



The Green Room
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Name of caravan: East Lodge

Job No.: 25-010

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Date: 17/11/25

Address:

Loading

Dead

Live

Roof:

SIP's with timber	=	0.25 kN/m ²
10mm gravel top coat	=	0.21 kN/m ²
Plasterboard	=	0.10 kN/m ²
Firestone EPDM	=	0.02 kN/m ²

TOTAL = 0.58 kN/m²

Imposed	=	0.75 kN/m ²
<u>TOTAL</u>	<u>=</u>	<u>0.75 kN/m²</u>

Floor:

SIP's with timber	=	0.25 kN/m ²
Flooring	=	0.10 kN/m ²

TOTAL = 0.37 kN/m²

Imposed loading	=	1.5 kN/m ²
Partition	=	0.5 kN/m ²

TOTAL = 2.0 kN/m²

External walls:

SIP's with timber	=	0.25 kN/m ²
Plasterboard	=	0.10 kN/m ²
Cedral cladding	=	0.19 kN/m ²

TOTAL = 0.54 kN/m²

Internal walls:

Timber construction	=	0.10 kN/m ²
Plasterboard 2 sides	=	0.20 kN/m ²

TOTAL = 0.30 kN/m²



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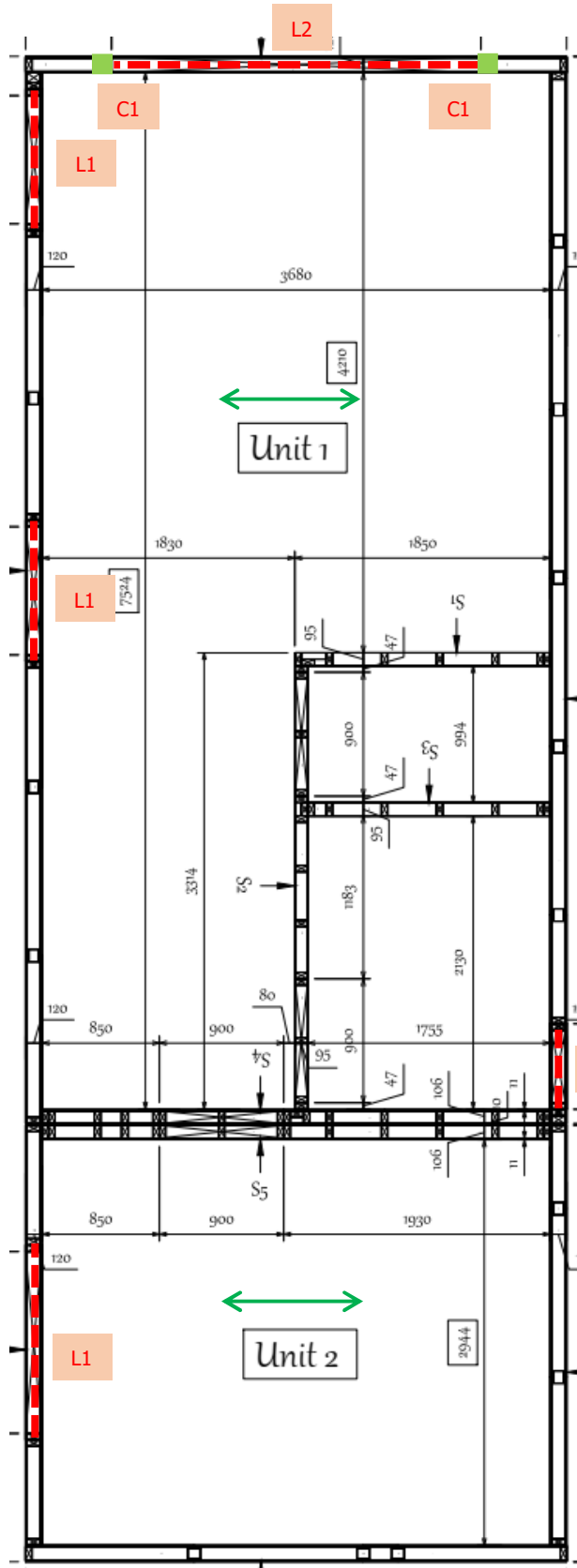
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Ground Floor Plan



Span of Sip Panel Floor & Roof

Min. Structural member sizes.

L1	120 SIP Box min 200mm height
L2	Min. double 47x220 C24 lintel nailed and glued
C1	Min 2No. 47x95 C24 timber glued & nailed



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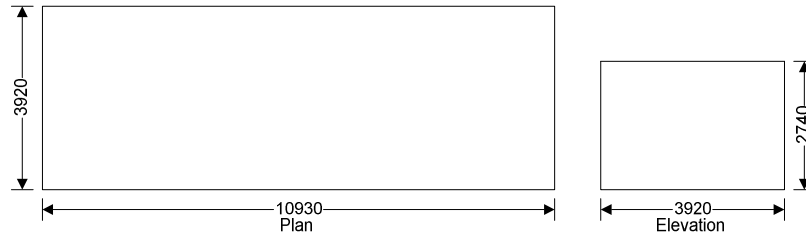
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Wind Load

WIND LOADING (BS6399)

In accordance with BS6399



Building data

Type of roof	Flat
Length of building	L = 10930 mm
Width of building	W = 3920 mm
Height to eaves	H = 2740 mm
Eaves type	Sharp
Reference height	H _r = 2740 mm

Dynamic classification

Building type factor (Table 1)	K _b = 1.0
Dynamic augmentation factor (1.6.1)	C _r = [K _b × (H _r / (0.1 m)) ^{0.75}] / (800 × log(H _r / (0.1 m))) = 0.01

Site wind speed

Location	unknown
Basic wind speed (Figure 6 BS6399:Pt 2)	V _b = 23.0 m/s
Site altitude	Δ _s = 20 m
Upwind distance from sea to site	d _{sea} = 50 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 × Δ _s / 1m = 1.02
Site wind speed	V _s = V _b × S _a × S _d × S _s × S _p = 23.5 m/s
Terrain category	Sea
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} = 2740mm
Effective height	H _e = max(H _{ref} - H _d , 0.4 × H _{ref}) = 2740mm
Fetch factor (Table 22)	S _c = 0.791
Turbulence factor (Table 22)	S _t = 0.209
Gust peak factor	g _t = 3.44
Terrain and building factor	S _b = S _c × (1 + (g _t × S _t) + S _h) = 1.36



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Effective wind speed

$$V_e = V_s \times S_b = \mathbf{31.9 \text{ m/s}}$$

Dynamic pressure

$$q_s = 0.613 \text{ kg/m}^3 \times V_e^2 = \mathbf{0.624 \text{ kN/m}^2}$$

Dynamic pressure - windward wall - Wind 90 deg and roof

Reference height (at which q is sought)

$$H_{ref} = \mathbf{2740 \text{ mm}}$$

Effective height

$$H_e = \max(H_{ref} - H_d, 0.4 \times H_{ref}) = \mathbf{2740 \text{ mm}}$$

Fetch factor (Table 22)

$$S_c = \mathbf{0.791}$$

Turbulence factor (Table 22)

$$S_t = \mathbf{0.209}$$

Gust peak factor

$$g_t = \mathbf{3.44}$$

Terrain and building factor

$$S_b = S_c \times (1 + (g_t \times S_t) + S_h) = \mathbf{1.36}$$

Effective wind speed

$$V_e = V_s \times S_b = \mathbf{31.9 \text{ m/s}}$$

Dynamic pressure

$$q_s = 0.613 \text{ kg/m}^3 \times V_e^2 = \mathbf{0.624 \text{ kN/m}^2}$$

Size effect factors

Diagonal dimension for gablewall

$$a_{eg} = \mathbf{5.2 \text{ m}}$$

External size effect factor gablewall

$$C_{aeg} = \mathbf{0.996}$$

Diagonal dimension for side wall

$$a_{es} = \mathbf{12.8 \text{ m}}$$

External size effect factor side wall

$$C_{aes} = \mathbf{0.915}$$

Diagonal dimension for roof

$$a_{er} = \mathbf{13.3 \text{ m}}$$

External size effect factor roof

$$C_{aer} = \mathbf{0.912}$$

Room/storey volume for internal size effect factor

$$V_i = \mathbf{0.125 \text{ m}^3}$$

Diagonal dimension for internal size effect factors

$$a_i = 10 \times (V_i)^{1/3} = \mathbf{5.000 \text{ m}}$$

Internal size effect factor

$$C_{ai} = \mathbf{1.000}$$

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, $C_{pi} -0.30$, - 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)	-2.00	0.62	0.912	-0.95	1.50	-1.43
B (-ve)	-1.40	0.62	0.912	-0.61	4.49	-2.74
C (-ve)	-0.70	0.62	0.912	-0.21	23.96	-5.06
D (-ve)	-0.20	0.62	0.912	0.07	12.90	0.95

