CALCULATION PACKAGE:

PROPOSED LOFT & POD CONVERSION:

JOB No – DD/2021/256:

Existing rear elevation

HOME TALES

CONTENTS

1. EXISTING/PROPOSED LOADING DATA:

1.1. WALL LOADING DATA:

		kN/sqm:	D.L.	L.L.
Dead:	Existing/Proposed cavity wall:		**4.20.**	
	Existing brick wall 215mm:		**5.00.**	
	Internal brick wall 103mm:		**2.50.**	
	Internal timber studs:		**0.50.**	
	External timber studs:		**1.00.**	
	Light blockwork: 10 kN/cum x 0.3m		**3.00.**	

1.2. PROPOSED FLOOR LOADING DATA:

		kN/sqm:	D.L.	L.L.
Dead:	Boards:	0.10.		
	Joists:	0.15.		
	Insulation:	0.05.		
	Partitions:	0.50.		
	Ceiling & services:	0.20.		
	Total =	1.00kN/sqm.	**1.00.**	

Live: Allow 1.50kN/m2 for residential loading case. **1.50.**

1.3. PROPOSED FLAT ROOF LOADING DATA:

		kN/sqm:	D.L.	L.L.
Dead:	Tiles:	0.50.		
	Battens & felt:	0.05.		
	Rafters:	0.10.		
	Insulation:	0.05.		
	Ceiling & services:	0.30.		
	Total =	1.00kN/sqm.	**1.00.**	

Live: Allow 0.75kN/m2 for snow. **0.75.**

1.4. EXISTING/PROPOSED MAIN ROOF (SLOPING) LOADING DATA:

		kN/sqm:	D.L.	L.L.
Dead:				
	Tiles:	0.50.		
	Battens & felt:	0.05.		
	Trusses:	0.10.		
	Insulation:	0.05.		
	Ceiling & services:	0.30.		
	Total =	1.00kN/sqm.		
Roof load on plan = 1.00 / Cos 32° = 1.18 kN/sqm.			**1.20.**	
Live: Allow 0.75kN/m2 for snow.				**0.75.**

1.5. EXISTING SIDE ROOF (SLOPING) LOADING DATA:

		kN/sqm:	D.L.	L.L.
Dead:				
	Tiles:	0.50.		
	Battens & felt:	0.05.		
	Trusses:	0.10.		
	Insulation:	0.05.		
	Ceiling & services:	0.30.		
	Total =	1.00kN/sqm.		
Roof load on plan = 1.00 / Cos 24° = 1.09 kN/sqm.			**1.10.**	
Live: Allow 0.75kN/m2 for snow.				**0.75.**

1.6. NEW MANSARD LOADING DATA:

		kN/sqm:	D.L.	L.L.
Dead:				
	Tiles:	0.50.		
	Battens & felt:	0.05.		
	Trusses:	0.10.		
	Insulation:	0.05.		
	Ceiling & services:	0.30.		
	Total =	1.00kN/sqm.		
Roof load on plan = 1.00 / Cos 70° = 2.92 kN/sqm.			**3.00.**	
Live: Allow 0.10kN/m2 for snow.				**0.10.**

2. STRUCTURAL CALCULATIONS

2.1. FLOOR JOISTS (FJ1):

Span = 3900 mm approximately.

See Tedd's computer calculations:

Adopt 50 x 200 mm joists @ 400 mm centres – Grade C24.

2.2. FLOOR JOISTS (FJ2):

Span = 4500 mm approximately.

See Tedd's computer calculations:

Adopt 50 x 200 mm joists @ 300 mm centres – Grade C24.

2.3. FLAT ROOF JOISTS (FRJ1):

Span = 4500 mm approximately.

See Tedd's computer calculations:

Adopt 50 x 200 mm joists @ 400 mm centres – Grade C24.

Triple up joists at the roof light and up to the openings.

Bolt multiple timber members together with 12.0 mm diameter bolts @ 450 mm centres staggered.

2.4. FLAT ROOF JOISTS (FRJ2):

Span = 4100 mm approximately.

See Tedd's computer calculations:

Adopt 50 x 200 mm joists @ 300 mm centres – Grade C24.

2.5. ROOF RAFTERS:

Span = 2100 mm approximately.

See Tedd's computer calculations:

Adopt 50 x 150 mm rafters @ 400mm centres – Grade C24.

Double up rafters at the roof light and up to the openings.

Bolt multiple timber members together with 12.0 mm diameter bolts @ 450 mm centres staggered.

2.6. MANSARD RAFTERS:

Span = 1000 mm approximately.

See Tedd's computer calculations:

Adopt 50 x 200 mm rafters @ 300mm centres – Grade C24.

2.7. BEAM B1:

Span = Approximately 2100 mm

Loading			D.L.	L.L.	
Proposed Floor Joists =	(1.00 + 1.50)	x 3.90 /2	1.95	2.93	kN/m
External Brick Wall =	5.00 x 0.60		3.00		kN/m
	Total =		4.95	2.93	kN/m
	Assumed =		**5.00**	**3.00**	**kN/m**

See Tedd's computer calculations:

Adopt 152 x 152 UC23 – GRADE S275.
Beam reaction = (5.50/3.20 + 5.50/3.20) kN

Beam reactions = 5.50 + 3.20 = 8.70 kN
Existing brickwork stress = 0.42 N/mm2
Padstone length = 8.70 x 1000 / 100 x 0.42 = **207** mm

Beam reactions = 5.50 + 3.20 = 8.70 kN
Existing brickwork stress = 0.42 N/mm2
Padstone length = 8.70 x 1000 / 215 x 0.42 = **96** mm

Adopt 450 x 100 x 225 mm DEEP CONCRETE PAD STONE ON ONE SIDE AND 215 x 215 x 225mm DEEP CONCRETE PAD STONE ON THE OTHER SIDE.

2.8. BEAM B2:

Span = Approximately 6000 mm

Loading				D.L.	L.L.	
Proposed Floor Joists =	(1.00 + 1.50)	x 3.90 /2		1.95	2.93	kN/m
Proposed Floor Joists =	(1.00 + 1.50)	x 2.80 /2		1.40	2.10	kN/m
Proposed Roof Rafters =	(1.20 + 0.75)	x 1.80 /2	x (1/3)	0.36	0.22	kN/m
Proposed Roof Rafters =	(1.20 + 0.75)	x 2.10 /2	x (1/3)	0.42	0.26	kN/m
Internal Stud Wall =	0.50 x 1.30		x (1/3)	0.21		kN/m
	Total =			4.34	5.51	kN/m
	Assumed =			**4.40**	**5.60**	**kN/m**
Reaction from Post P1 @5500 mm =				**10.30**	**6.70**	**kN**

See Tedd's computer calculations:

Adopt 203 x 203 UC46 – GRADE S275.
Beam reaction = (15.40/17.40 +24.00/22.90) kN

By inspection adopt 800 x 100 x 25.0 mm THICK STEEL SPREADER PLATE ON ONE SIDE AND 100 x 100 x 10.0 mm SHS SPREADER BEAM (1100 mm LONG) ON THE OTHER SIDE.

2.9. BEAM B3:

Span = Approximately 6000 mm

Loading				D.L.	L.L.	
Proposed Floor Joists =	(1.00 + 1.50)	x 2.80 /2		1.40	2.10	kN/m
Proposed Roof Rafters =	(1.20 + 0.75)	x 1.80 /2	x (2/3)	0.72	0.45	kN/m
Proposed Roof Rafters =	(1.20 + 0.75)	x 2.10 /2	x (2/3)	0.84	0.53	kN/m
Internal Stud Wall =	0.50 x 1.30		x (2/3)	0.44		kN/m
	Total =			3.40	3.08	kN/m
	Assumed =			**3.50**	**3.10**	**kN/m**

See Tedd's computer calculations:

Adopt 203 x 203 UC46 – GRADE S275.
Beam reaction = (11.90/9.30 + 11.90/9.30) kN

By inspection adopt 600 x 100 x 25.0 mm THICK STEEL SPREADER PLATE ON BOTH SIDES.

2.10. BEAM B4:

Span = Approximately 5600 mm

Loading			D.L.	L.L.	
Proposed Flat Roof Joists =	(1.00 + 0.75)	x 4.50 /2	2.25	1.69	kN/m
Proposed Roof Rafters =	(1.20 + 0.75)	x 1.80 /2	1.08	0.68	kN/m
	Total =		3.33	2.36	kN/m
	Assumed =		**3.40**	**2.40**	**kN/m**

See Tedd's computer calculations:

Adopt 152 x 152 UC30 – GRADE S275.
Beam reaction = (10.30/6.70 + 10.30/6.70) kN

Ultimate Reaction for Post P1 = 25.20 kN

By inspection adopt 600 x 100 x 25.0 mm THICK STEEL SPREADER PLATE ON ONE SIDE.

2.11. BEAM B5:

Span = Approximately	4200 mm			
Loading		**D.L.**	**L.L.**	
Proposed Flat Roof Joists =	(1.00 + 0.75) x 4.50 /2	2.25	1.69	kN/m
	Total =	2.25	1.69	kN/m
	Assumed =	**2.30**	**1.70**	**kN/m**

See Tedd's computer calculations:

PROVIDE 3 No 50 x 200 mm JOISTS + 2 No 8.0 mm x 195 mm DEEP STEEL PLATE TO FORM FLITCH BEAM. TIMBER TO BE GRADE C24. STEELWORK TO BE GRADE S275. MEMBERS BOLTED TOGETHER USING M12Ø BOLTS @ 450mm CRS STAGGERED.

Beam reaction = (5.60/3.60 + 5.60/3.60) kN

Ultimate Reaction for Timber Posts = 9.20 kN

By inspection adopt 600 x 100 x 25.0 mm THICK STEEL SPREADER PLATE ON ONE SIDE.

2.12. TIMBER POST (P):

Ultimate reaction = 9.20 kN

By inspection adopt 150 x 100 mm TIMBER POST – GRADE C24.

