<u>165 Englands Lane</u>

DESIGN PACK

Project no:	165ENL/07/26
Document no:	165ENL/07/26-D1
Revision:	S0
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Project Brief:
Loft conversion.

Reference Documents:

- Architect Drawings

Design Codes:

- BSEN1990 Basis of structure
- BSEN1991 Actions on structure
- BSEN1992 Design of concrete structures
- BSEN1993 Design of steel structures
- BSEN1996 Design of masonry structures
- BSEN1995 Design of timber structures

Reference Design Software:

- TEDDS

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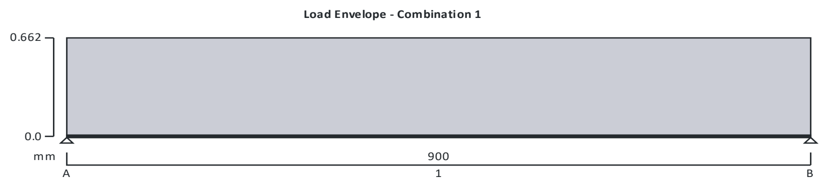
Steel beams design 29

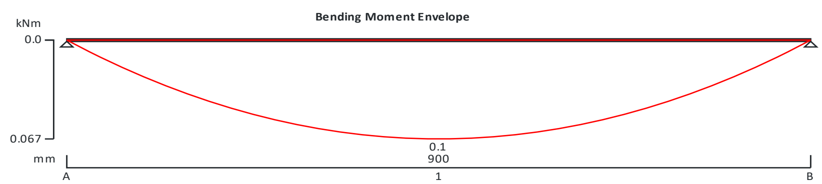
Timber Joists - Internal Floor Loads		
Item	**Unfactored Load (KN/m^2)**	**Load Type**
Floor joists	0.15	Dead
Floor boards	0.05	Dead
Plaster board and Skim	0.25	Dead
Finish	0.05	Dead
Residential Live Loading	1.5	Live
TOTAL FLOOR DEAD LOAD	**0.5 (KN/m2)**	
TOTAL FLOOR LIVE LOAD	**1.5 (KN/m2)**	

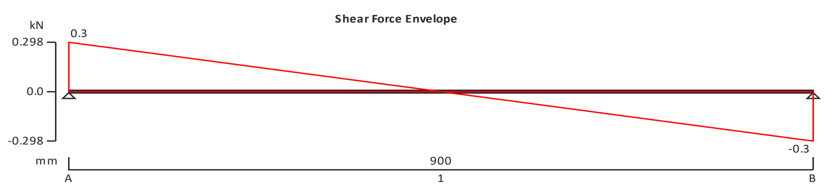
Proposed UDLs				
Item	**Span (m)**	**Dead Load (KN/m^2)**	**Live Load (KN/m^2)**	**Comments**
Loft Floor Loading	0.3	0.5	1.5	See Load Table For Internal Floor
	Total Dead		**0.15 (KN/m)**	
	Total Live		**0.45 (KN/m)**	

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.03







Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 0.150 kN/m
Imposed full UDL 0.450 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

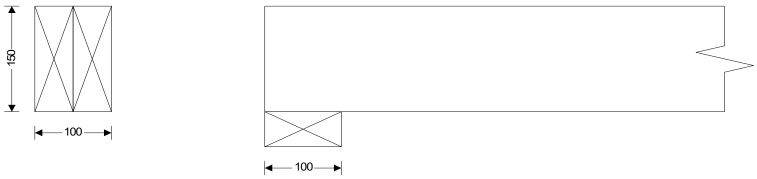
Maximum moment	M_{max} = **0.067** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **0.067** kNm	
Maximum shear	F_{max} = **0.298** kN	F_{min} = **-0.298** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **0.298** kN	
Total load on beam	W_{tot} = **0.596** kN	
Reactions at support A	R_{A_max} = **0.298** kN	R_{A_min} = **0.298** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **0.095** kN	

Unfactored imposed load reaction at support A $R_{A\_Imposed}$ = **0.203** kN

Reactions at support B R_{B_max} = **0.298** kN R_{B_min} = **0.298** kN

Unfactored dead load reaction at support B R_{B_Dead} = **0.095** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **0.203** kN



Timber section details

Breadth of sections b = **50** mm

Depth of sections h = **150** mm

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

Overall breadth of member $b_b = N \times b$ = **100** mm

Timber strength class **C24**

Member details

Service class of timber **1**

Load duration **Long term**

Length of span L_{s1} = **900** mm

Length of bearing L_b = **100** mm

Section properties

Cross sectional area of member $A = N \times b \times h$ = **15000** mm^2

Section modulus $Z_x = N \times b \times h^2 / 6$ = **375000** mm^3

$Z_y = h \times (N \times b)^2 / 6$ = **250000** mm^3

Second moment of area $I_x = N \times b \times h^3 / 12$ = **28125000** mm^4

$I_y = h \times (N \times b)^3 / 12$ = **12500000** mm^4

Radius of gyration $i_x = \sqrt{(I_x / A)}$ = **43.3** mm

$i_y = \sqrt{(I_y / A)}$ = **28.9** mm

Modification factors

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

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

Total depth of member - cl.2.10.6 $K_7 = (300 \text{ mm} / h)^{0.11}$ = **1.08**

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 $h / (N \times b)$ = **1.50**

PASS - Lateral support is adequate

Compression perpendicular to grain

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

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b)$ = **0.030** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.012**

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

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 =$ **8.094** N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x =$ **0.179** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm} =$ **0.022**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

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

Applied shear stress $\tau_a = 3 \times F / (2 \times A) =$ **0.030** N/mm^2

$\tau_a / \tau_{adm} =$ **0.042**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min} =$ **7200** N/mm^2

Permissible deflection $\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1}) =$ **2.700** mm

Bending deflection $\delta_{b\_s1} =$ **0.028** mm

Shear deflection $\delta_{v\_s1} =$ **0.012** mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} =$ **0.040** mm

$\delta_a / \delta_{adm} =$ **0.015**

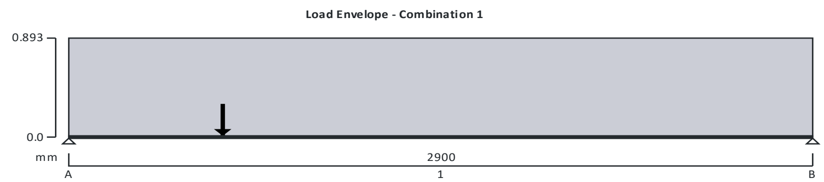
PASS - Total deflection is less than permissible deflection

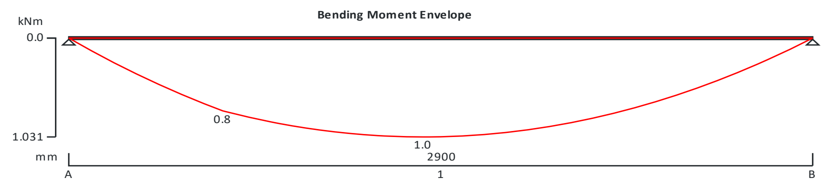
Timber Joists - Internal Floor Loads		
Item	**Unfactored Load (KN/m^2)**	**Load Type**
Floor joists	0.15	Dead
Floor boards	0.05	Dead
Plaster board and Skim	0.25	Dead
Finish	0.05	Dead
Residential Live Loading	1.5	Live
TOTAL FLOOR DEAD LOAD	**0.5 (KN/m2)**	
TOTAL FLOOR LIVE LOAD	**1.5 (KN/m2)**	

