Project: THE BUG FARM, ST DAVID'S NATURE RECOVERY CENTRE

Client: Dr S Beynon

Project No: 0208

Date: January 2024

SHEARWATER
Consulting Engineers Ltd

Contents:

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Design Approach.

An existing lean to shed is to be adapted for use as a Nature Recovery Centre. The new building will be accessible to the public.

The building roof comprises profiled steel sheets over 'I' beams that are supported on the adjacent stone wall and on columns to the outer wall.

The external walls comprise profiled steel sheet cladding on timber sheeting rails on steel columns.

Evidence of corrosion of the column bases was noted on site.

The building is braced by virtue of it's connection to the adjacent stone farm buildings. Further bracing will be provided by the ply lined internal stud wall to the perimeter walls.

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	Calcs for Nature Recovery Centre			Page No. 2	
	Calcs by RML	Date 1/24	Checked by RML	Date 1/24	Approved by RML	Date 1/24

<u>Summary.</u>

Purlins	-	Replace with 200 x 100 grade C24 at current positions.
Rafter check	-	Existing 178 x 102 UB 19 members are adequate. Provide bottom flange restraints at mid span. Provide 1.5m uplift restraint straps to rafter end.
Sheeting rails	-	Replace with 200 x 100 grade C24 at current positions.
Column check	-	Existing column adequate subject to repairing any corrosion.
Foundation check	-	Required foundation area = 0.2m2 per column. Review on site.

Loadings.

Timber purlins.

The existing purlins are 150 x 75mm. They are located at approximately 1.5m cts.

Span of timber purlins = 5.17m

Proposed new cladding to be an insulated panel.

Weight of cladding = 0.12 KN/m2

Dead load cladding on purlin = 0.12 x 1.5 = 0.2 KN/m

Live load on roof = 0.6 x 1.5 = 0.9 KN/m

Wind loads on building.

Building size = 20m x 13m

Height to eaves = 2.7m

Roof slope = 5 degrees.

Distance to sea = 4 km

Site altitude = 60m

Wind uplift on roof = -0.74 KN/m2 Case 1 zone C.

Wind suction on purlin = -0.74 x 1.5 = -1.11 KN/m2

Wind suction on walls = -0.75 KN/m2

Analysis of the existing purlins with the proposed loads shows that they are not adequate to support the applied loads (this has been checked for both C18 and C24 timber.)

Consider larger purlins.

Use 200 x 100 grade C24 purlins at 1.5m cts as existing. Refer to calculations.

Steel rafter analysis.

Span of rafters = 6.1m

The existing rafter members are 178 x 102 UB 19

Loaded width = 5.17m

Dead loads on beam				
Cladding	=	0.12 x 5.17	=	0.62 KN/m
Purlins	=	5.17 x 0.15 x 0.075 x 6 / 1.5	=	0.23 KN/m
Total dead load			=	0.85 KN/m

Live load on beam = 5.17 x 0.6 = 3.1 KN/m

Wind uplift on rafter = -0.74 x 5.17 = -3.83 KN/m

Analysis shows that the rafters are adequate to support the applied roof loads (Dead + Live load with full lateral restraint.)

The existing rafter is not adequate to support the wind uplift loads without lateral restraint. Providing lateral restraint to the bottom flange at mid span will increase the member capacity to a suitable level – refer to calculations.

Consider wind uplift at rafter support.

The rafter sits on the existing stone building wall.

There do not appear to be any uplift restraint straps at present and the roof has not moved.

Wind uplift load at support	=	-0.74 x 5.17 x 6.1 x 0.5	=	-11.7 KN
Weight of roof	=	0.85 x 6.1 x 0.5	=	2.6 KN
New uplift	=	(0.9 x 2.6) + (-11.7 x 1.4)	=	-14 KN
Weight of wall	=	0.9 x 0.45 x 20	=	8.1 KN/m2
Area to be mobilised	=	14 / 8.1	=	1.73m2

Provide 1.5m long uplift restraint straps to the steel rafter ends bolted to the stone wall.

<u>Sheeting rail check.</u>

Span of sheeting rails = 5.17m

Existing rails are 150 x 75 timber at about 1.5m cts

Lateral load from wind = -0.75 x 1.5 = 1.13 KN/m

The existing sheeting rails show signs of woodworm infestation on site. Analysis shows that they will be subject to excessive deflections. It is therefore proposed that the sheeting rails are replaced with new timbers.

Consider 200 x 100 grade C24 sheeting rails with notches top and bottom.

<u>Column check.</u>

Column length = 3.0m

Axial load from roof
Dead load = 3.2 KN
Live load = 9.5 KN
Total factored load = 19.6 KN

UDL on column (wind load bending) = -0.75 x 5.17 = 3.9 KN/m

Factored moment in column = $3.9 \times 1.4 \times 3.0^2 / 8$ = 6.15 KNm

Factored shear in post = 3.9 x 1.4 x 3.0 / 2 = 8.2 KN

<u>Foundation check.</u>

The vertical load from the column is 19.6 KN

Assuming a safe bearing pressure of 100 KN/m2

Required bearing area = 19.6 /100 = 0.2 m2

The nature of the existing foundations shall be reviewed on site.

Bracing to the existing building.

The building is a lean to structure to the older stone barn complex. The barn is considered adequate to resist wind loads on the lean to from the long north elevation.

Bracing should be provided to the narrow building end to both the roof and wall.

The walls will be braced by the proposed timber stud wall with plywood faces.

Purlins will transfer loads to the existing stone walls of adjacent buildings.

Wind post to gable / Door trimmer.

Post length = 3.0m

Loaded width = 1.5m

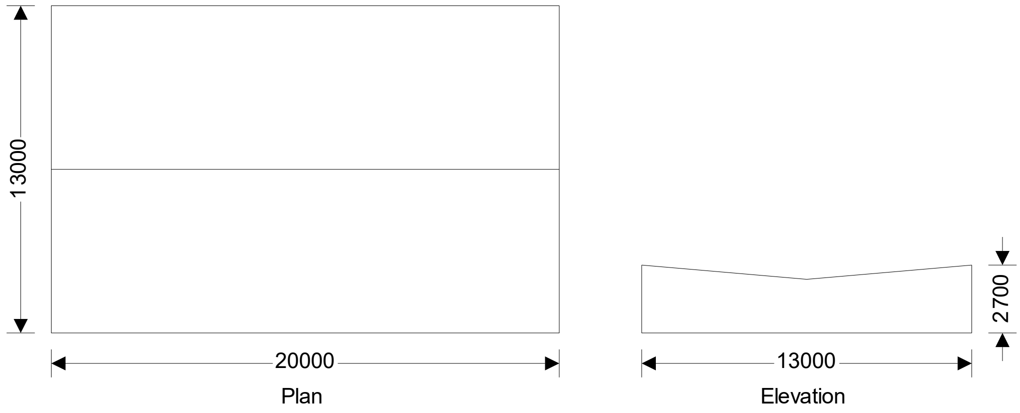
Wind load = 0.75 KN/m2

Load on post = 0.75 x 1.5 = 1.15 KN/m

WIND LOADING (BS6399)

In accordance with BS6399

Tedds calculation version 3.0.18



Building data

Type of roof	Duopitch
Length of building	L = **20000** mm
Width of building	W = **13000** mm
Height to eaves	H = **2700** mm
Pitch of roof	α_0 = **5.0** deg
Reference height	H_r = **2700** mm

Dynamic classification

Building type factor (Table 1)	K_b = **2.0**
Dynamic augmentation factor (1.6.1)	$C_r = [K_b \times (H_r / (0.1\ m))^{0.75}] / (800 \times \log(H_r / (0.1\ m))) = \mathbf{0.02}$