Total vertical net force

$$F_{w,v} = \mathbf{-8.27 \text{ kN}}$$

Total horizontal net force

$$F_{w,h} = \mathbf{0.00 \text{ kN}}$$



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Walls load case 1 - Wind 0, c_{pi} -0.30, - 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	-1.35	0.62	0.996	-0.65	3.00	-1.96
B	-0.82	0.62	0.996	-0.32	7.74	-2.48
w	0.81	0.62	0.915	0.65	29.95	19.53
I	-0.50	0.62	0.915	-0.10	29.95	-2.94

Overall loading

Equiv leeward net force for overall section

$$F_l = F_{w,wl} = -2.9 \text{ kN}$$

Net windward force for overall section

$$F_w = F_{w,ww} = 19.5 \text{ kN}$$

Overall loading overall section

$$F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h}) = 19.3 \text{ kN}$$

Roof load case 2 - Wind 90, c_{pi} -0.30, - 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)	-2.00	0.62	0.912	-0.95	0.77	-0.73
B (-ve)	-1.40	0.62	0.912	-0.61	0.77	-0.47
C (-ve)	-0.70	0.62	0.912	-0.21	6.15	-1.30
D (-ve)	-0.20	0.62	0.912	0.07	35.16	2.58

Total vertical net force

$$F_{w,v} = 0.09 \text{ kN}$$

Total horizontal net force

$$F_{w,h} = 0.00 \text{ kN}$$

Walls load case 2 - Wind 90, c_{pi} -0.30, - 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	-1.30	0.62	0.915	-0.56	2.15	-1.19
B	-0.80	0.62	0.915	-0.27	8.59	-2.32
C	-0.50	0.62	0.915	-0.10	19.21	-1.89
w	0.60	0.62	0.996	0.56	10.74	6.02
I	-0.50	0.62	0.996	-0.12	10.74	-1.33

Overall loading

Equiv leeward net force for overall section

$$F_l = F_{w,wl} = -1.3 \text{ kN}$$

Net windward force for overall section

$$F_w = F_{w,ww} = 6.0 \text{ kN}$$

Overall loading overall section

$$F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h}) = 6.3 \text{ kN}$$



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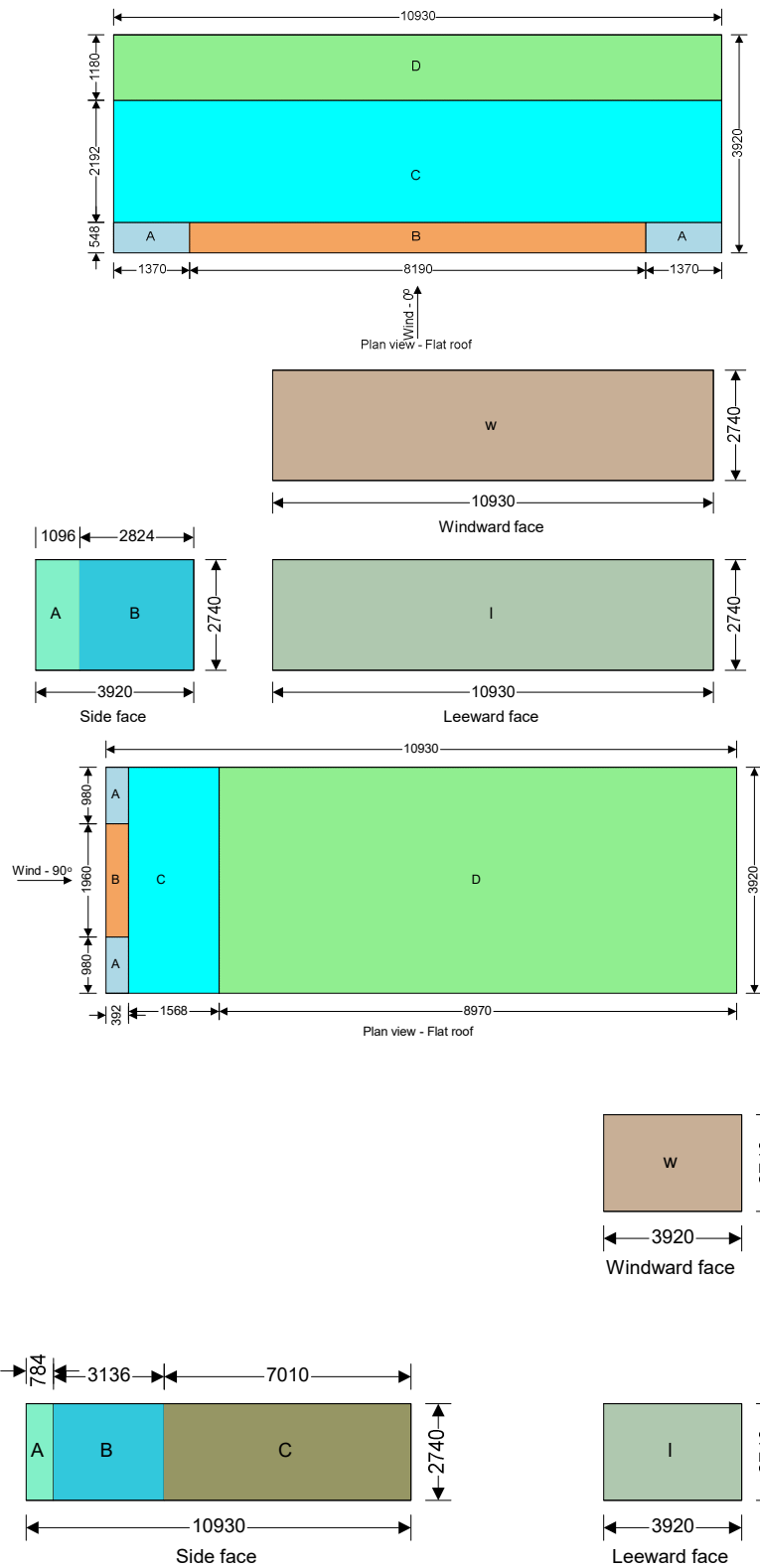
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Sip Design



Comparable OSB & Plywood Properties are set out below. For simplicity, adopt properties of Canadian Douglas Fir Ply (DFP) in calculations and factor accordingly i.e. OSB properties approx. $270/420\% = 64.3\%$ of that DFP. Therefore, 11.5mm OSB utilised which equates to $11.5 \times 64.3\% = 7.4\text{mm}$ DFP