2.13. STEEL POST P1:

Ultimate reaction = 25.20 kN

By inspection adopt 100 x 100 x 5.0 mm SHS POST – GRADE 275.

HOME TALES

3. TEKLA TEDDS CALCULATIONS

Contents:

TIMBER FLOOR JOIST (FJ1) DESIGN (BS5268)

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

Joist details

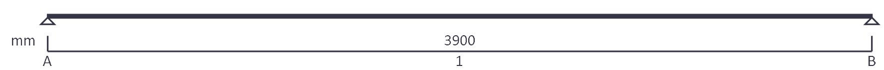
Joist breadth; $b = \mathbf{50}$ mm

Joist depth; $h = \mathbf{200}$ mm

Joist spacing; $s = \mathbf{400}$ mm

Timber strength class; **C24**

Service class of timber; **1**

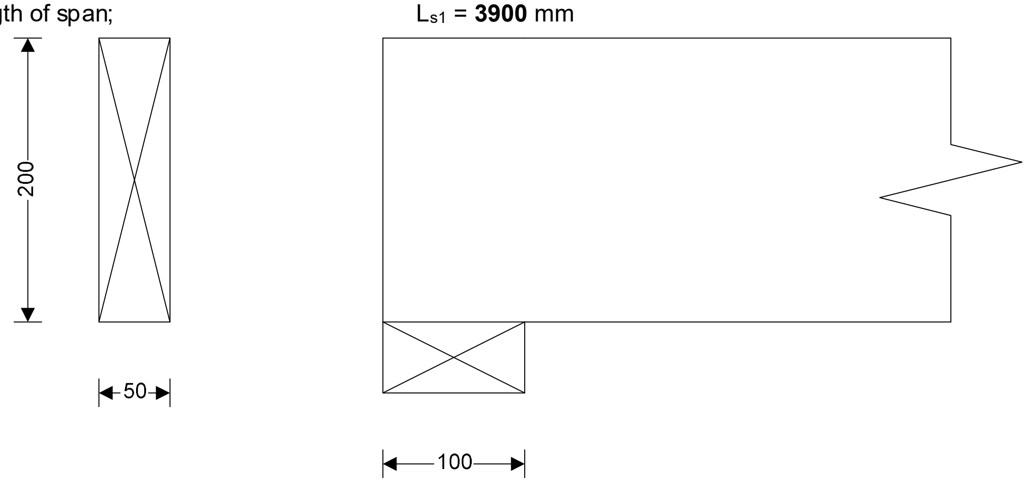


Span details

Number of spans; $N_{span} = \mathbf{1}$

Length of bearing; $L_b = \mathbf{100}$ mm

Effective length of span; $L_{s1} = \mathbf{3900}$ mm



Section properties

Second moment of area; $I = b \times h^3 / 12 = \mathbf{33333333}$ mm^4

Section modulus; $Z = b \times h^2 / 6 = \mathbf{333333}$ mm^3

Loading details

Joist self weight; $F_{swt} = b \times h \times \rho_{char} \times g_{acc} = \mathbf{0.03}$ kN/m

Dead load; $F_{d\_udl} = \mathbf{1.00}$ kN/m^2

Imposed UDL(Long term); $F_{i\_udl} = \mathbf{1.50}$ kN/m^2

Imposed point load (Medium term); $F_{i\_pt} = \mathbf{1.40}$ kN

Modification factors

Service class for bending parallel to grain K_{2m} = **1.00**

Service class for compression K_{2c} = **1.00**

Service class for shear parallel to grain K_{2s} = **1.00**

Service class for modulus of elasticity K_{2e} = **1.00**

Section depth factor; K_7 = **1.05**

Load sharing factor; K_8 = **1.10**

Consider long term loads

Load duration factor; K_3 = **1.00**

Maximum bending moment; M = **1.967** kNm

Maximum shear force; V = **2.017** kN

Maximum support reaction; R = **2.017** kN

Maximum deflection; δ = **9.004** mm

Check bending stress

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

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **8.626** N/mm^2

Applied bending stress; σ_{m_max} = M / Z = **5.900** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress; τ = **0.710** N/mm^2

Permissible shear stress; $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.781** N/mm^2

Applied shear stress; $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.303** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

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

Permissible bearing stress; $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **2.640** N/mm^2

Applied bearing stress; $\sigma_{c\_max} = R / (b \times L_b)$ = **0.403** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = \min(L_{s1} \times 0.003, 14\text{ mm})$ = **11.700** mm

Bending deflection (based on E_{mean}); $\delta_{bending}$ = **8.655** mm

Shear deflection; δ_{shear} = **0.350** mm

Total deflection; $\delta = \delta_{bending} + \delta_{shear}$ = **9.004** mm

PASS - Actual deflection within permissible limits

Consider medium term loads

Load duration factor; K_3 = **1.25**

Maximum bending moment; M = **2.191** kNm

Maximum shear force; V = **2.247** kN

Maximum support reaction; R = **2.247** kN

Maximum deflection; δ = **8.830** mm

Check bending stress

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

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **10.783** N/mm^2

Applied bending stress; $\sigma_{m\_max} = M / Z$ = **6.572** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress; τ = **0.710** N/mm^2

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

Applied shear stress; $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.337** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

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

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

Applied bearing stress; $\sigma_{c\_max} = R / (b \times L_b)$ = **0.449** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = \min(L_{s1} \times 0.003,\ 14\text{ mm})$ = **11.700** mm

Bending deflection (based on E_{mean}); $\delta_{bending}$ = **8.440** mm

Shear deflection; δ_{shear} = **0.389** mm

Total deflection; $\delta = \delta_{bending} + \delta_{shear}$ = **8.830** mm

PASS - Actual deflection within permissible limits

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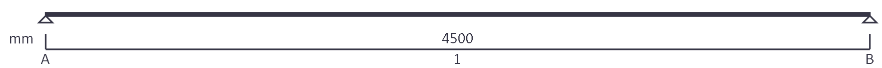
TIMBER FLOOR JOIST (FJ2) DESIGN (BS5268)

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

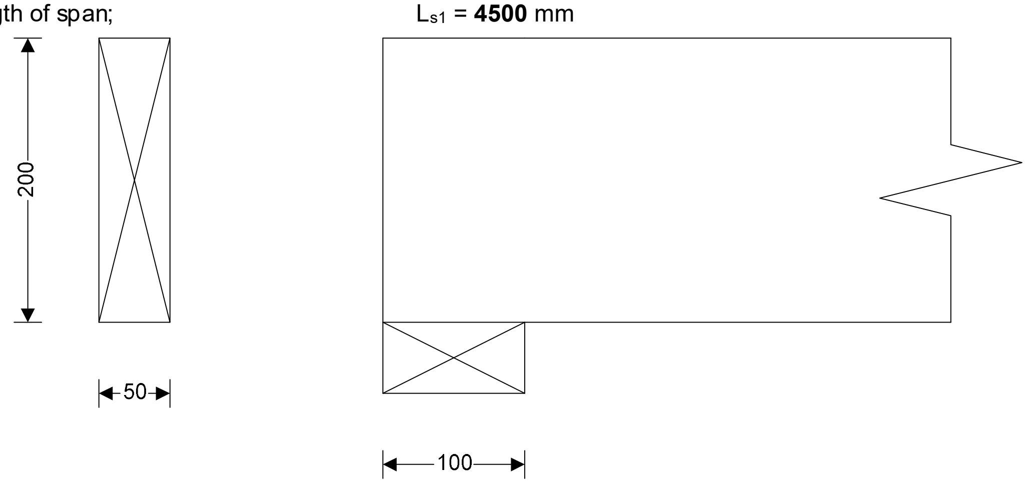
Joist details

Joist breadth;	b = **50** mm
Joist depth;	h = **200** mm
Joist spacing;	s = **300** mm
Timber strength class;	**C24**
Service class of timber;	**1**



Span details

Number of spans;	N_{span} = **1**
Length of bearing;	L_b = **100** mm
Effective length of span;	L_{s1} = **4500** mm



Section properties

Second moment of area;	$I = b \times h^3 / 12$ = **33333333** mm^4
Section modulus;	$Z = b \times h^2 / 6$ = **333333** mm^3

Loading details

Joist self weight;	$F_{swt} = b \times h \times \rho_{char} \times g_{acc}$ = **0.03** kN/m
Dead load;	F_{d_udl} = **1.00** kN/m^2
Imposed UDL(Long term);	F_{i_udl} = **1.50** kN/m^2

Imposed point load (Medium term); F_{i_pt} = **1.40** kN

Modification factors

Service class for bending parallel to grain K_{2m} = **1.00**

Service class for compression K_{2c} = **1.00**

Service class for shear parallel to grain K_{2s} = **1.00**

Service class for modulus of elasticity K_{2e} = **1.00**

Section depth factor; K_7 = **1.05**

Load sharing factor; K_8 = **1.10**

Consider long term loads

Load duration factor; K_3 = **1.00**

Maximum bending moment; M = **1.985** kNm

Maximum shear force; V = **1.765** kN

Maximum support reaction; R = **1.765** kN

Maximum deflection; δ = **11.986** mm

Check bending stress

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

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **8.626** N/mm^2

Applied bending stress; σ_{m_max} = M / Z = **5.956** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress; τ = **0.710** N/mm^2

Permissible shear stress; $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.781** N/mm^2

Applied shear stress; $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.265** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

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

Permissible bearing stress; $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **2.640** N/mm^2

Applied bearing stress; $\sigma_{c\_max} = R / (b \times L_b)$ = **0.353** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = \min(L_{s1} \times 0.003,\ 14\ \text{mm})$ = **13.500** mm

