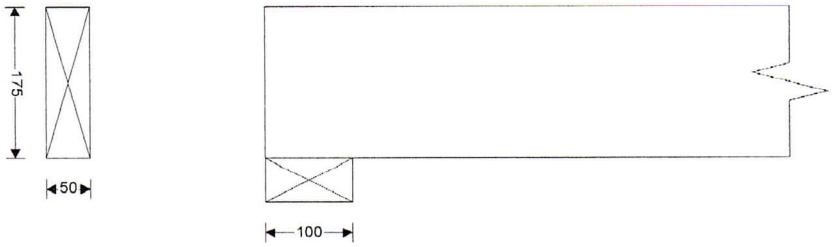
Analysis results

Maximum moment	M_{max} = **1.441** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **1.441** kNm	
Maximum shear	F_{max} = **0.947** kN	F_{min} = **-1.669** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **1.669** kN	
Total load on beam	W_{tot} = **2.616** kN	
Reactions at support A	R_{A_max} = **0.947** kN	R_{A_min} = **0.947** kN
Unfactored permanent load reaction at support A	$R_{A_Permanent}$ = **0.211** kN	
Unfactored variable load reaction at support A	$R_{A_Variable}$ = **0.441** kN	
Reactions at support B	R_{B_max} = **1.669** kN	R_{B_min} = **1.669** kN
Unfactored permanent load reaction at support B	$R_{B_Permanent}$ = **0.293** kN	
Unfactored variable load reaction at support B	$R_{B_Variable}$ = **0.849** kN	



Timber section details

Breadth of timber sections	b = **50** mm
Depth of timber sections	h = **175** mm
Number of timber sections in member	N = **1**
Overall breadth of timber member	b_b = N × b = **50** mm
Timber strength class - EN 338:2016 Table 1	**C24**

Member details

Load duration - cl.2.3.1.2	**Long-term**
Service class of timber - cl.2.3.1.3	**1**
Length of span	L_{s1} = **4100** mm
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	A = N × b × h = **8750** mm^2
Section modulus	W_y = N × b × h^2 / 6 = **255208** mm^3
	W_z = h × (N × b)^2 / 6 = **72917** mm^3
Second moment of area	I_y = N × b × h^3 / 12 = **22330729** mm^4
	I_z = h × (N × b)^3 / 12 = **1822917** mm^4
Radius of gyration	r_y = √(I_y / A) = **50.5** mm
	r_z = √(I_z / A) = **14.4** mm

Partial factor for material properties and resistances

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

Modification factors

Modification factor for load duration and moisture content - Table 3.1

k_{mod} = **0.700**

Deformation factor for service classes - Table 3.2	k_{def} = **0.600**

Depth factor for bending - exp.3.1	$k_{h.m}$ = **1.000**
Depth factor for tension - exp.3.1	$k_{h.t}$ = **1.000**
Bending stress re-distribution factor - cl.6.1.6(2)	k_m = **0.700**
Crack factor for shear resistance - cl.6.1.7(2)	k_{cr} = **0.670**
Load configuration factor - exp.6.4	$k_{c.90}$ = **1.000**
System strength factor - cl.6.6	k_{sys} = **1.000**
Lateral buckling factor - cl.6.3.3(5)	k_{crit} = **1.000**

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress	$\sigma_{c.90.d} = R_{B_max} / (N \times b \times L_b)$ = **0.334** N/mm²
Design compressive strength	$f_{c.90.d} = k_{mod} \times k_{sys} \times k_{c.90} \times f_{c.90.k} / \gamma_M$ = **1.346** N/mm²
	$\sigma_{c.90.d} / f_{c.90.d}$ = **0.248**

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress	$\sigma_{m.d} = M / W_y$ = **5.647** N/mm²
Design bending strength	$f_{m.d} = k_{h.m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m.k} / \gamma_M$ = **12.923** N/mm²
	$\sigma_{m.d} / f_{m.d}$ = **0.437**

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress	$\tau_d = 3 \times F / (2 \times k_{cr} \times A)$ = **0.427** N/mm²
Permissible shear stress	$f_{v.d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **2.154** N/mm²
	$\tau_d / f_{v.d}$ = **0.198**

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit	$\delta_{lim} = \min(14 \text{ mm}, 0.004 \times L_{s1})$ = **14.000** mm
Instantaneous deflection due to permanent load	δ_{instG} = **1.923** mm
Final deflection due to permanent load	$\delta_{finG} = \delta_{instG} \times (1 + k_{def})$ = **3.076** mm
Instantaneous deflection due to variable load	δ_{instQ} = **5.001** mm
Factor for quasi-permanent variable action	ψ_2 = **0.3**
Final deflection due to variable load	$\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def})$ = **5.901** mm
Total final deflection	$\delta_{fin} = \delta_{finG} + \delta_{finQ}$ = **8.978** mm
	$\delta_{fin} / \delta_{lim}$ = **0.641**

PASS - Total final deflection is less than the deflection limit

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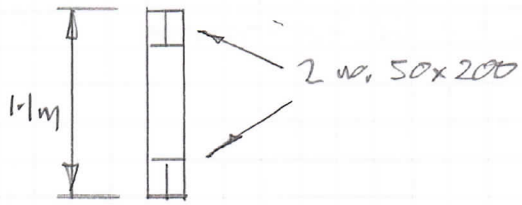
Job Title: 9 SANDY BROW		Date: MARCH 2026
Item: OPTION ④	Job No: 632	Sheet No: 16
Prepared by: RJP	Checked by:	

Ref:	Calculations:	Output:

OPTION ④

Consider ashlar wall as plyweb beam supporting dormer roof.