Plywood Engineering Properties

Comparative Product Data 12.5 mm

		Strength, Stiffness and Rigidity Capacities			
Engineering Property	Orientation	DFP	CSP	OSB Single-floor	OSB Sheathing
Bending (N·mm/mm)	0°	420	380	-	270
	90°	130	130	-	100
Axial Tension (N/mm)	0°	97	89	-	65
	90°	55	55	-	38
Axial Compression (N/mm)	0°	130	99	-	77
	90°	79	79	-	67
Shear-through-thickness (N/mm)	0°	30	30	-	50
	90°	30	30	-	50
Planar Shear-Bending (N/mm)	0°	5.5	5.5	-	4.8
	90°	2.8	2.8	-	3.0
Planar Shear-Shear in Plane (MPa)	0°	0.55	0.55	-	0.60
	90°	0.72	0.72	-	0.38
Bending Stiffness (N·mm ² /mm)	0°	1,700,000	1,200,000	-	1,100,000
	90°	190,000	190,000	-	220,000



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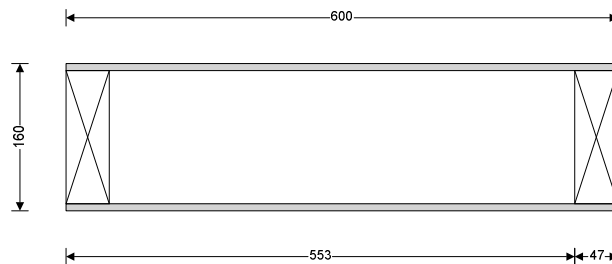
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Sip Roof Panel Design

STRESS SKIN PANEL DESIGN (BS5268-2:2002)

Double-skin panel details

Effective span of panel	$L_{ef} = 3900$ mm	Panel width	$b_{panel} = 600$ mm		
Web member depth	$h = 145$ mm	Web member breadth	$b = 47$ mm		
Web member spacing	$s = 553$ mm	No.of web members per panel	$N = 2$		
Timber strength class	C30	Panel depth	$t_{panel} = 159$ mm		
Top skin	7.5 mm x 3 ply Canadian Douglas fir plywood:Select tight face, select and sheathing				
grades:Unsanded					
Minimum thickness	$t_{mins_top} = 7.0$ mm				
Bottom skin	7.5 mm x 3 ply Canadian Douglas fir plywood:Select tight face, select and sheathing				
grades:Unsanded					
Minimum thickness	$t_{mins_bot} = 7.0$ mm				



Section properties

EA for top skin	$EA_{top} = 9160.795$ kN	EA for web member	$EA = 167649.000$ kN
EA for bottom skin	$EA_{bot} = 16482.620$ kN	Summation of EA for panel	$\Sigma EA = 193292$ kN
Neutral axis depth	$\bar{y} = 82.4$ mm	Bending rigidity of panel	$EI = 440$ kN/m ²

Loading details

Joist self weight	$F_{swt} = 0.10$ kN/m	Dead load	$F_{d_udl} = 0.50$ kN/m ²
Imposed UDL	$F_{i_udl} = 0.75$ kN/m ²	Imposed point load	$F_{i_pt} = 0.90$ kN

Modification factors

Section depth factor	$K_7 = 1.08$	Load sharing factor	$K_8 = 1.10$
Stress concentration factor	$K_{37} = 0.50$	Nail glue factor	$K_{70} = 0.90$

Consider medium term loads

Load duration factor (timber)	$K_3 = 1.25$	Load duration factor (plywood)	$K_{36} = 1.33$
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Check bending stresses

Maximum bending moment	$M = 1.619$ kNm		
Permiss.comp.stress top skin	$\sigma_{ms_top_adm} = 16.625$ N/mm ²	Compressive stress top skin	$\sigma_{ms_top} = 1.474$ N/mm ²
PASS - Compressive stress at extreme fibre of top skin is less than permissible stress			
Permiss.bend stress in web	$\sigma_{mw_adm} = 16.384$ N/mm ²	Bending stress top of web	$\sigma_{mw_top} = 3.409$ N/mm ²
PASS - Bending stress at upper extreme fibre of web is less than permissible stress			
		Bending stress bottom of web	$\sigma_{mw_bot} = 3.149$ N/mm ²
PASS - Bending stress at lower extreme fibre of web is less than permissible stress			
Perm.tens.stress bottom skin	$\sigma_{ms_bot_adm} = 10.999$ N/mm ²	Tensile stress bottom skin	$\sigma_{ms_bot} = 1.371$ N/mm ²
PASS - Tensile stress at extreme fibre of bottom skin is less than permissible stress			

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Check horizontal shear stresses in web members

Maximum shear force $V = 1.660$ kN

Permissible shear stress $\tau_{adm} = 1.650$ N/mm² Max.horizontal shear stress $\tau_{max} = 0.159$ N/mm²

PASS - Maximum horizontal shear stress is less than permissible shear stress

Check rolling shear stress between bottom skin and web members

Perm.roll shear stress bottom $\tau_{r_bot_adm} = 0.305$ N/mm² Max.rolling shear stress $\tau_{r_bot_max} = 0.048$ N/mm²

PASS - Maximum rolling shear stress at bottom skin is less than permissible rolling shear stress

Check deflection

Permissible deflection $\delta_{adm} = 11.700$ mm Total deflection $\delta_{max} = 5.987$ mm

PASS - Total deflection is less than permissible deflection

Consider short term loads

Load duration factor (timber) $K_3 = 1.50$ Load duration factor (plywood) $K_{36} = 1.50$

Check bending stresses

Maximum bending moment $M = 1.641$ kNm

Permiss.comp.stress top skin $\sigma_{ms_top_adm} = 18.750$ N/mm² Compressive stress top skin $\sigma_{ms_top} = 1.494$ N/mm²

PASS - Compressive stress at extreme fibre of top skin is less than permissible stress

Permiss.bend stress in web $\sigma_{mw_adm} = 19.661$ N/mm² Bending stress top of web $\sigma_{mw_top} = 3.456$ N/mm²

PASS - Bending stress at upper extreme fibre of web is less than permissible stress

Bending stress bottom of web $\sigma_{mw_bot} = 3.192$ N/mm²

PASS - Bending stress at lower extreme fibre of web is less than permissible stress

Perm.tens.stress bottom skin $\sigma_{ms_bot_adm} = 12.405$ N/mm² Tensile stress bottom skin $\sigma_{ms_bot} = 1.389$ N/mm²

PASS - Tensile stress at extreme fibre of bottom skin is less than permissible stress

Check horizontal shear stresses in web members

Maximum shear force $V = 1.233$ kN

Permissible shear stress $\tau_{adm} = 1.980$ N/mm² Max.horizontal shear stress $\tau_{max} = 0.118$ N/mm²

PASS - Maximum horizontal shear stress is less than permissible shear stress

Check rolling shear stress between bottom skin and web members

Perm.roll shear stress bottom $\tau_{r_bot_adm} = 0.344$ N/mm² Max.rolling shear stress $\tau_{r_bot_max} = 0.036$ N/mm²

PASS - Maximum rolling shear stress at bottom skin is less than permissible rolling shear stress

Check deflection

Permissible deflection $\delta_{adm} = 11.700$ mm Total deflection $\delta_{max} = 5.436$ mm

PASS - Total deflection is less than permissible deflection

Splice plate design

Min.length splice plate top $L_{top_splice} = 30$ mm Min.length splice plate bottom $L_{bot_splice} = 28$ mm

Provide:- minimum 170mm Sips Panel, 600mm wide (47 mm C24 timber)



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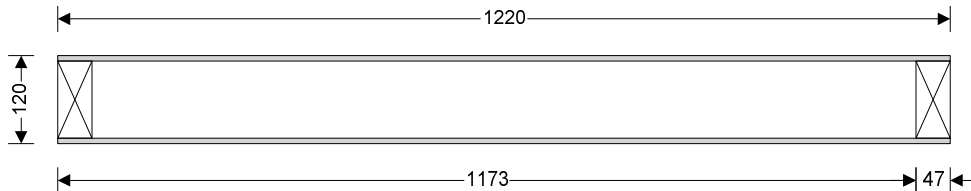
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SIP External Wall Design

