of Dormers

At All Bearings, Bear onto Existing Oak Frame or 3 No C16 47x97 Studs Positively Fixed Min.

Key

T = Truss - Rafters = C24 47x175's @ 400 Cr's
Tie = C24 47x147 @ 400 Crs
} Min sizes

R1 = C24 47x 97's @ 400 Cr's - Min size. (Check Roof / Insulation Build Up)

P1 = Flitch Beam
3 No C24 47x197's + 2 No 10mm Plates (S275) with Bolts M8 @ 300mm Vertically Staggered Cr's, 3 No Bolts Req'd @ Ends - See Attached.

P2 = Flitch Beam
2 No C24 47x197's + 1 No 10mm Plate (S275) with M8 Bolts @ 300mm Vertically Staggered Cr's, 3 No Bolts Req'd @ Ends - See Attached

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Project: WOODLANDS FARM	**Date:** JUNE 2023
Element: STRUCTURAL DESIGN	**Sheet:**

R1 MAX SPAN = 1.9m UDL = (1.5+0.85) x 0.4 = 0.94 kN/m SLS

Timber Beam Design

Input			Output		
Span of Timber	1.9	m	Deflection Actual	4.30	mm
UDL Load	0.94	kN/m	Deflection Limit	5.70	mm
Timber Grade	C24		Actual Stress	5.76	N/mm^2
Short/Med/Long Term Loading	Long		Permissible Stress	9.34	N/mm^2
Timbers at 600mm ctrs or less?	Yes		Beam Status	PASS	
Number of Timbers	>4				
Timber Width	47	mm			
Timber Depth	97	mm			

USE C24 47x97's @ 400 CR'S MIN

TRIMMER TO DORMER

MAX SPAN = 1.9m UDL = [(1.5+0.85) x 0.4] + [1 x 1.5] = 2.45 kN/m SLS

Timber Beam Design

Input			Output		
Span of Timber	1.9	m	Deflection Actual	4.63	mm
UDL Load	2.45	kN/m	Deflection Limit	5.70	mm
Timber Grade	C24		Actual Stress	5.00	N/mm^2
Short/Med/Long Term Loading	Long		Permissible Stress	9.34	N/mm^2
Timbers at 600mm ctrs or less?	No		Beam Status	PASS	
Number of Timbers	3				
Timber Width	47	mm			
Timber Depth	97	mm			

USE MIN 3 No C24 47x97's

PURLINS - P1

MAX SPAN = 5.2m UDL = (1.5+0.85) x [(5.2/2 - 0.65)] = 4.6 kN/m SLS

☆ LOAD TOO GREAT (JUST) FOR MULTIPLE TIMBERS ☆

SEE ATTACHED FOR FLITCH DESIGN.

TIMBER MEMBER ANALYSIS & DESIGN (EN1995-1-1:2004)

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 2.2.17

All results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm^2	11.3	0.2	0.022	PASS
Bending stress	N/mm^2	12.9	1.4	0.110	PASS
Shear stress	N/mm^2	2.2	0.2	0.080	PASS
Bending and axial force				0.110	PASS
Column stability check				0.125	PASS
Beam stability check				0.025	PASS
Deflection	mm	8.6	1.3	0.150	PASS
Member2 results summary	**Unit**	**Capacity**	**Maximum**	**Utilisation**	**Result**
Compressive stress	N/mm^2	11.3	0.2	0.022	PASS
Bending stress	N/mm^2	12.9	1.4	0.110	PASS
Shear stress	N/mm^2	2.2	0.2	0.080	PASS
Bending and axial force				0.110	PASS
Column stability check				0.125	PASS
Beam stability check				0.025	PASS
Deflection	mm	8.6	1.2	0.144	PASS

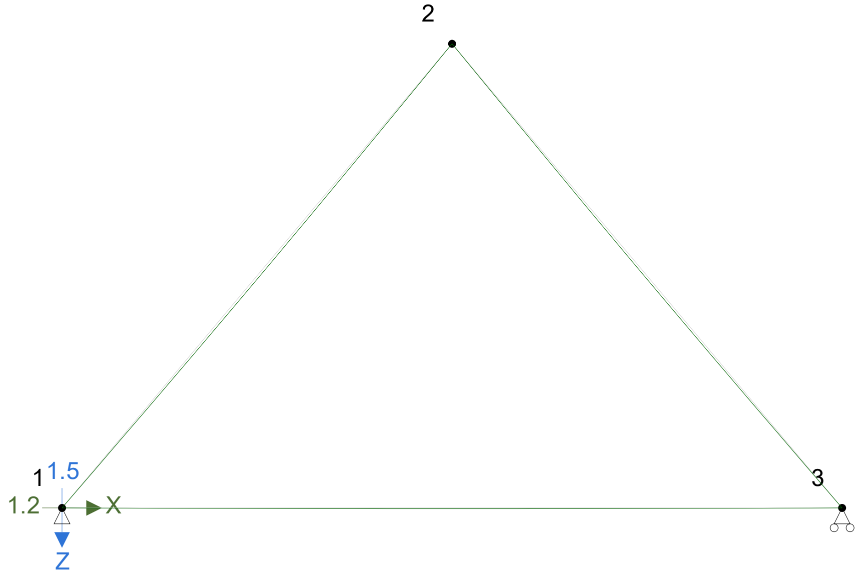
ANALYSIS

Tedds calculation version 1.0.37

Results

Total deflection

All (Strength) - Total deflection



Node deflections

Load combination: All (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0	
2	0	0.1	0	
3	0	0	0	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
All (Strength)	0	4.2

