

STRUCTURAL ANALYSIS AND DESIGN REPORT FOR A PROPOSED MOBILE HOUSE

27 Oakmont Drive, UK



PREPARED BY:

QUICK GARDENS

JANUARY 07, 2019



PROJECT: PROPOSED MOBILE HOUSE				Job Ref.	
STRUCTURAL CALCULATIONS: CONSTRUCTION MATERIAL DEAD LOADS				Sheet no./rev.	
Calc. by	Date	Chk'd by	Date	App'd by	Date

DEAD LOAD CONSTRUCTION

External wall construction

Material	Thickness (mm)	γ (kN/m ³)	Weight (kN/m ²)
Softwood 44x135mm C24 external leaf wall ; stacked horizontally	44;	5;	0.220
Celotex Insulation;	100	0.4	0.004
Softwood 44x135mm C24 inner leaf wall ; stacked horizontally	44;	5;	0.220
Totals;	208;		0.444

Ground floor construction

Material	Thickness (mm)	γ (kN/m ³)	Weight (kN/m ²)
Softwood Floor Boards;	20;	5;	0.100
Softwood 2NO. 70x140mm Floor Bearer @ 660mm ; Centers	140;	5;	0.700
Plywood	9;	7	0.063
Celotex Insulation;	100;	0.4	0.040
Totals;	269;		0.903

Roof Constructions

Material	Thickness (mm)	γ (kN/m ³)	Weight (kN/m ²)
Bitumen Roof Shingles;	3;	28;	0.084
Roof Boards;	20;	5;	0.100
Felt;	25;	2;	0.050
Softwood 2NO. 44x135mm C24 timber rafter @ ; 650mm Centers	44;	5;	0.220
Celotex insulation;	100;	0.4;	0.007
Oriented strand board	12	7	0.040
Tongue & Groove Board;	20;	9;	0.180
Totals;	224;		0.681

Internal Wall Construction

Material	Thickness (mm)	γ (kN/m ³)	Weight (kN/m ²)
Softwood 44x135mm C24 stacked horizontally	44;	5;	0.220
Totals;	44;		0.220

Snow Load = 0.28 KPa (See Calculation)

Wind Load = 0.37 KPa (See Calculation)

Floor load

Live Load = 1.5KPa



PROJECT: PROPOSED MOBILE HOUSE				Job Ref.	
STRUCTURAL CALCULATIONS: SNOW LOADING ANALYSIS PER EN1991				Sheet no./rev.	
Calc. by	Date	Chk'd by	Date	App'd by	Date

SNOW LOADING

In accordance with EN1991-1-3:2003+A1:2015 incorporating corrigenda dated December 2004 and March 2009 and the UK national annex NA+A1:2015 to BS EN 1991-1-3:2003+A1:2015 incorporating Corrigendum No.1

Tedds calculation version 1.0.10

Characteristic ground snow load

Location;	27 Oakmont Drive
Site altitude above sea level;	A = 20 m
Zone number;	Z = 3.0
Density of snow;	$\gamma = \mathbf{2.00}$ kN/m ³
Characteristic ground snow load;	$s_k = ((0.15 + (0.1 \times Z + 0.05)) + ((A - 100m) / 525m)) \times 1 \text{ kN/m}^2 = \mathbf{0.35}$ kN/m ²
Exposure coefficient (Normal);	$C_e = \mathbf{1.0}$
Thermal coefficient;	$C_t = \mathbf{1.0}$
Snow fence;	Not present

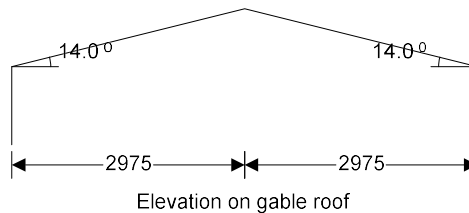
Building details

Roof type;	Duopitch
Width of roof (left on elevation);	$b_1 = \mathbf{2.98}$ m
Width of roof (right on elevation);	$b_2 = \mathbf{2.98}$ m
Slope of roof (left on elevation);	$\alpha_1 = \mathbf{14.00}$ deg
Slope of roof (right on elevation);	$\alpha_2 = \mathbf{14.00}$ deg

Shape coefficients

Shape coefficient roof (Table 5.2);	$\mu_{2,\alpha 1_T52} = \mathbf{0.80}$
Shape coefficient roof (Table 5.2);	$\mu_{2,\alpha 2_T52} = \mathbf{0.80}$
Shape coefficient roof (Table UK NA.2);	$\mu_{1,\alpha 1_NA2} = \mathbf{0.80}$
Shape coefficient roof (Table UK NA.2);	$\mu_{1,\alpha 2_NA2} = \mathbf{0.80}$

Case (i)	$\mu_{2,\alpha 1_T52}$	$\mu_{2,\alpha 2_T52}$	Shape coef	Coef	Loading (kN/m ²)
			$\mu_{2,\alpha 1_T52}$	0.800	0.28
Case (ii)		$\mu_{1,\alpha 2_NA2}$	$\mu_{2,\alpha 2_T52}$	0.800	0.28
			$\mu_{1,\alpha 1_NA2}$	0.800	0.28
Case (iii)	$\mu_{1,\alpha 1_NA2}$		$\mu_{1,\alpha 2_NA2}$	0.800	0.28



Loadcase 1 Table 5.2

Loading to roof 1 (LHS);	$s_{1_1} = \mu_{2,\alpha 1_T52} \times C_e \times C_t \times s_k = \mathbf{0.28}$ kN/m ²
Loading to roof 2 (RHS);	$s_{2_1} = \mu_{2,\alpha 2_T52} \times C_e \times C_t \times s_k = \mathbf{0.28}$ kN/m ²



PROJECT: PROPOSED MOBILE HOUSE				Job Ref.	
STRUCTURAL CALCULATIONS: SNOW LOADING ANALYSIS PER EN1991				Sheet no./rev.	
Calc. by	Date	Chk'd by	Date	App'd by	Date

Loadcase 2 UK NA.2

Loading to roof 1 (LHS);

$$s_{1_2} = 0 \times C_e \times C_t \times s_k = \mathbf{0.00 \text{ kN/m}^2}$$

Loading to roof 2 (RHS);

$$s_{2_2} = \mu_{1_2_NA2} \times C_e \times C_t \times s_k = \mathbf{0.28 \text{ kN/m}^2}$$

Loadcase 3 UK NA.2

Loading to roof 1 (LHS);

$$s_{1_3} = \mu_{1_1_NA2} \times C_e \times C_t \times s_k = \mathbf{0.28 \text{ kN/m}^2}$$

Loading to roof 2 (RHS);

$$s_{2_3} = 0 \times C_e \times C_t \times s_k = \mathbf{0.00 \text{ kN/m}^2}$$

;



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PROJECT:

PROPOSED MOBILE HOUSE

Job Ref.