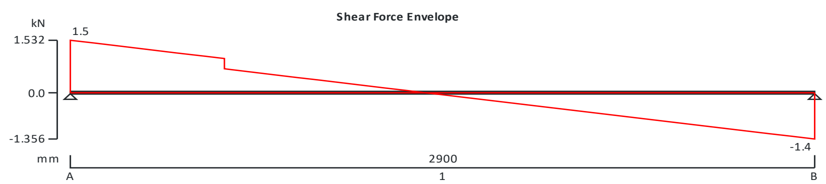
Proposed UDLs				
Item	**Span (m)**	**Dead Load (KN/m^2)**	**Live Load (KN/m^2)**	**Comments**
Loft Floor Loading	0.4	0.5	1.5	See Load Table For Internal Floor
T1 Reaction	-	-	-	0.3 KN @ 600mm
		Total Dead	**0.2 (KN/m)**	
		Total Live	**0.6 (KN/m)**	

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.03







Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 0.200 kN/m
Imposed full UDL 0.600 kN/m
Other point load 0.300 kN at 600 mm

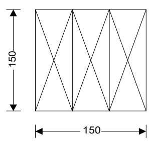
Load combinations

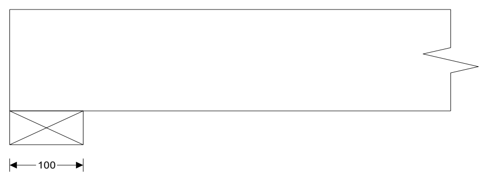
Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
		Other × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
		Other × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00
		Other × 1.00

Analysis results

Maximum moment	M_{max} = **1.031** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **1.031** kNm	
Maximum shear	F_{max} = **1.532** kN	F_{min} = **-1.356** kN

Design shear	$F = max(abs(F_{max}),abs(F_{min}))$ = **1.532** kN	
Total load on beam	W_{tot} = **2.889** kN	
Reactions at support A	R_{A_max} = **1.532** kN	R_{A_min} = **1.532** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **0.424** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **0.870** kN	
Unfactored other load reaction at support A	R_{A_Other} = **0.238** kN	
Reactions at support B	R_{B_max} = **1.356** kN	R_{B_min} = **1.356** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **0.424** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **0.870** kN	
Unfactored other load reaction at support B	R_{B_Other} = **0.062** kN	





Timber section details

Breadth of sections	b = **50** mm
Depth of sections	h = **150** mm
Number of sections in member	N = **3**
Overall breadth of member	$b_b = N \times b$ = **150** mm
Timber strength class	**C24**

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of span	L_{s1} = **2900** mm
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	$A = N \times b \times h$ = **22500** mm^2
Section modulus	$Z_x = N \times b \times h^2 / 6$ = **562500** mm^3
	$Z_y = h \times (N \times b)^2 / 6$ = **562500** mm^3
Second moment of area	$I_x = N \times b \times h^3 / 12$ = **42187500** mm^4
	$I_y = h \times (N \times b)^3 / 12$ = **42187500** mm^4
Radius of gyration	$i_x = \sqrt{(I_x / A)}$ = **43.3** mm
	$i_y = \sqrt{(I_y / A)}$ = **43.3** mm

Modification factors

Duration of loading - Table 17	K_3 = **1.00**
Bearing stress - Table 18	K_4 = **1.00**
Total depth of member - cl.2.10.6	$K_7 = (300\ mm / h)^{0.11}$ = **1.08**
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	$h / (N \times b)$ = **1.00**

PASS - Lateral support is adequate

Compression perpendicular to grain

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

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b) =$ **0.102** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm} =$ **0.043**

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

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 =$ **8.094** N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x =$ **1.832** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm} =$ **0.226**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

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

Applied shear stress $\tau_a = 3 \times F / (2 \times A) =$ **0.102** N/mm^2

$\tau_a / \tau_{adm} =$ **0.144**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min} =$ **7200** N/mm^2

Permissible deflection $\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1}) =$ **8.700** mm

Bending deflection $\delta_{b\_s1} =$ **3.006** mm

Shear deflection $\delta_{v\_s1} =$ **0.128** mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} =$ **3.134** mm

$\delta_a / \delta_{adm} =$ **0.360**

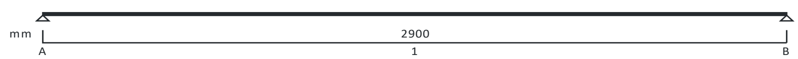
PASS - Total deflection is less than permissible deflection

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

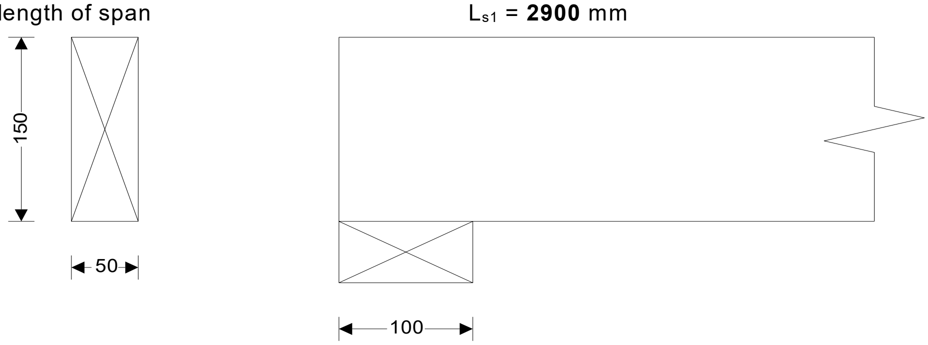
Joist details

Joist breadth	b = **50** mm
Joist depth	h = **150** mm
Joist spacing	s = **400** mm
Timber strength class	**C24**
Service class of timber	**1**



Span details

Number of spans	N_{span} = **1**
Length of bearing	L_b = **100** mm
Effective length of span	L_{s1} = **2900** mm



Section properties

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

Loading details

Joist self weight	$F_{swt} = b \times h \times \rho_{char} \times g_{acc} =$ **0.03** kN/m
Dead load	F_{d_udl} = **0.50** 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 = **1.08**
Load sharing factor	K_8 = **1.10**

Consider long term loads

Load duration factor	K_3 = **1.00**
Maximum bending moment	M = **0.868** kNm
Maximum shear force	V = **1.197** kN
Maximum support reaction	R = **1.197** kN
Maximum deflection	δ = **5.213** mm

Check bending stress

Bending stress	σ_m = **7.500** N/mm^2
Permissible bending stress	$\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **8.904** N/mm^2
Applied bending stress	σ_{m_max} = M / Z = **4.630** 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	$\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.239** 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	$\sigma_{c\_max} = R / (b \times L_b)$ = **0.239** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection	$\delta_{adm} = \min(L_{s1} \times 0.003, 14\text{ mm})$ = **8.700** mm
Bending deflection (based on E_{mean})	$\delta_{bending}$ = **5.007** mm
Shear deflection	δ_{shear} = **0.206** mm
Total deflection	$\delta = \delta_{bending} + \delta_{shear}$ = **5.213** mm

PASS - Actual deflection within permissible limits

Consider medium term loads

Load duration factor	K_3 = **1.25**
Maximum bending moment	M = **1.252** kNm
Maximum shear force	V = **1.727** kN
Maximum support reaction	R = **1.727** kN
Maximum deflection	δ = **6.349** mm

Check bending stress

Bending stress	σ_m = **7.500** N/mm^2
Permissible bending stress	$\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **11.130** N/mm^2
Applied bending stress	σ_{m_max} = M / Z = **6.679** 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	$\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.345** 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 $\sigma_{c\_max} = R / (b \times L_b)$ = **0.345** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = \min(L_{s1} \times 0.003, 14\ mm)$ = **8.700** mm

Bending deflection (based on E_{mean}) $\delta_{bending}$ = **6.053** mm

Shear deflection δ_{shear} = **0.297** mm

Total deflection $\delta = \delta_{bending} + \delta_{shear}$ = **6.349** mm

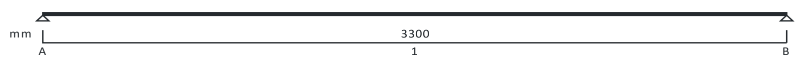
PASS - Actual deflection within permissible limits