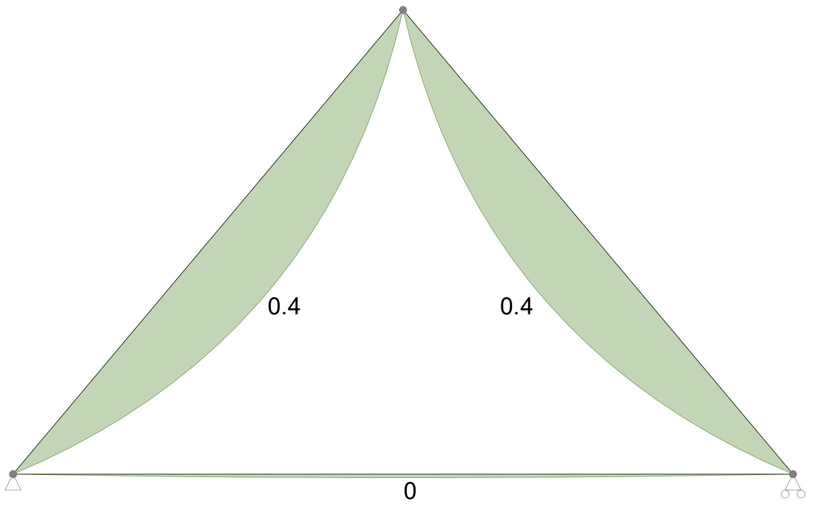
Element end forces

Load combination: All (Strength)

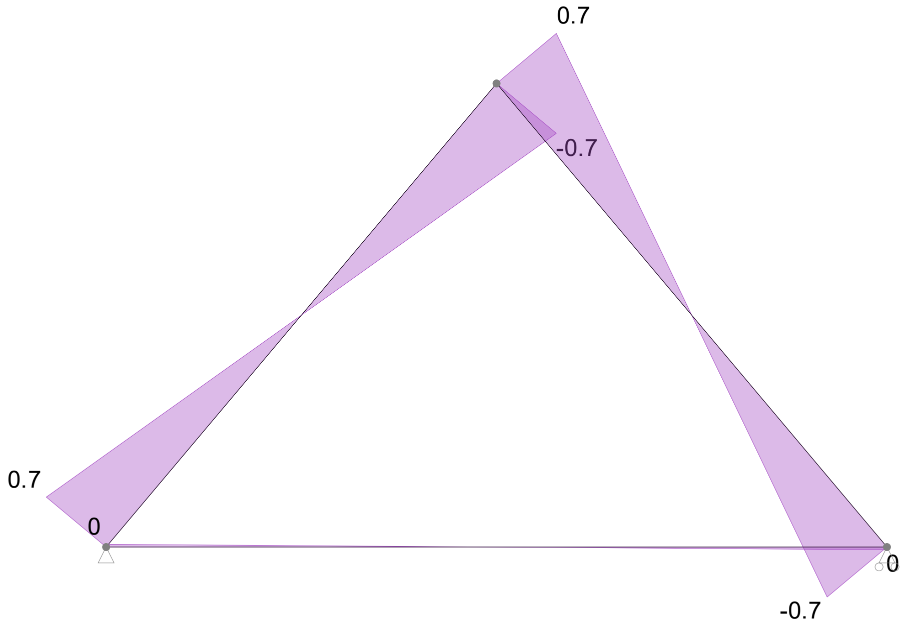
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.154	1	-2.2	-0.7	0
		2	0.6	-0.7	0
2	2.154	2	-0.6	-0.7	0
		3	2.2	-0.7	0
3	2.768	1	0.9	0	0
		3	-0.9	0	0

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Member results

Envelope - Strength combinations

Member	Shear force		Moment			
	Pos (m)	Max abs (kN)	Pos (m)	Max (kNm)	Pos (m)	Min (kNm)
All	0	0.7	1.077	0.4 (max)	0	0 (min)
Member2	2.154	-0.7 (max abs)	1.077	0.4 (max)	2.154	0 (min)

<u>All - Span 1</u>

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 γ_M = **1.300**

Member details

Load duration - cl.2.3.1.2 Long-term

Service class - cl.2.3.1.3 1

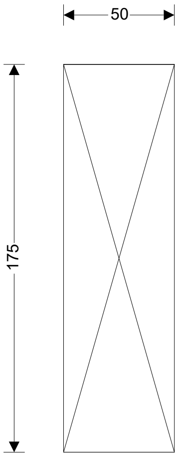
Timber section details

Number of timber sections in member N = **1**

Breadth of sections b = **50** mm

Depth of sections h = **175** mm

Timber strength class - EN 338:2016 Table 1 **C24**



50x175 timber section
Cross-sectional area, A, 8750 mm^2
Section modulus, W_y, 255208.3 mm^3
Section modulus, W_z, 72917 mm^3
Second moment of area, I_y, 22330729 mm^4
Second moment of area, I_z, 1822917 mm^4
Radius of gyration, i_y, 50.5 mm
Radius of gyration, i_z, 14.4 mm
Timber strength class C24
Characteristic bending strength, $f_{m.k}$, 24 N/mm^2
Characteristic shear strength, $f_{v.k}$, 4 N/mm^2
Characteristic compression strength parallel to grain, $f_{c.0.k}$, 21 N/mm^2
Characteristic compression strength perpendicular to grain, $f_{c.90.k}$, 2.5 N/mm^2
Characteristic tension strength parallel to grain, $f_{t.0.k}$, 14.5 N/mm^2
Mean modulus of elasticity, $E_{0.mean}$, 11000 N/mm^2
Fifth percentile modulus of elasticity, $E_{0.05}$, 7400 N/mm^2
Shear modulus of elasticity, G_{mean}, 690 N/mm^2
Characteristic density, ρ_k, 350 kg/m^3
Mean density, ρ_{mean}, 420 kg/m^3

Span details

Bearing length — L_b = **100** mm

Consider Combination 1 - All (Strength)

Modification factors

Duration of load and moisture content - Table 3.1 — k_{mod} = **0.7**
Deformation factor - Table 3.2 — k_{def} = **0.6**
Bending stress re-distribution factor - cl.6.1.6(2) — k_m = **0.7**
Crack factor for shear resistance - cl.6.1.7(2) — k_{cr} = **0.67**