Site wind speed

Location	St David's
Basic wind speed (Figure 6 BS6399:Pt 2)	V_b = **24.0** m/s
Site altitude	Δ_S = **60** m
Upwind distance from sea to site	d_{sea} = **4** km
Direction factor	S_d =**1.00**
Seasonal factor	S_s = **1.00**
Probability factor	S_p = **1.00**
Critical gap between buildings	g = **5000** mm
Topography not significant	
Altitude factor	$S_a = 1 + 0.001 \times \Delta_S / 1m = \mathbf{1.06}$
Site wind speed	$V_s = V_b \times S_a \times S_d \times S_s \times S_p = \mathbf{25.4}$ m/s
Terrain category	Country
Displacement height (sheltering effect excluded)	H_d = 0mm

The velocity pressure for the windward face of the building with a 0 degree wind is to be considered as 1 part as the height h is less than b (cl.2.2.3.2)

The velocity pressure for the windward face of the building with a 90 degree wind is to be considered as 1 part as the height h is less than b (cl.2.2.3.2)

Dynamic pressure - windward wall - Wind 0 deg and roof

Reference height (at which q is sought)	H_{ref} = **2700**mm

Effective height $H_e = \max(H_{ref} - H_d, 0.4 \times H_{ref})$ = **2700**mm
Fetch factor (Table 22) S_c = **0.830**
Turbulence factor (Table 22) S_t = **0.210**
Gust peak factor g_t = **3.44**
Terrain and building factor $S_b = S_c \times (1 + (g_t \times S_t) + S_h)$ = **1.43**
Effective wind speed $V_e = V_s \times S_b$ = **36.3** m/s
Dynamic pressure $q_s = 0.613\ kg/m^3 \times V_e^2$ = **0.809** kN/m²

Dynamic pressure - windward wall - Wind 90 deg and roof

Reference height (at which q is sought) H_{ref} = **2700**mm
Effective height $H_e = \max(H_{ref} - H_d, 0.4 \times H_{ref})$ = **2700**mm
Fetch factor (Table 22) S_c = **0.830**
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Size effect factors

Diagonal dimension for gablewall a_{eg} = **13.3** m
External size effect factor gablewall C_{aeg} = **0.926**
Diagonal dimension for side wall a_{es} = **20.2** m
External size effect factor side wall C_{aes} = **0.895**
Diagonal dimension for roof a_{er} = **21.0** m
External size effect factor roof C_{aer} = **0.892**
Room/storey volume for internal size effect factor V_i = **0.125** m³
Diagonal dimension for internal size effect factors $a_i = 10 \times (V_i)^{1/3}$ = **5.000** m
Internal size effect factor C_{ai} = **1.000**

Pressures and forces

Net pressure $p = q_s \times c_{pe} \times C_{ae} - q_s \times c_{pi} \times C_{ai}$
Net force $F_w = p \times A_{ref}$

Roof load case 1 - Wind 0, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m²)	External size factor, C_{ae}	Net Pressure, p (kN/m²)	Area, A_{ref} (m²)	Net force, F_w (kN)
A (-ve)	-2.40	0.81	0.892	-1.89	2.93	-5.54
B (-ve)	-1.20	0.81	0.892	-1.03	7.91	-8.13
C (-ve)	-0.80	0.81	0.892	-0.74	119.66	-88.45
E (-ve)	-0.50	0.81	0.892	-0.52	2.93	-1.53
F (-ve)	-0.30	0.81	0.892	-0.38	7.91	-2.99
G (-ve)	-0.50	0.81	0.892	-0.52	119.66	-62.54

Total vertical net force $F_{w,v}$ = **-168.55** kN
Total horizontal net force $F_{w,h}$ = **-3.06** kN

Walls load case 1 - Wind 0, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A	-1.34	0.81	0.926	-1.17	2.86	-3.35
B	-0.81	0.81	0.926	-0.77	10.44	-8.07
C	-0.56	0.81	0.926	-0.58	16.11	-9.36
w	0.60	0.81	0.895	0.27	54.00	14.72
l	-0.50	0.81	0.895	-0.52	54.00	-28.29

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl} = \mathbf{-28.3}$ kN

Net windward force for overall section $F_w = F_{w,ww} = \mathbf{14.7}$ kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h}) = \mathbf{34.7}$ kN

Roof load case 2 - Wind 90, c_{pi} 0.20, -c_{pe}

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-2.20	0.81	0.892	-1.75	3.52	-6.16
B (-ve)	-1.50	0.81	0.892	-1.24	3.52	-4.38
C (-ve)	-0.70	0.81	0.892	-0.67	28.19	-18.80
D (-ve)	-0.70	0.81	0.892	-0.67	225.76	-150.58

Total vertical net force $F_{w,v} = \mathbf{-179.25}$ kN

Total horizontal net force $F_{w,h} = \mathbf{0.00}$ kN

Walls load case 2 - Wind 90, c_{pi} 0.20, -c_{pe}

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A	-1.34	0.81	0.895	-1.14	2.92	-3.31
B	-0.81	0.81	0.895	-0.75	11.66	-8.77
C	-0.56	0.81	0.895	-0.57	39.42	-22.35
w	0.60	0.81	0.926	0.29	31.40	9.04
l	-0.50	0.81	0.926	-0.54	31.40	-16.86

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl} = \mathbf{-16.9}$ kN

Net windward force for overall section $F_w = F_{w,ww} = \mathbf{9.0}$ kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h}) = \mathbf{22.5}$ kN

Roof load case 3 - Wind 90, c_{pi} -0.3, -c_{pe}

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-2.20	0.81	0.892	-1.34	3.52	-4.74
B (-ve)	-1.50	0.81	0.892	-0.84	3.52	-2.96
C (-ve)	-0.70	0.81	0.892	-0.26	28.19	-7.39
D (-ve)	-0.70	0.81	0.892	-0.26	225.76	-59.22

Total vertical net force $F_{w,v} = \mathbf{-74.03}$ kN

Total horizontal net force $F_{w,h}$ = **0.00** kN

Walls load case 3 - Wind 90, c_{pi} -0.3, -c_{pe}

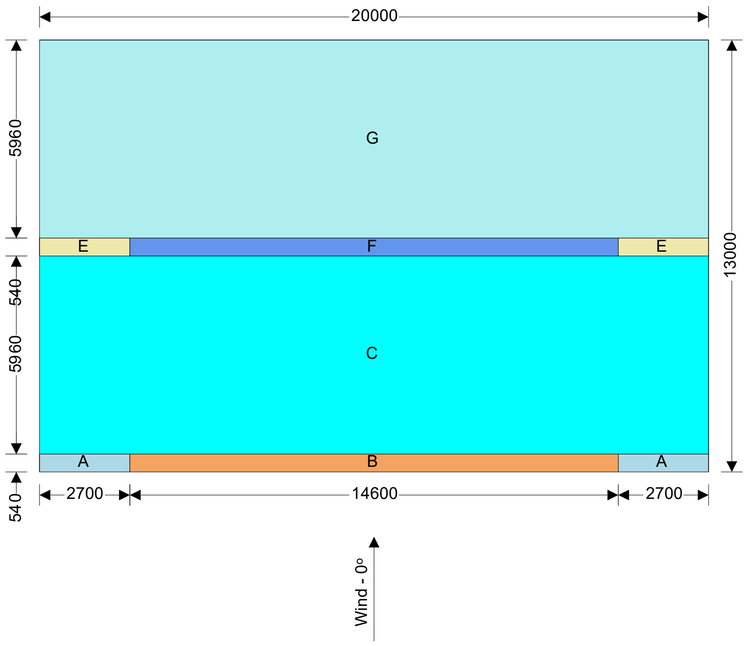
Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A	-1.34	0.81	0.895	-0.73	2.92	-2.13
B	-0.81	0.81	0.895	-0.35	11.66	-4.05
C	-0.56	0.81	0.895	-0.16	39.42	-6.39
w	0.60	0.81	0.926	0.69	31.40	21.75
l	-0.50	0.81	0.926	-0.13	31.40	-4.15