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

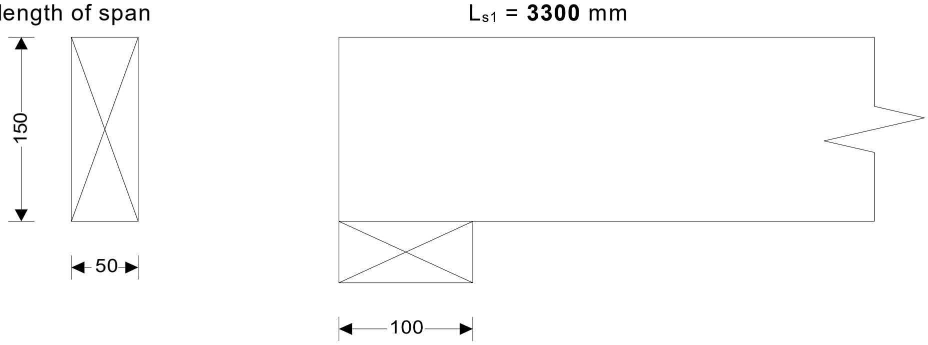
Joist details

Joist breadth	b = **50** mm
Joist depth	h = **150** mm
Joist spacing	s = **400** mm
Timber strength class	**C24**
Service class of timber	**1**



Span details

Number of spans	N_{span} = **1**
Length of bearing	L_b = **100** mm
Effective length of span	L_{s1} = **3300** mm



Section properties

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

Loading details

Joist self weight	$F_{swt} = b \times h \times \rho_{char} \times g_{acc}$ = **0.03** kN/m
Dead load	F_{d_udl} = **0.85** 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 = **1.08**
Load sharing factor	K_8 = **1.10**

Consider medium term loads

Load duration factor	K_3 = **1.25**
Maximum bending moment	M = **0.906** kNm
Maximum shear force	V = **1.098** kN
Maximum support reaction	R = **1.098** kN
Maximum deflection	δ = **6.984** mm

Check bending stress

Bending stress	σ_m = **7.500** N/mm^2
Permissible bending stress	$\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **11.130** N/mm^2
Applied bending stress	σ_{m_max} = M / Z = **4.833** 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	$\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.220** 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	$\sigma_{c\_max} = R / (b \times L_b)$ = **0.220** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection	$\delta_{adm} = \min(L_{s1} \times 0.003, 14\ mm)$ = **9.900** mm
Bending deflection (based on E_{mean})	$\delta_{bending}$ = **6.769** mm
Shear deflection	δ_{shear} = **0.215** mm
Total deflection	$\delta = \delta_{bending} + \delta_{shear}$ = **6.984** mm

PASS - Actual deflection within permissible limits

Consider short term loads

Load duration factor	K_3 = **1.50**
Maximum bending moment	M = **1.240** kNm
Maximum shear force	V = **1.503** kN
Maximum support reaction	R = **1.503** kN
Maximum deflection	δ = **8.449** mm

Check bending stress

Bending stress	σ_m = **7.500** N/mm^2
Permissible bending stress	$\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **13.355** N/mm^2
Applied bending stress	σ_{m_max} = M / Z = **6.615** 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	$\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.301** 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 $\sigma_{c\_max} = R / (b \times L_b)$ = **0.301** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = \min(L_{s1} \times 0.003, 14\text{ mm})$ = **9.900** mm

Bending deflection (based on E_{mean}) $\delta_{bending}$ = **8.155** mm

Shear deflection δ_{shear} = **0.294** mm

Total deflection $\delta = \delta_{bending} + \delta_{shear}$ = **8.449** mm

PASS - Actual deflection within permissible limits

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

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

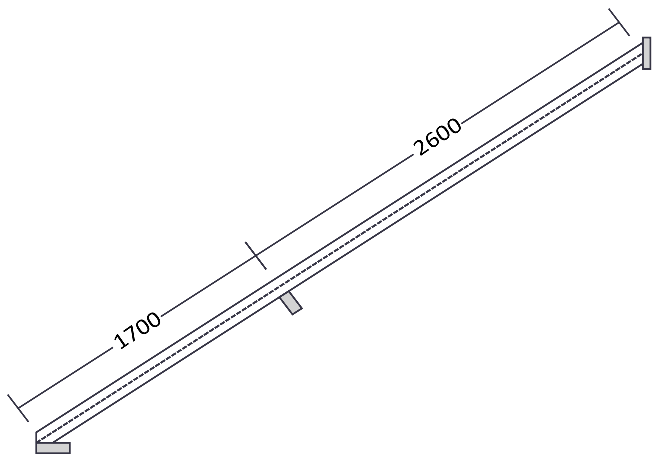
Tedds calculation version 1.0.09

Rafter details

Description 50 x 110 C16 timber rafters

Rafter spacing s_{Rafter} = **400** mm

Rafter inclination θ_{Rafter} = **35** deg



Forces input on Rafter

Permanent load on slope F_{G_Rafter}= **1.18** kN/m^2

Imposed load on plan F_{Q_Rafter}= **0.50** kN/m^2

Rafter loading details

Distributed loads

Permanent load on slope $p_G = F_{G\_Rafter} \times s_{Rafter}$= **0.47** kN/m

Imposed load on slope $p_Q = F_{Q\_Rafter} \times s_{Rafter} \times \cos(\theta_{Rafter})$= **0.16** kN/m

Member span 1 results summary	**Unit**	**Capacity**	**Maximum**	**Utilisation**	**Result**
Compressive stress	N/mm^2	11.5	0.4	0.035	PASS
Bearing stress	N/mm^2	1.5	0.3	0.229	PASS
Bending stress	N/mm^2	11.5	4.7	0.412	PASS
Shear stress	N/mm^2	2.2	0.4	0.171	PASS
Bending and axial force				0.413	PASS
Column stability check				0.457	PASS
Beam stability check				0.203	PASS
Deflection	mm	6.8	0.4	0.065	PASS

Member span 2 results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm^2	11.5	0.2	0.021	PASS
Bearing stress	N/mm^2	1.5	0.3	0.173	PASS
Bending stress	N/mm^2	11.5	4.7	0.412	PASS
Shear stress	N/mm^2	2.2	0.5	0.216	PASS
Bending and axial force				0.412	PASS
Column stability check				0.457	PASS
Beam stability check				0.190	PASS
Deflection	mm	10.4	7.4	0.716	PASS

ANALYSIS

Tedds calculation version 1.0.38

Loading

Self weight included (Permanent x 1)

Load combination factors

Load combination	Permanent	Imposed	Snow	Wind
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00

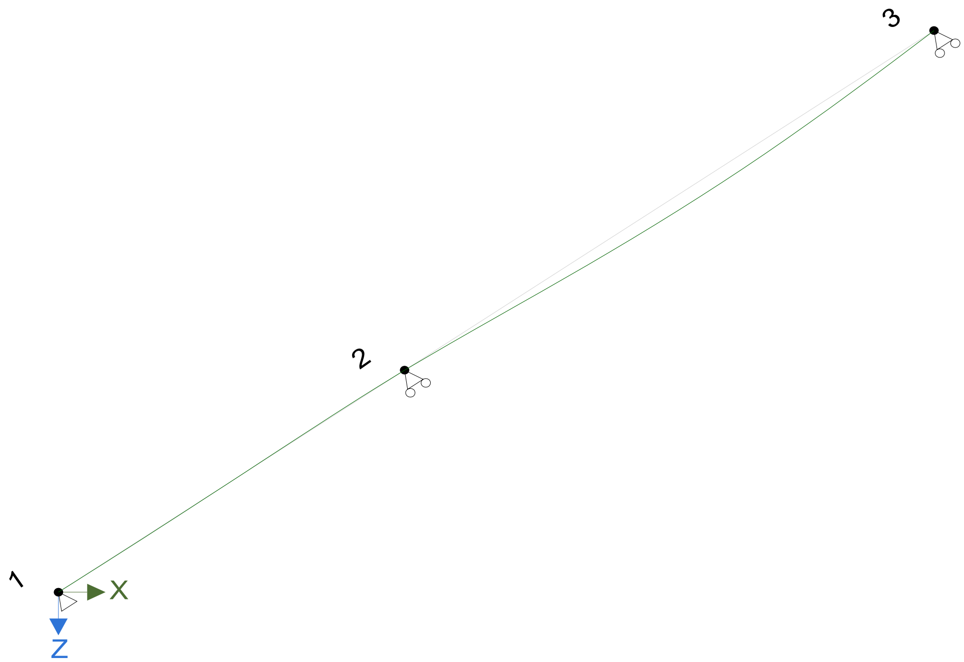
Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.47 kN/m at 0 m to 4.3 m
Member	Imposed	UDL	GlobalZ	0.16 kN/m at 0 m to 4.3 m