Check compression parallel to the grain - cl.6.1.4

Design axial compression — P_d = **2.163** kN
Design compressive stress — $\sigma_{c,0,d} = P_d / A$ = **0.247** N/mm^2
Design compressive strength — $f_{c,0,d} = k_{mod} \times f_{c.0.k} / \gamma_M$ = **11.308** N/mm^2
$\sigma_{c,0,d} / f_{c,0,d}$ = **0.022**

PASS - Design parallel compression strength exceeds design parallel compression stress

Check design at start of span

Check shear force - Section 6.1.7

Design shear force — $F_{y,d}$ = **0.671** kN
Design shear stress - exp.6.60 — $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.172** N/mm^2
Design shear strength — $f_{v,y,d} = k_{mod} \times f_{v.k} / \gamma_M$ = **2.154** N/mm^2
$\tau_{y,d} / f_{v,y,d}$ = **0.080**

PASS - Design shear strength exceeds design shear stress

Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis bending — $L_{e,y} = 0.9 \times 2154$ mm = **1939** mm
Slenderness ratio — $\lambda_y = L_{e,y} / i_y$ = **38.374**
Relative slenderness ratio - exp. 6.21 — $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})}$ = **0.651**
Effective length for z-axis bending — $L_{e,z}$ = **0** mm
Slenderness ratio — $\lambda_z = L_{e,z} / i_z$ = **0**
Relative slenderness ratio - exp. 6.22 — $\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})}$ = **0**

$\lambda_{rel,y}$ > 0.3 column stability check is required

Straightness factor β_c = **0.2**

Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) = \mathbf{0.747}$

$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) = \mathbf{0.470}$

$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) = \mathbf{0.898}$

$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) = \mathbf{1.064}$

Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) = \mathbf{0.024}$

$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = \mathbf{0.021}$

PASS - Column stability is acceptable

Check design 1077 mm along span

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d}$ = **0.361** kNm

Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y$ = **1.416** N/mm^2

Design bending strength $f_{m,y,d} = k_{mod} \times f_{m.k} / \gamma_M$ = **12.923** N/mm^2

$\sigma_{m,y,d} / f_{m,y,d} = \mathbf{0.11}$

PASS - Design bending strength exceeds design bending stress

Check combined bending and axial compression - Section 6.2.4

Combined loading checks - exp.6.19 & 6.20 $(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d} = \mathbf{0.110}$

$(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \times \sigma_{m,y,d} / f_{m,y,d} = \mathbf{0.077}$

PASS - Combined bending and axial compression utilisation is acceptable

Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis bending $L_{e,y}$ = 0.9 × 2154 mm = **1939** mm

Slenderness ratio $\lambda_y = L_{e,y} / i_y = \mathbf{38.374}$

Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})} = \mathbf{0.651}$

Effective length for z-axis bending $L_{e,z}$ = **0** mm

Slenderness ratio $\lambda_z = L_{e,z} / i_z = \mathbf{0}$

Relative slenderness ratio - exp. 6.22 $\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})} = \mathbf{0}$

$\lambda_{rel,y}$ > 0.3 column stability check is required

Straightness factor β_c = **0.2**

Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) = \mathbf{0.747}$

$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) = \mathbf{0.470}$

$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) = \mathbf{0.898}$

$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) = \mathbf{1.064}$

Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d} = \mathbf{0.134}$

$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) + k_m \times \sigma_{m,y,d} / f_{m,y,d} = \mathbf{0.097}$

PASS - Column stability is acceptable

Check beams subjected to either bending or combined bending and compression - cl.6.3.3

Lateral buckling factor - exp.6.34 k_{crit} = **1.000**

Beam stability check - exp.6.35 $(\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d}))^2 + \sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = \mathbf{0.033}$

PASS - Beam stability is acceptable

Check design 1093 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection δ_y = **0.8** mm

Quasi-permanent variable load factor ψ_2 = **0.3**

Final deflection with creep $\delta_{y,Final} = \delta_y \times (1 + k_{def})$ = **1.3** mm

Allowable deflection $\delta_{y,Allowable} = L_{m1\_s1} / 250$ = **8.6** mm

$\delta_{y,Final} / \delta_{y,Allowable}$ = **0.15**

PASS - Allowable deflection exceeds final deflection

Member2 - Span 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 γ_M = **1.300**

Member details

Load duration - cl.2.3.1.2 Long-term

Service class - cl.2.3.1.3 1

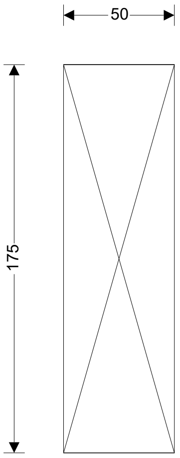
Timber section details

Number of timber sections in member N = **1**

Breadth of sections b = **50** mm

Depth of sections h = **175** mm

Timber strength class - EN 338:2016 Table 1 **C24**



50x175 timber section
Cross-sectional area, A, 8750 mm²
Section modulus, W_y, 255208.3 mm³
Section modulus, W_z, 72917 mm³
Second moment of area, I_y, 22330729 mm⁴
Second moment of area, I_z, 1822917 mm⁴
Radius of gyration, i_y, 50.5 mm
Radius of gyration, i_z, 14.4 mm
Timber strength class C24
Characteristic bending strength, $f_{m.k}$, 24 N/mm²
Characteristic shear strength, $f_{v.k}$, 4 N/mm²
Characteristic compression strength parallel to grain, $f_{c.0.k}$, 21 N/mm²
Characteristic compression strength perpendicular to grain, $f_{c.90.k}$, 2.5 N/mm²
Characteristic tension strength parallel to grain, $f_{t.0.k}$, 14.5 N/mm²
Mean modulus of elasticity, $E_{0.mean}$, 11000 N/mm²
Fifth percentile modulus of elasticity, $E_{0.05}$, 7400 N/mm²
Shear modulus of elasticity, G_{mean}, 690 N/mm²
Characteristic density, ρ_k, 350 kg/m³
Mean density, ρ_{mean}, 420 kg/m³

Span details

Bearing length L_b = **100** mm

Consider Combination 1 - All (Strength)