Overall loading

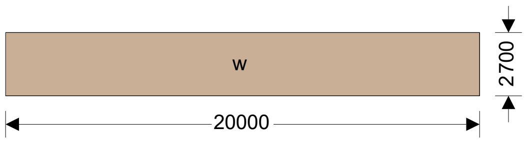
Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-4.1** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **21.8** kN

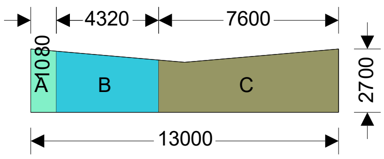
Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **22.5** kN



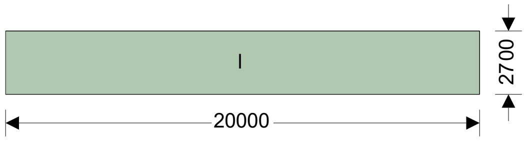
Plan view - Duopitch roof



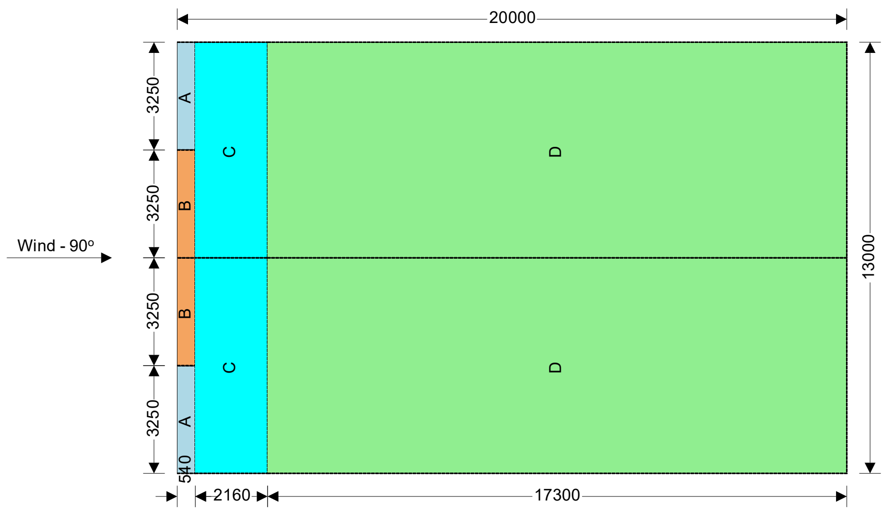
Windward face



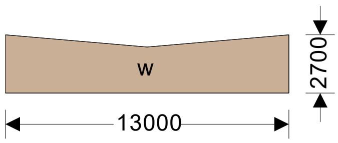
Side face



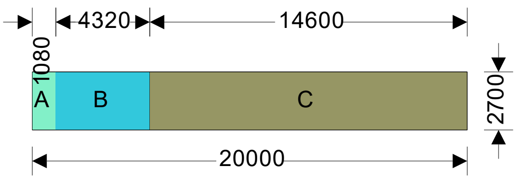
Leeward face



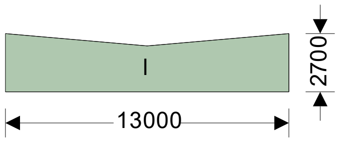
Plan view - Duopitch roof



Windward face



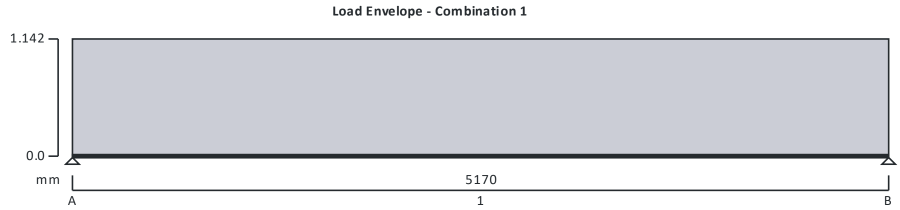
Side face

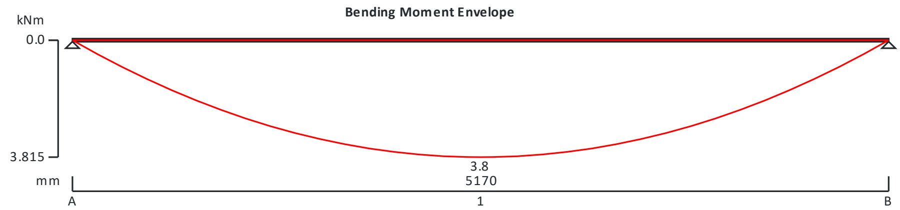


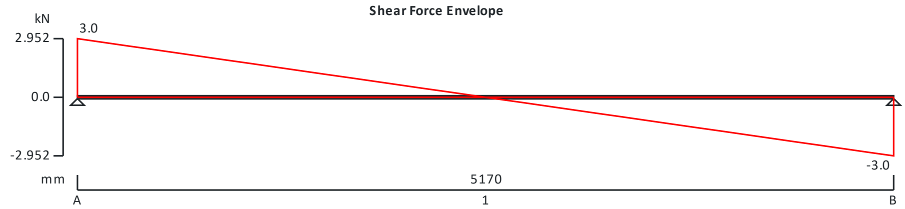
Leeward face

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02







Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 0.200 kN/m
Imposed full UDL 0.900 kN/m

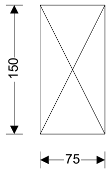
Load combinations

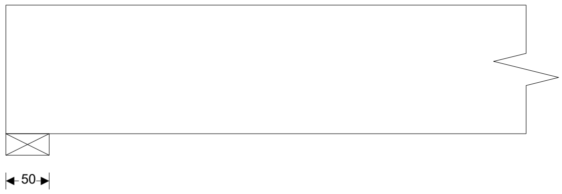
Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Maximum moment	M_{max} = **3.815** kNm	M_{min} = **0.000** kNm
Design moment	$M = max(abs(M_{max}),abs(M_{min}))$ = **3.815** kNm	
Maximum shear	F_{max} = **2.952** kN	F_{min} = **-2.952** kN
Design shear	$F = max(abs(F_{max}),abs(F_{min}))$ = **2.952** kN	
Total load on beam	W_{tot} = **5.904** kN	

Reactions at support A	R_{A_max} = **2.952** kN	R_{A_min} = **2.952** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **0.625** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **2.327** kN	
Reactions at support B	R_{B_max} = **2.952** kN	R_{B_min} = **2.952** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **0.625** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **2.327** kN	





Timber section details

Breadth of sections	b = **75** mm
Depth of sections	h = **150** mm
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **75** mm
Timber strength class	**C18**

Member details

Service class of timber	**1**
Load duration	**Medium term**
Length of span	L_{s1} = **5170** mm
Length of bearing	L_b = **50** mm

Section properties

Cross sectional area of member	$A = N \times b \times h$ = **11250** mm^2
Section modulus	$Z_x = N \times b \times h^2 / 6$ = **281250** mm^3
	$Z_y = h \times (N \times b)^2 / 6$ = **140625** mm^3
Second moment of area	$I_x = N \times b \times h^3 / 12$ = **21093750** mm^4
	$I_y = h \times (N \times b)^3 / 12$ = **5273437** mm^4
Radius of gyration	$i_x = \sqrt{(I_x / A)}$ = **43.3** mm
	$i_y = \sqrt{(I_y / A)}$ = **21.7** mm

Modification factors

Duration of loading - Table 17	K_3 = **1.25**
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	$h / (N \times b)$ = **2.00**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane)	$\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **2.750** N/mm^2
Applied bearing stress	$\sigma_{c\_a} = R_{B\_max} / (N \times b \times L_b)$ = **0.787** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.286**

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

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **7.824** N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x$ = **13.565** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm}$ = **1.734**

FAIL - Applied bending stress exceeds permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8$ = **0.838** N/mm^2

Applied shear stress $\tau_a = 3 \times F / (2 \times A)$ = **0.394** N/mm^2

τ_a / τ_{adm} = **0.470**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min}$ = **6000** N/mm^2

