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Contract: Parley Community Hub 275 Christchurch Road West Parley	**Job No:** 2024.159
	Sheet No: 1
Part Of Structure: General Loadings	**Date:** Jul '24

		D.L		L.L	
Flat Roof	L.L.			0.75	kN/m²
	Felt & Chippings	0.35	kN/m²		
	Boarding	0.20	kN/m²		
	Insulation	0.20	kN/m²		
	Timber	0.20	kN/m²		
	Ceiling	0.20	kN/m²		
		1.15	**kN/m²**	**0.75**	**kN/m²**

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Contract: Parley Community Hub, 275 Christchurch Road, West Parley

Job No. 2024.159

Sht. No. 2

Part of Structure

Date July '24

TJ1

Design of trimmers to new roof lights.

Span = 3.2m

Load on trimmers:		D.L.	L.L.	Total
		(kN)	(kN)	
Flat Roof	: Or 1.15 × 0.4 × 1/2	0.23		
	Or 0.75 × 0.4 × 1/2		0.15	
Roof lights	: Or 0.3 × 2.0/2	0.30		
	Or 0.75 × 2.0/2		0.75	
		0.53	0.90	

See computer printout for trimmer analysis (pages 3-5)

∴ Provide 2No. 225 × 47 – C24 Joists bolted together at 600 crs. (2No. bolts at bearing)

(By observation, provide same to small roof lights over New office & kitchen)

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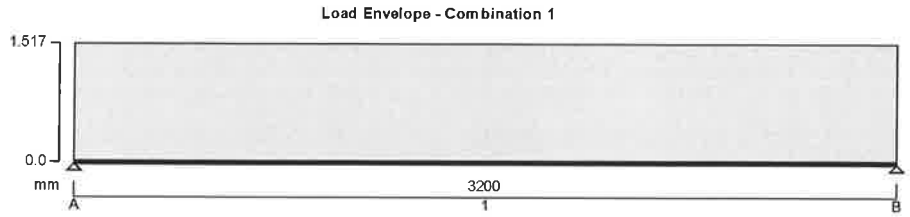
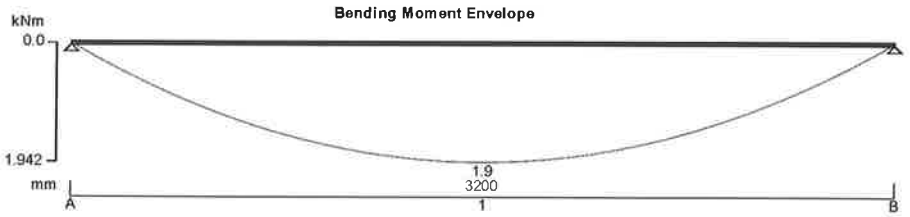
Project	Job no.
Parley Community Hub - 275 Christchurch Road, West	2024.159
Calcs for: TJ1	Start page no./Revision: 3

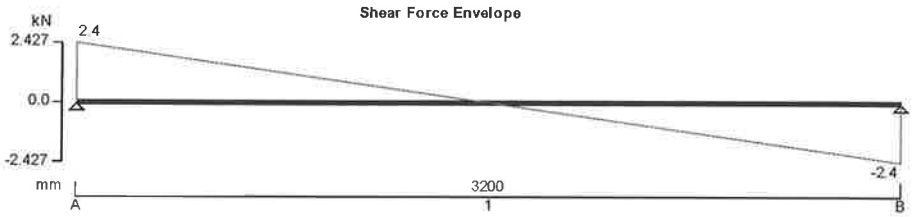
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
G.B.	03/07/2024				

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.5.07







Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 0.530 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} = **1.942** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **1.942** kNm	
Maximum shear	F_{max} = **2.427** kN	F_{min} = **-2.427** kN

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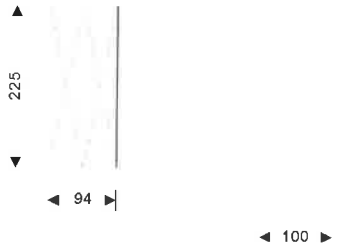
Project				Job no.	
Parley Community Hub - 275 Christchurch Road, West				2024.159	
Calcs for				Start page no./Revision	
TJ1				4	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
G.B.	03/07/2024				

Design shear	$F = \max(\text{abs}(F_{max}),\text{abs}(F_{min})) =$ **2.427** kN	
Total load on beam	$W_{tot} =$ **4.855** kN	
Reactions at support A	$R_{A\_max} =$ **2.427** kN	$R_{A\_min} =$ **2.427** kN
Unfactored dead load reaction at support A	$R_{A\_Dead} =$ **0.987** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed} =$ **1.440** kN	
Reactions at support B	$R_{B\_max} =$ **2.427** kN	$R_{B\_min} =$ **2.427** kN
Unfactored dead load reaction at support B	$R_{B\_Dead} =$ **0.987** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed} =$ **1.440** kN	



Timber section details

Breadth of sections	$b =$ **47** mm
Depth of sections	$h =$ **225** 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	**2**
Load duration	**Medium term**
Length of bearing	$L_b =$ **100** mm

Section properties

Cross sectional area of member	$A = N \times b \times h =$ **21150** mm^2
Section modulus	$Z_x = N \times b \times h^2 / 6 =$ **793125** mm^3
	$Z_y = h \times (N \times b)^2 / 6 =$ **331350** mm^3
Second moment of area	$I_x = N \times b \times h^3 / 12 =$ **89226562** mm^4
	$I_y = h \times (N \times b)^3 / 12 =$ **15573450** mm^4
Radius of gyration	$i_x = \sqrt{(I_x / A)} =$ **65.0** mm
	$i_y = \sqrt{(I_y / A)} =$ **27.1** 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	$K_7 = (300 \text{ mm} / h)^{0.11} =$ **1.03**
Load sharing - cl.2.10.11	$K_8 =$ **1.10**
Minimum modulus of elasticity - Table 20	$K_9 =$ **1.14**

Lateral support - cl.2.10.8

Ends held in position	
Permissible depth-to-breadth ratio - Table 19	**3.00**
Actual depth-to-breadth ratio	$h / (N \times b) =$ **2.39**

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PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 =$ **3.300** N/mm²

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b) =$ **0.258** N/mm²

$\sigma_{c\_a} / \sigma_{c\_adm} =$ **0.078**

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

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 =$ **10.644** N/mm²

Applied bending stress $\sigma_{m\_a} = M / Z_x =$ **2.448** N/mm²

$\sigma_{m\_a} / \sigma_{m\_adm} =$ **0.230**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8 =$ **0.976** N/mm²