Modification factors

Duration of load and moisture content - Table 3.1 k_{mod} = **0.7**

Deformation factor - Table 3.2 k_{def} = **0.6**

Bending stress re-distribution factor - cl.6.1.6(2) k_m = **0.7**

Crack factor for shear resistance - cl.6.1.7(2) k_{cr} = **0.67**

Check compression parallel to the grain - cl.6.1.4

Design axial compression P_d = **0.563** kN

Design compressive stress $\sigma_{c,0,d} = P_d / A$ = **0.064** N/mm²

Design compressive strength $f_{c,0,d} = k_{mod} \times f_{c.0.k} / \gamma_M$ = **11.308** N/mm²

$\sigma_{c,0,d} / f_{c,0,d}$ = **0.006**

PASS - Design parallel compression strength exceeds design parallel compression stress

Check design at start of span

Check shear force - Section 6.1.7

Design shear force $F_{y,d}$ = **0.671** kN

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.172** N/mm^2

Design shear strength $f_{v,y,d} = k_{mod} \times f_{v.k} / \gamma_M$ = **2.154** N/mm^2

$\tau_{y,d} / f_{v,y,d}$ = **0.080**

PASS - Design shear strength exceeds design shear stress

Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis bending $L_{e,y} = 0.9 \times 2154$ mm = **1939** mm

Slenderness ratio $\lambda_y = L_{e,y} / i_y$ = **38.374**

Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})}$ = **0.651**

Effective length for z-axis bending $L_{e,z}$ = **0** mm

Slenderness ratio $\lambda_z = L_{e,z} / i_z$ = **0**

Relative slenderness ratio - exp. 6.22 $\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})}$ = **0**

$\lambda_{rel,y}$ > 0.3 column stability check is required

Straightness factor β_c = **0.2**

Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2)$ = **0.747**

$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2)$ = **0.470**

$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)})$ = **0.898**

$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)})$ = **1.064**

Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d})$ = **0.006**

$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d})$ = **0.005**

PASS - Column stability is acceptable

Check design 1077 mm along span

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d}$ = **0.361** kNm

Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y$ = **1.416** N/mm^2

Design bending strength $f_{m,y,d} = k_{mod} \times f_{m.k} / \gamma_M$ = **12.923** N/mm^2

$\sigma_{m,y,d} / f_{m,y,d}$ = **0.11**

PASS - Design bending strength exceeds design bending stress

Check combined bending and axial compression - Section 6.2.4

Combined loading checks - exp.6.19 & 6.20 $(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d}$ = **0.110**

$(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \times \sigma_{m,y,d} / f_{m,y,d}$ = **0.077**

PASS - Combined bending and axial compression utilisation is acceptable

Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis bending $L_{e,y} = 0.9 \times 2154$ mm = **1939** mm

Slenderness ratio $\lambda_y = L_{e,y} / i_y$ = **38.374**

Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})}$ = **0.651**

Effective length for z-axis bending $L_{e,z}$ = **0** mm

Slenderness ratio $\lambda_z = L_{e,z} / i_z$ = **0**

Relative slenderness ratio - exp. 6.22 $\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})}$ = **0**

$\lambda_{rel,y}$ > 0.3 column stability check is required

Straightness factor β_c = **0.2**

Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) = \mathbf{0.747}$

$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) = \mathbf{0.470}$

$k_{c,y} = 1 / (k_y + \sqrt{}(k_y^2 - \lambda_{rel,y}^2)) = \mathbf{0.898}$

$k_{c,z} = 1 / (k_z + \sqrt{}(k_z^2 - \lambda_{rel,z}^2)) = \mathbf{1.064}$

Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d} = \mathbf{0.116}$

$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) + k_m \times \sigma_{m,y,d} / f_{m,y,d} = \mathbf{0.082}$

PASS - Column stability is acceptable

Check beams subjected to either bending or combined bending and compression - cl.6.3.3

Lateral buckling factor - exp.6.34 k_{crit} = **1.000**

Beam stability check - exp.6.35 $(\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d}))^2 + \sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = \mathbf{0.017}$

PASS - Beam stability is acceptable

Check design 1061 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection δ_y = **0.8** mm

Quasi-permanent variable load factor ψ_2 = **0.3**

Final deflection with creep $\delta_{y,Final} = \delta_y \times (1 + k_{def})$ = **1.2** mm

Allowable deflection $\delta_{y,Allowable} = L_{m2\_s1} / 250$ = **8.6** mm

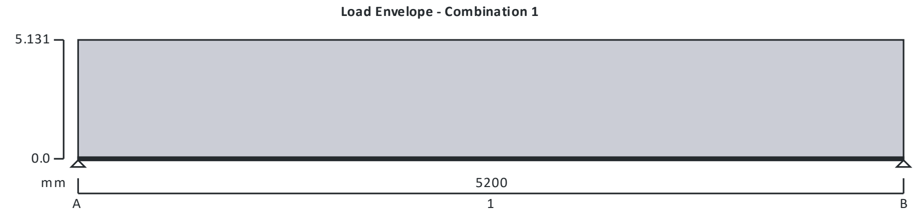
$\delta_{y,Final} / \delta_{y,Allowable}$ = **0.144**

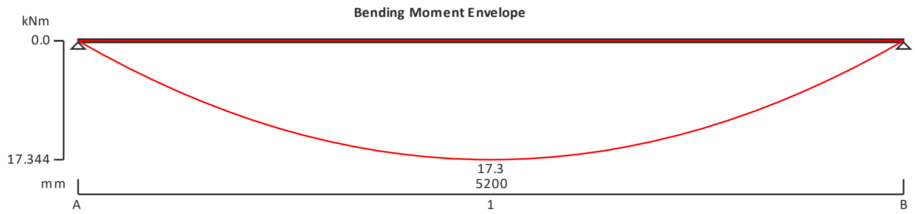
PASS - Allowable deflection exceeds final deflection