Span $5.2 + 0.1 + 0.1 = 5.4$ m



Loading - dormer roof DL $0.80\ KN/m^2 \times \frac{2.0}{2} = 0.8$

IL $0.75\ KN/m^2 \times \frac{2.0}{2} = 0.8$

roof DL $0.90\ KN/m^2 \times \frac{1.9}{2} = 0.9$

IL $0.60\ KN/m^2 \times \frac{1.9}{2} = 0.6$

wall $0.90\ KN/m^2 \times 2.4 = 2.2$

Σ DL 3.9 KN/m

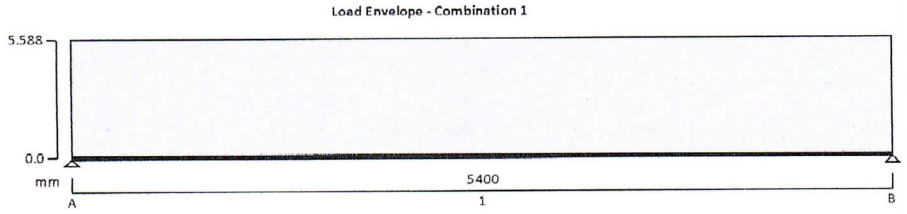
IL 1.4 KN/m

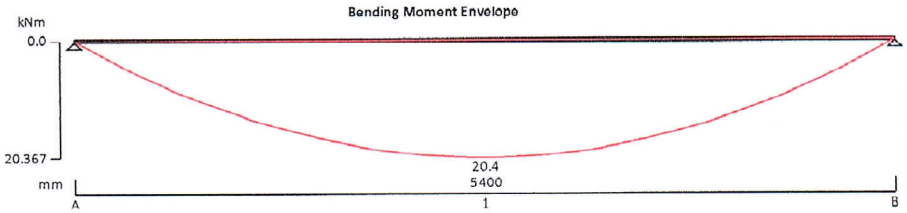
See computer print-out for analysis and design

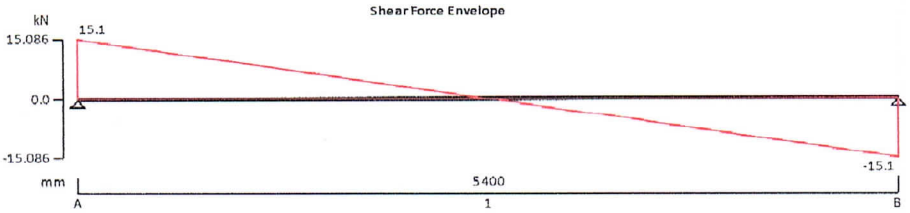
Plyweb beam is satisfactory

PLY WEB BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.03







Applied loading

Beam loads

Dead full UDL 3.900 kN/m
Imposed full UDL 1.400 kN/m
Dead self weight of beam × 1

Load combinations

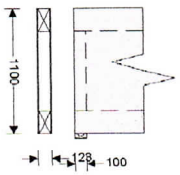
Load combination 1

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} = **20.367** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **20.367** kNm	
Maximum shear	F_{max} = **15.086** kN	F_{min} = **-15.086** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **15.086** kN	
Total load on beam	W_{tot} = **30.173** kN	

Reactions at support A	R_{A_max} = **15.086** kN	R_{A_min} = **15.086** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **11.306** kN	
Unfactored imposed load reaction at support A	$R_{A_Imposed}$ = **3.780** kN	
Reactions at support B	R_{B_max} = **15.086** kN	R_{B_min} = **15.086** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **11.306** kN	
Unfactored imposed load reaction at support B	$R_{B_Imposed}$ = **3.780** kN	



Timber section details

Breadth of timber sections	b = **100** mm
Depth of timber sections	h = **200** mm
Number of timber sections in beam	N = **2**
Timber strength class	**C16**

Ply web section details

Plywood type	**Finnish conifer plywood 1.4 mm veneer:Sanded**
Nominal ply thickness	t = **12.0** mm
Minimum ply thickness	t_{min} = **11.5** mm
Number of plies	ply = **9**
Depth of ply web	h_p = **1100** mm
Number of ply webs	N_p = **2**

Plywood properties

Modulus of elasticity in tension and compression	E_{t_perp} = **2950** N/mm²
Shear modulus	G = **270** N/mm²
Panel shear	τ_p = **3.740** N/mm²
Rolling shear	τ_{r_face} = **0.790** N/mm²
Plywood selfweight	s_{wt} = **6.6** kg/m²