Applied shear stress $\tau_a = 3 \times F / (2 \times A) =$ **0.172** N/mm²

$\tau_a / \tau_{adm} =$ **0.176**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min} \times K_9 =$ **8208** N/mm²

Permissible deflection $\delta_{adm} = \min(14\text{ mm}, 0.003 \times L_{s1}) =$ **9.600** mm

Bending deflection $\delta_{b\_s1} =$ **2.828** mm

Shear deflection $\delta_{v\_s1} =$ **0.215** mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} =$ **3.043** mm

$\delta_a / \delta_{adm} =$ **0.317**

PASS - Total deflection is less than permissible deflection

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Contract: Poole Community Hub
275 Christchurch Road
West Poole

Job No. 2024.159

Sht. No. 6

Part of Structure

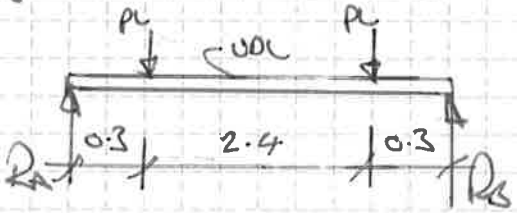
Date July '24

	D.L.	L.L.	Total

TJ2

Design of trimmer joist to support TJ1.

Span = 3.0 m.



Load on joists:

		D.L.	L.L.
		(kN)	(kN)
UDL: Flat Roof :	DL 1.15 x 0.4	0.46	
	LL 0.75 x 0.4		0.30
		(kN)	(kN)
Pt.Ld.: TJ1 (Page 4):	DL	0.99	
	LL		1.44

See computer printout for joist analysis (page 7-9)

∴ Provide 2No. 225x47 C24 joist bolted together at 600 ctrs. ctrs. (2No. bolts at bearings)

(By observation, provide some to small roof lights over New office & kitchen).

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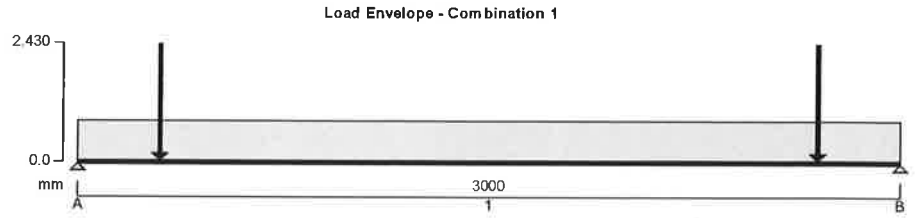
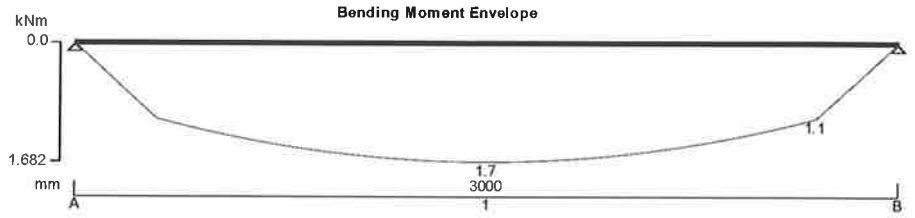
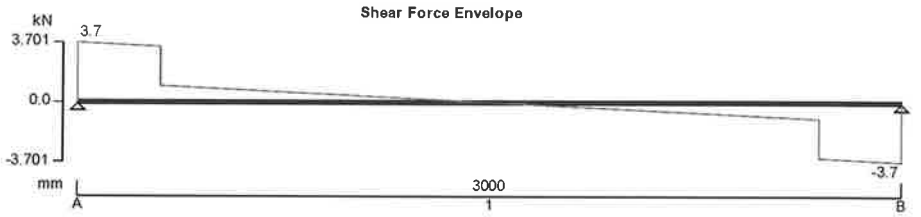
Project				Job no.	
Parley Community Hub - 275 Christchurch Road, West				2024.159	
Calcs for				Start page no./Revision	
TJ2				7	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
G.B.	03/07/2024				

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.5.07







Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 0.460 kN/m
Imposed full UDL 0.300 kN/m
Dead point load 0.990 kN at 300 mm
Imposed point load 1.440 kN at 300 mm
Dead point load 0.990 kN at 2700 mm
Imposed point load 1.440 kN at 2700 mm

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

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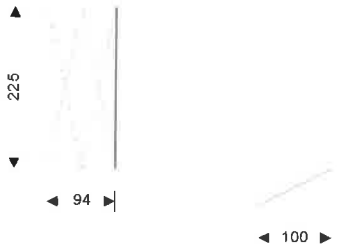
Project	Job no.
Parley Community Hub - 275 Christchurch Road, West	2024.159
Calcs for: TJ2	Start page no./Revision: 8

Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
G.B.	03/07/2024				

Analysis results

Maximum moment	M_{max} = **1.682** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **1.682** kNm	
Maximum shear	F_{max} = **3.701** kN	F_{min} = **-3.701** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **3.701** kN	
Total load on beam	W_{tot} = **7.401** kN	
Reactions at support A	R_{A_max} = **3.701** kN	R_{A_min} = **3.701** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **1.811** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **1.890** kN	
Reactions at support B	R_{B_max} = **3.701** kN	R_{B_min} = **3.701** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **1.811** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **1.890** kN	



Timber section details

Breadth of sections	b = **47** mm
Depth of sections	h = **225** mm
Number of sections in member	N = **2**
Overall breadth of member	b_b = N × b = **94** mm
Timber strength class	**C24**

Member details

Service class of timber	**2**
Load duration	**Medium term**
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of member	A = N × b × h = **21150** mm^2
Section modulus	Z_x = N × b × h^2 / 6 = **793125** mm^3
	Z_y = h × (N × b)^2 / 6 = **331350** mm^3
Second moment of area	I_x = N × b × h^3 / 12 = **89226562** mm^4
	I_y = h × (N × b)^3 / 12 = **15573450** mm^4
Radius of gyration	i_x = √(I_x / A) = **65.0** mm
	i_y = √(I_y / A) = **27.1** 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 mm / h)$^{0.11}$ = **1.03**
Load sharing - cl.2.10.11	K_8 = **1.10**
Minimum modulus of elasticity - Table 20	K_9 = **1.14**

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Lateral support - cl.2.10.8

Ends held in position

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

Actual depth-to-breadth ratio $h / (N \times b) =$ **2.39**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 =$ **3.300** N/mm²