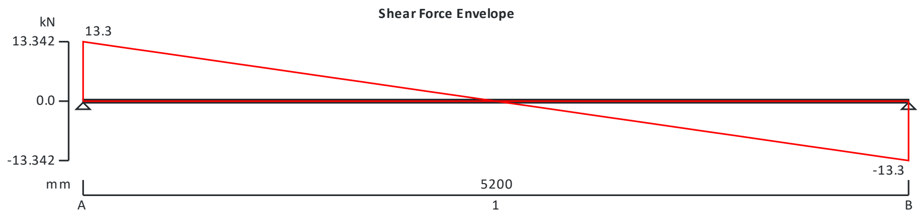
Tekla Tedds HJ Structural Engineers Ltd 5 Millers Close Hadlegh Suffolk, IP7 6GG	Project Woodlands Farm				Job no.	
	Calcs for P1 - Purlin				Start page no./Revision 1	
	Calcs by MAS	Calcs date 13/06/2023	Checked by	Checked date	Approved by	Approved date

FLITCH BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02







Applied loading

Beam loads

	Dead self weight of beam × 1
Roof	Dead full UDL 3.000 kN/m
Roof	Imposed full UDL 1.700 kN/m

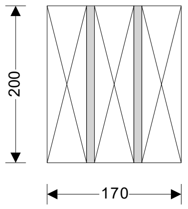
Load combinations

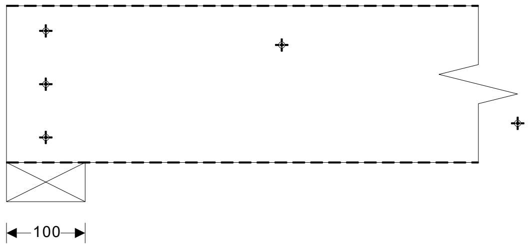
Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Maximum moment	M_{max} = **17.344** kNm	M_{min} = **0.000** kNm
Design moment	$M = \max(\text{abs}(M_{max}),\text{abs}(M_{min}))$ = **17.344** kNm	
Maximum shear	F_{max} = **13.342** kN	F_{min} = **-13.342** kN
Design shear	$F = \max(\text{abs}(F_{max}),\text{abs}(F_{min}))$ = **13.342** kN	
Total load on beam	W_{tot} = **26.684** kN	

Reactions at support A	R_{A_max} = **13.342** kN	R_{A_min} = **13.342** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **8.922** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **4.420** kN	
Reactions at support B	R_{B_max} = **13.342** kN	R_{B_min} = **13.342** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **8.922** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **4.420** kN	





Timber section details

Breadth of timber sections	b = **50** mm
Depth of timber sections	h = **200** mm
Number of timber sections in member	N = **3**
Timber strength class	**C24**

Steel section details

Breadth of steel plate	b_s = **10** mm
Depth of steel plate	h_s = **200** mm
Number of steel plates in beam	N_s = **2**
Steel stress	p_y = **230** N/mm^2
Bolt diameter	ϕ_b = **8** mm

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of span	L_{s1} = **5200** mm
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of beam	$A = N \times b \times h$ = **30000** mm^2
Timber section modulus	$Z_{xt} = N \times b \times h^2 / 6$ = **1000000** mm^3
Steel section modulus	$Z_{xs} = N_s \times b_s \times h_s^2 / 6$ = **133333** mm^3
Second moment of area of timber	$I_{xt} = N \times b \times h^3 / 12$ = **100000000** mm^4
Second moment of area of steel	$I_{xs} = N_s \times b_s \times h_s^3 / 12$ = **13333333** mm^4

Load proportions

Instant deflection under permanent actions	u_{instG} = **8.567** mm
Instant deflection under principal variable action	u_{instQ1} = **4.244** mm
	k_{def} = **0.6**
	ψ_2 = **0.3**

Final minimum modulus of elasticity

$E_{min,fin} = E_{min} \times (u_{instG} + u_{instQ1}) / (u_{instG} + u_{instQ1} + k_{def} \times (u_{instG} + \psi_2 \times u_{instQ1}))$ = **4929** N/mm^2

Proportion of applied load in timber	$k_t = E_{mean} \times I_{xt} / (E_{mean} \times I_{xt} + E_{S5950} \times I_{xs})$ = **0.283**
Proportion of applied load in steel	$k_s = 1.1 \times E_{S5950} \times I_{xs} / (E_{min,fin} \times I_{xt} + E_{S5950} \times I_{xs})$ = **0.932**

Modification factors

Duration of loading - Table 17 K_3 = **1.00**

Bearing stress - Table 18 K_4 = **1.00**

Total depth of member - cl.2.10.6 $K_7 = (300\text{ mm} / h)^{0.11}$ = **1.05**

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

Lateral support - cl.2.10.8

No lateral support

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

Actual depth-to-breadth ratio $h / (N \times b + N_s \times b_s)$ = **1.18**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **2.400** N/mm^2

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b)$ = **0.889** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.371**

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

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **7.842** N/mm^2

Applied timber bending stress $\sigma_{m\_a} = k_t \times M / Z_{xt}$ = **4.912** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.626**

PASS - Timber bending stress is less than permissible timber bending stress

Applied steel bending stress $\sigma_{m\_a\_s} = k_s \times M / Z_{xs}$ = **121.232** N/mm^2

$\sigma_{m\_a\_s} / p_y$ = **0.527**

PASS - Steel bending stress is less than permissible steel bending stress

Check beam in shear

Permissible shear stress $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.710** N/mm^2

Applied shear stress $\tau_a = 3 \times k_t \times F / (2 \times A)$ = **0.189** N/mm^2

τ_a / τ_{adm} = **0.266**

PASS - Shear stress within permissible limits

Deflection

Modulus of elasticity for deflection $E = E_{mean}$ = **10800** N/mm^2

Permissible deflection $\delta_{adm} = \min(0.551\text{ in}, 0.003 \times L_{s1})$ = **13.995** mm

Bending deflection δ_{b_s1} = **12.811** mm

Shear deflection δ_{v_s1} = **1.028** mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **13.839** mm