Bending deflection (based on E_{mean}); $\delta_{bending}$ = **11.633** mm

Shear deflection; δ_{shear} = **0.353** mm

Total deflection; $\delta = \delta_{bending} + \delta_{shear}$ = **11.986** mm

PASS - Actual deflection within permissible limits

Consider medium term loads

Load duration factor; K_3 = **1.25**

Maximum bending moment; M = **2.421** kNm

Maximum shear force; V = **2.152** kN

Maximum support reaction; R = **2.152** kN

Maximum deflection; δ = **12.772** mm

Check bending stress

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

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **10.783** N/mm^2

Applied bending stress; $\sigma_{m\_max} = M / Z$ = **7.264** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress; τ = **0.710** N/mm^2

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

Applied shear stress; $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.323** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

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

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

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

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = \min(L_{s1} \times 0.003,\ 14\text{ mm})$ = **13.500** mm

Bending deflection (based on E_{mean}); $\delta_{bending}$ = **12.341** mm

Shear deflection; δ_{shear} = **0.430** mm

Total deflection; $\delta = \delta_{bending} + \delta_{shear}$ = **12.772** mm

PASS - Actual deflection within permissible limits

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TIMBER FLAT ROOF JOIST (FRJ1) DESIGN (BS5268)

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

Joist details

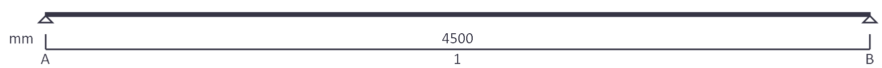
Joist breadth; b = **50** mm

Joist depth; h = **200** mm

Joist spacing; s = **400** mm

Timber strength class; **C24**

Service class of timber; **1**

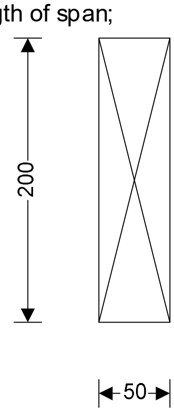


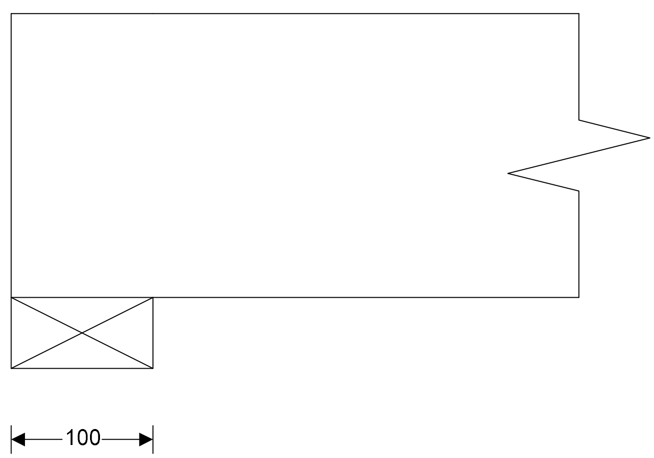
Span details

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

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

Effective length of span; L_{s1} = **4500** mm





Section properties

Second moment of area; $I = b \times h^3 / 12 =$ **33333333** mm^4

Section modulus; $Z = b \times h^2 / 6 =$ **333333** mm^3

Loading details

Joist self weight; $F_{swt} = b \times h \times \rho_{char} \times g_{acc} =$ **0.03** kN/m

Dead load; F_{d_udl} = **1.00** kN/m^2

Imposed UDL(Medium term); F_{i_udl} = **0.75** kN/m^2

Imposed point load (Short term); F_{i_pt} = **0.90** kN

Modification factors

Service class for bending parallel to grain K_{2m} = **1.00**

Service class for compression K_{2c} = **1.00**

Service class for shear parallel to grain K_{2s} = **1.00**

Service class for modulus of elasticity K_{2e} = **1.00**

Section depth factor; K_7 = **1.05**

Load sharing factor; K_8 = **1.10**

<u>Consider medium term loads</u>

Load duration factor; K_3 = **1.25**

Maximum bending moment; M = **1.859** kNm

Maximum shear force; V = **1.652** kN

Maximum support reaction; R = **1.652** kN

Maximum deflection; δ = **11.222** mm

Check bending stress

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

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **10.783** N/mm^2

Applied bending stress; σ_{m_max} = M / Z = **5.576** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress; τ = **0.710** N/mm^2

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

Applied shear stress; $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.248** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

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

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

Applied bearing stress; $\sigma_{c\_max} = R / (b \times L_b)$ = **0.330** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = \min(L_{s1} \times 0.003,\ 14\text{ mm})$ = **13.500** mm

Bending deflection (based on E_{mean}); $\delta_{bending}$ = **10.891** mm

Shear deflection; δ_{shear} = **0.330** mm

Total deflection; $\delta = \delta_{bending} + \delta_{shear}$ = **11.222** mm

PASS - Actual deflection within permissible limits

<u>Consider short term loads</u>

Load duration factor; K_3 = **1.50**

Maximum bending moment; M = **2.112** kNm

Maximum shear force; V = **1.877** kN

Maximum support reaction; R = **1.877** kN

Maximum deflection; δ = **11.563** mm

Check bending stress

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

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **12.939** N/mm^2

Applied bending stress; $\sigma_{m\_max} = M / Z$ = **6.336** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress; τ = **0.710** N/mm^2

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

Applied shear stress; $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.282** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

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

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

Applied bearing stress; $\sigma_{c\_max} = R / (b \times L_b)$ = **0.375** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = \min(L_{s1} \times 0.003,\ 14\text{ mm})$ = **13.500** mm

Bending deflection (based on E_{mean}); $\delta_{bending}$ = **11.188** mm

Shear deflection; δ_{shear} = **0.375** mm

Total deflection; $\delta = \delta_{bending} + \delta_{shear}$ = **11.563** mm

PASS - Actual deflection within permissible limits

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	Project			Job Ref.	
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	ID		ID			

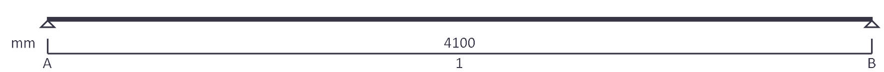
TIMBER FLAT ROOF JOIST (FRJ2) DESIGN (BS5268)

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

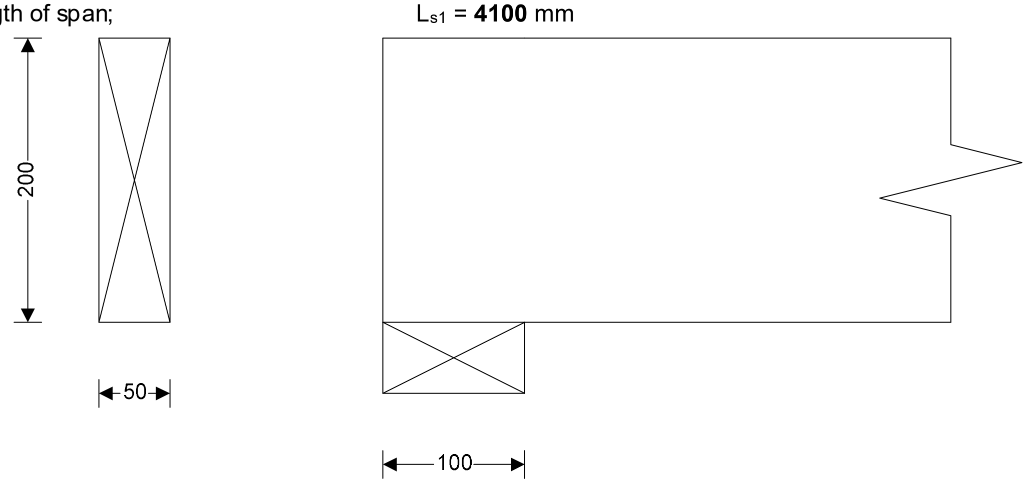
Joist details

Joist breadth;	b = **50** mm
Joist depth;	h = **200** mm
Joist spacing;	s = **300** mm
Timber strength class;	**C24**
Service class of timber;	**1**



Span details

Number of spans;	N_{span} = **1**
Length of bearing;	L_b = **100** mm
Effective length of span;	L_{s1} = **4100** mm



Section properties

Second moment of area;	$I = b \times h^3 / 12 =$ **33333333** mm^4
Section modulus;	$Z = b \times h^2 / 6 =$ **333333** mm^3

Loading details

Joist self weight;	$F_{swt} = b \times h \times \rho_{char} \times g_{acc} =$ **0.03** kN/m
Dead load;	F_{d_udl} = **1.00** kN/m^2
Imposed UDL(Medium term);	F_{i_udl} = **0.75** kN/m^2

Imposed point load (Short term); F_{i_pt} = **0.90** kN

Modification factors

Service class for bending parallel to grain K_{2m} = **1.00**

Service class for compression K_{2c} = **1.00**

Service class for shear parallel to grain K_{2s} = **1.00**

Service class for modulus of elasticity K_{2e} = **1.00**

Section depth factor; K_7 = **1.05**

Load sharing factor; K_8 = **1.10**

Consider medium term loads

Load duration factor; K_3 = **1.25**

Maximum bending moment; M = **1.175** kNm

Maximum shear force; V = **1.147** kN

Maximum support reaction; R = **1.147** kN

Maximum deflection; δ = **5.925** mm

Check bending stress

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

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **10.783** N/mm^2

Applied bending stress; σ_{m_max} = M / Z = **3.526** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress; τ = **0.710** N/mm^2

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

Applied shear stress; $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.172** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

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

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

Applied bearing stress; $\sigma_{c\_max} = R / (b \times L_b)$ = **0.229** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = \min(L_{s1} \times 0.003,\ 14\text{ mm})$ = **12.300** mm

Bending deflection (based on E_{mean}); $\delta_{bending}$ = **5.717** mm

Shear deflection; δ_{shear} = **0.209** mm

Total deflection; $\delta = \delta_{bending} + \delta_{shear}$ = **5.925** mm

PASS - Actual deflection within permissible limits

Consider short term loads

Load duration factor; K_3 = **1.50**

Maximum bending moment; M = **1.625** kNm

Maximum shear force; V = **1.585** kN

Maximum support reaction; R = **1.585** kN

Maximum deflection; δ = **7.295** mm

Check bending stress

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

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **12.939** N/mm^2

Applied bending stress; $\sigma_{m\_max} = M / Z$ = **4.875** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress; τ = **0.710** N/mm^2