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b) =$ **0.394** N/mm²

$\sigma_{c\_a} / \sigma_{c\_adm} =$ **0.119**

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

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 =$ **10.644** N/mm²

Applied bending stress $\sigma_{m\_a} = M / Z_x =$ **2.121** N/mm²

$\sigma_{m\_a} / \sigma_{m\_adm} =$ **0.199**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8 =$ **0.976** N/mm²

Applied shear stress $\tau_a = 3 \times F / (2 \times A) =$ **0.262** N/mm²

$\tau_a / \tau_{adm} =$ **0.269**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min} \times K_9 =$ **8208** N/mm²

Permissible deflection $\delta_{adm} = \min(14 \text{ mm}, 0.003 \times L_{s1}) =$ **9.000** mm

Bending deflection $\delta_{b\_s1} =$ **2.325** mm

Shear deflection $\delta_{v\_s1} =$ **0.186** mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} =$ **2.511** mm

$\delta_a / \delta_{adm} =$ **0.279**

PASS - Total deflection is less than permissible deflection

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Contract: Parley Community Hub
275 Christchurch Road
West Parley

Job No. 2024.159

Sht. No. 10

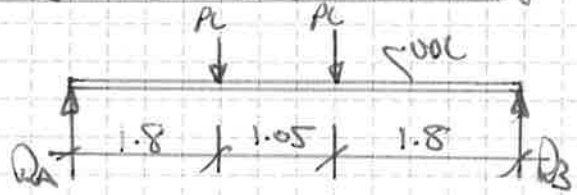
Part of Structure

Date July '24

	D.L.	L.L.	Total

T-J3

Design of trimmer joists over right hand bay.



Span = 4.85m

Load on joists:

		D.L.	L.L.
		(kN/m)	(kN/m)
UDL: Flat Roof	dl 1.15×0.4	0.46	
	ll 0.75×0.4		0.30
		(kN)	(kN)
Pt. Ld.: TJ1; Flat Roof	dl $(1.15 \times 1.8/2) \times 3.2/2$	1.66	
	ll $(0.75 \times 1.8/2) \times 3.2/2$		1.08
Roof light	dl $(0.3 \times 1.0/2) \times 3.2/2$	0.24	
	ll $(0.75 \times 1.0/2) \times 3.2/2$		0.60
		1.90	1.68

See computer printout for joist analysis (pages 11-14).

∴ Provide Timber Flitch Beam, as shown on page 12

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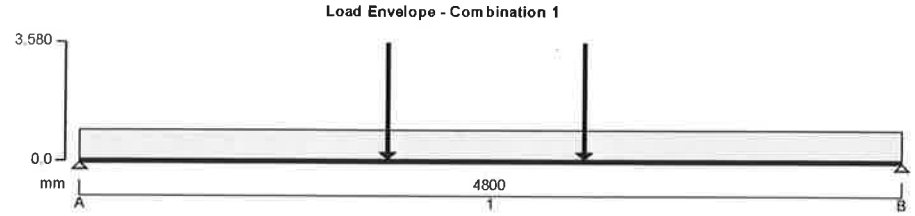
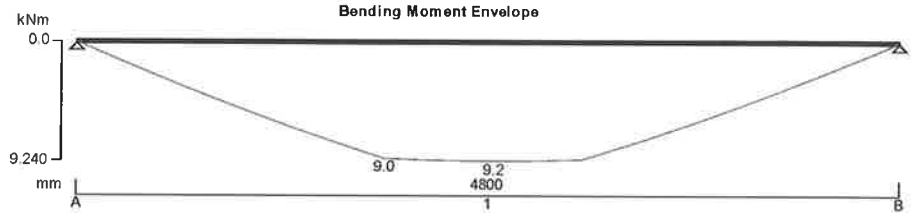
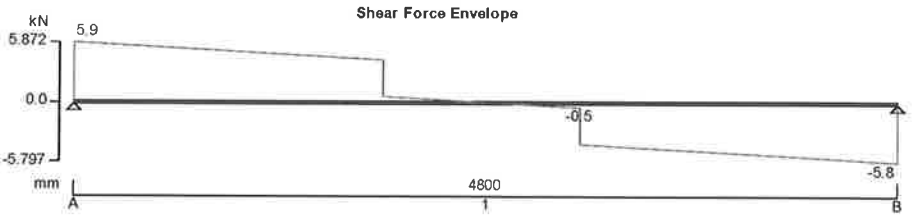
Project				Job no.	
Parley Community Hub - 275 Christchurch Road, West				2024.159	
Calcs for				Start page no./Revision	
TJ3				11	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
G.B.	01/07/2024				

FLITCH BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.5.07







Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 0.460 kN/m
Imposed full UDL 0.300 kN/m
Dead point load 1.900 kN at 1800 mm
Imposed point load 1.680 kN at 1800 mm
Dead point load 1.900 kN at 2950 mm
Imposed point load 1.680 kN at 2950 mm

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

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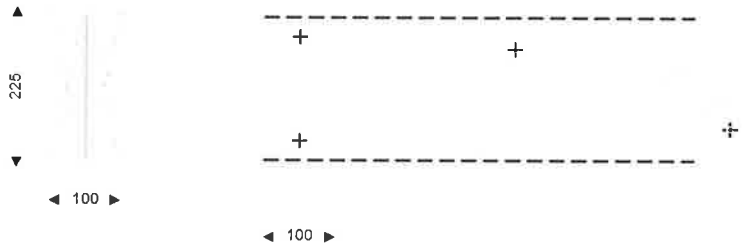
Project				Job no.	
Parley Community Hub - 275 Christchurch Road, West				2024.159	
Calcs for				Start page no./Revision	
TJ3				12	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
G.B.	01/07/2024				

Analysis results

Maximum moment	M_{max} = **9.240** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **9.240** kNm	
Maximum shear	F_{max} = **5.872** kN	F_{min} = **-5.797** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **5.872** kN	
Total load on beam	W_{tot} = **11.670** kN	
Reactions at support A	R_{A_max} = **5.872** kN	R_{A_min} = **5.872** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **3.455** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **2.418** kN	
Reactions at support B	R_{B_max} = **5.797** kN	R_{B_min} = **5.797** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **3.415** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **2.383** kN	



Timber section details

Breadth of timber sections	b = **47** mm
Depth of timber sections	h = **225** mm
Number of timber sections in member	N = **2**
Timber strength class	**C24**

Steel section details

Breadth of steel plate	b_s = **6** mm
Depth of steel plate	h_s = **200** mm
Number of steel plates in beam	N_s = **1**
Steel stress	p_y = **280** N/mm^2
Bolt diameter	ϕ_b = **10** mm