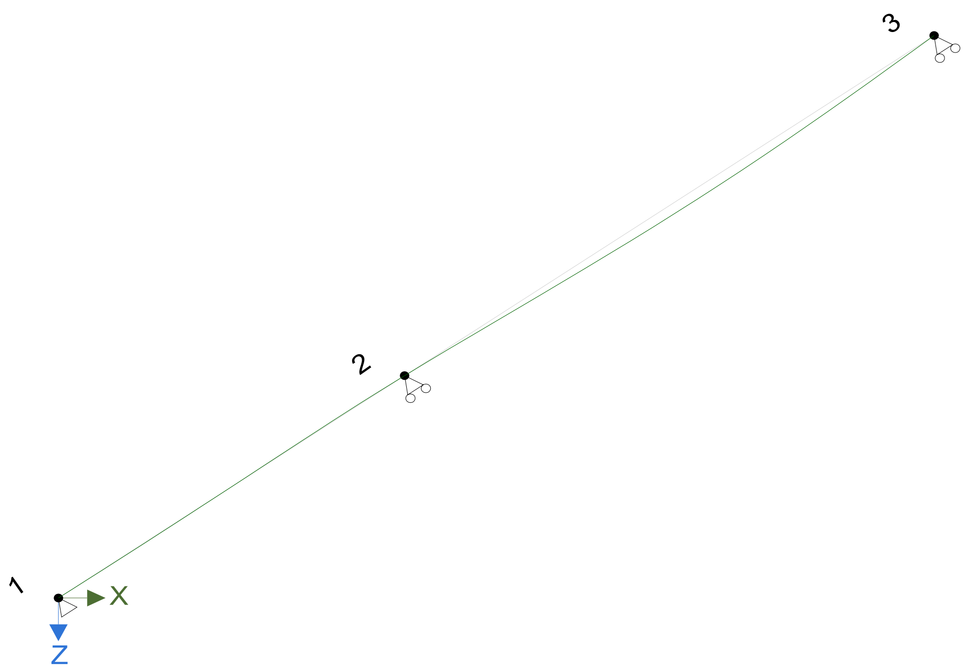
Results

Total deflection

1.35G + 1.50Q (Strength) - Total deflection



1.00G + 1.00Q (Service) - Total deflection



Node deflections

Load combination: 1.35G + 1.50Q (Strength)

Node	Deflection X (mm)	Deflection Z (mm)	Rotation (°)	Co-ordinate system
1	0	0	0.028	Member
2	-0.1	0	0.16121	Member
3	-0.1	0	-0.43827	Member

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection X (mm)	Deflection Z (mm)	Rotation (°)	Co-ordinate system
1	0	0	0.02018	Member
2	0	0	0.11617	Member
3	-0.1	0	-0.31583	Member

Total base reactions

Load case/combination	Force FX (kN)	Force FZ (kN)
1.35G + 1.50Q (Strength)	0	3.9
1.00G + 1.00Q (Service)	0	2.8

Element end forces

Load combination: 1.35G + 1.50Q (Strength)

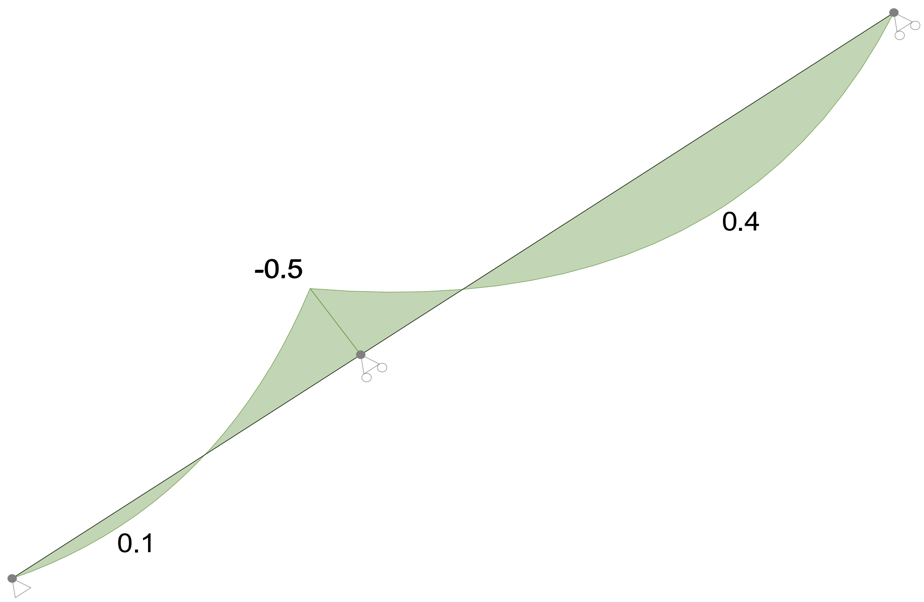
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	1.7	1	-2.2	-0.3	0
		2	1.4	-0.9	-0.5
2	2.6	2	-1.4	-1.1	0.5
		3	0	-0.8	0

Load combination: 1.00G + 1.00Q (Service)

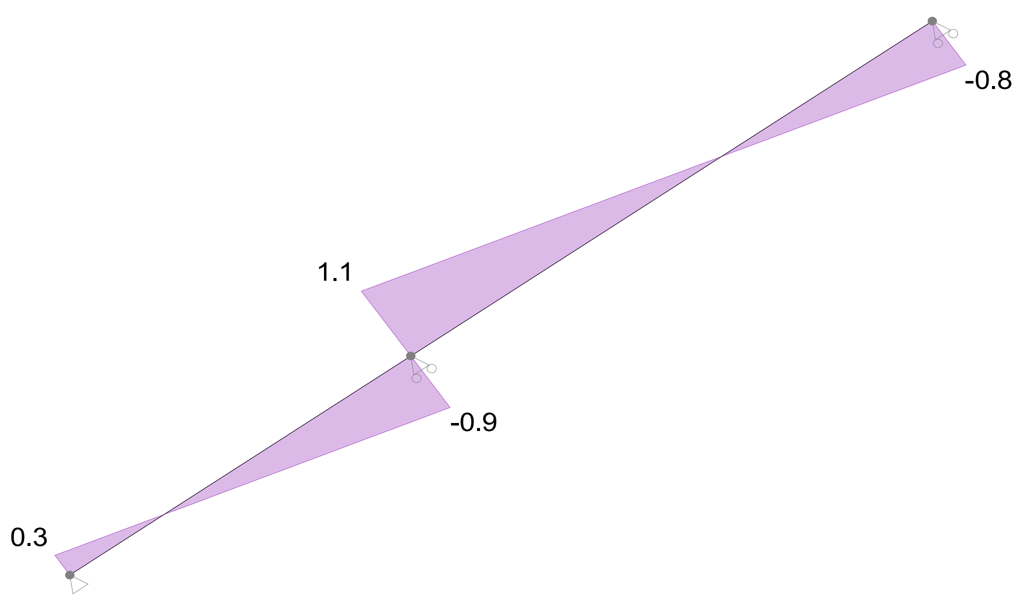
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	1.7	1	-1.6	-0.3	0
		2	1	-0.7	-0.3
2	2.6	2	-1	-0.8	0.3
		3	0	-0.6	0

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Member results

Envelope - Strength combinations

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Member	1.7	1.1 (max abs)	-0.9	-0.5 (min)	
	3.248	0		0.4 (max)	