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

Applied shear stress; $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.238** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

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

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

Applied bearing stress; $\sigma_{c\_max} = R / (b \times L_b)$ = **0.317** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = \min(L_{s1} \times 0.003,\ 14\text{ mm})$ = **12.300** mm

Bending deflection (based on E_{mean}); $\delta_{bending}$ = **7.007** mm

Shear deflection; δ_{shear} = **0.289** mm

Total deflection; $\delta = \delta_{bending} + \delta_{shear}$ = **7.295** mm

PASS - Actual deflection within permissible limits

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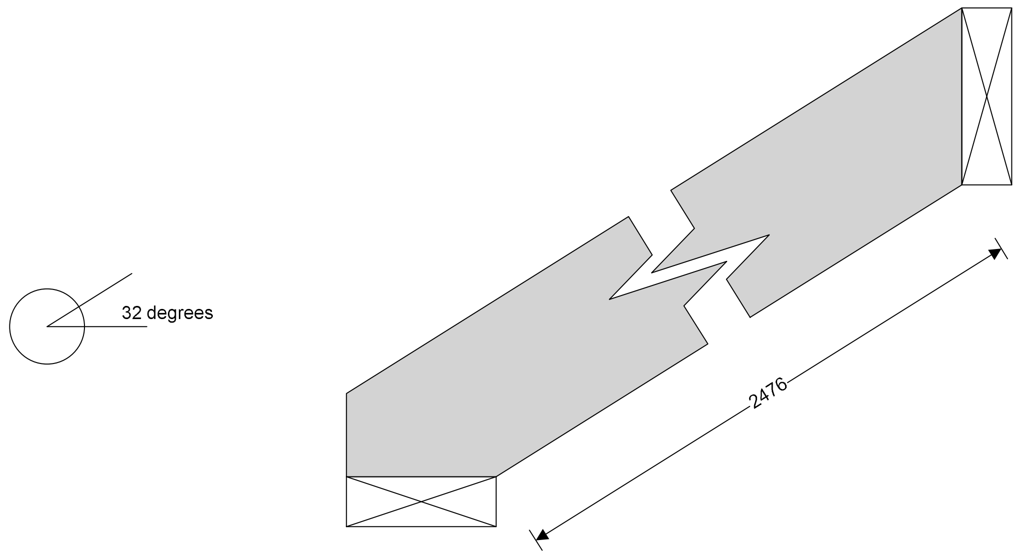
TIMBER ROOF RAFTER DESIGN (BS5268)

TIMBER RAFTER DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.03



Rafter details

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

Depth of timber sections; h = **150** mm

Rafter spacing; s = **400** mm

Rafter slope; α = **32.0** deg

Clear span of rafter on horizontal; L_{clh} = **2100** mm

Clear span of rafter on slope; $L_{cl} = L_{clh} / \cos(\alpha)$ = **2476** mm

Rafter span; **Single span**

Timber strength class; **C24**

Section properties

Cross sectional area of rafter; $A = b \times h$ = **7500** mm^2

Section modulus; $Z = b \times h^2 / 6$ = **187500** mm^3

Second moment of area; $I = b \times h^3 / 12$ = **14062500** mm^4

Radius of gyration; $r = \sqrt{(I / A)}$ = **43.3** mm

Loading details

Rafter self weight; $F_j = b \times h \times \rho_{char} \times g_{acc}$ = **0.03** kN/m

Dead load on slope; F_d = **1.00** kN/m^2

Imposed load on plan; F_u = **0.75** kN/m^2

Imposed point load; F_p = **0.90** kN

Modification factors

Section depth factor; $K_7 = (300\ \text{mm} / h)^{0.11}$ = **1.08**

Load sharing factor; K_8 = **1.10**

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

Load duration factor; $K_3 = \mathbf{1.00}$

Total UDL perpendicular to rafter; $F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = \mathbf{0.361}$ kN/m

Notional bearing length; $L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = \mathbf{3}$ mm

Effective span; $L_{eff} = L_{cl} + L_b = \mathbf{2480}$ mm

Check bending stress

Bending stress parallel to grain; $\sigma_m = \mathbf{7.500}$ N/mm^2

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{8.904}$ N/mm^2

Applied bending stress; $\sigma_{m\_max} = F \times L_{eff}^2 / (8 \times Z) = \mathbf{1.480}$ N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain; $\sigma_c = \mathbf{7.900}$ N/mm^2

Minimum modulus of elasticity; $E_{min} = \mathbf{7200}$ N/mm^2

Compression member factor; $K_{12} = \mathbf{0.69}$

Permissible compressive stress; $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \mathbf{5.960}$ N/mm^2

Applied compressive stress; $\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) = \mathbf{0.207}$ N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress; $\sigma_e = \pi^2 \times E_{min} / \lambda^2 = \mathbf{21.669}$ N/mm^2

Euler coefficient; $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e) = \mathbf{0.990}$

Combined axial compression and bending check; $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm} = \mathbf{0.203}$; **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain; $\tau = \mathbf{0.710}$ N/mm^2

Permissible shear stress; $\tau_{adm} = \tau \times K_3 \times K_8 = \mathbf{0.781}$ N/mm^2

Applied shear stress; $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) = \mathbf{0.090}$ N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = 0.003 \times L_{eff} = \mathbf{7.439}$ mm

Bending deflection; $\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I) = \mathbf{1.170}$ mm

Shear deflection; $\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) = \mathbf{0.066}$ mm

Total deflection; $\delta_{max} = \delta_b + \delta_s = \mathbf{1.236}$ mm

PASS - Total deflection within permissible limits

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

Load duration factor; $K_3 = \mathbf{1.25}$

Total UDL perpendicular to rafter; $F = [F_u \times \cos(\alpha)^2 + F_d \times \cos(\alpha)] \times s + F_j \times \cos(\alpha) = \mathbf{0.577}$ kN/m

Notional bearing length; $L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = \mathbf{5}$ mm

Effective span; $L_{eff} = L_{cl} + L_b = \mathbf{2482}$ mm

Check bending stress

Bending stress parallel to grain; $\sigma_m = \mathbf{7.500}$ N/mm^2

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{11.130}$ N/mm^2

Applied bending stress; $\sigma_{m\_max} = F \times L_{eff}^2 / (8 \times Z) = \mathbf{2.368}$ N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain; $\sigma_c = \mathbf{7.900}$ N/mm^2

Minimum modulus of elasticity; $E_{min} = \mathbf{7200}$ N/mm^2

Compression member factor; $K_{12} = \mathbf{0.66}$

Permissible compressive stress; $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \mathbf{7.141}$ N/mm^2

Applied compressive stress; $\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) = \mathbf{0.332}$ N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress; $\sigma_e = \pi^2 \times E_{min} / \lambda^2 = \mathbf{21.634}$ N/mm^2

Euler coefficient; $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e) = \mathbf{0.985}$

Combined axial compression and bending check; $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm} = \mathbf{0.262}$; **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain; $\tau = \mathbf{0.710}$ N/mm^2

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

Applied shear stress; $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) = \mathbf{0.143}$ N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = 0.003 \times L_{eff} = \mathbf{7.445}$ mm

Bending deflection; $\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I) = \mathbf{1.876}$ mm

Shear deflection; $\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) = \mathbf{0.105}$ mm

Total deflection; $\delta_{max} = \delta_b + \delta_s = \mathbf{1.981}$ mm

PASS - Total deflection within permissible limits

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

Load duration factor; $K_3 = \mathbf{1.50}$

Total UDL perpendicular to rafter; $F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = \mathbf{0.361}$ kN/m

Notional bearing length; $L_b = [F \times L_{cl} + F_p \times \cos(\alpha)] / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = \mathbf{6}$ mm

Effective span; $L_{eff} = L_{cl} + L_b = \mathbf{2483}$ mm

Check bending stress

Bending stress parallel to grain; $\sigma_m = \mathbf{7.500}$ N/mm^2

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{13.355}$ N/mm^2

Applied bending stress; $\sigma_{m\_max} = F \times L_{eff}^2/(8 \times Z) + F_p \times \cos(\alpha) \times L_{eff}/(4 \times Z) = \mathbf{4.010}$ N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain; $\sigma_c = \mathbf{7.900}$ N/mm^2

Minimum modulus of elasticity; $E_{min} = \mathbf{7200}$ N/mm^2

Compression member factor; $K_{12} = \mathbf{0.63}$

Permissible compressive stress; $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \mathbf{8.185}$ N/mm^2

Applied compressive stress; $\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha)+3 \times \tan(\alpha))/(2 \times A) + F_p \times \sin(\alpha)/A = \mathbf{0.271}$ N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress; $\sigma_e = \pi^2 \times E_{min} / \lambda^2 =$ **21.619** N/mm^2

Euler coefficient; $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e) =$ **0.988**

Combined axial compression and bending check; $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm} =$ **0.337**; **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain; $\tau =$ **0.710** N/mm^2

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

Applied shear stress; $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) + 3 \times F_p \times \cos(\alpha) / (2 \times A) =$ **0.242** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = 0.003 \times L_{eff} =$ **7.448** mm

Bending deflection; $\delta_b = L_{eff}^3 \times (5 \times F \times L_{eff} / 384 + F_p \times \cos(\alpha) / 48) / (E_{mean} \times I) =$ **2.778** mm

Shear deflection; $\delta_s = 12 \times L_{eff} \times (F \times L_{eff} + 2 \times F_p \times \cos(\alpha)) / (5 \times E_{mean} \times A) =$ **0.178** mm