STRESS SKIN PANEL DESIGN (BS5268-2:2002)

Double-skin panel details

Effective span of panel	$L_{ef} = 2400$ mm	Panel width	$b_{panel} = 1220$ mm		
Web member depth	$h = 100$ mm	Web member breadth	$b = 47$ mm		
Web member spacing	$s = 1173$ mm	No.of web members per panel	$N = 2$		
Timber strength class	C24	Panel depth	$t_{panel} = 114$ mm		
Top skin	7.5 mm x 3 ply Canadian Douglas fir plywood:Select tight face, select and sheathing				
grades:Unsanded					
Minimum thickness	$t_{mins_top} = 7.0$ mm				
Bottom skin	7.5 mm x 3 ply Canadian Douglas fir plywood:Select tight face, select and sheathing				
grades:Unsanded					
Minimum thickness	$t_{mins_bot} = 7.0$ mm				



Section properties

EA for top skin	$EA_{top} = 9160.795$ kN	EA for web member	$EA = 101520.000$ kN
EA for bottom skin	$EA_{bot} = 12396.020$ kN	Summation of EA for panel	$\Sigma EA = 123077$ kN
Neutral axis depth	$\bar{y} = 58.4$ mm	Bending rigidity of panel	$EI = 146$ kN/m ²

Loading details

Joist self weight	$F_{swt} = 0.12$ kN/m	Dead load	$F_{d_udl} = 0.30$ kN/m ²
Imposed UDL	$F_{i_udl} = 0.81$ kN/m ²		

Modification factors

Section depth factor	$K_7 = 1.13$	Load sharing factor	$K_8 = 1.00$
Stress concentration factor	$K_{37} = 0.50$	Nail glue factor	$K_{70} = 0.90$

Consider long term loads

Load duration factor (timber)	$K_3 = 1.00$	Load duration factor (plywood)	$K_{36} = 1.00$
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Check bending stresses

Maximum bending moment	$M = 1.343$ kNm		
Permiss.comp.stress top skin	$\sigma_{ms_top_adm} = 12.500$ N/mm ²	Compressive stress top skin	$\sigma_{ms_top} = 2.614$ N/mm ²
PASS - Compressive stress at extreme fibre of top skin is less than permissible stress			
Permiss.bend stress in web	$\sigma_{mw_adm} = 8.463$ N/mm ²	Bending stress top of web	$\sigma_{mw_top} = 5.107$ N/mm ²
PASS - Bending stress at upper extreme fibre of web is less than permissible stress			
		Bending stress bottom of web	$\sigma_{mw_bot} = 4.827$ N/mm ²
PASS - Bending stress at lower extreme fibre of web is less than permissible stress			
Perm.tens.stress bottom skin	$\sigma_{ms_bot_adm} = 8.270$ N/mm ²	Tensile stress bottom skin	$\sigma_{ms_bot} = 2.488$ N/mm ²
PASS - Tensile stress at extreme fibre of bottom skin is less than permissible stress			

Check horizontal shear stresses in web members

Maximum shear force	$V = 1.990$ kN		
Permissible shear stress	$\tau_{adm} = 0.710$ N/mm ²	Max.horizontal shear stress	$\tau_{max} = 0.264$ N/mm ²



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PASS - Maximum horizontal shear stress is less than permissible shear stress

Check rolling shear stress between bottom skin and web members

Perm.roll shear stress bottom $\tau_{r_bot_adm} = 0.230 \text{ N/mm}^2$ Max.rolling shear stress $\tau_{r_bot_max} = 0.094 \text{ N/mm}^2$

PASS - Maximum rolling shear stress at bottom skin is less than permissible rolling shear stress

Check deflection

Permissible deflection $\delta_{adm} = 8.100 \text{ mm}$ Total deflection $\delta_{max} = 7.194 \text{ mm}$

PASS - Total deflection is less than permissible deflection

Splice plate design

Min.length splice plate top $L_{top_splice} = 80 \text{ mm}$ Min.length splice plate bottom $L_{bot_splice} = 76 \text{ mm}$

Adopt:- 120mm Sips Panel, 1200mm wide



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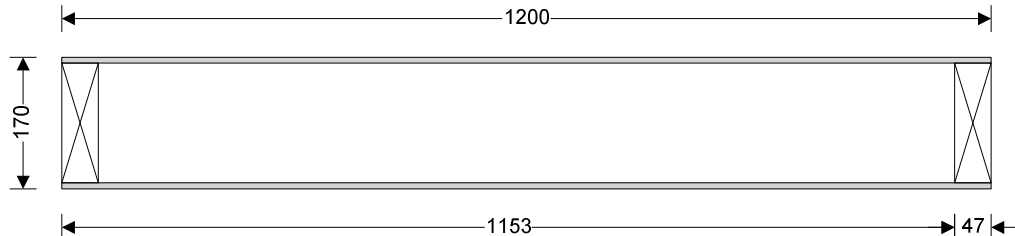
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SIP Floor panels

STRESS SKIN PANEL DESIGN (BS5268-2:2002)

Double-skin panel details

Effective span of panel	$L_{ef} = 1400$ mm	Panel width	$b_{panel} = 1200$ mm		
Web member depth	$h = 155$ mm	Web member breadth	$b = 47$ mm		
Web member spacing	$s = 1153$ mm	No.of web members per panel	$N = 2$		
Timber strength class	C24	Panel depth	$t_{panel} = 169$ mm		
Top skin	7.5 mm x 3 ply Canadian Douglas fir plywood:Select tight face, select and sheathing				
grades:Unsanded					
Minimum thickness	$t_{mins_top} = 7.0$ mm				
Bottom skin	7.5 mm x 3 ply Canadian Douglas fir plywood:Select tight face, select and sheathing				
grades:Unsanded					
Minimum thickness	$t_{mins_bot} = 7.0$ mm				



Section properties

EA for top skin	$EA_{top} = 9160.795$ kN	EA for web member	$EA = 157356.000$ kN
EA for bottom skin	$EA_{bot} = 10012.170$ kN	Summation of EA for panel	$\Sigma EA = 176529$ kN
Neutral axis depth	$\bar{y} = 84.9$ mm	Bending rigidity of panel	$EI = 441$ kN/m ²

Loading details

Joist self weight	$F_{swt} = 0.14$ kN/m	Dead load	$F_{d_udl} = 0.37$ kN/m ²
Imposed UDL	$F_{i_udl} = 2.00$ kN/m ²	Imposed point load	$F_{i_pt} = 1.40$ kN

Modification factors

Section depth factor	$K_7 = 1.08$	Load sharing factor	$K_8 = 1.00$
Stress concentration factor	$K_{37} = 0.50$	Nail glue factor	$K_{70} = 0.90$

Consider long term loads

Load duration factor (timber)	$K_3 = 1.00$	Load duration factor (plywood)	$K_{36} = 1.00$
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Check bending stresses

Maximum bending moment	$M = 1.492$ kNm		
Permiss.comp.stress top skin	$\sigma_{ms_top_adm} = 12.500$ N/mm ²	Compressive stress top skin	$\sigma_{ms_top} = 1.398$ N/mm ²
PASS - Compressive stress at extreme fibre of top skin is less than permissible stress			
Permiss.bend stress in web	$\sigma_{mw_adm} = 8.065$ N/mm ²	Bending stress top of web	$\sigma_{mw_top} = 2.847$ N/mm ²
PASS - Bending stress at upper extreme fibre of web is less than permissible stress			
		Bending stress bottom of web	$\sigma_{mw_bot} = 2.819$ N/mm ²
PASS - Bending stress at lower extreme fibre of web is less than permissible stress			
Perm.tens.stress bottom skin	$\sigma_{ms_bot_adm} = 8.270$ N/mm ²	Tensile stress bottom skin	$\sigma_{ms_bot} = 1.385$ N/mm ²
PASS - Tensile stress at extreme fibre of bottom skin is less than permissible stress			