Tedds calculation version 2.2.23

Member - 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 Medium-term

Service class - cl.2.3.1.3 2

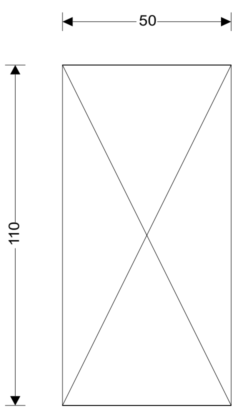
Timber section details

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

Breadth of sections b = **50** mm

Depth of sections h = **110** mm

Timber strength class - EN 338:2016 Table 1 **C16**



50x110 timber section
Cross-sectional area, A, 5500 mm^2
Section modulus, W_y, 100833.3 mm^3
Section modulus, W_z, 45833 mm^3
Second moment of area, I_y, 5545833 mm^4
Second moment of area, I_z, 1145833 mm^4
Radius of gyration, i_y, 31.8 mm
Radius of gyration, i_z, 14.4 mm
Timber strength class C16
Characteristic bending strength, $f_{m.k}$, 16 N/mm^2
Characteristic shear strength, $f_{v.k}$, 3.2 N/mm^2
Characteristic compression strength parallel to grain, $f_{c.0.k}$, 17 N/mm^2
Characteristic compression strength perpendicular to grain, $f_{c.90.k}$, 2.2 N/mm^2
Characteristic tension strength parallel to grain, $f_{t.0.k}$, 8.5 N/mm^2
Mean modulus of elasticity, $E_{0.mean}$, 8000 N/mm^2
Fifth percentile modulus of elasticity, $E_{0.05}$, 5400 N/mm^2
Shear modulus of elasticity, G_{mean}, 500 N/mm^2
Characteristic density, ρ_k, 310 kg/m^3
Mean density, ρ_{mean}, 370 kg/m^3

Span details

Bearing length L_b = **100** mm

Consider Combination 1 - 1.35G + 1.50Q (Strength)

Modification factors

Duration of load and moisture content - Table 3.1 k_{mod} = **0.8**

Deformation factor - Table 3.2 k_{def} = **0.8**

Depth factor for bending - Major axis - exp.3.1 $k_{h,m,y} = \min((150 \text{ mm} / h)^{0.2}, 1.3)$ = **1.064**

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**

System strength factor - cl.6.6 k_{sys} = **1.1**

Check compression parallel to the grain - cl.6.1.4

Design axial compression P_d = **2.233** kN

Design compressive stress $\sigma_{c,0,d} = P_d / A$ = **0.406** N/mm^2

Design compressive strength $f_{c,0,d} = k_{mod} \times k_{sys} \times f_{c.0.k} / \gamma_M$ = **11.508** N/mm^2

$\sigma_{c,0,d} / f_{c,0,d}$ = **0.035**

PASS - Design parallel compression strength exceeds design parallel compression stress

<u>Check design at start of span</u>

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d}$ = **2.218** kN

Effective contact length $L_{b,ef} = L_b + \min(L_b, 30\text{ mm})$ = **130** mm

Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef})$ = **0.341** N/mm^2

Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c.90.k} / \gamma_M$ = **1.489** N/mm^2

$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d})$ = **0.229**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d}$ = **0.349** kN

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.142** N/mm^2

Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **2.166** N/mm^2

$\tau_{y,d} / f_{v,y,d}$ = **0.066**

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 $L_{e,y} = 0.9 \times 1700$ mm = **1530** mm

Slenderness ratio $\lambda_y = L_{e,y} / i_y$ = **48.183**

Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})}$ = **0.861**

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 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2)$ = **0.926**

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

$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)})$ = **0.788**

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

Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d})$ = **0.045**

$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d})$ = **0.033**

PASS - Column stability is acceptable

<u>Check design at end of span</u>

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d}$ = **2.06** kN

Effective contact length $L_{b,ef} = \min(L_b, 30\text{ mm}) + L_b + \min(L_b, 30\text{ mm})$ = **160** mm

Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef})$ = **0.258** N/mm^2

Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c.90.k} / \gamma_M$ = **1.489** N/mm^2

$\sigma_{c,y,90,d}$ / ($k_{c,90}$ × $f_{c,y,90,d}$) = **0.173**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force	$F_{y,d}$ = **0.912** kN
Design shear stress - exp.6.60	$\tau_{y,d}$ = 1.5 × $F_{y,d}$ / (k_{cr} × b × h) = **0.371** N/mm^2
Design shear strength	$f_{v,y,d}$ = k_{mod} × k_{sys} × $f_{v.k}$ / γ_M = **2.166** N/mm^2
	$\tau_{y,d}$ / $f_{v,y,d}$ = **0.171**

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending moment	$M_{y,d}$ = **0.479** kNm
Design bending stress	$\sigma_{m,y,d}$ = $M_{y,d}$ / W_y = **4.747** N/mm^2
Design bending strength	$f_{m,y,d}$ = $k_{h,m,y}$ × k_{mod} × k_{sys} × $f_{m.k}$ / γ_M = **11.524** N/mm^2
	$\sigma_{m,y,d}$ / $f_{m,y,d}$ = **0.412**

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 + $\sigma_{m,y,d}$ / $f_{m,y,d}$ = **0.413**
	($\sigma_{c,0,d}$ / $f_{c,0,d}$)^2 + k_m × $\sigma_{m,y,d}$ / $f_{m,y,d}$ = **0.290**

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 × 1700 mm = **1530** mm
Slenderness ratio	λ_y = $L_{e,y}$ / i_y = **48.183**
Relative slenderness ratio - exp. 6.21	$\lambda_{rel,y}$ = λ_y / π × $\sqrt{}$($f_{c.0.k}$ / $E_{0.05}$) = **0.861**
Effective length for z-axis bending	$L_{e,z}$ = **0** mm
Slenderness ratio	λ_z = $L_{e,z}$ / i_z = **0**
Relative slenderness ratio - exp. 6.22	$\lambda_{rel,z}$ = λ_z / π × $\sqrt{}$($f_{c.0.k}$ / $E_{0.05}$) = **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 × (1 + β_c × ($\lambda_{rel,y}$ - 0.3) + $\lambda_{rel,y}^2$) = **0.926**
	k_z = 0.5 × (1 + β_c × ($\lambda_{rel,z}$ - 0.3) + $\lambda_{rel,z}^2$) = **0.470**
	$k_{c,y}$ = 1 / (k_y + $\sqrt{}$(k_y^2 - $\lambda_{rel,y}^2$)) = **0.788**
	$k_{c,z}$ = 1 / (k_z + $\sqrt{}$(k_z^2 - $\lambda_{rel,z}^2$)) = **1.064**
Column stability checks - exp.6.23 & 6.24	$\sigma_{c,0,d}$ / ($k_{c,y}$ × $f_{c,0,d}$) + $\sigma_{m,y,d}$ / $f_{m,y,d}$ = **0.457**
	$\sigma_{c,0,d}$ / ($k_{c,z}$ × $f_{c,0,d}$) + k_m × $\sigma_{m,y,d}$ / $f_{m,y,d}$ = **0.322**

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} × $f_{m,y,d}$))^2 + $\sigma_{c,0,d}$ / ($k_{c,z}$ × $f_{c,0,d}$) = **0.203**

PASS - Beam stability is acceptable

Consider Combination 2 - 1.00G + 1.00Q (Service)

Check design 1374 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection	δ_y = **0.2** mm

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

Final deflection with creep $\delta_{y,Final} = \delta_y \times (1 + k_{def})$ = **0.4** mm