Total deflection; $\delta_{max} = \delta_b + \delta_s =$ **2.956** mm

PASS - Total deflection within permissible limits

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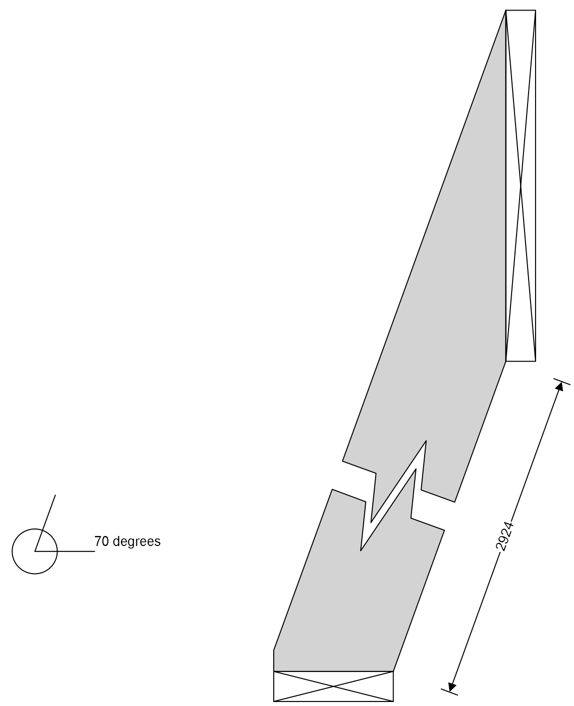
TIMBER MANSARD RAFTER DESIGN (BS5268)

TIMBER RAFTER DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.03



Rafter details

Breadth of timber sections;	$b = \mathbf{50}$ mm
Depth of timber sections;	$h = \mathbf{200}$ mm
Rafter spacing;	$s = \mathbf{300}$ mm
Rafter slope;	$\alpha = \mathbf{70.0}$ deg
Clear span of rafter on horizontal;	$L_{clh} = \mathbf{1000}$ mm
Clear span of rafter on slope;	$L_{cl} = L_{clh} / \cos(\alpha) = \mathbf{2924}$ mm
Rafter span;	**Single span**
Timber strength class;	**C24**

Section properties

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

Loading details

Rafter self weight;	$F_j = b \times h \times \rho_{char} \times g_{acc} = \mathbf{0.03}$ kN/m
Dead load on slope;	$F_d = \mathbf{1.00}$ kN/m^2
Imposed load on plan;	$F_u = \mathbf{0.10}$ kN/m^2

Imposed point load; F_p = **0.90** kN

Modification factors

Section depth factor; $K_7 = (300\text{ mm} / h)^{0.11} = \mathbf{1.05}$

Load sharing factor; $K_8 = \mathbf{1.10}$

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

Load duration factor; $K_3 = \mathbf{1.00}$

Total UDL perpendicular to rafter; $F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = \mathbf{0.114}$ kN/m

Notional bearing length; $L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = \mathbf{1}$ mm

Effective span; $L_{eff} = L_{cl} + L_b = \mathbf{2925}$ mm

Check bending stress

Bending stress parallel to grain; $\sigma_m = \mathbf{7.500}$ N/mm^2

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{8.626}$ N/mm^2

Applied bending stress; $\sigma_{m\_max} = F \times L_{eff}^2 / (8 \times Z) = \mathbf{0.367}$ N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain; $\sigma_c = \mathbf{7.900}$ N/mm^2

Minimum modulus of elasticity; $E_{min} = \mathbf{7200}$ N/mm^2

Compression member factor; $K_{12} = \mathbf{0.73}$

Permissible compressive stress; $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \mathbf{6.350}$ N/mm^2

Applied compressive stress; $\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) = \mathbf{0.144}$ N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress; $\sigma_e = \pi^2 \times E_{min} / \lambda^2 = \mathbf{27.685}$ N/mm^2

Euler coefficient; $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e) = \mathbf{0.994}$

Combined axial compression and bending check; $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm} = \mathbf{0.065}$; **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain; $\tau = \mathbf{0.710}$ N/mm^2

Permissible shear stress; $\tau_{adm} = \tau \times K_3 \times K_8 = \mathbf{0.781}$ N/mm^2

Applied shear stress; $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) = \mathbf{0.025}$ N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = 0.003 \times L_{eff} = \mathbf{8.775}$ mm

Bending deflection; $\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I) = \mathbf{0.303}$ mm

Shear deflection; $\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) = \mathbf{0.022}$ mm

Total deflection; $\delta_{max} = \delta_b + \delta_s = \mathbf{0.325}$ mm

PASS - Total deflection within permissible limits

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

Load duration factor; $K_3 = \mathbf{1.25}$

Total UDL perpendicular to rafter; $F = [F_u \times \cos(\alpha)^2 + F_d \times \cos(\alpha)] \times s + F_j \times \cos(\alpha) = \mathbf{0.118}$ kN/m

Notional bearing length; $L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = \mathbf{1}$ mm

Effective span; $L_{eff} = L_{cl} + L_b =$ **2925** mm

Check bending stress

Bending stress parallel to grain; $\sigma_m =$ **7.500** N/mm^2

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 =$ **10.783** N/mm^2

Applied bending stress; $\sigma_{m\_max} = F \times L_{eff}^2 / (8 \times Z) =$ **0.378** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain; $\sigma_c =$ **7.900** N/mm^2

Minimum modulus of elasticity; $E_{min} =$ **7200** N/mm^2

Compression member factor; $K_{12} =$ **0.71**

Permissible compressive stress; $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} =$ **7.712** N/mm^2

Applied compressive stress; $\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) =$ **0.148** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress; $\sigma_e = \pi^2 \times E_{min} / \lambda^2 =$ **27.684** N/mm^2

Euler coefficient; $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e) =$ **0.994**

Combined axial compression and bending check; $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm} =$ **0.055**; **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain; $\tau =$ **0.710** N/mm^2

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

Applied shear stress; $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) =$ **0.026** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = 0.003 \times L_{eff} =$ **8.775** mm

Bending deflection; $\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I) =$ **0.312** mm

Shear deflection; $\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) =$ **0.022** mm

Total deflection; $\delta_{max} = \delta_b + \delta_s =$ **0.334** mm

PASS - Total deflection within permissible limits

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

Load duration factor; $K_3 =$ **1.50**

Total UDL perpendicular to rafter; $F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) =$ **0.114** kN/m

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

Effective span; $L_{eff} = L_{cl} + L_b =$ **2926** mm

Check bending stress

Bending stress parallel to grain; $\sigma_m =$ **7.500** N/mm^2

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 =$ **12.939** N/mm^2

Applied bending stress; $\sigma_{m\_max} = F \times L_{eff}^2/(8 \times Z) + F_p \times \cos(\alpha) \times L_{eff}/(4 \times Z) =$ **1.043** N/mm^2

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain; $\sigma_c =$ **7.900** N/mm^2

Minimum modulus of elasticity; E_{min} = **7200** N/mm^2

Compression member factor; K_{12} = **0.69**

Permissible compressive stress; $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **8.964** N/mm^2

Applied compressive stress; $\sigma_{c\_max} = F \times L_{eff} \times (\cot(\alpha)+3 \times \tan(\alpha))/(2 \times A)+F_p \times \sin(\alpha)/A$ = **0.229** N/mm^2

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress; $\sigma_e = \pi^2 \times E_{min} / \lambda^2$ = **27.663** N/mm^2

Euler coefficient; $K_{eu} = 1 - (1.5 \times \sigma_{c\_max} \times K_{12} / \sigma_e)$ = **0.991**

Combined axial compression and bending check; $\sigma_{m\_max} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_max} / \sigma_{c\_adm}$ = **0.107**; **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain; τ = **0.710** N/mm^2

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

Applied shear stress; $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) + 3 \times F_p \times \cos(\alpha) / (2 \times A)$ = **0.071** N/mm^2

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = 0.003 \times L_{eff}$ = **8.779** mm

Bending deflection; $\delta_b = L_{eff}^3 \times (5 \times F \times L_{eff} / 384 + F_p \times \cos(\alpha)/ 48) / (E_{mean} \times I)$ = **0.750** mm

Shear deflection; $\delta_s = 12 \times L_{eff} \times (F \times L_{eff} + 2 \times F_p \times \cos(\alpha))/(5 \times E_{mean} \times A)$ = **0.062** mm

Total deflection; $\delta_{max} = \delta_b + \delta_s$ = **0.811** mm

PASS - Total deflection within permissible limits

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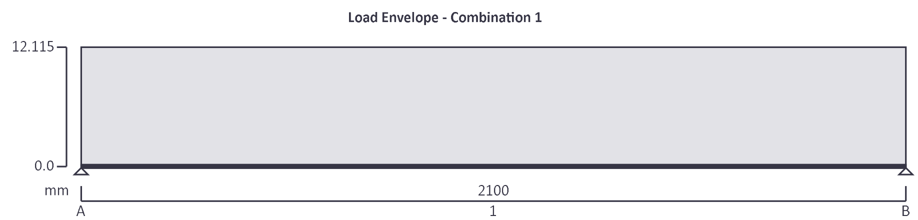
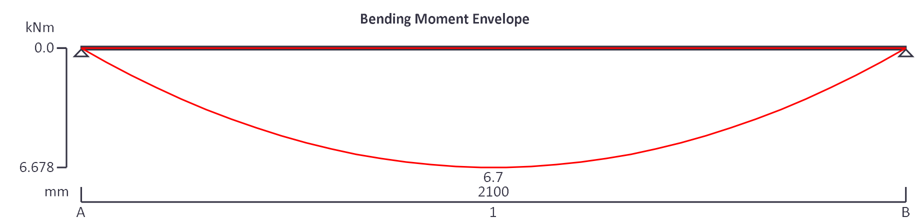
STEEL BEAM B1 ANALYSIS & DESIGN (BS5950)