Check horizontal shear stresses in web members

Maximum shear force	$V = 2.984$ kN
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Permissible shear stress $\tau_{adm} = 0.710 \text{ N/mm}^2$

Max.horizontal shear stress $\tau_{max} = 0.273 \text{ N/mm}^2$

PASS - Maximum horizontal shear stress is less than permissible shear stress

Check rolling shear stress between bottom skin and web members

Perm.roll shear stress bottom $\tau_{r_bot_adm} = 0.230 \text{ N/mm}^2$

Max.rolling shear stress $\tau_{r_bot_max} = 0.058 \text{ N/mm}^2$

PASS - Maximum rolling shear stress at bottom skin is less than permissible rolling shear stress

Check deflection

Permissible deflection $\delta_{adm} = 6.000 \text{ mm}$

Total deflection $\delta_{max} = 1.573 \text{ mm}$

PASS - Total deflection is less than permissible deflection

Splice plate design

Min.length splice plate top $L_{top_splice} = 43 \text{ mm}$

Min.length splice plate bottom $L_{bot_splice} = 42 \text{ mm}$

Provide:- minimum 170mm Sips Panel, 1200mm wide (47mm C24 timber)



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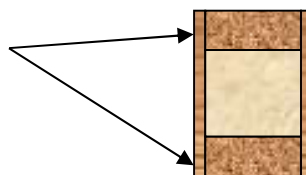
Lintel L1

1.4m Eaves Opening: Roof - Dead full UDL 1.28 kN/m;
Total: 3 kN/m

Snow - Imposed full UDL 1.65 kN/m

Maximum Loads for SIPs Box Beam Lintels (120mm)								
Beam Depth (mm)	Opening Span (mm)							
	1000	1200	1400	1600	1800	2000	2200	2400
200	6.1	5.1	4.3	3.8	3.4	3.0	2.7	2.5
250	8.2	6.8	5.8	5.1	4.5	4.1	3.7	3.4
300	10.4	8.7	7.4	6.5	5.7	5.2	4.7	4.3
350	12.7	10.6	9.1	7.9	7.0	6.3	5.7	5.2
400	15.2	12.6	10.8	9.4	8.4	7.5	6.8	6.2
450	17.7	14.7	12.6	11.0	9.8	8.8	8.0	7.3

95x47 C24



Beam Depth (mm)

Adopt: min. 200 x 120 SIP Box lintel over windows with 47mm C24 battens Top & Bottom



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Lintel L2

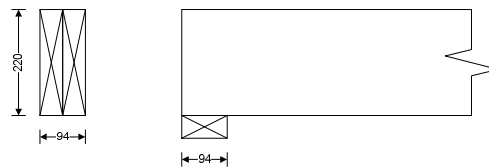
TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

Applied loading

Roof : Dead full UDL 1.243 kN/m; Snow: Imposed full UDL 1.695 kN/m

Analysis results

Design moment	$M = 3.723 \text{ kNm}$	Design shear	$F = 5.376 \text{ kN}$
Total load on beam	$W_{\text{tot}} = 10.752 \text{ kN}$		
Reactions at support A	$R_{A_{\text{max}}} = 5.376 \text{ kN}$	$R_{A_{\text{min}}} = 5.376 \text{ kN}$	
Unfactored dead load reaction at support A	$R_{A_{\text{Dead}}} = 1.722 \text{ kN}$		
Unfactored imposed load reaction at support A	$R_{A_{\text{Imposed}}} = 2.348 \text{ kN}$		
Reactions at support B	$R_{B_{\text{max}}} = 5.376 \text{ kN}$	$R_{B_{\text{min}}} = 5.376 \text{ kN}$	
Unfactored dead load reaction at support B	$R_{B_{\text{Dead}}} = 1.722 \text{ kN}$		
Unfactored imposed load reaction at support B	$R_{B_{\text{Imposed}}} = 2.348 \text{ kN}$		



Timber section details

Breadth of section	$b = 47 \text{ mm}$	Depth of section	$h = 220 \text{ mm}$
Number of sections	$N = 2$	Breadth of beam	$b_b = 94 \text{ mm}$
Timber strength class	C24		

Member details

Service class of timber	1	Load duration	Long term
Length of span	$L_{s1} = 2770 \text{ mm}$		
Length of bearing	$L_b = 94 \text{ mm}$		

Lateral support - cl.2.10.8: Permiss.depth-to-breadth ratio **4.00** Actual depth-to-breadth ratio **2.34**

PASS - Lateral support is adequate

Check bearing stress

Permissible bearing stress	$\sigma_{c_{\text{adm}}} = 2.640 \text{ N/mm}^2$	Applied bearing stress	$\sigma_{c_a} = 0.608 \text{ N/mm}^2$
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PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress	$\sigma_{m_{\text{adm}}} = 8.536 \text{ N/mm}^2$	Applied bending stress	$\sigma_{m_a} = 4.910 \text{ N/mm}^2$
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PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain: Permissible shear stress $\tau_{\text{adm}} = 0.781 \text{ N/mm}^2$ Applied shear stress $\tau_a = 0.390 \text{ N/mm}^2$

PASS - Applied shear stress is less than permissible shear stress

Deflection: Permissible deflection $\delta_{\text{adm}} = 5.004 \text{ mm}$ Total deflection $\delta_a = 3.608 \text{ mm}$

PASS - Total deflection is less than permissible deflection

Adopt: Min 2x47x220 C24 lintel nailed and glued together



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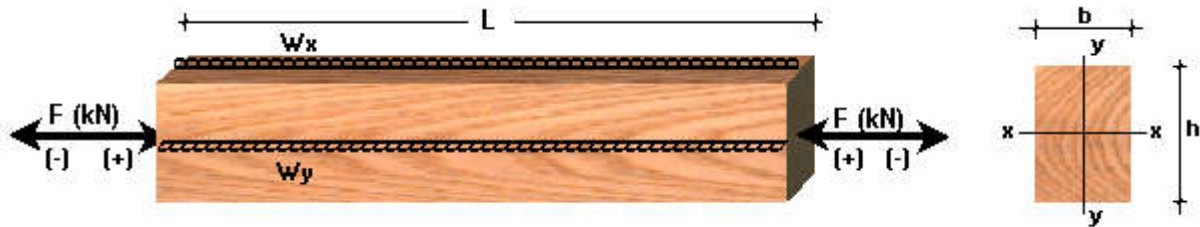
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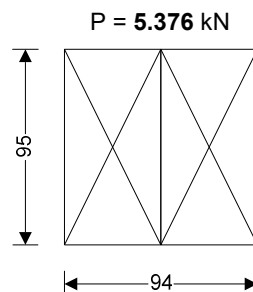
Column C1



TIMBER MEMBER DESIGN TO BS5268-2:2002

Analysis results

Design axial compression



Timber section details

Breadth of section $b = 47 \text{ mm}$
Number of sections $N = 2$
Timber strength class **C24**

Depth of section $h = 95 \text{ mm}$
Breadth of beam $b_b = 94 \text{ mm}$

Member details

Service class of timber **2**
Unbraced length in x-axis $L_x = 2100 \text{ mm}$
length factor in x-axis $K_x = 0.7$
Effective length in x-axis $L_{ex} = 1470 \text{ mm}$

Load duration **Long term**
Unbraced length in y-axis $L_y = 2100 \text{ mm}$ Effective
Effective length factor in y-axis $K_y = 0.7$
Effective length in y-axis $L_{ey} = 1470 \text{ mm}$