STRUCTURAL CALCULATIONS:

WIND LOADING ANALYSIS PER EN1991

Sheet no./rev.

Calc. by

Date

Chk'd by

Date

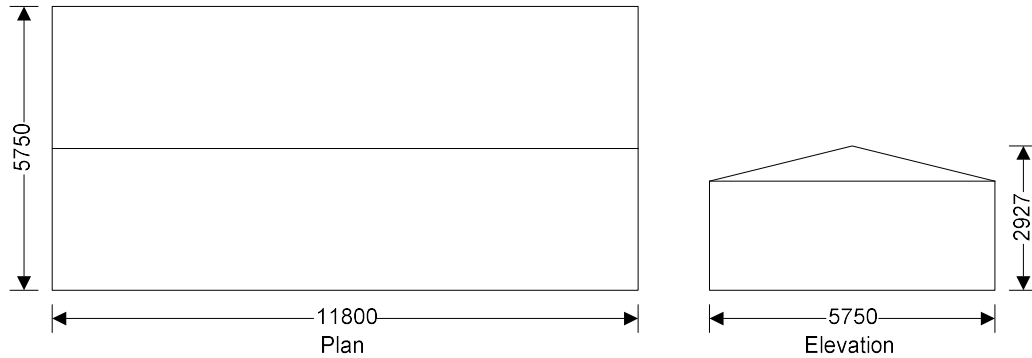
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WIND LOADING

In accordance with EN1991-1-4:2005+A1:2010 and the UK national annex

Tedds calculation version 3.0.23



Building data

Type of roof; Duopitch
Length of building; L = **11800** mm
Width of building; W = **5750** mm
Height to eaves; H = **2210** mm
Pitch of roof; α_0 = **14.0** deg
Total height; h = **2927** mm

Basic values

Location; 27 Oakmont Drive
Wind speed velocity (FigureNA.1); $V_{b,map}$ = **21.5** m/s
Distance to shore; L_{shore} = **66.00** km
Altitude above sea level; A_{alt} = **20.0**m
Altitude factor; C_{alt} = $A_{alt}/1m \times 0.001 + 1$ = **1.020**
Fundamental basic wind velocity; $V_{b,0} = V_{b,map} \times C_{alt}$ = **21.9** m/s
Direction factor; C_{dir} = **1.00**
Season factor; C_{season} = **1.00**
Shape parameter K; K = **0.2**
Exponent n; n = **0.5**
Air density; ρ = **1.226** kg/m³
Probability factor; $C_{prob} = [(1 - K \times \ln(-\ln(1-p)))/(1 - K \times \ln(-\ln(0.98)))]^n$ = **1.00**
Basic wind velocity (Exp. 4.1); $V_b = C_{dir} \times C_{season} \times V_{b,0} \times C_{prob}$ = **21.9** m/s
Reference mean velocity pressure; $q_b = 0.5 \times \rho \times V_b^2$ = **0.295** kN/m²

Orography

Orography factor not significant; C_o = **1.0**
Terrain category; Country
Displacement height (sheltering effect excluded); h_{dis} = **0**mm

	PROJECT:				Job Ref.	
	PROPOSED MOBILE HOUSE				Sheet no./rev.	
	STRUCTURAL CALCULATIONS: WIND LOADING ANALYSIS PER EN1991					
	Calc. by	Date	Chk'd by	Date	App'd by	Date

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.7.2.2)

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.7.2.2)

Peak velocity pressure - windward wall - Wind 0 deg and roof

Reference height (at which q is sought); $z = 2210\text{mm}$
Displacement height (sheltering effects excluded); $h_{dis} = 0\text{ mm}$
Exposure factor (Figure NA.7); $C_e = 1.47$
Peak velocity pressure; $q_p = C_e \times q_b = 0.43\text{ kN/m}^2$

Structural factor

Structural damping; $\delta_s = 0.100$
Height of element; $h_{part} = 2210\text{ mm}$
Size factor (Table NA.3); $C_s = 0.928$
Dynamic factor (Figure NA.9); $C_d = 1.002$
Structural factor; $C_{sCd} = C_s \times C_d = 0.930$

Peak velocity pressure - windward wall - Wind 90 deg and roof

Reference height (at which q is sought); $z = 2927\text{mm}$
Displacement height (sheltering effects excluded); $h_{dis} = 0\text{ mm}$
Exposure factor (Figure NA.7); $C_e = 1.62$
Peak velocity pressure; $q_p = C_e \times q_b = 0.48\text{ kN/m}^2$

Structural factor

Structural damping; $\delta_s = 0.100$
Height of element; $h_{part} = 2927\text{ mm}$
Size factor (Table NA.3); $C_s = 0.956$
Dynamic factor (Figure NA.9); $C_d = 1.015$
Structural factor; $C_{sCd} = C_s \times C_d = 0.971$

Structural factor - roof 0 deg

Structural damping; $\delta_s = 0.100$
Height of element; $h_{part} = 2927\text{ mm}$
Size factor (Table NA.3); $C_s = 0.926$
Dynamic factor (Figure NA.9); $C_d = 1.002$
Structural factor; $C_{sCd} = C_s \times C_d = 0.928$

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.); $q_{p,i} = 0.48\text{ kN/m}^2$

Pressures and forces

Net pressure; $p = C_{sCd} \times q_p \times C_{pe} - q_{p,i} \times C_{pi}$
Net force; $F_w = p_w \times A_{ref}$

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



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PROJECT:

PROPOSED MOBILE HOUSE

Job Ref.