Allowable deflection $\delta_{y,Allowable} = L_{m1\_s1} / 250$ = **6.8** mm

$\delta_{y,Final} / \delta_{y,Allowable}$ = **0.065**

PASS - Allowable deflection exceeds final deflection

Member - Span 2

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 Medium-term

Service class - cl.2.3.1.3 2

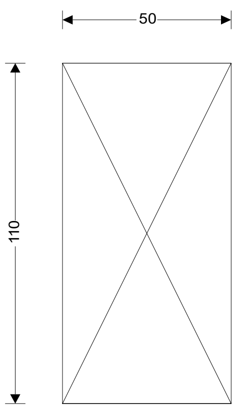
Timber section details

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

Breadth of sections b = **50** mm

Depth of sections h = **110** mm

Timber strength class - EN 338:2016 Table 1 **C16**



50x110 timber section
Cross-sectional area, A, 5500 mm^2
Section modulus, W_y, 100833.3 mm^3
Section modulus, W_z, 45833 mm^3
Second moment of area, I_y, 5545833 mm^4
Second moment of area, I_z, 1145833 mm^4
Radius of gyration, i_y, 31.8 mm
Radius of gyration, i_z, 14.4 mm
Timber strength class C16
Characteristic bending strength, $f_{m.k}$, 16 N/mm^2
Characteristic shear strength, $f_{v.k}$, 3.2 N/mm^2
Characteristic compression strength parallel to grain, $f_{c.0.k}$, 17 N/mm^2
Characteristic compression strength perpendicular to grain, $f_{c.90.k}$, 2.2 N/mm^2
Characteristic tension strength parallel to grain, $f_{t.0.k}$, 8.5 N/mm^2
Mean modulus of elasticity, $E_{0.mean}$, 8000 N/mm^2
Fifth percentile modulus of elasticity, $E_{0.05}$, 5400 N/mm^2
Shear modulus of elasticity, G_{mean}, 500 N/mm^2
Characteristic density, ρ_k, 310 kg/m^3
Mean density, ρ_{mean}, 370 kg/m^3

Span details

Bearing length L_b = **100** mm

Consider Combination 1 - 1.35G + 1.50Q (Strength)

Modification factors

Duration of load and moisture content - Table 3.1 k_{mod} = **0.8**

Deformation factor - Table 3.2 k_{def} = **0.8**

Depth factor for bending - Major axis - exp.3.1 $k_{h,m,y} = \min((150 \text{ mm} / h)^{0.2}, 1.3)$ = **1.064**

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**

System strength factor - cl.6.6 k_{sys} = **1.1**

Check compression parallel to the grain - cl.6.1.4

Design axial compression P_d = **1.35** kN

Design compressive stress $\sigma_{c,0,d} = P_d / A$ = **0.246** N/mm^2

Design compressive strength $f_{c,0,d} = k_{mod} \times k_{sys} \times f_{c.0.k} / \gamma_M$ = **11.508** N/mm^2

$\sigma_{c,0,d} / f_{c,0,d}$ = **0.021**

PASS - Design parallel compression strength exceeds design parallel compression stress

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d}$ = **2.06** kN

Effective contact length $L_{b,ef} = \min(L_b, 30\ mm) + L_b + \min(L_b, 30\ mm)$ = **160** mm

Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef})$ = **0.258** N/mm^2

Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c.90.k} / \gamma_M$ = **1.489** N/mm^2

$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d})$ = **0.173**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d}$ = **1.148** kN

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.467** N/mm^2

Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **2.166** N/mm^2

$\tau_{y,d} / f_{v,y,d}$ = **0.216**

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d}$ = **0.479** kNm

Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y$ = **4.747** N/mm^2

Design bending strength $f_{m,y,d} = k_{h,m,y} \times k_{mod} \times k_{sys} \times f_{m.k} / \gamma_M$ = **11.524** N/mm^2

$\sigma_{m,y,d} / f_{m,y,d}$ = **0.412**

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 $(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d}$ = **0.412**

$(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \times \sigma_{m,y,d} / f_{m,y,d}$ = **0.289**

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 $L_{e,y} = 0.9 \times 2600$ mm = **2340** mm

Slenderness ratio $\lambda_y = L_{e,y} / i_y$ = **73.691**

Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})}$ = **1.316**

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 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2)$ = **1.468**

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

$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)})$ = **0.472**

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

Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d}$ = **0.457**

$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) + k_m \times \sigma_{m,y,d} / f_{m,y,d}$ = **0.308**

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 $(\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d}))^2 + \sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d})$ = **0.19**

PASS - Beam stability is acceptable

Consider Combination 2 - 1.00G + 1.00Q (Service)

Check design 1426 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection δ_y = **4.1** mm

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

Final deflection with creep $\delta_{y,Final} = \delta_y \times (1 + k_{def})$ = **7.4** mm

Allowable deflection $\delta_{y,Allowable} = L_{m1\_s2} / 250$ = **10.4** mm

$\delta_{y,Final} / \delta_{y,Allowable}$ = **0.716**

PASS - Allowable deflection exceeds final deflection

Flat Roof Loads		
Item	**Unfactored Load (KN/m^2)**	**Load Type**
Chippings and Felt	0.3	Dead
Board, Joists & Firrings	0.3	Dead
Ceiling & Insulation	0.25	Dead
ROOF DEAD LOAD ON PLAN	**0.85 (KN/m2)**	
ROOF LIVE LOAD ON PLAN	**0.75 (KN/m2)**	

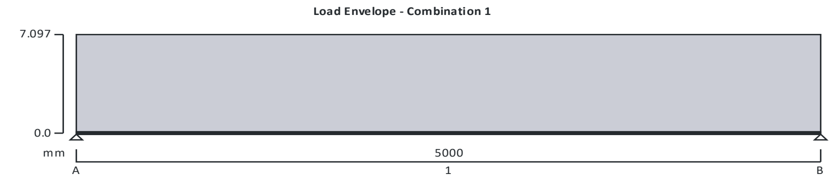
Pitched Roof Loads		
Item	**Unfactored Load (KN/m^2)**	**Load Type**
Roof Tiles	0.6	Dead
Rafters	0.15	Dead
Felt & Battens	0.1	Dead
Insulation	0.1	Dead
Plasterboard	0.15	Dead
ROOF DEAD LOAD ON PLAN	**1.36 (KN/m2)**	
ROOF LIVE LOAD ON PLAN	**0.5 (KN/m2)**	

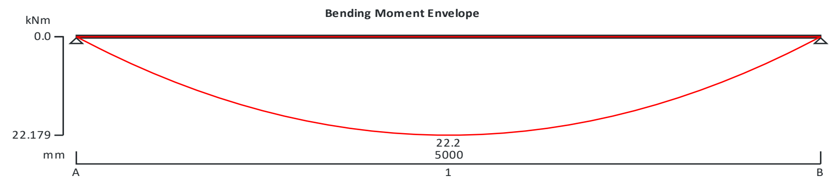
Proposed UDLs				
Item	**Span (m)**	**Dead Load (KN/m^2)**	**Live Load (KN/m^2)**	**Comments**
Flat Roof	1.7	0.85	0.75	See Load Table For Flat Roof
Pitched Roof	1	1.36	0.5	See Load Table For Pitched Roof
		Total Dead	**2.81 (KN/m)**	
		Total Live	**1.78 (KN/m)**	

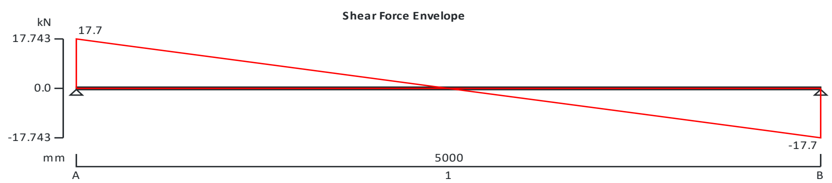
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.08







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 2.81 kN/m
	Imposed full UDL 1.78 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} = **22.2** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **17.7** kN	V_{min} = **-17.7** kN

Deflection	δ_{max} = **15.3** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **17.7** kN	R_{A_min} = **17.7** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **7.6** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **4.5** kN	
Maximum reaction at support B	R_{B_max} = **17.7** kN	R_{B_min} = **17.7** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **7.6** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **4.5** kN	