Slenderness ratio - cl.2.11.4

Slenderness ratio $\lambda = 54.173$

Permissible slenderness ratio $\lambda_{max} = 180$

PASS - Slenderness ratio is less than permissible slenderness ratio

Compression parallel to grain

Permissible comp.stress $\sigma_{c_adm} = 5.586 \text{ N/mm}^2$

Applied compressive stress $\sigma_{c_a} = 0.336 \text{ N/mm}^2$

PASS - Applied compressive stress is less than permissible compressive stress

Adopt: double 47x95 C24 Timber post, nailed and glued together

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Ring beam under external load bearing wall

L- max 1.2m

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

Applied loading

Beam loads

Wall	Dead full UDL 1.296 kN/m
Roof&Floor	Dead full UDL 1.635 kN/m
Roof&Floor	Imposed full UDL 3.725 kN/m

Analysis results

Maximum moment	$M_{max} = 1.584$ kNm	$M_{min} = 0.000$ kNm
Design moment	$M = \max(\text{abs}(M_{max}), \text{abs}(M_{min})) = 1.584$ kNm	
Maximum shear	$F_{max} = 5.280$ kN	$F_{min} = -5.280$ kN
Design shear	$F = \max(\text{abs}(F_{max}), \text{abs}(F_{min})) = 5.280$ kN	
Total load on beam	$W_{tot} = 10.559$ kN	
Reactions at support A	$R_{A_{max}} = 5.280$ kN	$R_{A_{min}} = 5.280$ kN
Unfactored dead load reaction at support A	$R_{A_{Dead}} = 1.759$ kN	
Unfactored imposed load reaction at support A	$R_{A_{Imposed}} = 2.235$ kN	
Reactions at support B	$R_{B_{max}} = 5.280$ kN	$R_{B_{min}} = 5.280$ kN
Unfactored dead load reaction at support B	$R_{B_{Dead}} = 1.759$ kN	
Unfactored imposed load reaction at support B	$R_{B_{Imposed}} = 2.235$ kN	

Timber section details

Breadth of sections	$b = 47$ mm
Depth of sections	$h = 145$ 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	Long term
Length of span	$L_{s1} = 1200$ mm
Length of bearing	$L_b = 100$ mm

Section properties

Cross sectional area of member	$A = N \times b \times h = 13630$ mm ²
Section modulus	$Z_x = N \times b \times h^2 / 6 = 329392$ mm ³
	$Z_y = h \times (N \times b)^2 / 6 = 213537$ mm ³
Second moment of area	$I_x = N \times b \times h^3 / 12 = 23880896$ mm ⁴
	$I_y = h \times (N \times b)^3 / 12 = 10036223$ mm ⁴
Radius of gyration	$i_x = \sqrt{I_x / A} = 41.9$ 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	$K_7 = (300 \text{ mm} / h)^{0.11} = 1.08$

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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 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	$h / (N \times b) = 1.54$
PASS - Lateral support is adequate	
Compression perpendicular to grain	
Permissible bearing stress (no wane)	$\sigma_{c_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 = 2.640 \text{ N/mm}^2$
Applied bearing stress	$\sigma_{c_a} = R_{B_max} / (N \times b \times L_b) = 0.562 \text{ N/mm}^2$
	$\sigma_{c_a} / \sigma_{c_adm} = 0.213$
PASS - Applied compressive stress is less than permissible compressive stress at bearing	
Bending parallel to grain	
Permissible bending stress	$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 8.937 \text{ N/mm}^2$
Applied bending stress	$\sigma_{m_a} = M / Z_x = 4.809 \text{ N/mm}^2$
	$\sigma_{m_a} / \sigma_{m_adm} = 0.538$
PASS - Applied bending stress is less than permissible bending stress	
Shear parallel to grain	
Permissible shear stress	$\tau_{adm} = \tau \times K_3 \times K_8 = 0.781 \text{ N/mm}^2$
Applied shear stress	$\tau_a = 3 \times F / (2 \times A) = 0.581 \text{ N/mm}^2$
	$\tau_a / \tau_{adm} = 0.744$
PASS - Applied shear stress is less than permissible shear stress	
Deflection	
Modulus of elasticity for deflection	$E = E_{min} \times K_9 = 8208 \text{ N/mm}^2$
Permissible deflection	$\delta_{adm} = \min(0.157 \text{ in}, 0.003 \times L_{s1}) = 3.600 \text{ mm}$
Bending deflection	$\delta_{b_s1} = 0.917 \text{ mm}$
Shear deflection	$\delta_{v_s1} = 0.206 \text{ mm}$
Total deflection	$\delta_a = \delta_{b_s1} + \delta_{v_s1} = 1.122 \text{ mm}$
	$\delta_a / \delta_{adm} = 0.312$
PASS - Total deflection is less than permissible deflection	

Adopt: minimum 47x170 + 47x145 C24 Timbers (nailed and glued together)



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Ring beam under external non - load bearing wall

L- max 1.3m

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

Applied loading

Beam loads

Wall	Dead full UDL 1.265 kN/m
Floor&Roof	Dead full UDL 0.984 kN/m
Floor&Roof	Imposed full UDL 1.650 kN/m

Analysis results

Maximum moment	$M_{max} = 1.269 \text{ kNm}$	$M_{min} = 0.000 \text{ kNm}$
Design moment	$M = \max(\text{abs}(M_{max}), \text{abs}(M_{min})) = 1.269 \text{ kNm}$	
Maximum shear	$F_{max} = 3.627 \text{ kN}$	$F_{min} = -3.627 \text{ kN}$
Design shear	$F = \max(\text{abs}(F_{max}), \text{abs}(F_{min})) = 3.627 \text{ kN}$	
Total load on beam	$W_{tot} = 7.254 \text{ kN}$	
Reactions at support A	$R_{A_max} = 3.627 \text{ kN}$	$R_{A_min} = 3.627 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_Dead} = 1.574 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_Imposed} = 1.155 \text{ kN}$	
Reactions at support B	$R_{B_max} = 3.627 \text{ kN}$	$R_{B_min} = 3.627 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_Dead} = 1.574 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_Imposed} = 1.155 \text{ kN}$	

Timber section details

Breadth of sections	$b = 47 \text{ mm}$
Depth of sections	$h = 145 \text{ mm}$
Number of sections in member	$N = 2$
Overall breadth of member	$b_b = N \times b = 94 \text{ mm}$
Timber strength class	C24

Member details

Service class of timber	2
Load duration	Long term
Length of span	$L_{s1} = 1500 \text{ mm}$
Length of bearing	$L_b = 100 \text{ mm}$

Section properties

Cross sectional area of member	$A = N \times b \times h = 13630 \text{ mm}^2$
Section modulus	$Z_x = N \times b \times h^2 / 6 = 329392 \text{ mm}^3$
	$Z_y = h \times (N \times b)^2 / 6 = 213537 \text{ mm}^3$
Second moment of area	$I_x = N \times b \times h^3 / 12 = 23880896 \text{ mm}^4$
	$I_y = h \times (N \times b)^3 / 12 = 10036223 \text{ mm}^4$
Radius of gyration	$i_x = \sqrt{I_x / A} = 41.9 \text{ mm}$
	$i_y = \sqrt{I_y / A} = 27.1 \text{ mm}$

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	$K_7 = (300 \text{ mm} / h)^{0.11} = 1.08$