STRUCTURAL CALCULATIONS:

WIND LOADING ANALYSIS PER EN1991

Sheet no./rev.

Calc. by

Date

Chk'd by

Date

App'd by

Date

Zone	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (kN/m ²)	Net pressure p (kN/m ²)	Area A_{ref} (m ²)	Net force F_w (kN)
F (-ve)	-1.17	0.48	-0.62	1.77	-1.09
G (-ve)	-0.84	0.48	-0.47	5.35	-2.51
H (-ve)	-0.42	0.48	-0.28	27.84	-7.86
I (-ve)	-0.49	0.48	-0.31	27.84	-8.73
J (-ve)	-1.26	0.48	-0.66	7.12	-4.67

Total vertical net force; $F_{w,v} = -24.11$ kN

Total horizontal net force; $F_{w,h} = 0.47$ kN

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

Zone	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (kN/m ²)	Net pressure p (kN/m ²)	Area A_{ref} (m ²)	Net force F_w (kN)
A	-1.20	0.48	-0.63	2.76	-1.74
B	-0.80	0.48	-0.45	12.01	-5.43
D	0.73	0.43	0.20	26.08	5.25
E	-0.37	0.43	-0.24	26.08	-6.39

Overall loading

Equiv leeward net force for overall section; $F_l = F_{w,wE} = -6.4$ kN

Net windward force for overall section; $F_w = F_{w,wD} = 5.3$ kN

Lack of correlation (cl.7.2.2(3) – Note); $f_{corr} = 0.85$; as h/W is 0.509

Overall loading overall section; $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h}) = 10.3$ kN

Roof load case 2 - Wind 0, c_{pi} -0.3, $+c_{pe}$

Zone	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (kN/m ²)	Net pressure p (kN/m ²)	Area A_{ref} (m ²)	Net force F_w (kN)
F (+ve)	0.18	0.48	0.22	1.77	0.39
G (+ve)	0.18	0.48	0.22	5.35	1.20
H (+ve)	0.18	0.48	0.22	27.84	6.22
I (+ve)	-0.49	0.48	-0.07	27.84	-2.06
J (+ve)	-1.26	0.48	-0.42	7.12	-2.96

Total vertical net force; $F_{w,v} = 2.71$ kN

Total horizontal net force; $F_{w,h} = 3.10$ kN

Walls load case 2 - Wind 0, c_{pi} -0.3, $+c_{pe}$



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PROJECT:

PROPOSED MOBILE HOUSE

Job Ref.

STRUCTURAL CALCULATIONS:

WIND LOADING ANALYSIS PER EN1991

Sheet no./rev.

Calc. by

Date

Chk'd by

Date

App'd by

Date

Zone	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (kN/m ²)	Net pressure p (kN/m ²)	Area A_{ref} (m ²)	Net force F_w (kN)
A	-1.20	0.48	-0.39	2.76	-1.08
B	-0.80	0.48	-0.21	12.01	-2.55
D	0.73	0.43	0.44	26.08	11.49
E	-0.37	0.43	-0.01	26.08	-0.15

Overall loading

Equiv leeward net force for overall section;

$$F_l = F_{w,WE} = -0.1 \text{ kN}$$

Net windward force for overall section;

$$F_w = F_{w,WD} = 11.5 \text{ kN}$$

Lack of correlation (cl.7.2.2(3) – Note);

$$f_{corr} = 0.85; \text{ as } h/W \text{ is } 0.509$$

Overall loading overall section;

$$F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h}) = 12.5 \text{ kN}$$

Roof load case 3 - Wind 90, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (kN/m ²)	Net pressure p (kN/m ²)	Area A_{ref} (m ²)	Net force F_w (kN)
F (-ve)	-1.64	0.48	-0.86	1.70	-1.46
G (-ve)	-1.46	0.48	-0.77	1.70	-1.32
H (-ve)	-0.60	0.48	-0.37	13.63	-5.10
I (-ve)	-0.41	0.48	-0.29	52.89	-15.14

Total vertical net force;

$$F_{w,v} = -22.33 \text{ kN}$$

Total horizontal net force;

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

Walls load case 3 - Wind 90, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (kN/m ²)	Net pressure p (kN/m ²)	Area A_{ref} (m ²)	Net force F_w (kN)
A	-1.20	0.43	-0.60	2.54	-1.53
B	-0.80	0.43	-0.43	10.17	-4.41
C	-0.50	0.43	-0.31	13.37	-4.10
D	0.70	0.48	0.23	14.77	3.39
E	-0.30	0.48	-0.24	14.77	-3.47

Overall loading

Equiv leeward net force for overall section;

$$F_l = F_{w,WE} = -3.5 \text{ kN}$$

Net windward force for overall section;

$$F_w = F_{w,WD} = 3.4 \text{ kN}$$

Lack of correlation (cl.7.2.2(3) – Note);

$$f_{corr} = 0.85; \text{ as } h/L \text{ is } 0.248$$

Overall loading overall section;

$$F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h}) = 5.8 \text{ kN}$$

Roof load case 4 - Wind 90, c_{pi} -0.3, $+c_{pe}$



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PROJECT:

PROPOSED MOBILE HOUSE

Job Ref.

STRUCTURAL CALCULATIONS:

WIND LOADING ANALYSIS PER EN1991

Sheet no./rev.