Section details

Section type — **UC 152x152x23 (British Steel Section Range 2022 (BS4-1))**

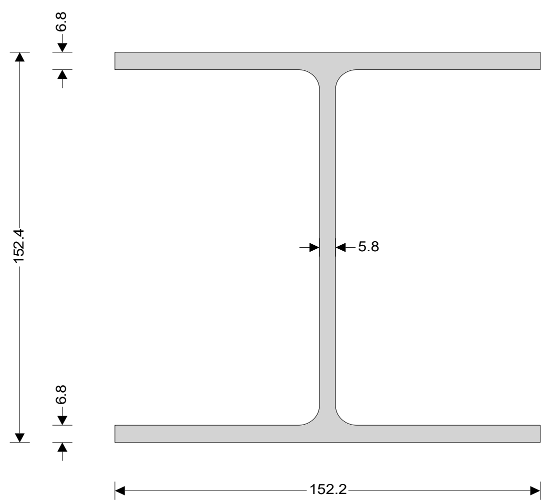
Steel grade — **S275**

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** + 2 × D

$K_{LT.B}$ = **1.00** + 2 × D

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section — d = **123.6** mm

$d / t = 21.3 \times \varepsilon <= 80 \times \varepsilon$ — Class 1 plastic

Outstand flanges - Table 11

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

$b / T = 11.2 \times \varepsilon <= 15 \times \varepsilon$ Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **17.7** kN

$d / t < 70 \times \varepsilon$

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 $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **22.2** 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 $L_E = 1.0 \times L_{s1} + 2 \times D$ = **5305** mm

Slenderness ratio $\lambda = L_E / r_{yy}$ = **143.454**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter u = **0.840**

Torsional index x = **20.701**

Slenderness factor $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25}$ = **0.736**

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

Equivalent slenderness - cl.4.3.6.7 $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **87.277**

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 $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.371**

Euler stress $p_E = \pi^2 \times E / \lambda_{LT}^2$ = **265.6** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2$ = **319.5** N/mm^2

Bending strength - Annex B.2.1 $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5})$ = **149.1** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment M_2 = **16.6** kNm

Moment at centre-line of segment M_3 = **22.2** kNm

Moment at three quarter point of segment M_4 = **16.6** kNm

Maximum moment in segment M_{abs} = **22.2** kNm

Maximum moment governing buckling resistance $M_{LT} = M_{abs}$ = **22.2** 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 $M_b = p_b \times S_{eff} = \mathbf{26.3}$ kNm

$M_b / m_{LT} = \mathbf{28.4}$ 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 $\delta_{lim} = L_{s1} / 250 = \mathbf{20}$ mm

Maximum deflection span 1 $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = \mathbf{15.294}$ mm

PASS - Maximum deflection does not exceed deflection limit

Pitched Roof Loads		
Item	**Unfactored Load (KN/m^2)**	**Load Type**
Roof Tiles	0.6	Dead
Rafters	0.15	Dead
Felt & Battens	0.1	Dead
Insulation	0.1	Dead
Plasterboard	0.15	Dead
ROOF DEAD LOAD ON PLAN	**1.36 (KN/m2)**	
ROOF LIVE LOAD ON PLAN	**0.5 (KN/m2)**	

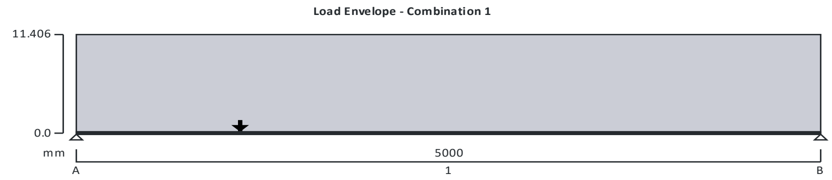
Timber Joists - Internal Floor Loads		
Item	**Unfactored Load (KN/m^2)**	**Load Type**
Floor joists	0.15	Dead
Floor boards	0.05	Dead
Plaster board and Skim	0.25	Dead
Finish	0.05	Dead
Residential Live Loading	1.5	Live
TOTAL FLOOR DEAD LOAD	**0.5 (KN/m2)**	
TOTAL FLOOR LIVE LOAD	**1.5 (KN/m2)**	

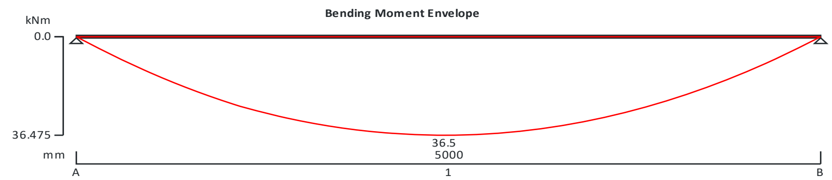
Proposed UDLs				
Item	**Span (m)**	**Dead Load (KN/m^2)**	**Live Load (KN/m^2)**	**Comments**
Pitched Roof	1.7	1.36	0.5	See Load Table For Pitched Roof
Loft Floor Loading	1.45	0.5	1.5	See Load Table For Internal Floor
Timber Walls	1.35	1	0	1 KN/m2 assumed for timber construction
T2 Reaction	-	-	-	1.5 KN @ 1100mm
		Total Dead	**4.39 (KN/m)**	
		Total Live	**3.03 (KN/m)**	

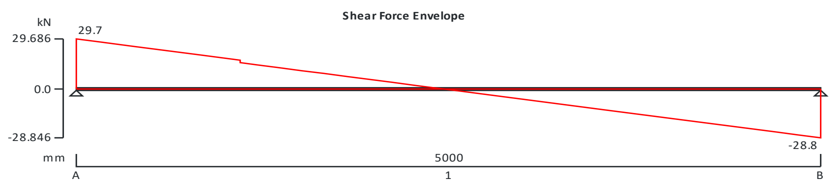
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.08







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 4.39 kN/m
	Imposed full UDL 3.03 kN/m
	Other point load 1.5 kN at 1100 mm

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Other × 1.00
		Dead × 1.40
		Imposed × 1.60
		Other × 1.00
	Support B	Dead × 1.40
		Imposed × 1.60

Other × 1.00

Analysis results

Maximum moment	M_{max} = **36.5** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **29.7** kN	V_{min} = **-28.8** kN
Deflection	δ_{max} = **18.2** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **29.7** kN	R_{A_min} = **29.7** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **11.7** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **7.6** kN	
Unfactored other load reaction at support A	R_{A_Other} = **1.2** kN	
Maximum reaction at support B	R_{B_max} = **28.8** kN	R_{B_min} = **28.8** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **11.7** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **7.6** kN	
Unfactored other load reaction at support B	R_{B_Other} = **0.3** kN	

Section details

Section type **UC 152x152x30 (British Steel Section Range 2022 (BS4-1))**

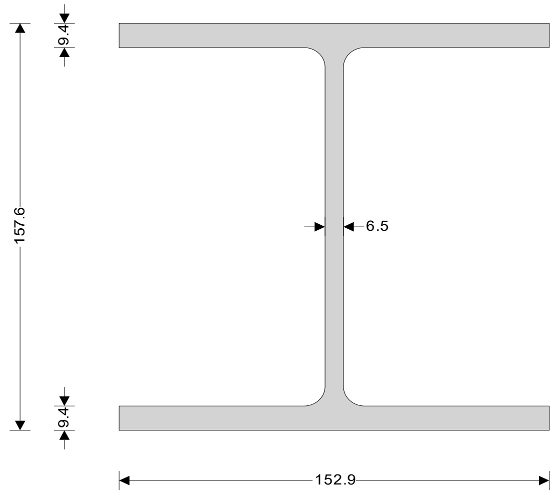
Steel grade **S275**

From table 9: Design strength p_y

Thickness of element max(T, t) = **9.4** mm

Design strength p_y = **275** N/mm²

Modulus of elasticity E = **205000** N/mm²



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** + 2 × D
	$K_{LT.B}$ = **1.00** + 2 × D

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section — $d = \mathbf{123.6}$ 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 — $F_v = \max(\mathrm{abs}(V_{max}), \mathrm{abs}(V_{min})) = \mathbf{29.7}$ 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 — $M = \max(\mathrm{abs}(M_{s1\_max}), \mathrm{abs}(M_{s1\_min})) = \mathbf{36.5}$ kNm