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

Bending deflection δ_{b_s1} = **83.933** mm

Shear deflection δ_{v_s1} = **1.085** mm

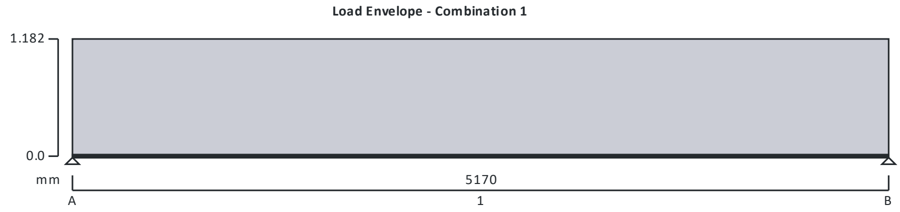
Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **85.018** mm

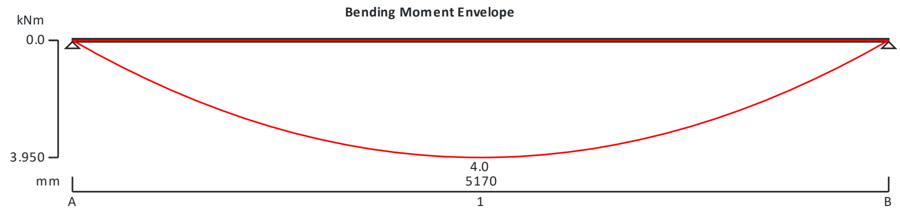
δ_a / δ_{adm} = **6.075**

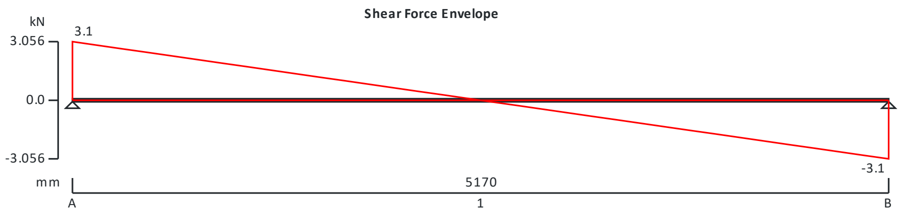
FAIL - Total deflection exceeds permissible deflection

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02







Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 0.200 kN/m
Imposed full UDL 0.900 kN/m

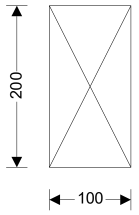
Load combinations

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} = **3.950** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **3.950** kNm	
Maximum shear	F_{max} = **3.056** kN	F_{min} = **-3.056** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **3.056** kN	
Total load on beam	W_{tot} = **6.113** kN	

Reactions at support A	R_{A_max} = **3.056** kN	R_{A_min} = **3.056** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **0.730** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **2.327** kN	
Reactions at support B	R_{B_max} = **3.056** kN	R_{B_min} = **3.056** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **0.730** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **2.327** kN	



Timber section details

Breadth of sections — b = **100** mm

Depth of sections — h = **200** mm

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

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

Timber strength class — **C24**

Member details

Service class of timber — **1**

Load duration — **Medium term**

Length of span — L_{s1} = **5170** mm

Length of bearing — L_b = **50** mm

Underside of beam notched at all supports

Beam depth at notch — h_e = **150** mm

Section properties

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

Section modulus — $Z_x = N \times b \times h^2 / 6$ = **666667** mm^3

$Z_y = h \times (N \times b)^2 / 6$ = **333333** mm^3

Second moment of area — $I_x = N \times b \times h^3 / 12$ = **66666667** mm^4

$I_y = h \times (N \times b)^3 / 12$ = **16666667** mm^4

Radius of gyration — $i_x = \sqrt{(I_x / A)}$ = **57.7** mm

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

Modification factors

Duration of loading - Table 17 — K_3 = **1.25**

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

Shear at notched ends - cl.2.10.4 — $K_5 = h_e / h$ = **0.75**

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 — $h / (N \times b)$ = **2.00**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **3.000** N/mm^2

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b)$ = **0.611** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.204**

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

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **9.803** N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x$ = **5.926** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.605**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain at notched support

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_5 \times K_8$ = **0.666** N/mm^2

Applied shear stress $\tau_a = 3 \times F / (2 \times N \times b \times h_e)$ = **0.306** N/mm^2

τ_a / τ_{adm} = **0.459**

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 δ_{adm} = min(0.984 in, 0.005 × L_{s1}) = **24.994** mm

Bending deflection δ_{b_s1} = **22.915** mm

Shear deflection δ_{v_s1} = **0.527** mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **23.441** mm

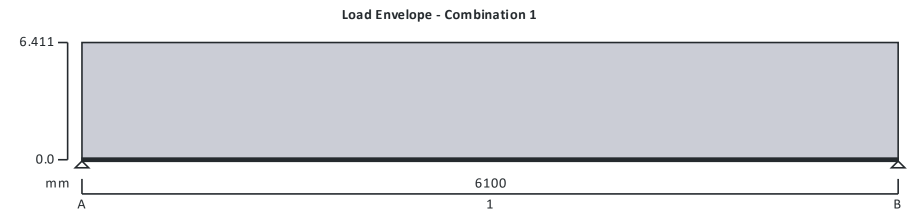
δ_a / δ_{adm} = **0.938**

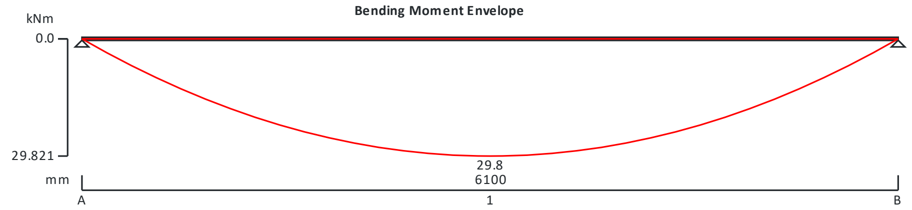
PASS - Total deflection is less than permissible deflection

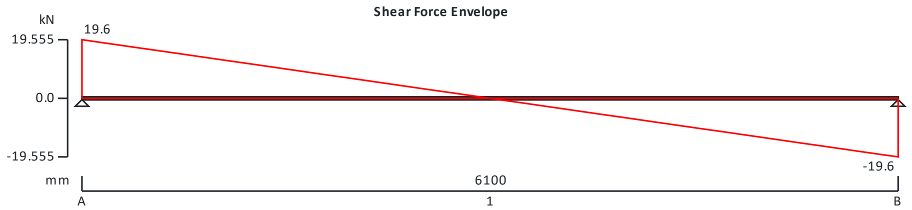
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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 0.85 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} = **29.8** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **19.6** kN	V_{min} = **-19.6** kN
Deflection	δ_{max} = **20.1** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **19.6** kN	R_{A_min} = **19.6** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **3.2** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **9.5** kN	
Maximum reaction at support B	R_{B_max} = **19.6** kN	R_{B_min} = **19.6** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **3.2** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **9.5** kN	

Section details

Section type — **UKB 178x102x19 (Tata Steel Advance)**

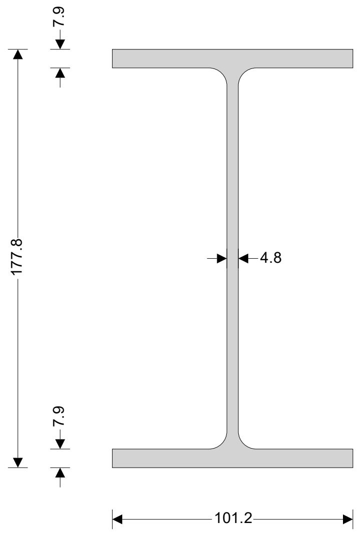
Steel grade — **S275**

From table 9: Design strength p_y

Thickness of element — max(T, t) = **7.9** mm

Design strength — p_y = **275** N/mm^2

Modulus of elasticity — E = **205000** N/mm^2



Lateral restraint

Span 1 has full lateral restraint

Effective length factors

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

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

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

$K_{LT.B}$ = **1.20** + 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 = **146.8** mm