Calc. by

Date

Chk'd by

Date

App'd by

Date

Zone	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (kN/m ²)	Net pressure p (kN/m ²)	Area A_{ref} (m ²)	Net force F_w (kN)
F (+ve)	0.18	0.48	0.23	1.70	0.39
G (+ve)	0.18	0.48	0.23	1.70	0.39
H (+ve)	0.18	0.48	0.23	13.63	3.10
I (+ve)	0.18	0.48	0.23	52.89	12.02

Total vertical net force; $F_{w,v} = 15.41$ kN

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

Walls load case 4 - Wind 90, $C_{pi} -0.3$, $+C_{pe}$

Zone	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (kN/m ²)	Net pressure p (kN/m ²)	Area A_{ref} (m ²)	Net force F_w (kN)
A	-1.20	0.43	-0.36	2.54	-0.92
B	-0.80	0.43	-0.19	10.17	-1.97
C	-0.50	0.43	-0.07	13.37	-0.90
D	0.70	0.48	0.47	14.77	6.92
E	-0.30	0.48	0.00	14.77	0.06

Overall loading

Equiv leeward net force for overall section;

$$F_l = F_{w,wE} = 0.1 \text{ kN}$$

Net windward force for overall section;

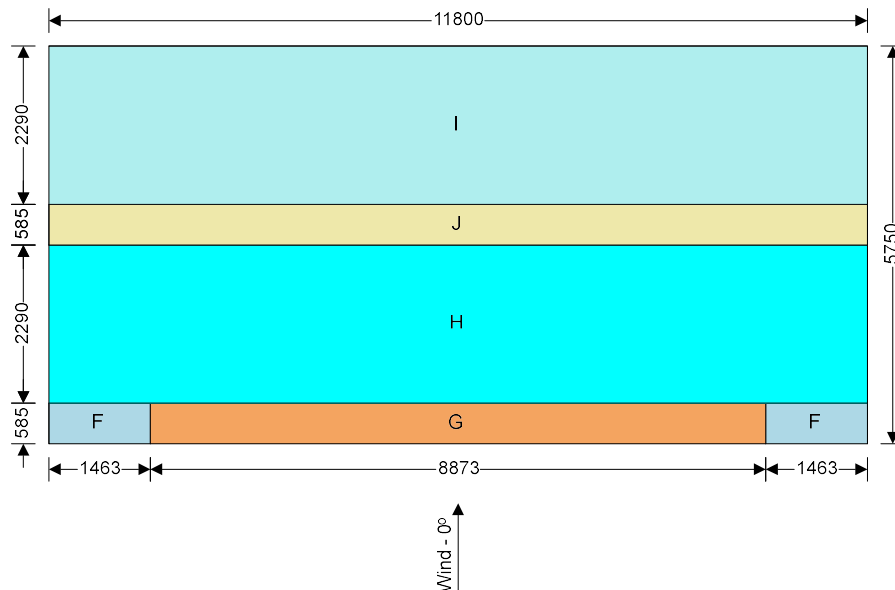
$$F_w = F_{w,wD} = 6.9 \text{ kN}$$

Lack of correlation (cl.7.2.2(3) – Note);

$$f_{corr} = 0.85; \text{ as } h/L \text{ is } 0.248$$

Overall loading overall section;

$$F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h}) = 5.8 \text{ kN}$$



Plan view - Duopitch roof



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PROJECT:

PROPOSED MOBILE HOUSE

Job Ref.

STRUCTURAL CALCULATIONS:

WIND LOADING ANALYSIS PER EN1991

Sheet no./rev.

Calc. by

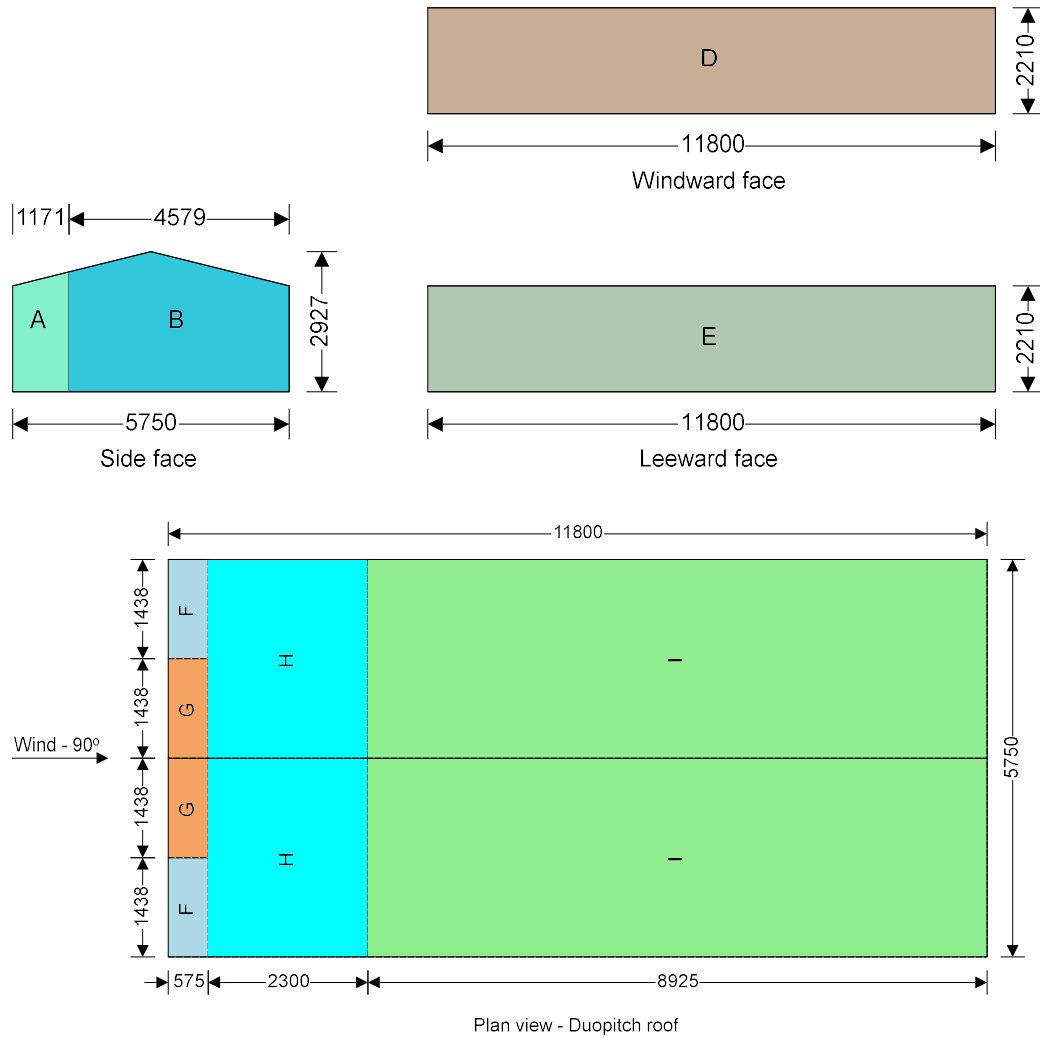
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PROJECT:

PROPOSED MOBILE HOUSE

Job Ref.