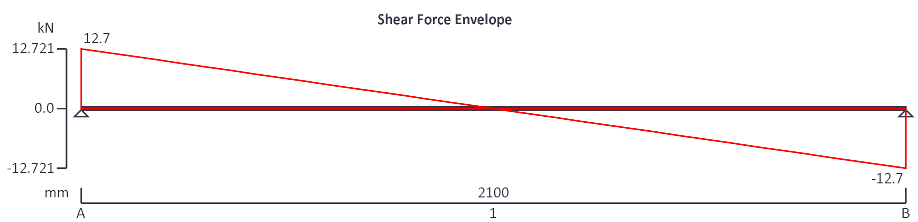
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 5 kN/m
	Imposed full UDL 3 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60

		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

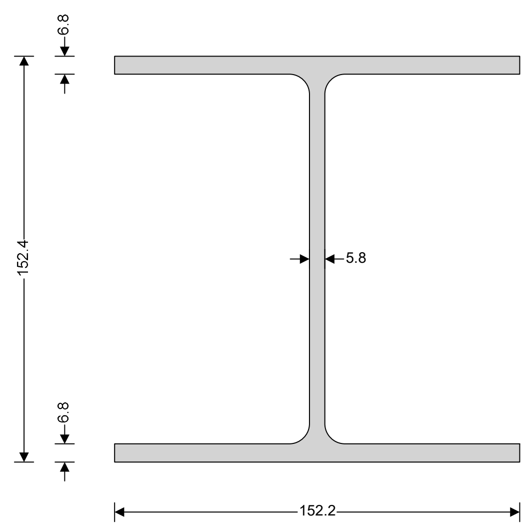
Maximum moment;	M_{max} = **6.7** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **12.7** kN;	V_{min} = **-12.7** kN
Deflection;	δ_{max} = **0.8** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **12.7** kN;	R_{A_min} = **12.7** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **5.5** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **3.2** kN	
Maximum reaction at support B;	R_{B_max} = **12.7** kN;	R_{B_min} = **12.7** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **5.5** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **3.2** kN	

Section details

Section type;	**UC 152x152x23 (BS4-1)**
Steel grade;	**S275**

From table 9: Design strength p_y

Thickness of element;	max(T, t) = **6.8** mm
Design strength;	p_y = **275** N/mm^2
Modulus of elasticity;	E = **205000** N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis;	K_x = **1.00**
Effective length factor in minor axis;	K_y = **1.00**
Effective length factor for lateral-torsional buckling;	$K_{LT.A}$ = **1.00**;
	$K_{LT.B}$ = **1.00**;

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section; d = **123.6** mm

d / t = 21.3 × ε <= 80 × ε; Class 1 plastic

Outstand flanges - Table 11

Width of section; b = B / 2 = **76.1** mm

b / T = 11.2 × ε <= 15 × ε; Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **12.7** kN

d / t < 70 × ε

Web does not need to be checked for shear buckling

Shear area; $A_v = t \times D$ = **884** mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v$ = **145.8** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **6.7** kNm

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange; $\beta_{2f} = 10 \times \varepsilon$ = **10**

Limiting value for class 3 semi-compact flange; $\beta_{3f} = 15 \times \varepsilon$ = **15**

Limiting value for class 2 compact web; $\beta_{2w} = 100 \times \varepsilon$ = **100**

Limiting value for class 3 semi-compact web; $\beta_{3w} = 120 \times \varepsilon$ = **120**

Effective plastic modulus - cl.3.5.6.2

$S_{eff} = \min(Z_{xx} + (S_{xx} - Z_{xx}) \times \min([((\beta_{3w} / (d / t))^2 - 1) / ((\beta_{3w} / \beta_{2w})^2 - 1)], [(\beta_{3f} / (b / T) - 1) / (\beta_{3f} / \beta_{2f} - 1)]), S_{xx})$ = **176245** mm^3

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{eff}, 1.2 \times p_y \times Z_{xx})$ = **48.5** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.0 \times L_{s1}$ = **2100** mm

Slenderness ratio; $\lambda = L_E / r_{yy}$ = **56.789**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; u = **0.840**

Torsional index; x = **20.701**

Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25}$ = **0.923**

Ratio - cl.4.3.6.9; $\beta_W = S_{eff} / S_{xx}$ = **0.968**

Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **43.319**

Limiting slenderness - Annex B.2.2; $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5}$ = **34.310**

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

Bending strength - Section 4.3.6.5

Robertson constant; α_{LT} = **7.0**

Perry factor; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.063**

Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2$ = **1078.2** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \mathbf{710.6}$ N/mm^2

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = \mathbf{254}$ N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 = \mathbf{5}$ kNm

Moment at centre-line of segment; $M_3 = \mathbf{6.7}$ kNm

Moment at three quarter point of segment; $M_4 = \mathbf{5}$ kNm

Maximum moment in segment; $M_{abs} = \mathbf{6.7}$ kNm

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} = \mathbf{6.7}$ kNm

Equivalent uniform moment factor for lateral-torsional buckling;

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

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{eff} = \mathbf{44.8}$ kNm

$M_b / m_{LT} = \mathbf{48.4}$ kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection; $\delta_{lim} = \min(12\text{ mm}, L_{s1} / 360) = \mathbf{5.833}$ mm

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{0.813}$ mm

PASS - Maximum deflection does not exceed deflection limit

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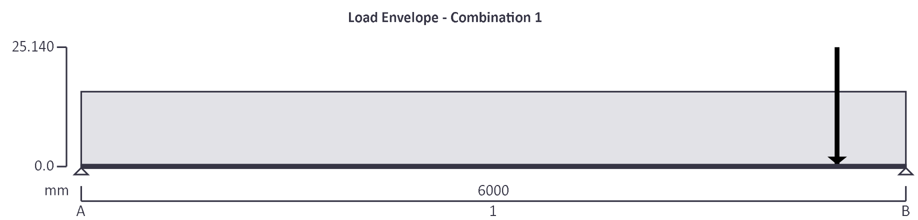
HOME TALES	Project				Job Ref. DD/2021/256	
	Section TEKLA TEDDS ANNEX				Sheet no./rev. 27	
	Calc. by ID	Date	Chk'd by ID	Date	App'd by	Date

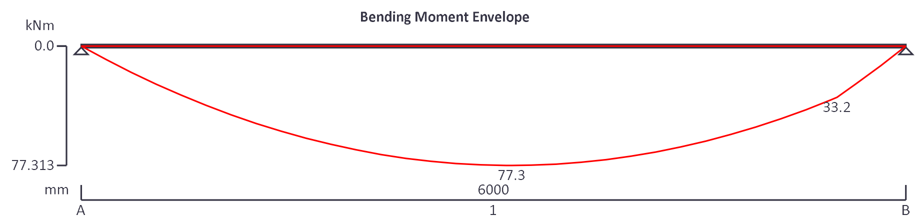
STEEL BEAM B2 ANALYSIS & DESIGN (BS5950)

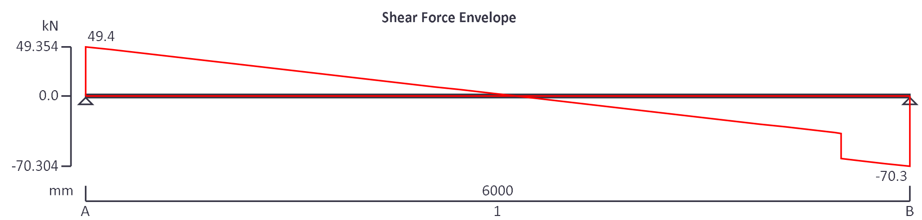
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 4.4 kN/m
	Imposed full UDL 5.6 kN/m
	Dead point load 10.3 kN at 5500 mm
	Imposed point load 6.7 kN at 5500 mm

Load combinations

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

Analysis results

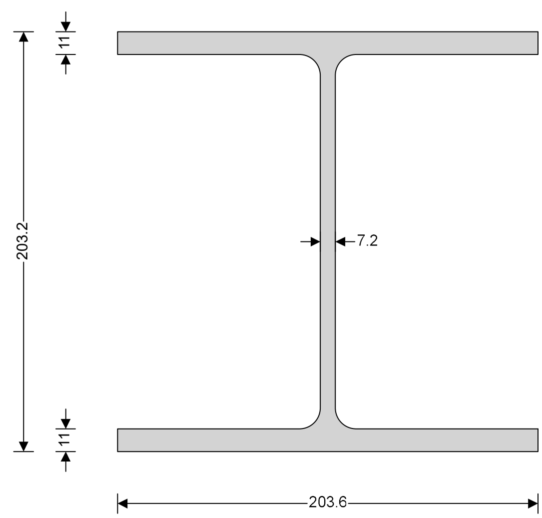
Maximum moment;	M_{max} = **77.3** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **49.4** kN;	V_{min} = **-70.3** kN
Deflection;	δ_{max} = **10.9** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **49.4** kN;	R_{A_min} = **49.4** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **15.4** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **17.4** kN	
Maximum reaction at support B;	R_{B_max} = **70.3** kN;	R_{B_min} = **70.3** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **24** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **22.9** kN	

Section details

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

From table 9: Design strength p_y

Thickness of element;	max(T, t) = **11.0** mm
Design strength;	p_y = **275** N/mm^2
Modulus of elasticity;	E = **205000** N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis;	K_x = **1.00**

Effective length factor in minor axis; K_y = **1.00**

Effective length factor for lateral-torsional buckling; $K_{LT.A}$ = **1.00**;

$K_{LT.B}$ = **1.00**;

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section; d = **160.8** mm

$d / t = 22.3 \times \varepsilon <= 80 \times \varepsilon$; Class 1 plastic

Outstand flanges - Table 11

Width of section; b = B / 2 = **101.8** mm

$b / T = 9.3 \times \varepsilon <= 10 \times \varepsilon$; Class 2 compact

Section is class 2 compact

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **70.3** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area; $A_v = t \times D$ = **1463** mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v$ = **241.4** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **77.3** kNm

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx})$ = **136.8** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.0 \times L_{s1}$ = **6000** mm