$d / t = 30.6 \times \varepsilon <= 80 \times \varepsilon$ — Class 1 plastic

Outstand flanges - Table 11

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

$b / T = 6.4 \times \varepsilon <= 9 \times \varepsilon$ Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) =$ **19.6** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

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

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

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) =$ **29.8** kNm

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

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection $\delta_{lim} = L_{s1} / 200 =$ **30.5** mm

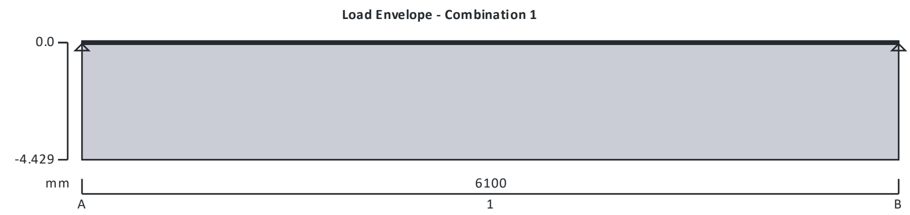
Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **20.105** mm

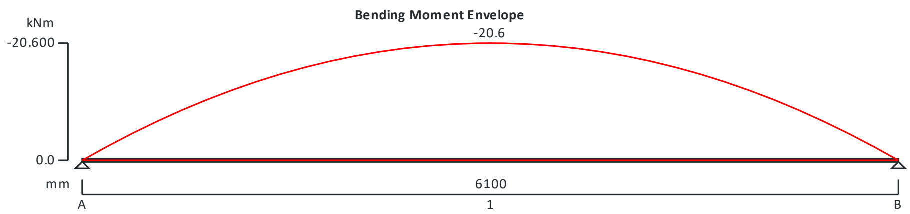
PASS - Maximum deflection does not exceed deflection limit

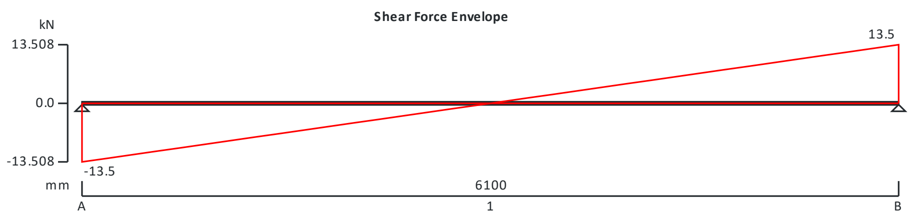
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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 0.85 kN/m
	Wind full UDL -3.83 kN/m

Load combinations

Load combination 1	Support A	Dead × 0.90
		Imposed × 1.60
		Wind × 1.40
		Dead × 0.90
		Imposed × 1.60
		Wind × 1.40
	Support B	Dead × 0.90

Imposed × 1.60
Wind × 1.40

Analysis results

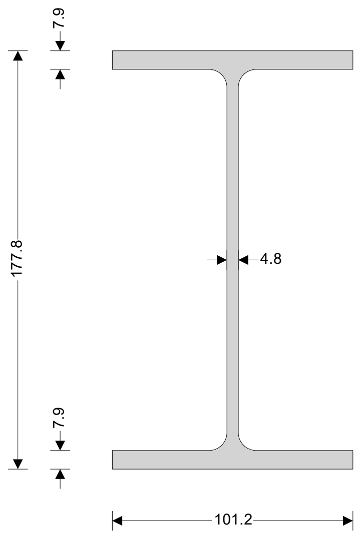
Maximum moment	M_{max} = **0** kNm	M_{min} = **-20.6** kNm
Maximum moment span 1 segment 1	$M_{s1\_seg1\_max}$ = **0** kNm	$M_{s1\_seg1\_min}$ = **-20.6** kNm
Maximum moment span 1 segment 2	$M_{s1\_seg2\_max}$ = **0** kNm	$M_{s1\_seg2\_min}$ = **-20.6** kNm
Maximum shear	V_{max} = **13.5** kN	V_{min} = **-13.5** kN
Maximum shear span 1 segment 1	$V_{s1\_seg1\_max}$ = **0** kN	$V_{s1\_seg1\_min}$ = **-13.5** kN
Maximum shear span 1 segment 2	$V_{s1\_seg2\_max}$ = **13.5** kN	$V_{s1\_seg2\_min}$ = **0** kN
Deflection segment 3	δ_{max} = **0** mm	δ_{min} = **24.8** mm
Maximum reaction at support A	R_{A_max} = **-13.5** kN	R_{A_min} = **-13.5** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **3.2** kN	
Unfactored wind load reaction at support A	R_{A_Wind} = **-11.7** kN	
Maximum reaction at support B	R_{B_max} = **-13.5** kN	R_{B_min} = **-13.5** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **3.2** kN	
Unfactored wind load reaction at support B	R_{B_Wind} = **-11.7** kN	

Section details

Section type	**UKB 178x102x19 (Tata Steel Advance)**
Steel grade	**S275**

From table 9: Design strength p_y

Thickness of element	max(T, t) = **7.9** mm
Design strength	p_y = **275** N/mm^2
Modulus of elasticity	E = **205000** N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports plus midspan

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**
	$K_{LT.B}$ = **1.20** + 2 × D

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: $d = \mathbf{146.8}$ mm

$d / t = 30.6 \times \varepsilon <= 80 \times \varepsilon$ Class 1 plastic

Outstand flanges - Table 11

Width of section: $b = B / 2 = \mathbf{50.6}$ mm

$b / T = 6.4 \times \varepsilon <= 9 \times \varepsilon$ Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force: $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{13.5}$ kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area: $A_v = t \times D = \mathbf{853}$ mm^2

Design shear resistance: $P_v = 0.6 \times p_y \times A_v = \mathbf{140.8}$ kN

PASS - Design shear resistance exceeds design shear force

Moment capacity at span 1 segment 2 - Section 4.2.5

Design bending moment: $M = \max(\text{abs}(M_{s1\_seg2\_max}), \text{abs}(M_{s1\_seg2\_min})) = \mathbf{20.6}$ kNm

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

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling: $L_E = ((1.0 + 1.2) \times L_{s1\_seg2} + 2 \times D) / 2 = \mathbf{3533}$ mm

Slenderness ratio: $\lambda = L_E / r_{yy} = \mathbf{148.827}$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter: $u = \mathbf{0.888}$

Torsional index: $x = \mathbf{22.560}$

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: $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} = \mathbf{98.959}$

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: $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = \mathbf{0.453}$

Euler stress: $p_E = \pi^2 \times E / \lambda_{LT}^2 = \mathbf{206.6}$ N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \mathbf{287.6}$ N/mm^2

Bending strength - Annex B.2.1: $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = \mathbf{126.7}$ N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment: $M_2 = \mathbf{19.3}$ kNm

Moment at centre-line of segment: $M_3 = \mathbf{15.4}$ kNm

Moment at three quarter point of segment: $M_4 = \mathbf{9}$ kNm

Maximum moment in segment: $M_{abs} = \mathbf{20.6}$ kNm

Maximum moment governing buckling resistance: $M_{LT} = M_{abs} = \mathbf{20.6}$ kNm

Equivalent uniform moment factor for lateral-torsional buckling

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = \mathbf{0.781}$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment $M_b = p_b \times S_{xx} =$ **21.7** kNm

$M_b / m_{LT} =$ **27.8** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed and wind loads