STRUCTURAL CALCULATIONS:

WIND LOADING ANALYSIS PER EN1991

Sheet no./rev.

Calc. by

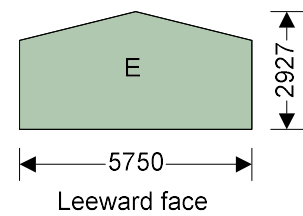
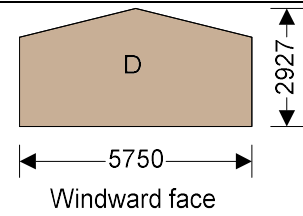
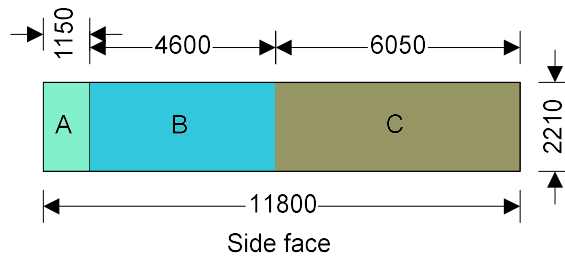
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PROJECT:

PROPOSED MOBILE HOUSE

Job Ref.

STRUCTURAL CALCULATIONS:

TIMBER RAFTER DESIGN CHECK PER EN1995

Sheet no./rev.

Calc. by

Date

Chk'd by

Date

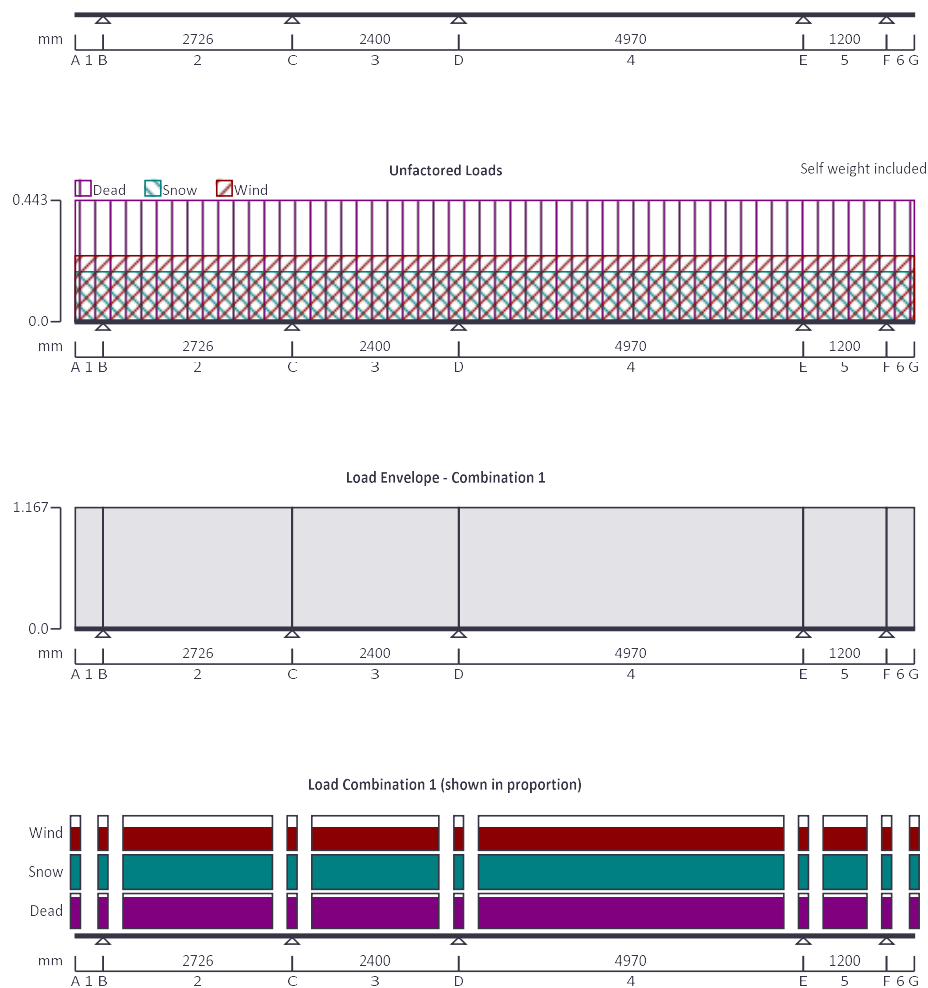
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TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

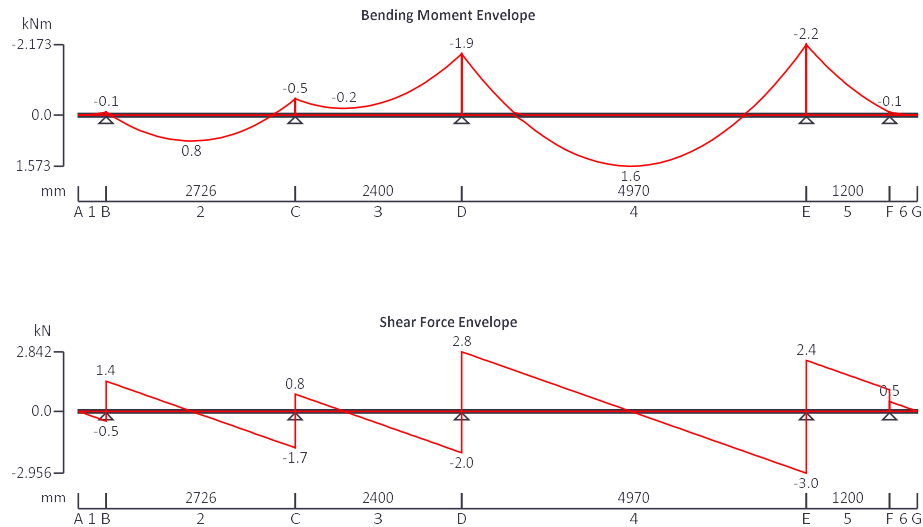
Tedds calculation version 1.7.04





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PROJECT: PROPOSED MOBILE HOUSE				Job Ref.	
STRUCTURAL CALCULATIONS: TIMBER RAFTER DESIGN CHECK PER EN1995				Sheet no./rev.	
Calc. by	Date	Chk'd by	Date	App'd by	Date