Member details

Service class of timber	**2**
Load duration	**Medium term**
Length of bearing	L_b = **100** mm

Section properties

Cross sectional area of beam	A = N × b × h = **21150** mm^2
Timber section modulus	Z_{xt} = N × b × h^2 / 6 = **793125** mm^3
Steel section modulus	Z_{xs} = N_s × b_s × h_s^2 / 6 = **40000** mm^3
Second moment of area of timber	I_{xt} = N × b × h^3 / 12 = **89226562** mm^4
Second moment of area of steel	I_{xs} = N_s × b_s × h_s^3 / 12 = **4000000** mm^4

Load proportions

Instant deflection under permanent actions	u_{instG} = **6.997** mm
Instant deflection under principal variable action	u_{instQ1} = **5.158** mm

k_{def} = **0.8**

ψ_2 = **0.3**

Final minimum modulus of elasticity

$E_{min,fin} = E_{min} \times (u_{instG} + u_{instQ1}) / (u_{instG} + u_{instQ1} + k_{def} \times (u_{instG} + \psi_2 \times u_{instQ1}))$ = **4608** N/mm²

Proportion of applied load in timber — $k_t = E_{mean} \times I_{xt} / (E_{mean} \times I_{xt} + E_{S5950} \times I_{xs})$ = **0.540**

Proportion of applied load in steel — $k_s = 1.1 \times E_{S5950} \times I_{xs} / (E_{min,fin} \times I_{xt} + E_{S5950} \times I_{xs})$ = **0.733**

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 — $K_7 = (300 \text{ mm} / h)^{0.11}$ = **1.03**

Load sharing - cl.2.10.11 — K_8 = **1.10**

Minimum modulus of elasticity - Table 20 — K_9 = **1.14**

Lateral support - cl.2.10.8

Ends held in position

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

Actual depth-to-breadth ratio — $h / (N \times b + N_s \times b_s)$ = **2.25**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) — $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **3.300** N/mm²

Applied bearing stress — $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b)$ = **0.625** N/mm²

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.189**

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

Bending parallel to grain

Permissible bending stress — $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **10.644** N/mm²

Applied timber bending stress — $\sigma_{m\_a} = k_t \times M / Z_{xt}$ = **6.294** N/mm²

$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.591**

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

Applied steel bending stress — $\sigma_{m\_a\_s} = k_s \times M / Z_{xs}$ = **169.235** N/mm²

$\sigma_{m\_a\_s} / p_y$ = **0.604**

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

Check beam in shear

Permissible shear stress — $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.976** N/mm²

Applied shear stress — $\tau_a = 3 \times k_t \times F / (2 \times A)$ = **0.225** N/mm²

τ_a / τ_{adm} = **0.230**

PASS - Shear stress within permissible limits

Deflection

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

Permissible deflection — $\delta_{adm} = \min(14 \text{ mm}, 0.003 \times L_{s1})$ = **14.000** mm

Bending deflection — δ_{b_s1} = **12.156** mm

Shear deflection — δ_{v_s1} = **0.777** mm

Total deflection — $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **12.932** mm

δ_a / δ_{adm} = **0.924**

PASS - Total deflection is less than permissible deflection

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Flitch plate bolting requirements

Total load on beam	W_{tot} = **11.670** kN
Total load taken by steel	$W_s = k_s \times W_{tot}$ = **8.549** kN
Basic bolt shear load - Table 71	v_{90} = **1.940** kN
Number of interfaces	$N_{int} = (N + N_s) - 1$ = **2**
Number of bolts required at supports	$N_{be} = \max(k_s \times R_{A\_max} / (N_{int} \times v_{90}), 2)$ = **2**
Limiting bolt spacing	$S_{limit} = \min(2.5 \times h, 600\ mm)$ = **563** mm
Maximum bolt spacing	S_{max} = **300** mm
Minimum number of bolts along length of beam	$N_{bl} = W_s / (N_{int} \times v_{90})$ = **2.203**

- Provide a minimum of 2 No.10 mm diameter bolts at each support
- Provide 10 mm diameter bolts at maximum 300 mm centres staggered 55 mm alternately above and below the centre line

Minimum bolt spacings

Minimum end spacing	$S_{end} = 4 \times \phi_b$ = **40** mm
Minimum edge spacing	$S_{edge} = 4 \times \phi_b$ = **40** mm
Minimum bolt spacing	$S_{bolt} = 4 \times \phi_b$ = **40** mm
Minimum washer diameter	$\phi_w = 3 \times \phi_b$ = **30** mm
Minimum washer thickness	$t_w = 0.25 \times \phi_b$ = **2.5** mm

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Contract: Poley Community Hub, 275 Christchurch Road, West Poley

Job No. 2024.159

Sht. No. 15

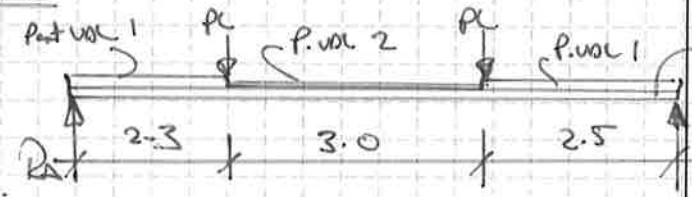
Part of Structure

Date July 2024

	D.L.	L.L.	Total

Check adequacy of existing steel beams to support trimmer joists (consider 254 x 114 RSJ)

Span = 7.8m.



Load on joists :

			D.L. (kN)	L.L. (kN)
Full wdl : Flat Roof	: dl	$1.15 \times 4.3/2$	2.47	
	ll	$0.75 \times 4.3/2$		1.61
Part wdl 1 : Flat Roof	: dl	$1.15 \times 3.0/2$	1.73	
	ll	$0.75 \times 3.0/2$		1.13
Part wdl 2 : Flat Roof	: dl	$1.15 \times 0.3/2$	0.35	
	ll	$0.75 \times 0.3/2$		0.11
Pt. Load 1 : TJ2 (Page 8)	: dl		1.81	
	ll			1.89

See computer printout for beam analysis (pages 16-18)

∴ Exist. beam are inadequate to support new loads in deflection.

∴ Check for reduced load, if we introduce a second beam.

∴ Load on new beam :

As above without the full wdl.

See computer printout for beam analysis (pages 22-24)

∴ Provide New 254 x 146 x 31 UB alongside existing beam

Also check existing beam for reduced load

Load on beam = Full wdl above only

See computer printout for beam analysis (pages 19 - 21)

∴ Existing 254 x 114 RSJ is adequate.