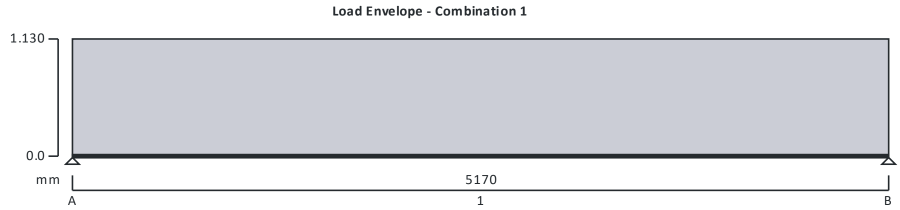
Limiting deflection $\delta_{lim} = L_{s1} / 200 =$ **30.5** mm

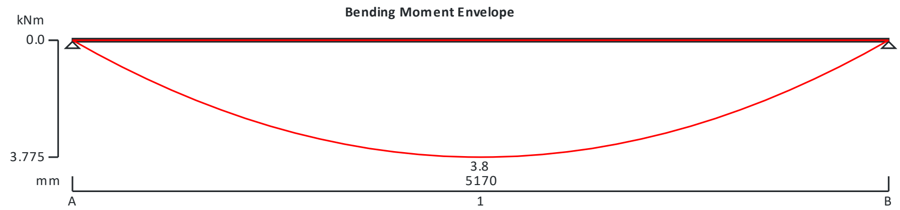
Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) =$ **24.84** mm

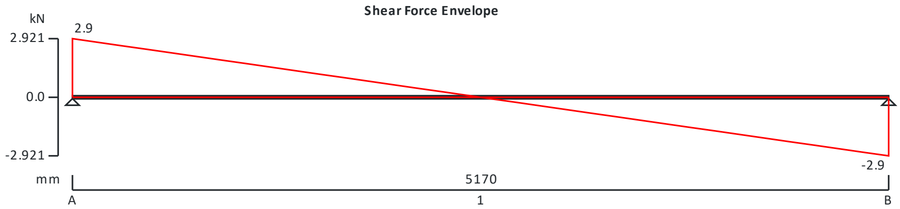
PASS - Maximum deflection does not exceed deflection limit

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02







Applied loading

Beam loads

Dead self weight of beam × 1

Wind full UDL 1.130 kN/m

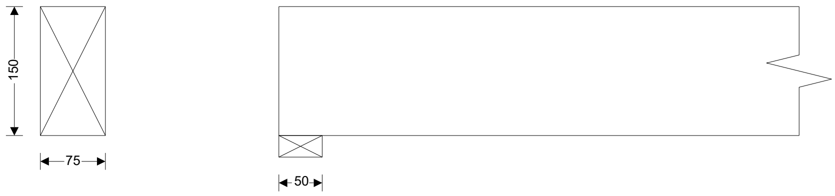
Load combinations

Load combination 1	Support A	Dead × 0.00
		Imposed × 1.00
		Wind × 1.00
	Span 1	Dead × 0.00
		Imposed × 1.00
		Wind × 1.00
	Support B	Dead × 0.00
		Imposed × 1.00
		Wind × 1.00

Analysis results

Maximum moment	M_{max} = **3.775** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **3.775** kNm	
Maximum shear	F_{max} = **2.921** kN	F_{min} = **-2.921** kN

Design shear	$F = max(abs(F_{max}),abs(F_{min}))$ = **2.921** kN	
Total load on beam	W_{tot} = **5.842** kN	
Reactions at support A	R_{A_max} = **2.921** kN	R_{A_min} = **2.921** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **0.120** kN	
Unfactored wind load reaction at support A	R_{A_Wind} = **2.921** kN	
Reactions at support B	R_{B_max} = **2.921** kN	R_{B_min} = **2.921** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **0.120** kN	
Unfactored wind load reaction at support B	R_{B_Wind} = **2.921** kN	



Timber section details

Breadth of sections — b = **75** mm

Depth of sections — h = **150** mm

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

Overall breadth of member — $b_b = N \times b$ = **75** mm

Timber strength class — **C24**

Member details

Service class of timber — **1**

Load duration — **Very short term**

Length of span — L_{s1} = **5170** mm

Length of bearing — L_b = **50** mm

Section properties

Cross sectional area of member — $A = N \times b \times h$ = **11250** mm^2

Section modulus — $Z_x = N \times b \times h^2 / 6$ = **281250** mm^3

$Z_y = h \times (N \times b)^2 / 6$ = **140625** mm^3

Second moment of area — $I_x = N \times b \times h^3 / 12$ = **21093750** mm^4

$I_y = h \times (N \times b)^3 / 12$ = **5273437** mm^4

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

$i_y = \sqrt{(I_y / A)}$ = **21.7** mm

Modification factors

Duration of loading - Table 17 — K_3 = **1.75**

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 — $h / (N \times b)$ = **2.00**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) — $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **4.200** N/mm^2

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b) =$ **0.779** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm} =$ **0.185**

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

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 =$ **14.165** N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x =$ **13.424** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm} =$ **0.948**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8 =$ **1.243** N/mm^2

Applied shear stress $\tau_a = 3 \times F / (2 \times A) =$ **0.389** N/mm^2

$\tau_a / \tau_{adm} =$ **0.313**

PASS - Applied shear stress is less than permissible shear stress

Deflection

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

Permissible deflection $\delta_{adm} = \min(0.984 \text{ in}, 0.005 \times L_{s1}) =$ **24.994** mm

Bending deflection $\delta_{b\_s1} =$ **72.052** mm

Shear deflection $\delta_{v\_s1} =$ **0.932** mm

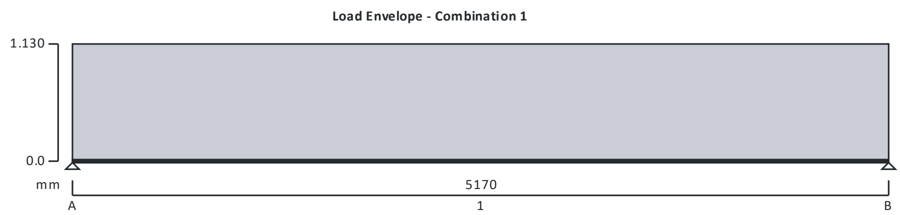
Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} =$ **72.984** mm

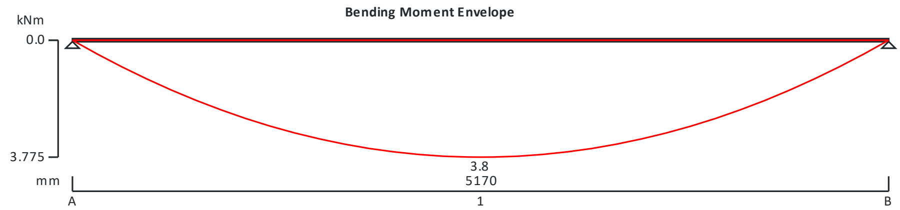
$\delta_a / \delta_{adm} =$ **2.920**

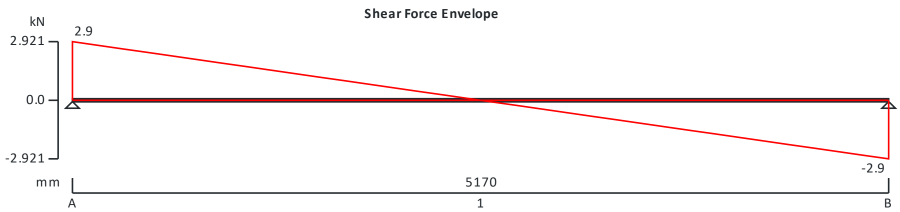
FAIL - Total deflection exceeds permissible deflection

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02







Applied loading

Beam loads

Dead self weight of beam × 1

Wind full UDL 1.130 kN/m

Load combinations

Load combination 1	Support A	Dead × 0.00
		Imposed × 1.00
		Wind × 1.00
	Span 1	Dead × 0.00
		Imposed × 1.00
		Wind × 1.00
	Support B	Dead × 0.00
		Imposed × 1.00
		Wind × 1.00

Analysis results

Maximum moment	M_{max} = **3.775** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **3.775** kNm	
Maximum shear	F_{max} = **2.921** kN	F_{min} = **-2.921** kN