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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	$h / (N \times b) = 1.54$
PASS - Lateral support is adequate	
Compression perpendicular to grain	
Permissible bearing stress (no wane)	$\sigma_{c_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 = 2.400 \text{ N/mm}^2$
Applied bearing stress	$\sigma_{c_a} = R_{A_max} / (N \times b \times L_b) = 0.386 \text{ N/mm}^2$
	$\sigma_{c_a} / \sigma_{c_adm} = 0.161$
PASS - Applied compressive stress is less than permissible compressive stress at bearing	
Bending parallel to grain	
Permissible bending stress	$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 8.124 \text{ N/mm}^2$
Applied bending stress	$\sigma_{m_a} = M / Z_x = 3.854 \text{ N/mm}^2$
	$\sigma_{m_a} / \sigma_{m_adm} = 0.474$
PASS - Applied bending stress is less than permissible bending stress	
Shear parallel to grain	
Permissible shear stress	$\tau_{adm} = \tau \times K_3 \times K_8 = 0.710 \text{ N/mm}^2$
Applied shear stress	$\tau_a = 3 \times F / (2 \times A) = 0.399 \text{ N/mm}^2$
	$\tau_a / \tau_{adm} = 0.562$
PASS - Applied shear stress is less than permissible shear stress	
Deflection	
Modulus of elasticity for deflection	$E = E_{min} = 7200 \text{ N/mm}^2$
Permissible deflection	$\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1}) = 4.200 \text{ mm}$
Bending deflection	$\delta_{b_s1} = 1.134 \text{ mm}$
Shear deflection	$\delta_{v_s1} = 0.187 \text{ mm}$
Total deflection	$\delta_a = \delta_{b_s1} + \delta_{v_s1} = 1.321 \text{ mm}$
	$\delta_a / \delta_{adm} = 0.315$
PASS - Total deflection is less than permissible deflection	

Adopt: minimum 47x170 + 47x145 C24 Timbers (nailed and glued together)



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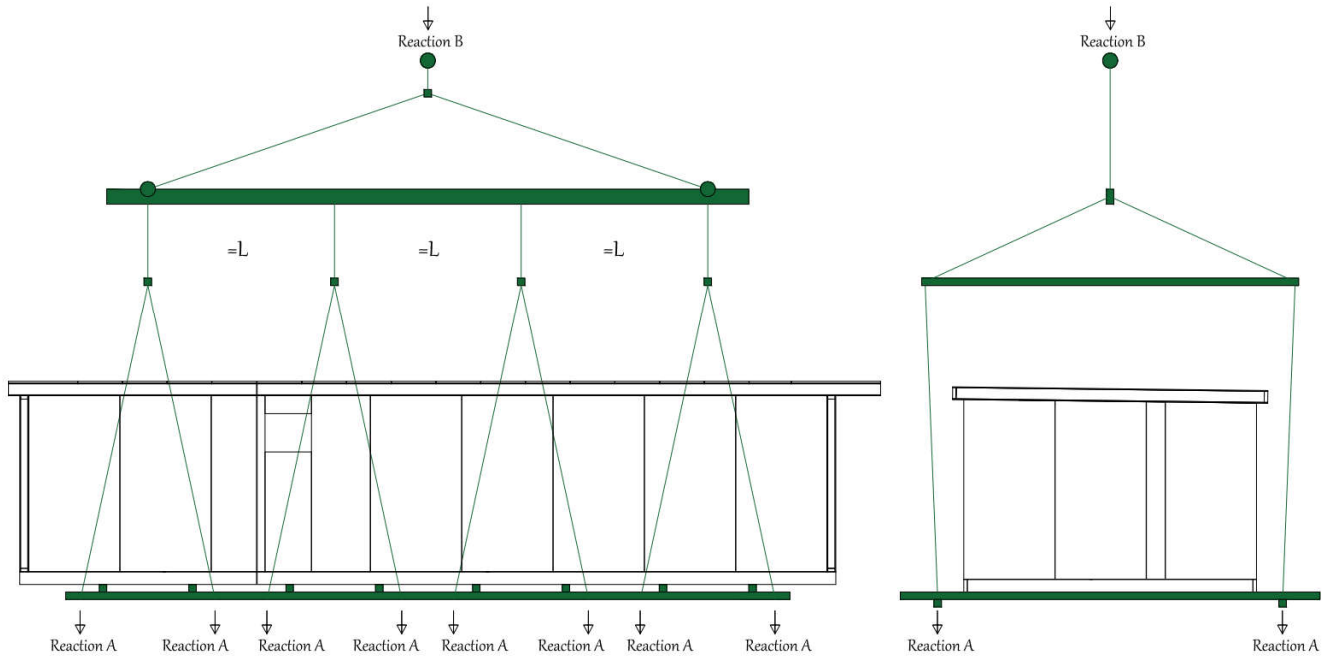
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Lifting scheme



Lifting Reaction B = 286 kN

Note: Lifting equipment, lifting beams and straps etc. to be supplied by the specialist crane company.



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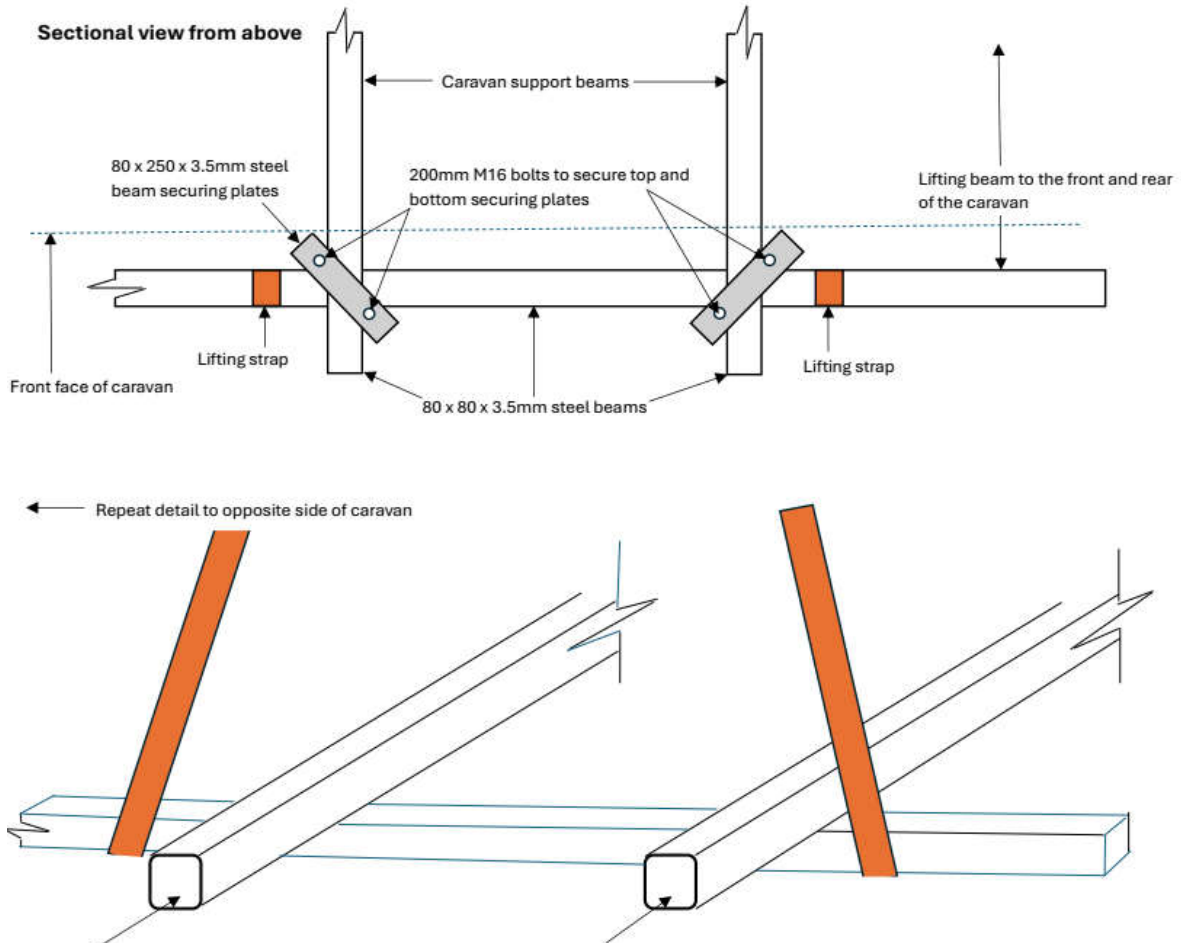
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General Caravan Lifting Cradle Detail



Caravan support beams are inserted under the caravan and secured to the lifting beam by securing plates and bolts as shown above.