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Existing steel beam check	16

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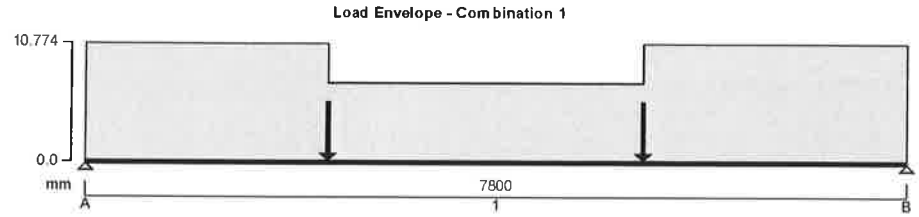
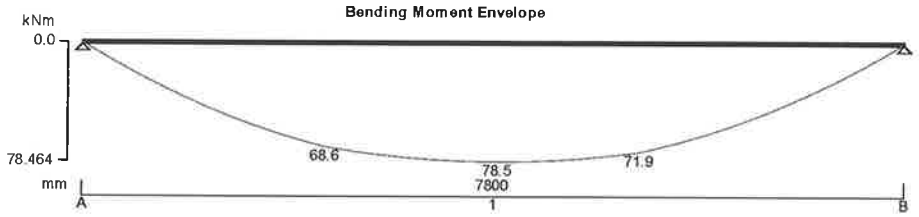
G.B.	08/07/2024				

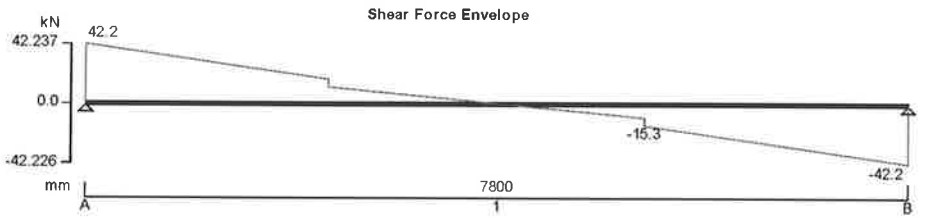
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.05







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 2.47 kN/m
Imposed full UDL 1.61 kN/m
Dead partial UDL 1.73 kN/m from 0 mm to 2300 mm
Imposed partial UDL 1.13 kN/m from 0 mm to 2300 mm
Dead partial UDL 0.35 kN/m from 2300 mm to 5300 mm
Imposed partial UDL 0.11 kN/m from 2300 mm to 5300 mm
Dead partial UDL 1.73 kN/m from 5300 mm to 7800 mm
Imposed partial UDL 1.13 kN/m from 5300 mm to 7800 mm
Dead point load 1.81 kN at 2300 mm

Imposed point load 1.89 kN at 2300 mm
Dead point load 1.81 kN at 5300 mm
Imposed point load 1.89 kN at 5300 mm

Load combinations

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

Analysis results

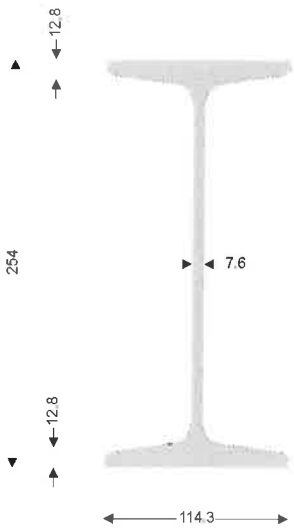
Maximum moment	M_{max} = **78.5** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **42.2** kN	V_{min} = **-42.2** kN
Deflection	δ_{max} = **33.1** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **42.2** kN	R_{A_min} = **42.2** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **17.5** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **11.1** kN	
Maximum reaction at support B	R_{B_max} = **42.2** kN	R_{B_min} = **42.2** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **17.5** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **11** kN	

Section details

Section type	**RSJ 254x114x37 (BS4-1)**
Steel grade	**S275**

From table 9: Design strength p_y

Thickness of element	max(T, t) = **12.8** mm
Design strength	p_y = **275** N/mm²
Modulus of elasticity	E = **205000** N/mm²



Lateral restraint

Span 1 has full lateral restraint

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G.B.	08/07/2024				

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

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section d = **199.3** mm

d / t = 26.2 × ε <= 80 × ε Class 1 plastic

Outstand flanges - Table 11

Width of section b = B / 2 = **57.2** mm

b / T = 4.5 × ε <= 9 × ε Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force $F_v = max(abs(V_{max}), abs(V_{min}))$ = **42.2** kN

d / t < 70 × ε

Web does not need to be checked for shear buckling

Shear area $A_v = t \times D$ = **1930** mm^2

Design shear resistance $P_v = 0.6 \times p_y \times A_v$ = **318.5** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = max(abs(M_{s1\_max}), abs(M_{s1\_min}))$ = **78.5** kNm

Moment capacity low shear - cl.4.2.5.2 $M_c = min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx})$ = **126.2** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection $\delta_{lim} = L_{s1} / 360$ = **21.667** mm

Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min}))$ = **33.083** mm

FAIL - Deflection exceeds deflection limit

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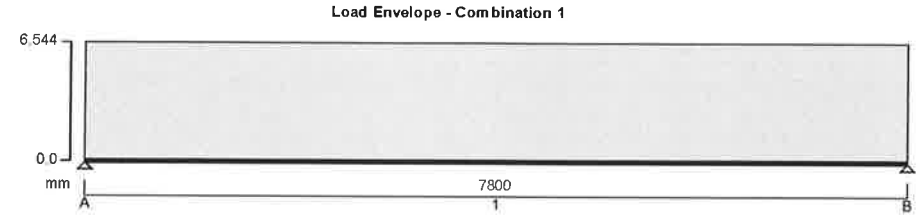
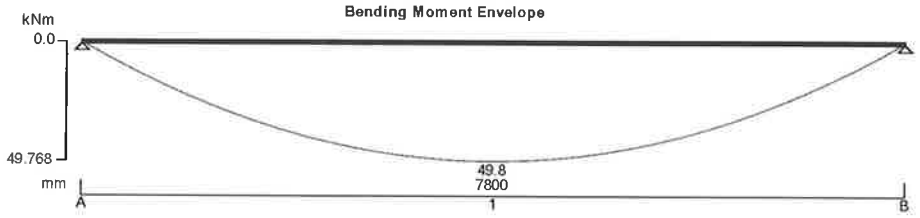
G.B.	03/07/2024				