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of span	L_{s1} = **5400** mm
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of timber	$A = N \times b \times h$ = **40000** mm²
Cross sectional area of ply web	$A_p = N_p \times t_{min} \times h_p$ = **25300** mm²
Second moment of area of ply web	$I_{xp} = N_p \times t_{min} \times h_p^3 / 12$ = **2551083333** mm⁴
	$I_{yp} = h_p \times [(b + N_p \times t_{min})^3 - b^3] / 12$ = **78912808** mm⁴
Second moment of area of timber	$I_x = N \times b \times [h_p^3 - (h_p - 2 \times h)^3] / 24$ = **8233333333** mm⁴
	$I_y = N \times h \times b^3 / 12$ = **33333333** mm⁴
Section modulus of timber flange	$Z_x = 2 \times I_x / h_p$ = **14969697** mm³

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 — $K_7 = (300\ mm / h)^{0.11}$ = **1.05**

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

Bending, tension and shear parallel to grain - Table 25

K_{27} = **1.00**

Modulus of elasticity, compression parallel to grain - Table 25

K_{28} = **1.14**

Modification factor for grade stresses - Table 39 — K_{36s} = **1.00**

Modification factor for moduli - Table 39 — K_{36m} = **1.00**

Stress concentration factor - cl.4.7 — K_{37} = **0.50**

Bonding pressure factor - cl.6.10.1.4 — K_{70} = **0.90**

Check lateral stability of beam

Compression flange fully restrained

Permissible I_x to I_y ratio — **More than 40**

Actual I_x to I_y ratio — $(I_x + I_{xp}) / (I_y + I_{yp})$ = **96.08**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) — $\sigma_{c_adm} = \sigma_{cp1} \times K_3 \times K_4$ = **2.200** N/mm²

Applied bearing stress — $\sigma_{c_a} = 2 \times R_{A_max} / (N \times b \times L_b)$ = **1.509** N/mm²

$\sigma_{c_a} / \sigma_{c_adm}$ = **0.686**

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

Bending parallel to grain

Permissible bending stress — $\sigma_{m_adm} = \sigma_m \times K_3 \times K_{27}$ = **5.300** N/mm²

Applied bending stress — $\sigma_{m_a} = M / Z_x$ = **1.361** N/mm²

$\sigma_{m_a} / \sigma_{m_adm}$ = **0.257**

PASS - Applied bending stress is less than permissible bending stress

Panel shear

Permissible panel shear stress — $\tau_{p_adm} = \tau_p \times K_{36s}$ = **3.740** N/mm²

First moment of area of timber above x-x axis — $Q = N \times b \times h \times (h_p - h) / 4$ = **9000000** mm³

First moment of area of ply web above x-x axis — $Q_p = N_p \times t_{min} \times h_p^2 / 8$ = **3478750** mm³

Total moment of area of beam above x-x axis — $Q_{tot} = Q + Q_p$ = **12478750** mm³

Applied panel shear stress — $\tau_{p_a} = F \times Q_{tot} / [N_p \times t_{min} \times (I_x + I_{xp})]$ = **0.759** N/mm²

$\tau_{p_a} / \tau_{p_adm}$ = **0.203**

PASS - Panel shear stress is less than permissible panel shear stress

Web-flange interface shear

Permissible interface stress — $\tau_{r_adm} = \tau_{r_face} \times K_{36s} \times K_{37} \times K_{70}$ = **0.356** N/mm²

Applied interface stress — $\tau_{r_a} = F \times Q / [2 \times h \times (I_x + I_{xp})]$ = **0.031** N/mm²

$\tau_{r_a} / \tau_{r_adm}$ = **0.089**

PASS - Web-flange interface stress is less than permissible interface stress

Deflection

Modulus of elasticity for deflection — $E = [(E_{min} \times K_{28} \times I_x) + (E_{t_perp} \times K_{36m} \times I_{xp})] / I$ = **5746** N/mm²

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

Bending deflection	δ_{b_s1} = **0.998** mm
Shear deflection	δ_{v_s1} = **2.981** mm
Total deflection	$\delta_a = \delta_{b_s1} + \delta_{v_s1}$ = **3.980** mm
	δ_a / δ_{adm} = **0.284**
	PASS - Total deflection is less than permissible deflection

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Job Title: 9 SANDY BROW		Date: MARCH 2026
Item: OPTION (5)	Job No: 632	Sheet No: 21
Prepared by: RJP	Checked by:	

Ref:	Calculations:	Output:

OPTION (5)

Consider ashlar wall as plyweb beam supporting pitched roof with Velux windows.

Loading - roof DL $1.05\ KN/m^2 \times \frac{5.9}{2} = 3.1\ KN/m$

IL $0.60\ KN/m^2 \times \frac{5.9}{2} = 1.8\ KN/m$

By inspection of option (4)

Plyweb beam will work satisfactorily