Slenderness ratio; $\lambda = L_E / r_{yy}$ = **116.862**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; u = **0.847**

Torsional index; x = **17.713**

Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25}$ = **0.749**

Ratio - cl.4.3.6.9; β_W = **1.000**

Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **74.100**

Limiting slenderness - Annex B.2.2; $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5}$ = **34.310**

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

Bending strength - Section 4.3.6.5

Robertson constant; α_{LT} = **7.0**

Perry factor; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.279**

Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2$ = **368.5** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2$ = **373.1** N/mm^2

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5})$ = **178.5** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; M_2 = **56.3** kNm

Moment at centre-line of segment; M_3 = **77.2** kNm

Moment at three quarter point of segment; M_4 = **62.6** kNm

Maximum moment in segment; M_{abs} = **77.3** kNm

Maximum moment governing buckling resistance; $M_{LT} = M_{abs}$ = **77.3** kNm

Equivalent uniform moment factor for lateral-torsional buckling;

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44)$ = **0.930**

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx}$ = **88.8** kNm

M_b / m_{LT} = **95.5** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360$ = **16.667** mm

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **10.891** mm

PASS - Maximum deflection does not exceed deflection limit

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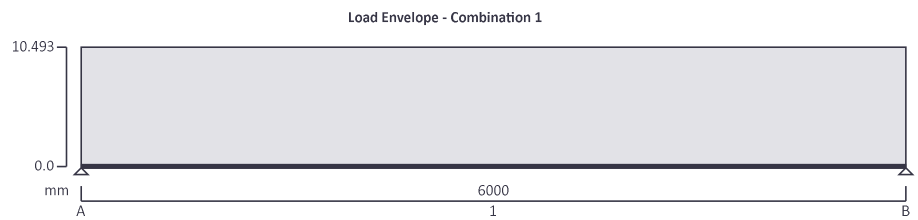
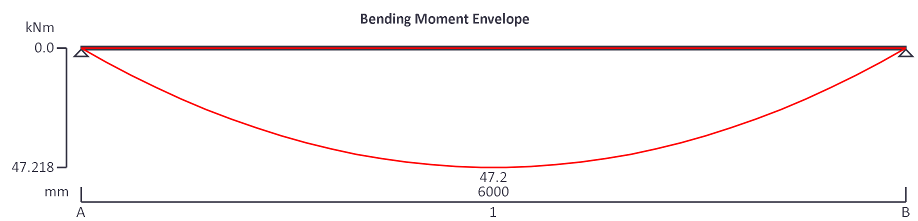
STEEL BEAM B3 ANALYSIS & DESIGN (BS5950)

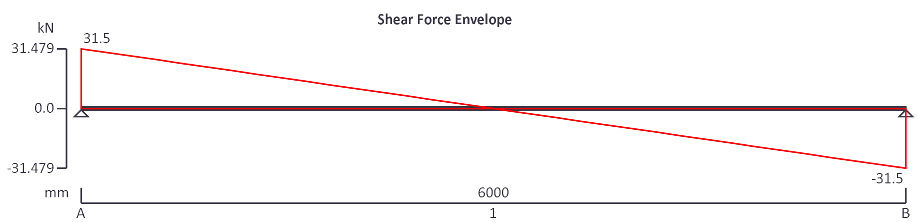
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 3.5 kN/m
	Imposed full UDL 3.1 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60

		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

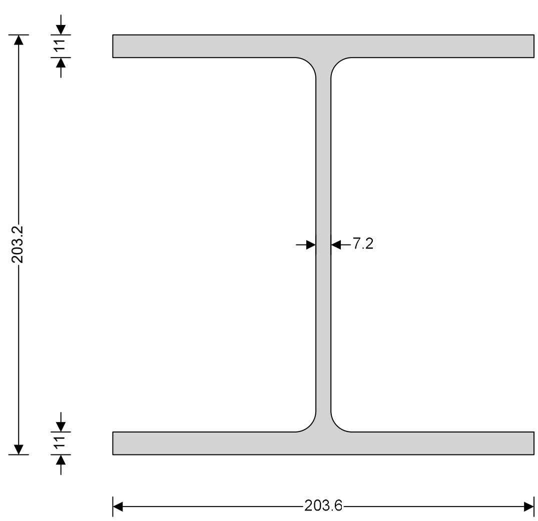
Maximum moment;	M_{max} = **47.2** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **31.5** kN;	V_{min} = **-31.5** kN
Deflection;	δ_{max} = **5.6** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **31.5** kN;	R_{A_min} = **31.5** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **11.9** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **9.3** kN	
Maximum reaction at support B;	R_{B_max} = **31.5** kN;	R_{B_min} = **31.5** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **11.9** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **9.3** kN	

Section details

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

From table 9: Design strength p_y

Thickness of element;	max(T, t) = **11.0** mm
Design strength;	p_y = **275** N/mm^2
Modulus of elasticity;	E = **205000** N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis;	K_x = **1.00**
Effective length factor in minor axis;	K_y = **1.00**
Effective length factor for lateral-torsional buckling;	$K_{LT.A}$ = **1.00**;
	$K_{LT.B}$ = **1.00**;

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section; $d = \mathbf{160.8}$ mm

$d / t = 22.3 \times \varepsilon <= 80 \times \varepsilon$; Class 1 plastic

Outstand flanges - Table 11

Width of section; $b = B / 2 = \mathbf{101.8}$ mm

$b / T = 9.3 \times \varepsilon <= 10 \times \varepsilon$; Class 2 compact

Section is class 2 compact

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{31.5}$ kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area; $A_v = t \times D = \mathbf{1463}$ mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v = \mathbf{241.4}$ kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{47.2}$ kNm

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = \mathbf{136.8}$ kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.0 \times L_{s1} = \mathbf{6000}$ mm

Slenderness ratio; $\lambda = L_E / r_{yy} = \mathbf{116.862}$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u = \mathbf{0.847}$

Torsional index; $x = \mathbf{17.713}$

Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = \mathbf{0.749}$

Ratio - cl.4.3.6.9; $\beta_W = \mathbf{1.000}$

Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} = \mathbf{74.100}$

Limiting slenderness - Annex B.2.2; $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = \mathbf{34.310}$

λ_{LT} > λ_{L0} - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant; $\alpha_{LT} = \mathbf{7.0}$

Perry factor; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = \mathbf{0.279}$

Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2 = \mathbf{368.5}$ N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \mathbf{373.1}$ N/mm^2

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = \mathbf{178.5}$ N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 = \mathbf{35.4}$ kNm

Moment at centre-line of segment; $M_3 = \mathbf{47.2}$ kNm

Moment at three quarter point of segment; $M_4 = \mathbf{35.4}$ kNm

Maximum moment in segment; $M_{abs} = \mathbf{47.2}$ kNm

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} = \mathbf{47.2}$ kNm

Equivalent uniform moment factor for lateral-torsional buckling;

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

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx} = \mathbf{88.8}$ kNm

$M_b / m_{LT} = \mathbf{96}$ kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360 = \mathbf{16.667}$ mm

Maximum deflection span 1; $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = \mathbf{5.587}$ mm

PASS - Maximum deflection does not exceed deflection limit

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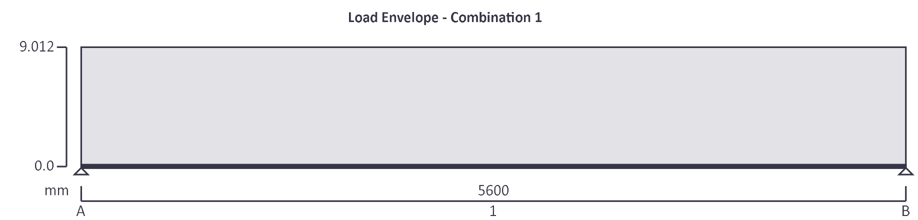
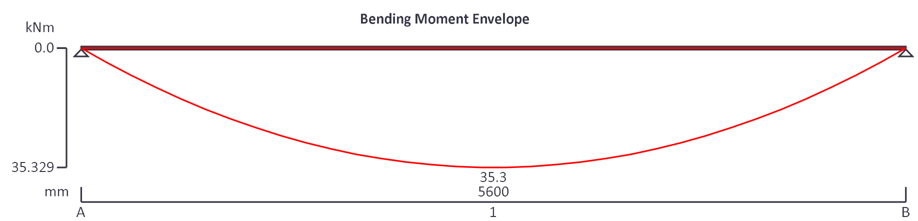
STEEL BEAM B4 ANALYSIS & DESIGN (BS5950)

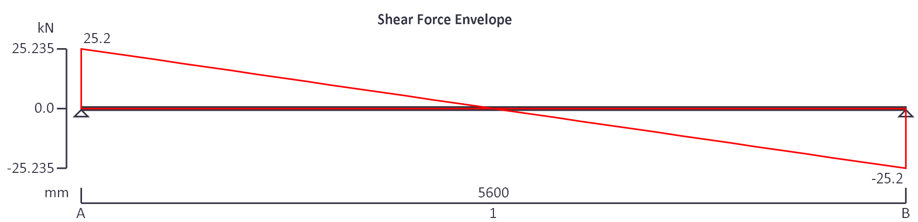
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 3.4 kN/m
	Imposed full UDL 2.4 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60