δ_a / δ_{adm} = **0.989**

PASS - Total deflection is less than permissible deflection

Flitch plate bolting requirements

Total load on beam W_{tot} = **26.684** kN

Total load taken by steel $W_s = k_s \times W_{tot}$ = **24.868** kN

Basic bolt shear load - Table 77 v_{90} = **1.066** kN

Number of interfaces $N_{int} = (N + N_s) - 1$ = **4**

Number of bolts required at supports $N_{be} = \max(k_s \times R_{A\_max} / (N_{int} \times v_{90}), 2)$ = **2.916**

Limiting bolt spacing $S_{limit} = \min(2.5 \times h, 600\text{ mm})$ = **500** mm

Maximum bolt spacing S_{max} = **300** mm

Minimum number of bolts along length of beam $N_{bl} = W_s / (N_{int} \times v_{90}) = \mathbf{5.831}$

- Provide a minimum of 3 No.8 mm diameter bolts at each support
- Provide 8 mm diameter bolts at maximum 300 mm centres staggered 50 mm alternately above and below the centre line

Minimum bolt spacings

Minimum end spacing $S_{end} = 4 \times \phi_b = \mathbf{32}$ mm

Minimum edge spacing $S_{edge} = 4 \times \phi_b = \mathbf{32}$ mm

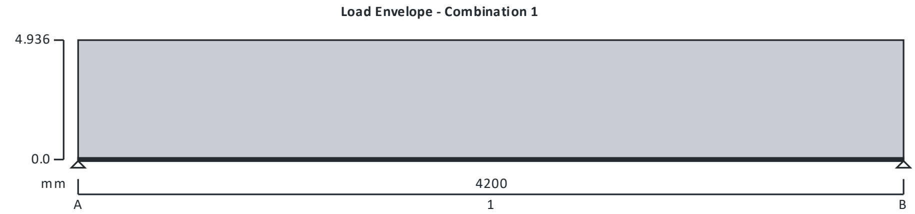
Minimum bolt spacing $S_{bolt} = 4 \times \phi_b = \mathbf{32}$ mm

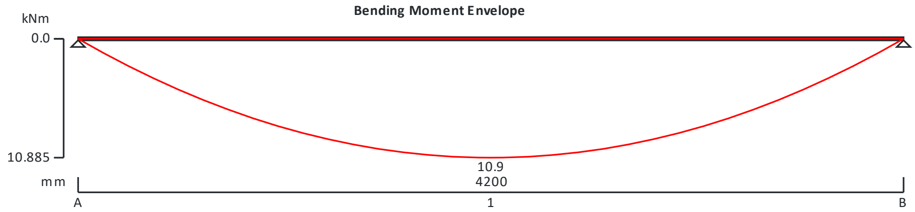
Minimum washer diameter $\phi_w = 3 \times \phi_b = \mathbf{24}$ mm

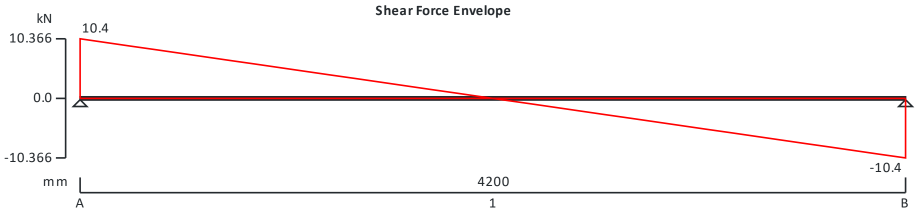
Minimum washer thickness $t_w = 0.25 \times \phi_b = \mathbf{2}$ mm

FLITCH BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02







Applied loading

Beam loads

	Dead self weight of beam × 1
Roof	Dead full UDL 3.000 kN/m
Roof	Imposed full UDL 1.700 kN/m

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} = **10.885** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **10.885** kNm	
Maximum shear	F_{max} = **10.366** kN	F_{min} = **-10.366** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **10.366** kN	
Total load on beam	W_{tot} = **20.733** kN	

Reactions at support A — R_{A_max} = **10.366** kN — R_{A_min} = **10.366** kN

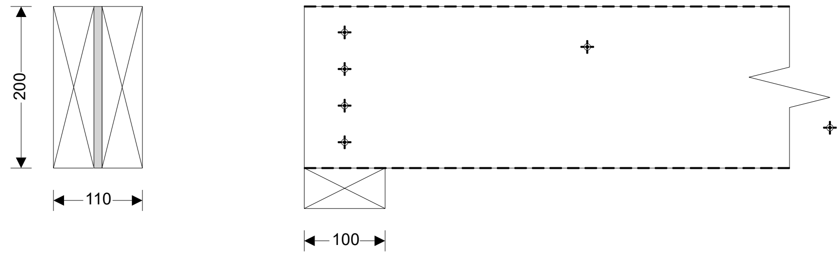
Unfactored dead load reaction at support A — R_{A_Dead} = **6.796** kN

Unfactored imposed load reaction at support A — $R_{A\_Imposed}$ = **3.570** kN

Reactions at support B — R_{B_max} = **10.366** kN — R_{B_min} = **10.366** kN

Unfactored dead load reaction at support B — R_{B_Dead} = **6.796** kN

Unfactored imposed load reaction at support B — $R_{B\_Imposed}$ = **3.570** kN



Timber section details

Breadth of timber sections — b = **50** mm

Depth of timber sections — h = **200** mm

Number of timber sections in member — N = **2**

Timber strength class — **C24**

Steel section details

Breadth of steel plate — b_s = **10** mm

Depth of steel plate — h_s = **200** mm

Number of steel plates in beam — N_s = **1**

Steel stress — p_y = **230** N/mm^2

Bolt diameter — ϕ_b = **8** mm

Member details

Service class of timber — **1**

Load duration — **Long term**

Length of span — L_{s1} = **4200** mm

Length of bearing — L_b = **100** mm

Section properties

Cross sectional area of beam — $A = N \times b \times h$ = **20000** mm^2

Timber section modulus — $Z_{xt} = N \times b \times h^2 / 6$ = **666667** mm^3

Steel section modulus — $Z_{xs} = N_s \times b_s \times h_s^2 / 6$ = **66667** mm^3