Design shear — $F = max(abs(F_{max}),abs(F_{min}))$ = **2.921** kN

Total load on beam — W_{tot} = **5.842** kN

Reactions at support A — R_{A_max} = **2.921** kN — R_{A_min} = **2.921** kN

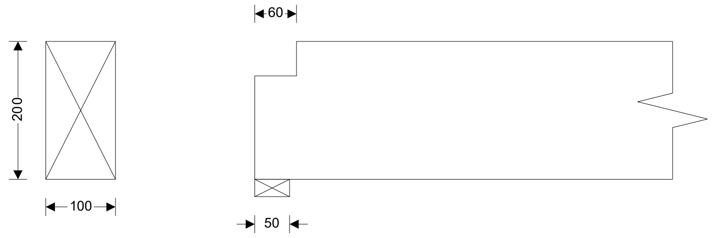
Unfactored dead load reaction at support A — R_{A_Dead} = **0.213** kN

Unfactored wind load reaction at support A — R_{A_Wind} = **2.921** kN

Reactions at support B — R_{B_max} = **2.921** kN — R_{B_min} = **2.921** kN

Unfactored dead load reaction at support B — R_{B_Dead} = **0.213** kN

Unfactored wind load reaction at support B — R_{B_Wind} = **2.921** kN



Timber section details

Breadth of sections — b = **100** mm

Depth of sections — h = **200** mm

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

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

Timber strength class — **C24**

Member details

Service class of timber — **1**

Load duration — **Very short term**

Length of span — L_{s1} = **5170** mm

Length of bearing — L_b = **50** mm

Top edge of beam notched at all supports

Beam depth at notch — h_e = **150** mm

Length of notch beyond bearing — a = **10** mm

Section properties

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

Section modulus — $Z_x = N \times b \times h^2 / 6$ = **666667** mm^3

$Z_y = h \times (N \times b)^2 / 6$ = **333333** mm^3

Second moment of area — $I_x = N \times b \times h^3 / 12$ = **66666667** mm^4

$I_y = h \times (N \times b)^3 / 12$ = **16666667** mm^4

Radius of gyration — $i_x = \sqrt{(I_x / A)}$ = **57.7** mm

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

Modification factors

Duration of loading - Table 17 — K_3 = **1.75**

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

Shear at notched ends - cl.2.10.4 — $K_5 = (h \times (h_e - a) + a \times h_e) / h_e^2$ = **1.31**

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

Ends held in position

Permissible depth-to-breadth ratio - Table 19 **3.00**

Actual depth-to-breadth ratio $h / (N \times b) = \mathbf{2.00}$

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 = \mathbf{4.200}$ N/mm^2

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b) = \mathbf{0.584}$ N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm} = \mathbf{0.139}$

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

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{13.724}$ N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x = \mathbf{5.663}$ N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm} = \mathbf{0.413}$

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain at notched support

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_5 \times K_8 = \mathbf{1.629}$ N/mm^2

Applied shear stress $\tau_a = 3 \times F / (2 \times N \times b \times h_e) = \mathbf{0.292}$ N/mm^2

$\tau_a / \tau_{adm} = \mathbf{0.179}$

PASS - Applied shear stress is less than permissible shear stress

Deflection

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

Permissible deflection $\delta_{adm} = \min(0.984\text{ in}, 0.005 \times L_{s1}) = \mathbf{24.994}$ mm

Bending deflection $\delta_{b\_s1} = \mathbf{23.496}$ mm

Shear deflection $\delta_{v\_s1} = \mathbf{0.540}$ mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} = \mathbf{24.036}$ mm

$\delta_a / \delta_{adm} = \mathbf{0.962}$

PASS - Total deflection is less than permissible deflection

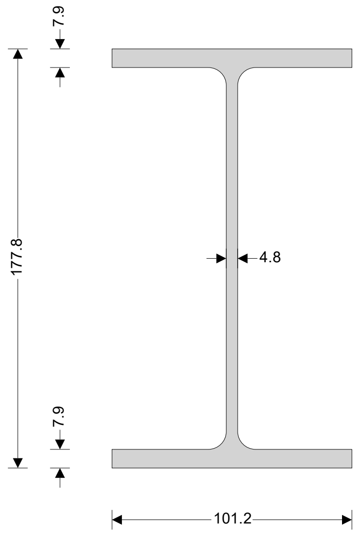
STEEL MEMBER DESIGN (BS5950)

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

TEDDS calculation version 3.0.07

Section details

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



Lateral restraint

Distance between major axis restraints	L_x = **3000** mm
Distance between minor axis restraints	L_y = **3000** mm

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} = **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 = **146.8** mm	
Stress ratios	$r1 = \min(F_c / (d \times t \times p_{yw}), 1)$ = **0.101**	
	$r2 = F_c / (A \times p_{yw})$ = **0.029**	
	$d / t = 30.6 \times \varepsilon <= \max(80 \times \varepsilon / (1 + r1), 40 \times \varepsilon)$	Class 1 plastic

Outstand flanges - Table 11

Width of section	b = B / 2 = **50.6** mm	
	$b / T = 6.4 \times \varepsilon <= 9 \times \varepsilon$	Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force $F_{y,v}$ = **8.2** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

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

Design shear resistance $P_{y,v} = 0.6 \times p_y \times A_v$ = **140.8** kN

PASS - Design shear resistance exceeds design shear force

Shear capacity - Section 4.2.3

Design shear force $F_{x,v}$ = **0** kN

Moment capacity - Section 4.2.5

Design bending moment M = **6.2** kNm

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

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling $L_E = 1.0 \times L_y$ = **3000** mm

Slenderness ratio $\lambda = L_E / r_{yy}$ = **126.382**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter u = **0.888**

Torsional index x = **22.560**

Slenderness factor $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25}$ = **0.790**

Ratio - cl.4.3.6.9 β_W = **1.000**

Equivalent slenderness - cl.4.3.6.7 $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **88.610**

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

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

Bending strength - Section 4.3.6.5

Robertson constant α_{LT} = **7.0**

Perry factor $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.380**

Euler stress $p_E = \pi^2 \times E / \lambda_{LT}^2$ = **257.7** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2$ = **315.3** N/mm^2

Bending strength - Annex B.2.1 $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5})$ = **146.3** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Equivalent uniform moment factor for LTB m_{LT} = **1.000**

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment $M_b = p_b \times S_{xx}$ = **25.1** kNm

M_b / m_{LT} = **25.1** kNm

PASS - Buckling resistance moment exceeds design bending moment

Compression members - Section 4.7

Design compression force F_c = **19.6** kN

Effective length for major (x-x) axis buckling - Section 4.7.3

Effective length for buckling $L_{Ex} = L_x \times K_x$ = **3000** mm

Slenderness ratio - cl.4.7.2 $\lambda_x = L_{Ex} / r_{xx}$ = **40.129**

Compressive strength - Section 4.7.5

Limiting slenderness $\lambda_0 = 0.2 \times (\pi^2 \times E / p_y)^{0.5}$ = **17.155**

Strut curve - Table 23 a

Robertson constant α_x = **2.0**

Perry factor $\eta_x = \alpha_x \times (\lambda_x - \lambda_0) / 1000$ = **0.046**

Euler stress $p_{Ex} = \pi^2 \times E / \lambda_x^2$ = **1256.5** N/mm^2

$\phi_x = (p_y + (\eta_x + 1) \times p_{Ex}) / 2$ = **794.6** N/mm^2

Compressive strength - Annex C.1 $p_{cx} = p_{Ex} \times p_y / (\phi_x + (\phi_x^2 - p_{Ex} \times p_y)^{0.5})$ = **259.9** N/mm^2

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4 $P_{cx} = A \times p_{cx}$ = **630.7** kN