NB. The above information shows the general principle for a steel lifting cradle only.

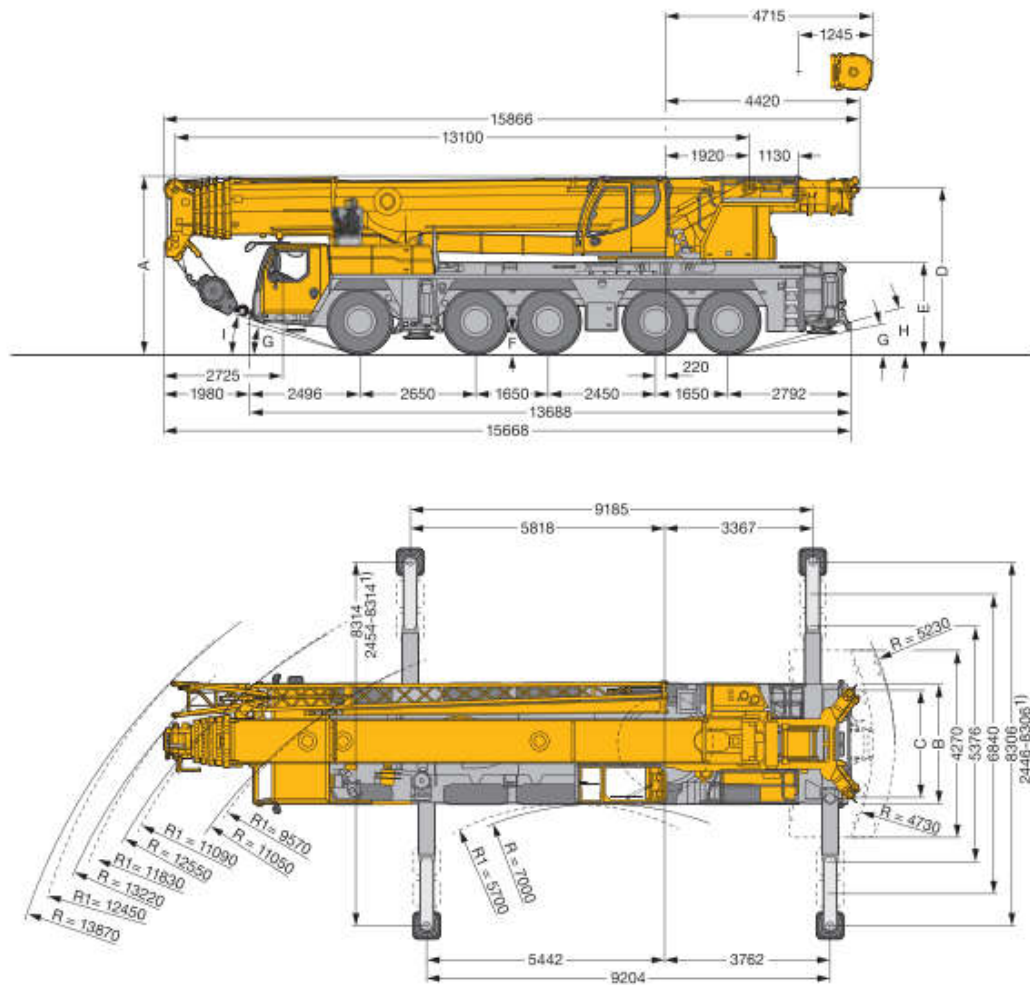
Each caravan model will require individual engineering calculations as the beam dimensions and thickness will vary. Site specific crane size calculations will be required.

<https://www.cadmancranes.com/services/crane-size-calculator/>

NOT TO SCALE

Suggested crane: Liebherr LTM 1160-5.2 (To be confirmed by crane company due to site conditions and lifting distance)

Dimensions





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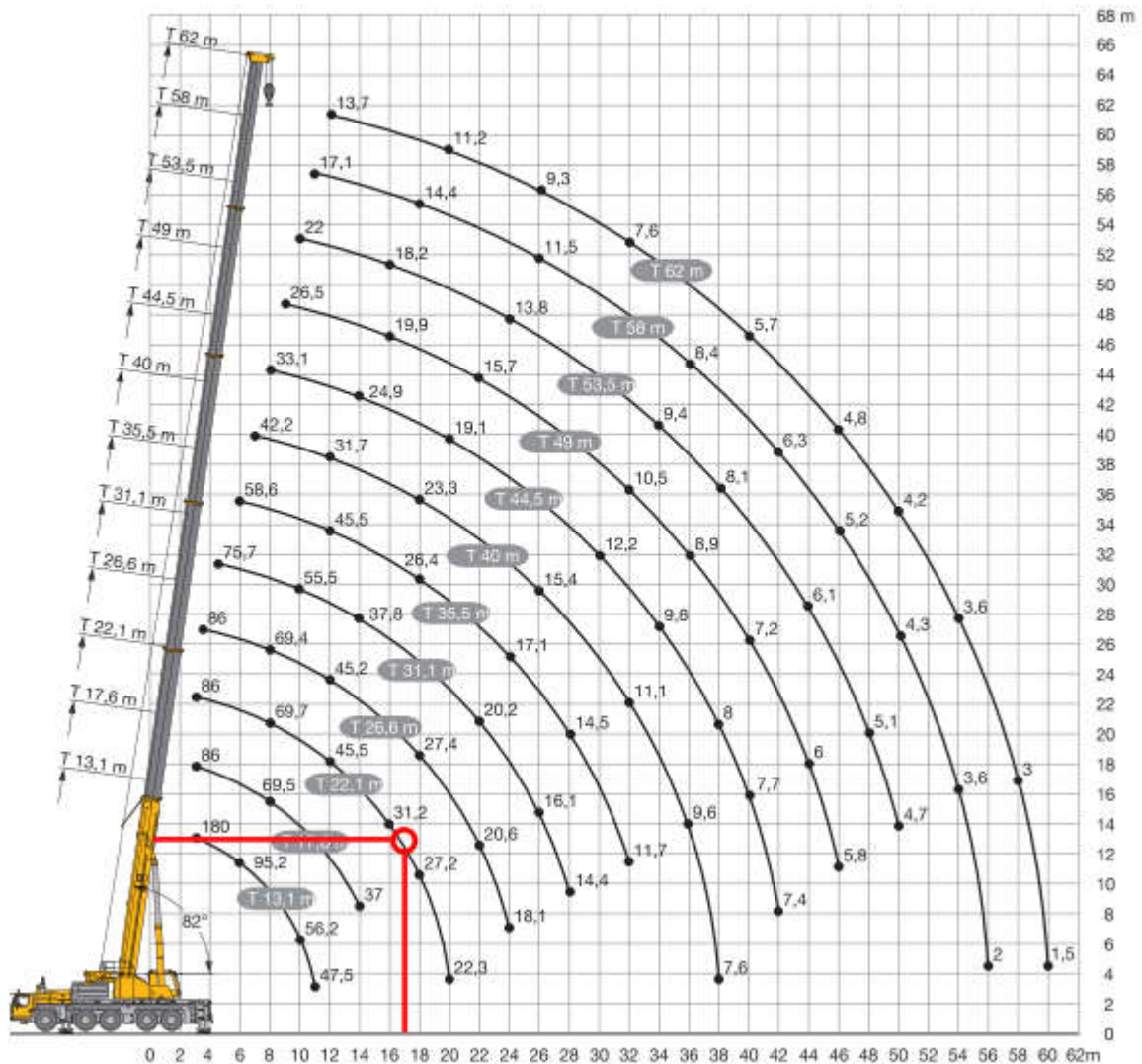
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Lifting heights



Lifting height – approx 13m

Suggested max. reach distance – approx 17m