Second moment of area of timber — $I_{xt} = N \times b \times h^3 / 12$ = **66666667** mm^4

Second moment of area of steel — $I_{xs} = N_s \times b_s \times h_s^3 / 12$ = **6666667** mm^4

Load proportions

Instant deflection under permanent actions — u_{instG} = **6.284** mm

Instant deflection under principal variable action — u_{instQ1} = **3.301** mm

k_{def} = **0.6**

ψ_2 = **0.3**

Final minimum modulus of elasticity

$E_{min,fin} = E_{min} \times (u_{instG} + u_{instQ1}) / (u_{instG} + u_{instQ1} + k_{def} \times (u_{instG} + \psi_2 \times u_{instQ1}))$ = **4947** N/mm^2

Proportion of applied load in timber — $k_t = E_{mean} \times I_{xt} / (E_{mean} \times I_{xt} + E_{S5950} \times I_{xs})$ = **0.345**

Proportion of applied load in steel — $k_s = 1.1 \times E_{S5950} \times I_{xs} / (E_{min,fin} \times I_{xt} + E_{S5950} \times I_{xs})$ = **0.886**

Modification factors

Duration of loading - Table 17 — K_3 = **1.00**

Bearing stress - Table 18 — K_4 = **1.00**

Total depth of member - cl.2.10.6 — $K_7 = (300\text{ mm} / h)^{0.11}$ = **1.05**

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

Lateral support - cl.2.10.8

No lateral support

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

Actual depth-to-breadth ratio — $h / (N \times b + N_s \times b_s)$ = **1.82**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) — $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **2.400** N/mm²

Applied bearing stress — $\sigma_{c\_a} = R_{B\_max} / (N \times b \times L_b)$ = **1.037** N/mm²

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.432**

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

Bending parallel to grain

Permissible bending stress — $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **7.842** N/mm²

Applied timber bending stress — $\sigma_{m\_a} = k_t \times M / Z_{xt}$ = **5.634** N/mm²

$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.718**

PASS - Timber bending stress is less than permissible timber bending stress

Applied steel bending stress — $\sigma_{m\_a\_s} = k_s \times M / Z_{xs}$ = **144.681** N/mm²

$\sigma_{m\_a\_s} / p_y$ = **0.629**

PASS - Steel bending stress is less than permissible steel bending stress

Check beam in shear

Permissible shear stress — $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.710** N/mm²

Applied shear stress — $\tau_a = 3 \times k_t \times F / (2 \times A)$ = **0.268** N/mm²

τ_a / τ_{adm} = **0.378**

PASS - Shear stress within permissible limits

Deflection

Modulus of elasticity for deflection — $E = E_{mean}$ = **10800** N/mm²

Permissible deflection — $\delta_{adm} = \min(0.551\text{ in}, 0.003 \times L_{s1})$ = **12.600** mm

Bending deflection — δ_{b_s1} = **9.585** mm

Shear deflection — δ_{v_s1} = **0.968** mm

Total deflection — $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **10.552** mm

δ_a / δ_{adm} = **0.837**

PASS - Total deflection is less than permissible deflection

Flitch plate bolting requirements

Total load on beam — W_{tot} = **20.733** kN

Total load taken by steel — $W_s = k_s \times W_{tot}$ = **18.372** kN

Basic bolt shear load - Table 71 — v_{90} = **1.269** kN

Number of interfaces — $N_{int} = (N + N_s) - 1$ = **2**

Number of bolts required at supports — $N_{be} = \max(k_s \times R_{B\_max} / (N_{int} \times v_{90}), 2)$ = **3.619**

Limiting bolt spacing — $S_{limit} = \min(2.5 \times h, 600\text{ mm})$ = **500** mm

Maximum bolt spacing — S_{max} = **300** mm

Minimum number of bolts along length of beam $N_{bl} = W_s / (N_{int} \times v_{90}) = \mathbf{7.238}$

- Provide a minimum of 4 No.8 mm diameter bolts at each support
- Provide 8 mm diameter bolts at maximum 300 mm centres staggered 50 mm alternately above and below the centre line

Minimum bolt spacings

Minimum end spacing $S_{end} = 4 \times \phi_b = \mathbf{32}$ mm

Minimum edge spacing $S_{edge} = 4 \times \phi_b = \mathbf{32}$ mm

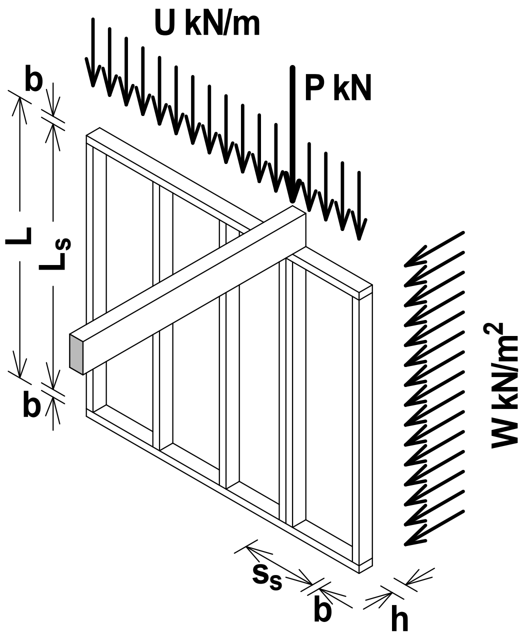
Minimum bolt spacing $S_{bolt} = 4 \times \phi_b = \mathbf{32}$ mm

Minimum washer diameter $\phi_w = 3 \times \phi_b = \mathbf{24}$ mm

Minimum washer thickness $t_w = 0.25 \times \phi_b = \mathbf{2}$ mm

TIMBER STUD DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.05



Stud details

Stud breadth	b = **50** mm
Stud depth	h = **100** mm
Number of studs	N_s = **3**

Strength class C16 timber (Table 8 BS5268:Pt 2:2002)