Applied loading

Beam loads

0.681KPa x 0.650m Tributary Width
0.28KPa x 0.650m Tributary Width
0.37KPa x 0.650m Tributary Width

Dead self weight of beam x 1
Dead full UDL 0.443 kN/m
Snow full UDL 0.182 kN/m
Wind full UDL 0.241 kN/m

Load combinations

Load combination 1

Support A	Dead x 1.35 Snow x 1.50 Wind x 1.00
Span 1	Dead x 1.35 Snow x 1.50 Wind x 1.00
Support B	Dead x 1.35 Snow x 1.50 Wind x 1.00
Span 2	Dead x 1.35 Snow x 1.50 Wind x 1.00
Support C	Dead x 1.35 Snow x 1.50 Wind x 1.00
Span 3	Dead x 1.35 Snow x 1.50 Wind x 1.00
Support D	Dead x 1.35



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PROJECT: PROPOSED MOBILE HOUSE				Job Ref.	
STRUCTURAL CALCULATIONS: TIMBER RAFTER DESIGN CHECK PER EN1995				Sheet no./rev.	
Calc. by	Date	Chk'd by	Date	App'd by	Date

	Snow $\times 1.50$
	Wind $\times 1.00$
Span 4	Dead $\times 1.35$
	Snow $\times 1.50$
	Wind $\times 1.00$
Support E	Dead $\times 1.35$
	Snow $\times 1.50$
	Wind $\times 1.00$
Span 5	Dead $\times 1.35$
	Snow $\times 1.50$
	Wind $\times 1.00$
Support F	Dead $\times 1.35$
	Snow $\times 1.50$
	Wind $\times 1.00$
Span 6	Dead $\times 1.35$
	Snow $\times 1.50$
	Wind $\times 1.00$
Support G	Dead $\times 1.35$
	Snow $\times 1.50$
	Wind $\times 1.00$

Analysis results

Maximum moment;

$$M_{\max} = 1.573 \text{ kNm}; \quad M_{\min} = -2.173 \text{ kNm}$$

Design moment;

$$M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 2.173 \text{ kNm}$$

Maximum shear;

$$F_{\max} = 2.842 \text{ kN}; \quad F_{\min} = -2.956 \text{ kN}$$

Design shear;

$$F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = 2.956 \text{ kN}$$

Total load on beam;

$$W_{\text{tot}} = 14.111 \text{ kN}$$

Reactions at support A;

$$R_{A_{\max}} = 0.000 \text{ kN}; \quad R_{A_{\min}} = 0.000 \text{ kN}$$

Unfactored dead load reaction at support A;

$$R_{A_{\text{Dead}}} = 0.000 \text{ kN}$$

Unfactored snow load reaction at support A;

$$R_{A_{\text{Snow}}} = 0.000 \text{ kN}$$

Unfactored wind load reaction at support A;

$$R_{A_{\text{Wind}}} = 0.000 \text{ kN}$$

Reactions at support B;

$$R_{B_{\max}} = 1.907 \text{ kN}; \quad R_{B_{\min}} = 1.907 \text{ kN}$$

Unfactored dead load reaction at support B;

$$R_{B_{\text{Dead}}} = 0.790 \text{ kN}$$

Unfactored snow load reaction at support B;

$$R_{B_{\text{Snow}}} = 0.297 \text{ kN}$$

Unfactored wind load reaction at support B;

$$R_{B_{\text{Wind}}} = 0.394 \text{ kN}$$

Reactions at support C;

$$R_{C_{\max}} = 2.563 \text{ kN}; \quad R_{C_{\min}} = 2.563 \text{ kN}$$

Unfactored dead load reaction at support C;

$$R_{C_{\text{Dead}}} = 1.062 \text{ kN}$$

Unfactored snow load reaction at support C;

$$R_{C_{\text{Snow}}} = 0.400 \text{ kN}$$

Unfactored wind load reaction at support C;

$$R_{C_{\text{Wind}}} = 0.529 \text{ kN}$$

Reactions at support D;

$$R_{D_{\max}} = 4.819 \text{ kN}; \quad R_{D_{\min}} = 4.819 \text{ kN}$$

Unfactored dead load reaction at support D;

$$R_{D_{\text{Dead}}} = 1.997 \text{ kN}$$

Unfactored snow load reaction at support D;

$$R_{D_{\text{Snow}}} = 0.752 \text{ kN}$$

Unfactored wind load reaction at support D;

$$R_{D_{\text{Wind}}} = 0.995 \text{ kN}$$

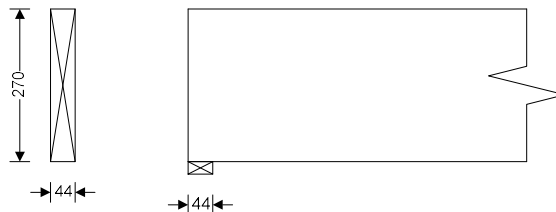
Reactions at support E;

$$R_{E_{\max}} = 5.389 \text{ kN}; \quad R_{E_{\min}} = 5.389 \text{ kN}$$



PROJECT: PROPOSED MOBILE HOUSE				Job Ref.	
STRUCTURAL CALCULATIONS: TIMBER RAFTER DESIGN CHECK PER EN1995				Sheet no./rev.	
Calc. by	Date	Chk'd by	Date	App'd by	Date