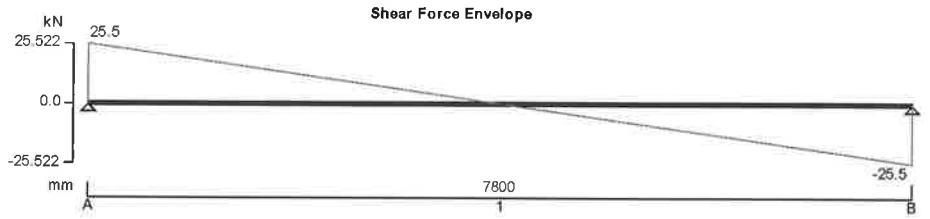
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.05







Support conditions

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

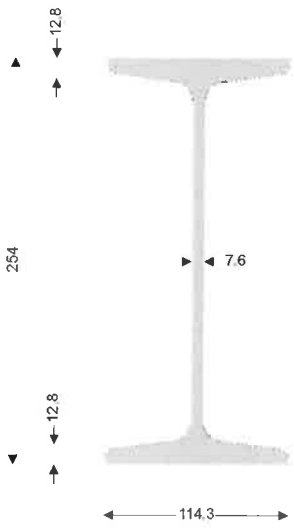
Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 2.47 kN/m
	Imposed full UDL 1.61 kN/m

Load combinations

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

		Imposed × 1.60
Analysis results		
Maximum moment	M_{max} = **49.8** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **25.5** kN	V_{min} = **-25.5** kN
Deflection	δ_{max} = **20.6** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **25.5** kN	R_{A_min} = **25.5** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **11.1** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **6.3** kN	
Maximum reaction at support B	R_{B_max} = **25.5** kN	R_{B_min} = **25.5** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **11.1** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **6.3** kN	
Section details		
Section type	**RSJ 254x114x37 (BS4-1)**	
Steel grade	**S275**	
From table 9: Design strength p_y		
Thickness of element	max(T, t) = **12.8** mm	
Design strength	p_y = **275** N/mm²	
Modulus of elasticity	E = **205000** N/mm²	



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** + 2 × D	
Classification of cross sections - Section 3.5		
	$\varepsilon = \sqrt{[275\ N/mm^2 / p_y]}$ = **1.00**	
Internal compression parts - Table 11		
Depth of section	d = **199.3** mm	
	d / t = 26.2 × ε <= 80 × ε	Class 1 plastic

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Outstand flanges - Table 11

Width of section

$b = B / 2 =$ **57.2** mm

$b / T = 4.5 \times \varepsilon <= 9 \times \varepsilon$ Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) =$ **25.5** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area

$A_v = t \times D =$ **1930** mm^2

Design shear resistance

$P_v = 0.6 \times p_y \times A_v =$ **318.5** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) =$ **49.8** kNm

Moment capacity low shear - cl.4.2.5.2

$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) =$ **126.2** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection

$\delta_{lim} = L_{s1} / 360 =$ **21.667** mm

Maximum deflection span 1

$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **20.562** mm

PASS - Maximum deflection does not exceed deflection limit

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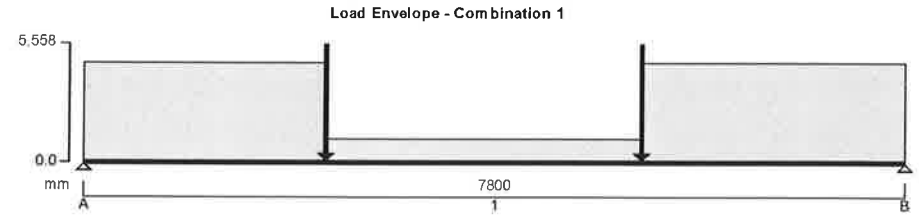
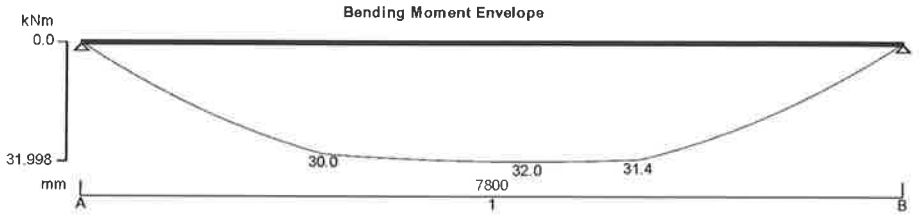
G.B.	08/07/2024				

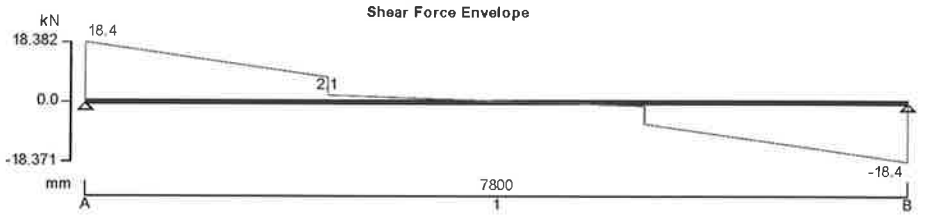
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.05







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 partial UDL 1.73 kN/m from 0 mm to 2300 mm
	Imposed partial UDL 1.13 kN/m from 0 mm to 2300 mm
	Dead partial UDL 0.35 kN/m from 2300 mm to 5300 mm
	Imposed partial UDL 0.11 kN/m from 2300 mm to 5300 mm
	Dead partial UDL 1.73 kN/m from 5300 mm to 7800 mm
	Imposed partial UDL 1.13 kN/m from 5300 mm to 7800 mm
	Dead point load 1.81 kN at 2300 mm
	Imposed point load 1.89 kN at 2300 mm
	Dead point load 1.81 kN at 5300 mm

Imposed point load 1.89 kN at 5300 mm

Load combinations

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

Analysis results

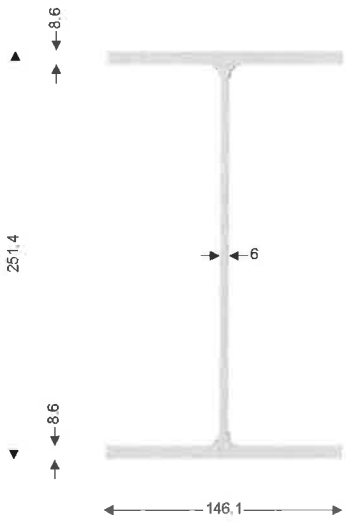
Maximum moment	M_{max} = **32** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **18.4** kN	V_{min} = **-18.4** kN
Deflection	δ_{max} = **16** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **18.4** kN	R_{A_min} = **18.4** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **7.7** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **4.8** kN	
Maximum reaction at support B	R_{B_max} = **18.4** kN	R_{B_min} = **18.4** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **7.7** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **4.8** kN	