		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

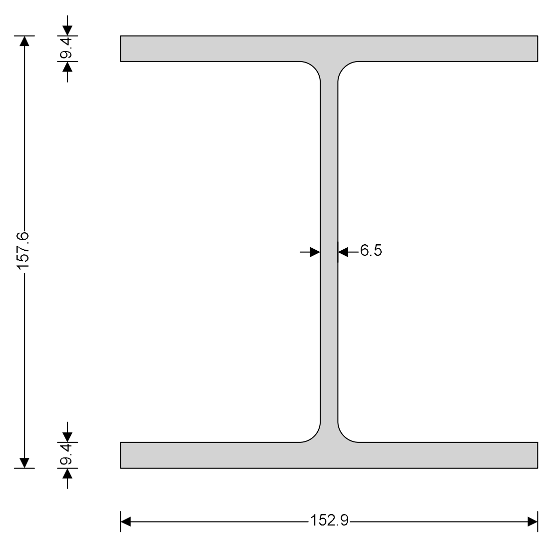
Maximum moment;	M_{max} = **35.3** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **25.2** kN;	V_{min} = **-25.2** kN
Deflection;	δ_{max} = **8.6** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **25.2** kN;	R_{A_min} = **25.2** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **10.3** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **6.7** kN	
Maximum reaction at support B;	R_{B_max} = **25.2** kN;	R_{B_min} = **25.2** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **10.3** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **6.7** kN	

Section details

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

From table 9: Design strength p_y

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



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis;	K_x = **1.00**
Effective length factor in minor axis;	K_y = **1.00**
Effective length factor for lateral-torsional buckling;	$K_{LT.A}$ = **1.00**;
	$K_{LT.B}$ = **1.00**;

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section; $d = \mathbf{123.6}$ mm

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

Outstand flanges - Table 11

Width of section; $b = B / 2 = \mathbf{76.5}$ mm

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

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{25.2}$ kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area; $A_v = t \times D = \mathbf{1024}$ mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v = \mathbf{169}$ kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{35.3}$ kNm

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

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.0 \times L_{s1} = \mathbf{5600}$ mm

Slenderness ratio; $\lambda = L_E / r_{yy} = \mathbf{146.320}$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u = \mathbf{0.849}$

Torsional index; $x = \mathbf{15.999}$

Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = \mathbf{0.663}$

Ratio - cl.4.3.6.9; $\beta_W = \mathbf{1.000}$

Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} = \mathbf{82.288}$

Limiting slenderness - Annex B.2.2; $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = \mathbf{34.310}$

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

Bending strength - Section 4.3.6.5

Robertson constant; $\alpha_{LT} = \mathbf{7.0}$

Perry factor; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = \mathbf{0.336}$

Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2 = \mathbf{298.8}$ N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \mathbf{337.1}$ N/mm^2

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = \mathbf{159.7}$ N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 = \mathbf{26.5}$ kNm

Moment at centre-line of segment; $M_3 = \mathbf{35.3}$ kNm

Moment at three quarter point of segment; $M_4 = \mathbf{26.5}$ kNm

Maximum moment in segment; $M_{abs} = \mathbf{35.3}$ kNm

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} =$ **35.3** kNm

Equivalent uniform moment factor for lateral-torsional buckling;

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

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx} =$ **39.6** kNm

$M_b / m_{LT} =$ **42.8** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360 =$ **15.556** mm

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **8.576** mm

PASS - Maximum deflection does not exceed deflection limit

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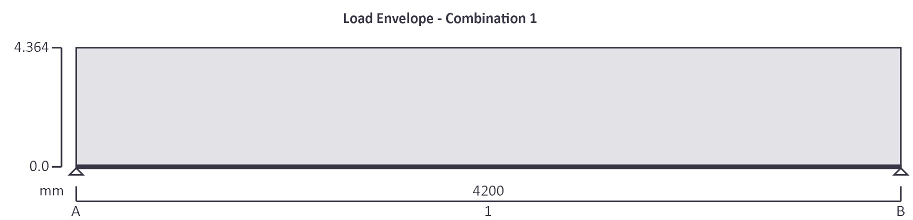
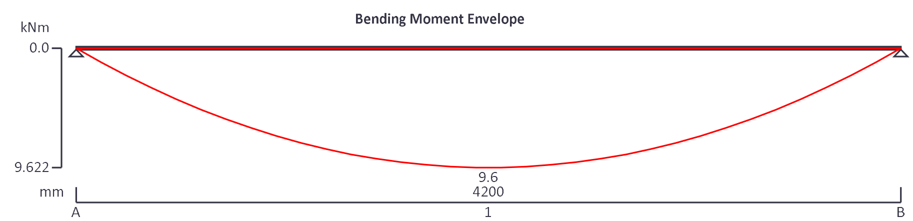
	Project				Job Ref.	
HOME TALES					DD/2021/256	
	Section				Sheet no./rev.	
	TEKLA TEDDS ANNEX				39	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	ID		ID			

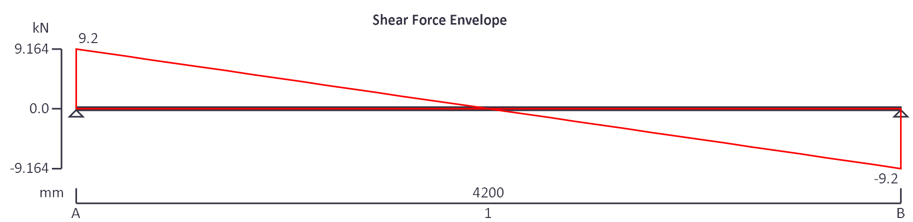
FLITCH BEAM B5 ANALYSIS & DESIGN (BS5268)

FLITCH BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.01







Applied loading

Beam loads

Dead self weight of beam × 1

Dead full UDL 2.300 kN/m

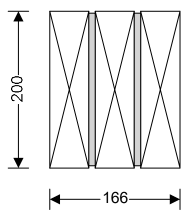
Imposed full UDL 1.700 kN/m

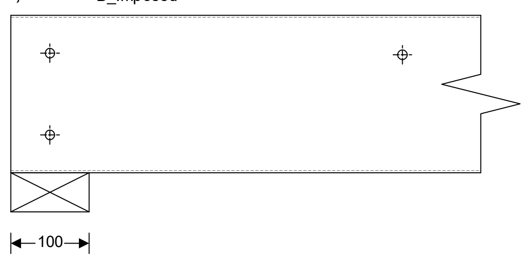
Load combinations

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} = **9.622** kNm;	M_{min} = **0.000** kNm
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **9.622** kNm	
Maximum shear;	F_{max} = **9.164** kN;	F_{min} = **-9.164** kN
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **9.164** kN	
Total load on beam;	W_{tot} = **18.328** kN	
Reactions at support A;	R_{A_max} = **9.164** kN;	R_{A_min} = **9.164** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **5.594** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **3.570** kN	
Reactions at support B;	R_{B_max} = **9.164** kN;	R_{B_min} = **9.164** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **5.594** 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 = **3**
Timber strength class;	**C24**

Steel section details

Breadth of steel plate;	b_s = **8** mm
Depth of steel plate;	h_s = **195** mm
Number of steel plates in beam;	N_s = **2**
Steel stress;	p_y = **165** N/mm^2
Bolt diameter;	ϕ_b = **12** 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;	$A = N \times b \times h$ = **30000** mm^2
Timber section modulus;	$Z_{xt} = N \times b \times h^2 / 6$ = **1000000** mm^3
Steel section modulus;	$Z_{xs} = N_s \times b_s \times h_s^2 / 6$ = **101400** 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;	$I_{xs} = N_s \times b_s \times h_s^3 / 12$ = **9886500** mm^4

Load proportions

Instant deflection under permanent actions; u_{instG} = **3.474** mm

Instant deflection under principal variable action; u_{instQ1} = **2.217** mm

k_{def} = **0.6**

ψ_2 = **0.3**

Final minimum modulus of elasticity

$E_{min,fin} = E_{min} \times (u_{instG} + u_{instQ1}) / (u_{instG} + u_{instQ1} + k_{def} \times (u_{instG} + \psi_2 \times u_{instQ1}))$ = **5013** N/mm^2

Proportion of applied load in timber; $k_t = E_{mean} \times I_{xt} / (E_{mean} \times I_{xt} + E_{S5950} \times I_{xs})$ = **0.348**

Proportion of applied load in steel; $k_s = 1.1 \times E_{S5950} \times I_{xs} / (E_{min,fin} \times I_{xt} + E_{S5950} \times I_{xs})$ = **0.882**

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; $h / (N \times b + N_s \times b_s)$ = **1.20**

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; $\sigma_{c\_a} = R_{B\_max} / (N \times b \times L_b)$ = **0.611** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.255**

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; $\sigma_{m\_a} = k_t \times M / Z_{xt}$ = **3.345** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.427**

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

Applied steel bending stress; $\sigma_{m\_a\_s} = k_s \times M / Z_{xs}$ = **83.684** N/mm^2

$\sigma_{m\_a\_s} / p_y$ = **0.507**

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; $\tau_a = 3 \times k_t \times F / (2 \times A)$ = **0.159** N/mm^2

τ_a / τ_{adm} = **0.224**

PASS - Shear stress within permissible limits

Deflection

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

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

Bending deflection; δ_{b_s1} = **5.691** mm

Shear deflection; δ_{v_s1} = **0.570** mm

Total deflection; $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **6.261** mm

δ_a / δ_{adm} = **0.497**

PASS - Total deflection is less than permissible deflection

Flitch plate bolting requirements

Total load on beam; W_{tot} = **18.328** kN

Total load taken by steel; $W_s = k_s \times W_{tot}$ = **16.163** kN

Basic bolt shear load - Table 77; v_{90} = **1.668** kN

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

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

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

Maximum bolt spacing; S_{max} = **450** mm

Minimum number of bolts along length of beam; $N_{bl} = W_s / (N_{int} \times v_{90})$ = **2.423**

- Provide a minimum of 2 No.12 mm diameter bolts at each support
- Provide 12 mm diameter bolts at maximum 450 mm centres staggered 50 mm alternately above and below the centre line

Minimum bolt spacings

Minimum end spacing; $S_{end} = 4 \times \phi_b$ = **48** mm

Minimum edge spacing; $S_{edge} = 4 \times \phi_b$ = **48** mm

Minimum bolt spacing; $S_{bolt} = 4 \times \phi_b$ = **48** mm

Minimum washer diameter; $\phi_w = 3 \times \phi_b$ = **36** mm

Minimum washer thickness; $t_w = 0.25 \times \phi_b$ = **3** mm

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