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

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling — $L_E = 1.0 \times L_{s1} + 2 \times D = \mathbf{5315}$ mm

Slenderness ratio — $\lambda = L_E / r_{yy} = \mathbf{138.879}$

Equivalent slenderness - Section 4.3.6.7

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

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

Slenderness factor — $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = \mathbf{0.677}$

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

Equivalent slenderness - cl.4.3.6.7 — $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} = \mathbf{79.748}$

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 — $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = \mathbf{0.318}$

Euler stress — $p_E = \pi^2 \times E / \lambda_{LT}^2 = \mathbf{318.1}$ N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \mathbf{347.2}$ N/mm^2

Bending strength - Annex B.2.1 — $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = \mathbf{165.4}$ N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment — $M_2 = \mathbf{28}$ kNm

Moment at centre-line of segment — $M_3 = \mathbf{36.5}$ kNm

Moment at three quarter point of segment — $M_4 = \mathbf{27.1}$ kNm

Maximum moment in segment — $M_{abs} = \mathbf{36.5}$ kNm

Maximum moment governing buckling resistance — $M_{LT} = M_{abs} = \mathbf{36.5}$ kNm

Equivalent uniform moment factor for lateral-torsional buckling

$$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = \mathbf{0.927}$$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment $M_b = p_b \times S_{xx} =$ **41** kNm

$M_b / m_{LT} =$ **44.2** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead, imposed and other loads

Limiting deflection $\delta_{lim} = L_{s1} / 250 =$ **20** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **18.205** mm

PASS - Maximum deflection does not exceed deflection limit

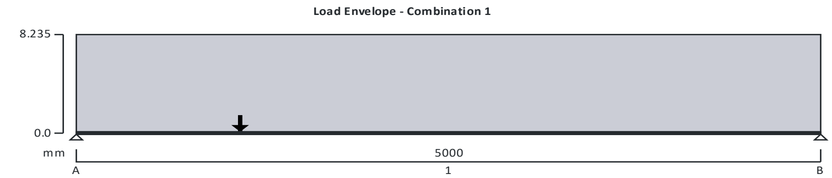
Timber Joists - Internal Floor Loads		
Item	**Unfactored Load (KN/m^2)**	**Load Type**
Floor joists	0.15	Dead
Floor boards	0.05	Dead
Plaster board and Skim	0.25	Dead
Finish	0.05	Dead
Residential Live Loading	1.5	Live
TOTAL FLOOR DEAD LOAD	**0.5 (KN/m2)**	
TOTAL FLOOR LIVE LOAD	**1.5 (KN/m2)**	

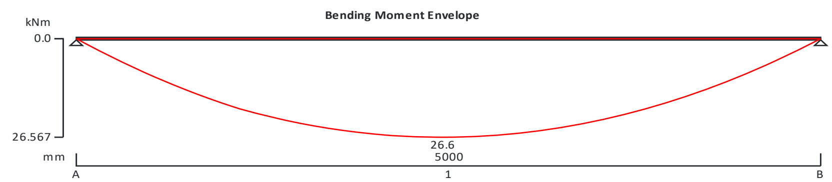
Proposed UDLs				
Item	**Span (m)**	**Dead Load (KN/m^2)**	**Live Load (KN/m^2)**	**Comments**
Loft Floor Loading	2.55	0.5	1.5	See Load Table For Internal Floor
T2 Reaction	-	-	-	1.5 KN @ 1100mm
		Total Dead	**1.28 (KN/m)**	
		Total Live	**3.83 (KN/m)**	

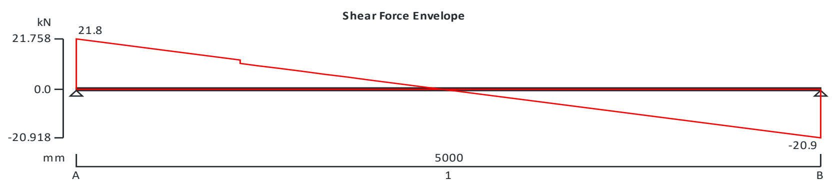
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.08







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 1.28 kN/m
	Imposed full UDL 3.83 kN/m
	Other point load 1.5 kN at 1100 mm

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Other × 1.00
		Dead × 1.40
		Imposed × 1.60
		Other × 1.00
	Support B	Dead × 1.40
		Imposed × 1.60

Other × 1.00

Analysis results

Maximum moment	M_{max} = **26.6** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **21.8** kN	V_{min} = **-20.9** kN
Deflection	δ_{max} = **17.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **21.8** kN	R_{A_min} = **21.8** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **3.8** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **9.6** kN	
Unfactored other load reaction at support A	R_{A_Other} = **1.2** kN	
Maximum reaction at support B	R_{B_max} = **20.9** kN	R_{B_min} = **20.9** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **3.8** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **9.6** kN	
Unfactored other load reaction at support B	R_{B_Other} = **0.3** kN	

Section details

Section type **UC 152x152x23 (British Steel Section Range 2022 (BS4-1))**

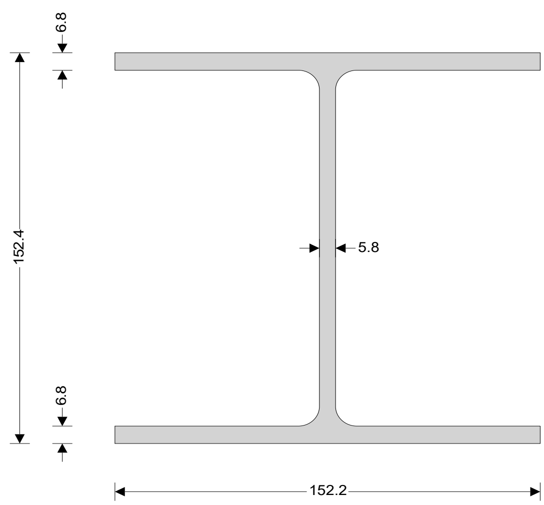
Steel grade **S275**

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** + 2 × D

$K_{LT.B}$ = **1.00** + 2 × D

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section — d = **123.6** mm

$d / t = 21.3 \times \varepsilon <= 80 \times \varepsilon$ — Class 1 plastic

Outstand flanges - Table 11

Width of section — $b = B / 2$ = **76.1** mm

$b / T = 11.2 \times \varepsilon <= 15 \times \varepsilon$ — Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force — $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **21.8** kN

$d / t < 70 \times \varepsilon$

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 — $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **26.6** 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 — $L_E = 1.0 \times L_{s1} + 2 \times D$ = **5305** mm

Slenderness ratio — $\lambda = L_E / r_{yy}$ = **143.454**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter — u = **0.840**

Torsional index — x = **20.701**

Slenderness factor — $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25}$ = **0.736**

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

Equivalent slenderness - cl.4.3.6.7 — $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **87.277**

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 — $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.371**

Euler stress — $p_E = \pi^2 \times E / \lambda_{LT}^2$ = **265.6** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2$ = **319.5** N/mm^2

Bending strength - Annex B.2.1 — $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5})$ = **149.1** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment M_2 = **20.5** kNm

Moment at centre-line of segment M_3 = **26.6** kNm

Moment at three quarter point of segment M_4 = **19.7** kNm

Maximum moment in segment M_{abs} = **26.6** kNm

Maximum moment governing buckling resistance $M_{LT} = M_{abs}$ = **26.6** kNm

Equivalent uniform moment factor for lateral-torsional buckling

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44)$ = **0.927**

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment $M_b = p_b \times S_{eff}$ = **26.3** kNm

M_b / m_{LT} = **28.3** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead, imposed and other loads

Limiting deflection $\delta_{lim} = L_{s1} / 250$ = **20** mm

Maximum deflection span 1 $\delta = \max(abs(\delta_{max}), abs(\delta_{min}))$ = **17.905** mm

PASS - Maximum deflection does not exceed deflection limit