PASS - Compression resistance exceeds design compression force

Effective length for minor (y-y) axis buckling - Section 4.7.3

Effective length for buckling $L_{Ey} = L_y \times K_y$ = **3000** mm

Slenderness ratio - cl.4.7.2 $\lambda_y = L_{Ey} / r_{yy}$ = **126.382**

Compressive strength - Section 4.7.5

Limiting slenderness $\lambda_0 = 0.2 \times (\pi^2 \times E / p_y)^{0.5}$ = **17.155**

Strut curve - Table 23 b

Robertson constant α_y = **3.5**

Perry factor $\eta_y = \alpha_y \times (\lambda_y - \lambda_0) / 1000$ = **0.382**

Euler stress $p_{Ey} = \pi^2 \times E / \lambda_y^2$ = **126.7** N/mm^2

$\phi_y = (p_y + (\eta_y + 1) \times p_{Ey}) / 2$ = **225** N/mm^2

Compressive strength - Annex C.1 $p_{cy} = p_{Ey} \times p_y / (\phi_y + (\phi_y^2 - p_{Ey} \times p_y)^{0.5})$ = **99.3** N/mm^2

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4 $P_{cy} = A \times p_{cy}$ = **240.9** kN

PASS - Compression resistance exceeds design compression force

Compression members with moments - Section 4.8.3

Comb.compression & bending check - cl.4.8.3.2 $F_c / (A \times p_y) + M / M_c$ = **0.160**

PASS - Combined bending and compression check is satisfied

Member buckling resistance - Section 4.8.3.3

Max major axis moment governing M_b $M_{LT} = M_x$ = **6.15** kNm

Equivalent uniform moment factor for major axis flexural buckling

m_x = **1.000**

m_y = **1.000**

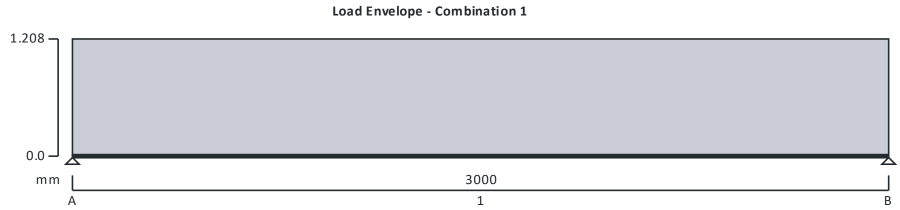
Buckling resistance checks - cl.4.8.3.3.2 $F_c / P_{cx} + m_x \times M / M_c \times (1 + 0.5 \times F_c / P_{cx})$ = **0.164**

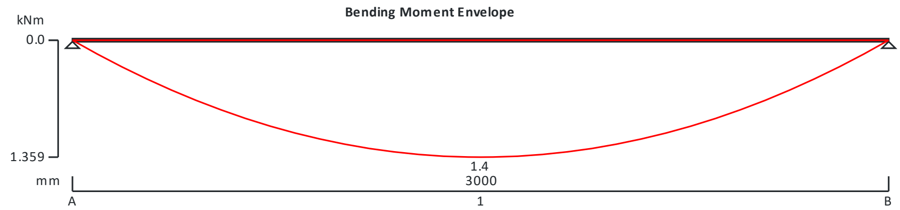
$F_c / P_{cy} + m_{LT} \times M_{LT} / M_b$ = **0.327**

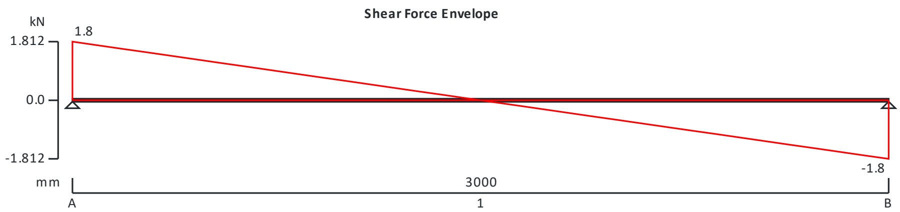
PASS - Member buckling resistance checks are satisfied

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02







Applied loading

Beam loads

Dead self weight of beam × 1

Dead full UDL 1.150 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

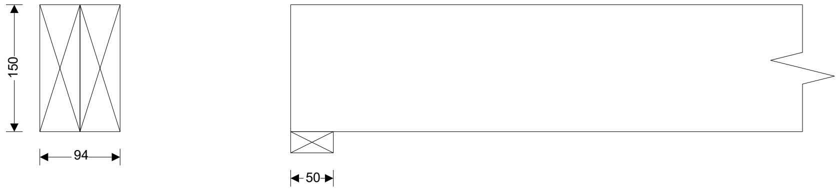
Analysis results

Maximum moment	M_{max} = **1.359** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **1.359** kNm	
Maximum shear	F_{max} = **1.812** kN	F_{min} = **-1.812** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **1.812** kN	
Total load on beam	W_{tot} = **3.624** kN	
Reactions at support A	R_{A_max} = **1.812** kN	R_{A_min} = **1.812** kN

Unfactored dead load reaction at support A R_{A_Dead} = **1.812** kN

Reactions at support B R_{B_max} = **1.812** kN R_{B_min} = **1.812** kN

Unfactored dead load reaction at support B R_{B_Dead} = **1.812** kN



Timber section details

Breadth of sections b = **47** mm

Depth of sections h = **150** mm

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

Overall breadth of member $b_b = N \times b$ = **94** mm

Timber strength class **C24**

Member details

Service class of timber **1**

Load duration **Long term**

Length of span L_{s1} = **3000** mm

Length of bearing L_b = **50** mm

Section properties

Cross sectional area of member $A = N \times b \times h$ = **14100** mm^2

Section modulus $Z_x = N \times b \times h^2 / 6$ = **352500** mm^3

$Z_y = h \times (N \times b)^2 / 6$ = **220900** mm^3

Second moment of area $I_x = N \times b \times h^3 / 12$ = **26437500** mm^4

$I_y = h \times (N \times b)^3 / 12$ = **10382300** mm^4

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

$i_y = \sqrt{(I_y / A)}$ = **27.1** 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

Ends held in position and members held in line, as by purlins or tie rods at centres not more than 30 times the breadth of the member

Permissible depth-to-breadth ratio - Table 19 **4.00**

Actual depth-to-breadth ratio $h / (N \times b)$ = **1.60**

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 $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b)$ = **0.386** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.161**

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

Bending parallel to grain

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

Applied bending stress: $\sigma_{m\_a} = M / Z_x = \mathbf{3.856}$ N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm} = \mathbf{0.476}$

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

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

Applied shear stress: $\tau_a = 3 \times F / (2 \times A) = \mathbf{0.193}$ N/mm^2

$\tau_a / \tau_{adm} = \mathbf{0.272}$

PASS - Applied shear stress is less than permissible shear stress

Deflection

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

Permissible deflection: $\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1}) = \mathbf{9.000}$ mm

Bending deflection: $\delta_{b\_s1} = \mathbf{6.694}$ mm

Shear deflection: $\delta_{v\_s1} = \mathbf{0.257}$ mm

Total deflection: $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} = \mathbf{6.951}$ mm

$\delta_a / \delta_{adm} = \mathbf{0.772}$

PASS - Total deflection is less than permissible deflection

SHEARWATER
Consulting Engineers Ltd

Construction (Design and Management) Regulations 2015

Non Domestic Projects.

The CDM Regulations place duties on various parties to manage health, safety and welfare during the design and construction of construction projects.

Detailed guidance on the CDM Regulations 2015 can be found on the Health and Safety Executive (HSE) website here:-
www.hse.gov.uk/construction/cdm/2015/index.htm

Shearwater Consulting Engineers Ltd will undertake the role of 'designer' for the structural elements of the project that they have been engaged to design. The table below sets out any residual risks that the 'contractor' will need to consider during the works. Please note that we have only included risks that are specific to this project, we have not listed risks that a competent contractor would be expected to be familiar with.

Register of residual design risks to be taken into account by the contractor.

Item.	Description.	Mitigation Measures.
1.	None	
2.		
3.		
4.		
5.		
6.		