Section details

Section type	**UB 254x146x31 (BS4-1)**
Steel grade	**S275**

From table 9: Design strength p_y

Thickness of element	max(T, t) = **8.6** 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**

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Project				Job no.	
Parley Community Hub - 275 Christchurch Road, West				2024.159	
Calcs for				Start page no./Revision	
New SB1				24	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
G.B.	08/07/2024				

Effective length factor for lateral-torsional buckling $K_{LT,A}$ = **1.00** + 2 × D

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section

d = **219** mm

d / t = 36.5 × ε <= 80 × ε Class 1 plastic

Outstand flanges - Table 11

Width of section

b = B / 2 = **73.1** mm

b / T = 8.5 × ε <= 9 × ε Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$F_v = max(abs(V_{max}), abs(V_{min}))$ = **18.4** kN

d / t < 70 × ε

Web does not need to be checked for shear buckling

Shear area

$A_v = t \times D$ = **1508** mm^2

Design shear resistance

$P_v = 0.6 \times p_y \times A_v$ = **248.9** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$M = max(abs(M_{s1\_max}), abs(M_{s1\_min}))$ = **32** kNm

Moment capacity low shear - cl.4.2.5.2

$M_c = min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx})$ = **108.1** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection

$\delta_{lim} = L_{s1} / 360$ = **21.667** mm

Maximum deflection span 1

$\delta = max(abs(\delta_{max}), abs(\delta_{min}))$ = **16.044** mm

PASS - Maximum deflection does not exceed deflection limit

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Contract: Parley Community Hub
275 Christchurch Road
West Parley

Job No. 2024.159

Sht. No. 25

Part of Structure

Date July 2024

Check reactions of existing beam & loads against new beam & existing (reduced load) beam to determine if supporting beams & columns need reassessing.

		D.L.	L.L.	Total
Check existing beam for existing loads:		(kN)	(kN)	
Full UDL: Flat Roof:	DL 1.15 × 7.2/2	4.14		
	LL 0.75 × 7.2/2		2.70	
SW of Beam:	DL	0.37		
Reaction at supporting beam:		(kN)	(kN)	
RA = RB:	DL (4.14 + 0.37) × 7.8/2	17.59		
	LL 2.7 × 7.8/2		10.53	28.12
Reaction from reduced load beam:		(kN)	(kN)	
(Page 20)	DL	11.10		
	LL		6.30	
Reaction from new steel beam:				
(Page 23)	DL	6.10		
	LL		3.70	
		17.20	10.00	27.20

∴ Extg. reactions are greater than proposed therefore no further checks are required. (i.e. 28.12kN > 27.20kN)

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Contract: Parley Community Hub, 275 Christchurch Road, West Parley

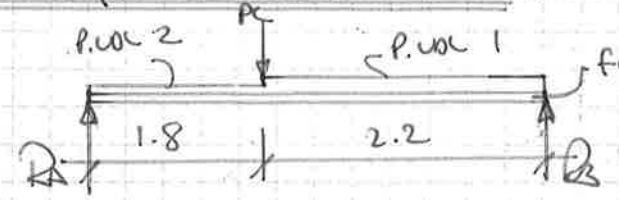
Job No. 2024.159

Sht. No. 26

Part of Structure

Date July 2024

Check existing beam for new loads over Bar.



Span = 4.0m.

Load on beam:

		D.L.	L.L.	Total
		(kN/m)	(kN/m)	
full UDL : Flat Roof :	DL 1.15 × 4.1/2	2.35		
	LL 0.75 × 4.1/2		1.54	
Part UDL 1 : Flat Roof :	DL 1.15 × 4.8/2	2.76		
	LL 0.75 × 4.8/2		1.80	
Part UDL 2 : Flat Roof :	DL 1.15 × 1.9/2	1.09		
	LL 0.75 × 1.9/2		0.71	
		(kN)	(kN)	
Point Load : T.S2 (Page 12) :	DL	3.46		
	LL		2.42	
		(kN)	(kN)	

R_A = DL $(2.35 \times 4.0/2) + (2.76 \times 2.2 \times 1.1/4.0) + (1.09 \times 1.8 \times 3.1/4.0) + (3.46 \times 2.2/4.0)$ = 9.79

LL $(1.54 \times 4.0/2) + (1.8 \times 2.2 \times 1.1/4.0) + (0.71 \times 1.8 \times 3.1/4.0) + (2.42 \times 2.2/4.0)$ = 6.49

R_B = DL $(2.35 \times 4.0/2) + (2.76 \times 2.2 \times 2.9/4.0) + (1.09 \times 1.8 \times 0.9/4.0) + (3.46 \times 1.8/4.0)$ = 11.10

LL $(1.54 \times 4.0/2) + (1.8 \times 2.2 \times 2.9/4.0) + (0.71 \times 1.8 \times 0.9/4.0) + (2.42 \times 1.8/4.0)$ = 7.32

		(kNm)	(kNm)	
BM max.	DL $(11.1 \times 2.18) - ((2.35+2.76) \times 2.18 \times 2.18/2)$	12.06		
	LL $(7.32 \times 2.18) - ((1.54+1.8) \times 2.18 \times 2.18/2)$		8.02	

Equiv UDL = $(12.06 + 8.02) \times 8/4^2$ = 10.04 kN/m

∴ Min I req'd = $\dfrac{5 \times 10.04 \times 4^3 \times 355 \times 10^3}{384 \times 5800}$ = 480.4 cm⁴

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Contract: Parley Community Hub, 275 Christchurch Road, West Parley

Job No. 2024.159

Sht. No. 27

Part of Structure

Date July 2024

	D.L.	L.L.	Total
Check design of exiting timber beam over Bar.			
Span = 4.0m.			
Load on beam: (IGNORE ANY 1ST FLOOR LOADS).	(kN/m)	(kN/m)	
Flat Roof : DL 1.15 × 9.0/2	5.18		
LL 0.75 × 9.0/2		3.38	
$R_A = R_B$: DL 5.18 × 4.0/2	(kN) 10.36	(kN)	
LL 3.38 × 4.0/2		6.76	
BMmax. : DL 5.18 × 4.0²/8	(kNm) 10.36	(kNm)	
LL 3.38 × 4.0²/8		6.76	
min I req'd : DL+LL $\frac{5(5.18+3.38)\times 4.0^3 \times 333\times 10^3}{384\times 5800} = 409.6\,cm^4$			
Compare against new loading situation for new roof light.			
on previous page:			

		Exist. Lds		New Loads	
Check reaction (on timber part)	: DL+LL	10.36+6.76 = 17.12	>	9.79+6.49 = 16.28	∴ Ok
Check BM	: DL+LL	10.36+6.76 = 17.12	<	12.06+8.02 = 20.08	∴ Design beam for additional BM.
Check I	: DL+LL	409.6cm⁴	<	480.4cm⁴	∴ Design beam for additional deflection.