Unfactored dead load reaction at support E;	$R_{E_Dead} = 2.233 \text{ kN}$	
Unfactored snow load reaction at support E;	$R_{E_Snow} = 0.841 \text{ kN}$	
Unfactored wind load reaction at support E;	$R_{E_Wind} = 1.113 \text{ kN}$	
Reactions at support F;	$R_{F_max} = -0.566 \text{ kN};$	$R_{F_min} = -0.566 \text{ kN}$
Unfactored dead load reaction at support F;	$R_{F_Dead} = -0.235 \text{ kN}$	
Unfactored snow load reaction at support F;	$R_{F_Snow} = -0.088 \text{ kN}$	
Unfactored wind load reaction at support F;	$R_{F_Wind} = -0.117 \text{ kN}$	
Reactions at support G;	$R_{G_max} = 0.000 \text{ kN};$	$R_{G_min} = 0.000 \text{ kN}$
Unfactored dead load reaction at support G;	$R_{G_Dead} = 0.000 \text{ kN}$	
Unfactored snow load reaction at support G;	$R_{G_Snow} = 0.000 \text{ kN}$	
Unfactored wind load reaction at support G;	$R_{G_Wind} = 0.000 \text{ kN}$	



Timber section details

Breadth of timber sections;	$b = 44 \text{ mm}$
Depth of timber sections;	$h = 270 \text{ mm}$
Number of timber sections in member;	$N = 1$
Overall breadth of timber member;	$b_b = N \times b = 44 \text{ mm}$
Timber strength class - EN 338:2016 Table 1;	C24

Member details

Load duration - cl.2.3.1.2;	Short-term
Service class of timber - cl.2.3.1.3;	1
Length of span 1;	$L_{s1} = 400 \text{ mm}$
Length of span 2;	$L_{s2} = 2726 \text{ mm}$
Length of span 3;	$L_{s3} = 2400 \text{ mm}$
Length of span 4;	$L_{s4} = 4970 \text{ mm}$
Length of span 5;	$L_{s5} = 1200 \text{ mm}$
Length of span 6;	$L_{s6} = 400 \text{ mm}$
Length of bearing;	$L_b = 44 \text{ mm}$

Section properties

Cross sectional area of member;	$A = N \times b \times h = 11880 \text{ mm}^2$
Section modulus;	$W_y = N \times b \times h^2 / 6 = 534600 \text{ mm}^3$
	$W_z = h \times (N \times b)^2 / 6 = 87120 \text{ mm}^3$
Second moment of area;	$I_y = N \times b \times h^3 / 12 = 72171000 \text{ mm}^4$
	$I_z = h \times (N \times b)^3 / 12 = 1916640 \text{ mm}^4$
Radius of gyration;	$r_y = \sqrt{I_y / A} = 77.9 \text{ mm}$
	$r_z = \sqrt{I_z / A} = 12.7 \text{ mm}$



PROJECT: PROPOSED MOBILE HOUSE				Job Ref.	
STRUCTURAL CALCULATIONS: TIMBER RAFTER DESIGN CHECK PER EN1995				Sheet no./rev.	
Calc. by	Date	Chk'd by	Date	App'd by	Date

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3; $\gamma_M = 1.300$

Modification factors

Modification factor for load duration and moisture content - Table 3.1

$$k_{mod} = 0.900$$

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; $k_{c,90} = 1.500$

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; $\sigma_{c,90,d} = R_{E_max} / (N \times b \times (L_b + 2 \times \min(L_b, 30 \text{ mm}))) = 1.178 \text{ N/mm}^2$

Design compressive strength; $f_{c,90,d} = k_{mod} \times k_{sys} \times k_{c,90} \times f_{c,90,k} / \gamma_M = 2.596 \text{ N/mm}^2$

$$\sigma_{c,90,d} / f_{c,90,d} = 0.454$$

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress; $\sigma_{m,d} = M / W_y = 4.064 \text{ N/mm}^2$

Design bending strength; $f_{m,d} = k_{h,m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m,k} / \gamma_M = 16.615 \text{ N/mm}^2$

$$\sigma_{m,d} / f_{m,d} = 0.245$$

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress; $\tau_d = 3 \times F / (2 \times k_{cr} \times A) = 0.557 \text{ N/mm}^2$

Permissible shear stress; $f_{v,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.769 \text{ N/mm}^2$

$$\tau_d / f_{v,d} = 0.201$$

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit; $\delta_{lim} = 0.003 \times L_{s1} = 1.200 \text{ mm}$

Instantaneous deflection due to permanent load; $\delta_{instG} = -0.138 \text{ mm}$

Final deflection due to permanent load; $\delta_{finG} = \delta_{instG} \times (1 + k_{def}) = -0.221 \text{ mm}$

Instantaneous deflection due to variable load; $\delta_{instQ} = -0.121 \text{ mm}$

Factor for quasi-permanent variable action; $\psi_2 = 0.3$

Final deflection due to variable load; $\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def}) = -0.143 \text{ mm}$

Total final deflection; $\delta_{fin} = \delta_{finG} + \delta_{finQ} = -0.364 \text{ mm}$

$$\text{abs}(\delta_{fin}) / \delta_{lim} = 0.303$$

PASS - Total final deflection is less than the deflection limit

;



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PROJECT:

PROPOSED MOBILE HOUSE

Job Ref.

STRUCTURAL CALCULATIONS:

TIMBER ROOF BEAM DESIGN CHECK PER EN1995

Sheet no./rev.