Section properties

Cross sectional area	$A = N_s \times b \times h$ = **15000** mm^2
Section modulus	$Z = N_s \times b \times h^2 / 6$ = **250000** mm^3
Moment of inertia in the major axis	$I_x = N_s \times b \times h^3 / 12$ = **12500000** mm^4
Moment of inertia in the minor axis	$I_y = N_s \times h \times b^3 / 12$ = **3125000** mm^4
Radius of gyration in the major axis	$r_x = \sqrt{(I_x / A)}$ = **28.9** mm
Radius of gyration in the minor axis	$r_y = \sqrt{(I_y / A)}$ = **14.4** mm

Panel details - Studs restrained by sheathing in the plane of the panel

Panel height	L = **2600** mm
Stud length	$L_s = L - (2 \times b)$ = **2500** mm
Standard stud spacing	s_s = **400** mm
Panel opening	O = **0** mm
Loaded panel length	$s = \max(s_s, (O + s_s) / 2)$ = **400** mm
Effective length in the major axis	$L_{ex} = 0.85 \times L_s$ = **2125** mm
Slenderness ratio	$\lambda = L_{ex} / r_x$ = **73.61**

Vertical loading details	**Dead loads**	**Imposed loads**
Wall UDL	U_{w_d} = **3.00** kN/m	
Roof UDL	U_{r_d} = **1.50** kN/m	U_{r_i} = **0.85** kN/m

Roof point loads P_{r_d} = **7.00** kN P_{r_i} = **7.00** kN

Lateral loading details

Wind loading W = **0.50** kN/m^2

Wind load duration **Very short term**

Modification factors

Section depth factor $K_7 = (300 \text{ mm} / h)^{0.11}$ = **1.13**

Load sharing factor K_8 = **1.10**

Consider combined axial compression and bending under very short term loads

Load duration factor K_3 = **1.75**

Vertical loading $F = (U_{w\_d} + U_{r\_d} + U_{r\_i}) \times s + P_{r\_d} + P_{r\_i}$ = **16.14** kN

Check bending stress

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

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **11.513** N/mm^2

Bending moment $M_{max} = W \times s \times L^2 / 8$ = **0.169** kNm

Applied bending stress $\sigma_{m\_max} = M_{max} / Z$ = **0.676** N/mm^2

PASS - Applied bending stress under very short term loads is within permissible limits

Check compressive stress on stud

Compression member factor K_{12} = **0.43**

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

Permissible compressive stress $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **5.609** N/mm^2

Applied compressive stress $\sigma_{c\_max} = F / (N_s \times b \times h)$ = **1.076** N/mm^2

PASS - Applied compressive stress under very short term loads is within permissible limits

Check compressive stress on rail

Bearing stress modification factor K_4 = **1.00**

Compression perpendicular to grain (no wane) σ_{cp1} = **2.200** N/mm^2

Permissible compressive stress $\sigma_{cp1\_adm} = \sigma_{cp1} \times K_3 \times K_4$ = **3.850** N/mm^2

Applied compressive stress $\sigma_{cp1\_max} = F / (N_s \times b \times h)$ = **1.076** N/mm^2

PASS - Applied compressive stress under very short term loads is within permissible limits

Check combined axial compression and bending

Euler critical stress $\sigma_e = (\pi^2 \times E_{min}) / \lambda^2$ = **10.564** N/mm^2

Euler coefficient $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e)$ = **0.935**

Combined axial compression and bending value $K = \sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm}$ = **0.255** **< 1**

PASS - Combined compressive and bending stresses under very short term loads are within permissible limits

Check stud deflection

Euler critical stress $\sigma_e = (\pi^2 \times E_{min}) / \lambda^2$ = **10.564** N/mm^2

Maximum deflection $\delta_{adm} = \min(7.5 \text{ mm}, 0.003 \times (L - 2 \times b))$ = **7.500** mm

Bending deflection $\delta_{max} = 5 \times W \times s \times L_s^4 / (384 \times E_{mean} \times I_x)$ = **0.925** mm

PASS - Deflection due to wind loading is less than permissible limit

Consider axial compression without bending under medium term loads

Load duration factor K_3 = **1.25**

Vertical loading $F = (U_{w\_d} + U_{r\_d} + U_{r\_i}) \times s + P_{r\_d} + P_{r\_i}$ = **16.14** kN

Check compressive stress on stud

Compression member factor K_{12} = **0.51**

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

Permissible compressive stress $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **4.773** N/mm^2

Applied compressive stress $\sigma_{c\_max} = F / (N_s \times b \times h)$ = **1.076** N/mm^2

PASS - Applied compressive stress under medium term loads is within permissible limits

Check compressive stress on rail

Bearing stress modification factor K_4 = **1.00**

Compression perpendicular to grain (no wane) σ_{cp1} = **2.200** N/mm^2

Permissible compressive stress $\sigma_{cp1\_adm} = \sigma_{cp1} \times K_3 \times K_4$ = **2.750** N/mm^2

Applied compressive stress $\sigma_{cp1\_max} = F / (N_s \times b \times h)$ = **1.076** N/mm^2

PASS - Applied compressive stress under medium term loads is within permissible limits

Consider axial compression without bending under long term loads

Load duration factor K_3 = **1.00**

Vertical loading $F = (U_{w\_d} + U_{r\_d}) \times s + P_{r\_d}$ = **8.80** kN

Check compressive stress on stud

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

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

Permissible compressive stress $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **4.165** N/mm^2

Applied compressive stress $\sigma_{c\_max} = F / (N_s \times b \times h)$ = **0.587** N/mm^2

PASS - Applied compressive stress under long term loads is within permissible limits

Check compressive stress on rail

Bearing stress modification factor K_4 = **1.00**

Compression perpendicular to grain (no wane) σ_{cp1} = **2.200** N/mm^2

Permissible compressive stress $\sigma_{cp1\_adm} = \sigma_{cp1} \times K_3 \times K_4$ = **2.200** N/mm^2

Applied compressive stress $\sigma_{cp1\_max} = F / (N_s \times b \times h)$ = **0.587** N/mm^2

PASS - Applied compressive stress under long term loads is within permissible limits