∴ Strengthening or replacement of exiting beam will be required - subject to further investigation.

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Contract: Poley Community Hub
275 Christchurch Rd
West Poley

Job No. 2024.159

Sht. No. 28

Part of Structure

Date July 2024

	D.L.	L.L.	Total

Design of padstone for new beam
(to replace existing padstone).

Consider 600 x 100 x 215 dp cast padstone.

Ult. load on padstone:

(New & existing beam - Pos 17):

DL+LL = 42kN

Stress at U/S of padstone = $\frac{42 \times 10^3}{600 \times 100}$ = 0.7N/mm²

Load bearing capacity $\left(1.25 f_{bk} / \gamma_m\right)$ = $\frac{1.25 \times 3.5}{3.5}$ = 1.25N/mm² > 0.7N/mm²
∴ Ok

∴ Provide 600 x 100 x 215 dp cast padstone.

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Contract: Parley Community Hub, 275 Christchurch Road, West Parley

Job No. 2024.159

Sht. No. 29

Part of Structure

Date July '24

Check adequacy of existing flat roof joints (consider 225 x 50 mm joists at 450 mm crs.).

Span = 4.8m. (max)

Load on joints:

	D.L. (kN/m²)	L.L. (kN/m²)	Total
Max Flat Roof : DC	1.15		
" LL		0.75	

See computer printout for joint analysis (pages 30-32)

∴ Extg. 225 x 50 mm C24 joists at 450 mm crs. are adequate.

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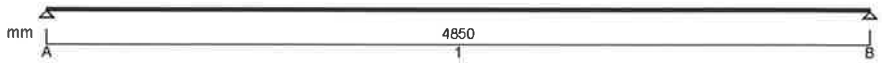
Project				Job no.	
Parley Community Centre - 275 Christchurch Road, West				2024.159	
Calcs for				Start page no./Revision	
Existing flat roof joists				30	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
G.B.	04/07/2024				

TIMBER JOIST DESIGN (BS5268-2:2002)

TEDDS calculation version 1.1.02

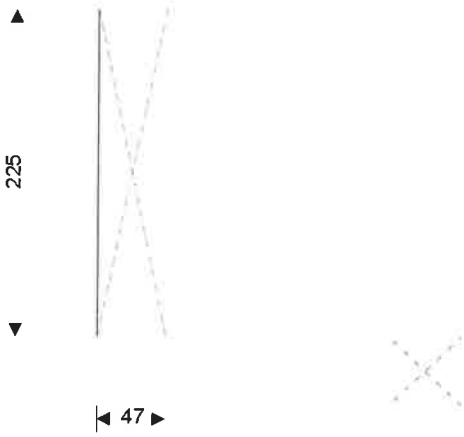
Joist details

Joist breadth	b = **47** mm
Joist depth	h = **225** mm
Joist spacing	s = **450** mm
Timber strength class	**C24**
Service class of timber	**2**



Span details

Number of spans	N_{span} = **1**
Length of bearing	L_b = **50** mm
Effective length of span	L_{s1} = **4850** mm



Section properties

Second moment of area	$I = b \times h^3 / 12$ = **44613281** mm^4
Section modulus	$Z = b \times h^2 / 6$ = **396563** mm^3

Loading details

Joist self weight	$F_{swt} = b \times h \times \rho_{char} \times g_{acc}$ = **0.04** kN/m
Dead load	F_{d_udl} = **1.15** 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

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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 = **1.03**
Load sharing factor	K_8 = **1.10**

Consider medium term loads

Load duration factor	K_3 = **1.25**
Maximum bending moment	M = **2.621** kNm
Maximum shear force	V = **2.161** kN
Maximum support reaction	R = **2.161** kN
Maximum deflection	δ = **13.768** mm

Check bending stress

Bending stress	σ_m = **7.500** N/mm²
Permissible bending stress	$\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **10.644** N/mm²
Applied bending stress	$\sigma_{m\_max} = M / Z$ = **6.609** N/mm²

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress	τ = **0.710** N/mm²
Permissible shear stress	$\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.976** N/mm²
Applied shear stress	$\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.307** N/mm²

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane)	σ_{cp1} = **2.400** N/mm²
Permissible bearing stress	$\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **3.300** N/mm²
Applied bearing stress	$\sigma_{c\_max} = R / (b \times L_b)$ = **0.920** N/mm²

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection	$\delta_{adm} = \min(L_{s1} \times 0.003, 14\text{ mm})$ = **14.000** mm
Bending deflection (based on E_{mean})	$\delta_{bending}$ = **13.327** mm
Shear deflection	δ_{shear} = **0.441** mm
Total deflection	$\delta = \delta_{bending} + \delta_{shear}$ = **13.768** mm

PASS - Actual deflection within permissible limits

Consider short term loads

Load duration factor	K_3 = **1.50**
Maximum bending moment	M = **2.720** kNm
Maximum shear force	V = **2.243** kN
Maximum support reaction	R = **2.243** kN
Maximum deflection	δ = **13.177** mm

Check bending stress

Bending stress	σ_m = **7.500** N/mm²
Permissible bending stress	$\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **12.773** N/mm²

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Applied bending stress — $\sigma_{m\_max} = M / Z =$ **6.858** N/mm²

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress — $\tau =$ **0.710** N/mm²

Permissible shear stress — $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 =$ **1.172** N/mm²

Applied shear stress — $\tau_{max} = 3 \times V / (2 \times b \times h) =$ **0.318** N/mm²

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane) — $\sigma_{cp1} =$ **2.400** N/mm²

Permissible bearing stress — $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 =$ **3.960** N/mm²

Applied bearing stress — $\sigma_{c\_max} = R / (b \times L_b) =$ **0.954** N/mm²

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection — $\delta_{adm} = \min(L_{s1} \times 0.003, 14\text{ mm}) =$ **14.000** mm

Bending deflection (based on E_{mean}) — $\delta_{bending} =$ **12.720** mm

Shear deflection — $\delta_{shear} =$ **0.457** mm

Total deflection — $\delta = \delta_{bending} + \delta_{shear} =$ **13.177** mm

PASS - Actual deflection within permissible limits