Calc. by

Date

Chk'd by

Date

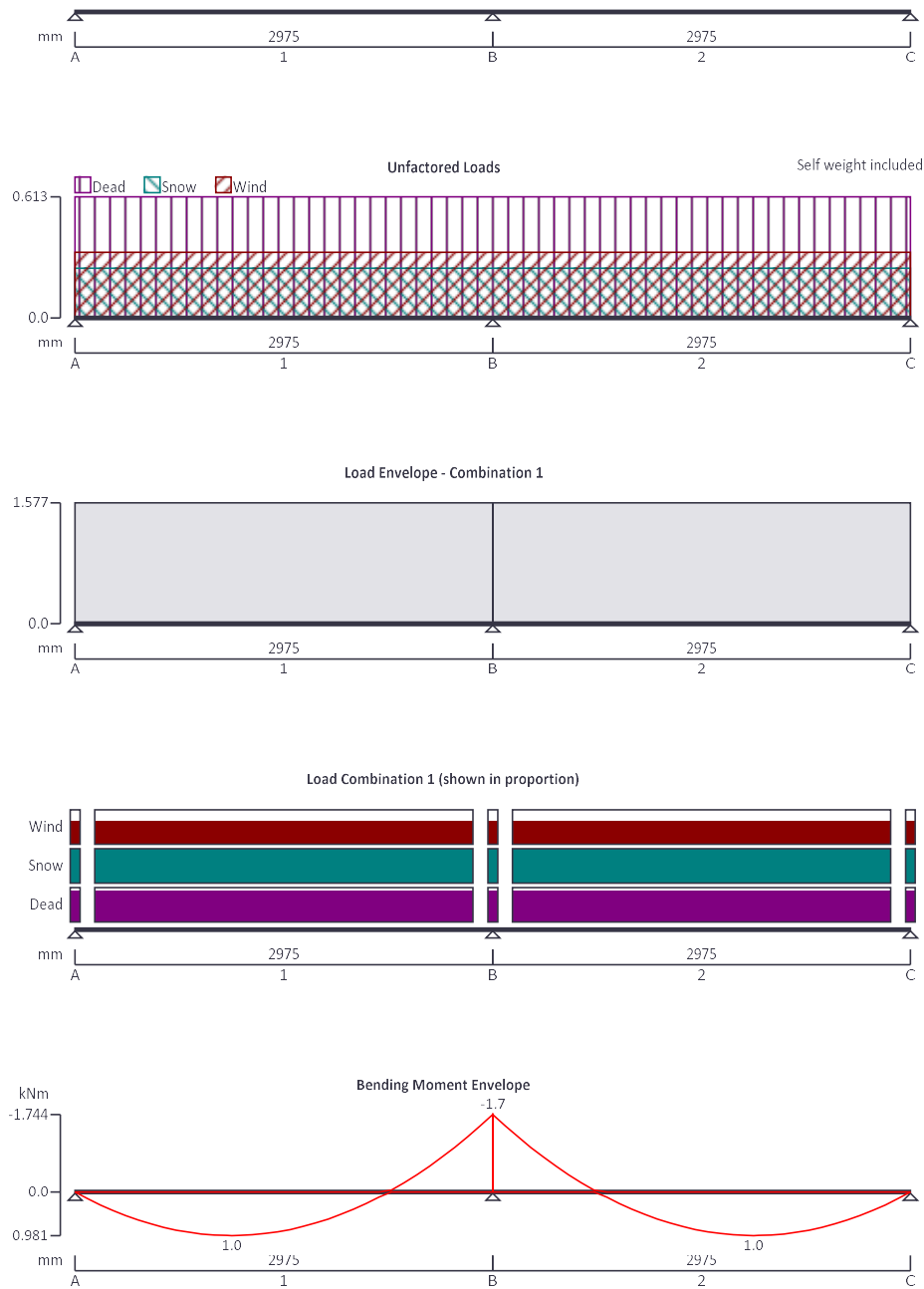
App'd by

Date

TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the recommended values

Tedds calculation version 1.7.04





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PROJECT:

PROPOSED MOBILE HOUSE

Job Ref.

STRUCTURAL CALCULATIONS:

TIMBER ROOF BEAM DESIGN CHECK PER EN1995

Sheet no./rev.

Calc. by

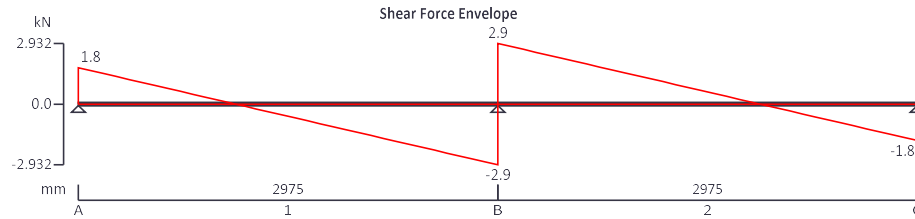
Date

Chk'd by

Date

App'd by

Date



Applied loading

Beam loads

0.681KPa x 0.9m Tributary Width

0.28KPa x 0.9m Tributary Width

0.37KPa x 0.9m Tributary Width

Dead self weight of beam $\times 1$

Dead full UDL 0.613 kN/m

Snow full UDL 0.252 kN/m

Wind full UDL 0.333 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.35$

Snow $\times 1.50$

Wind $\times 1.00$

Span 1

Dead $\times 1.35$

Snow $\times 1.50$

Wind $\times 1.00$

Support B

Dead $\times 1.35$

Snow $\times 1.50$

Wind $\times 1.00$

Span 2

Dead $\times 1.35$

Snow $\times 1.50$

Wind $\times 1.00$

Support C

Dead $\times 1.35$

Snow $\times 1.50$

Wind $\times 1.00$

Analysis results

Maximum moment;

$M_{\max} = 0.981 \text{ kNm};$

$M_{\min} = -1.744 \text{ kNm}$

Design moment;

$M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 1.744 \text{ kNm}$

Maximum shear;

$F_{\max} = 2.932 \text{ kN};$

$F_{\min} = -2.932 \text{ kN}$

Design shear;

$F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = 2.932 \text{ kN}$

Total load on beam;

$W_{\text{tot}} = 9.381 \text{ kN}$

Reactions at support A;

$R_{A_{\max}} = 1.759 \text{ kN};$

$R_{A_{\min}} = 1.759 \text{ kN}$

Unfactored dead load reaction at support A;

$R_{A_{\text{Dead}}} = 0.715 \text{ kN}$

Unfactored snow load reaction at support A;

$R_{A_{\text{Snow}}} = 0.281 \text{ kN}$

Unfactored wind load reaction at support A;

$R_{A_{\text{Wind}}} = 0.372 \text{ kN}$

Reactions at support B;

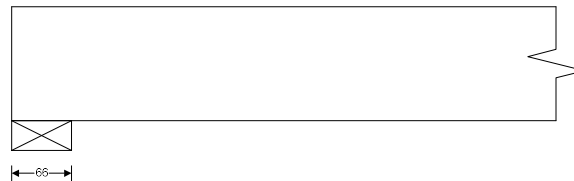
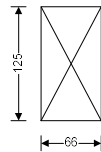
$R_{B_{\max}} = 5.863 \text{ kN};$

$R_{B_{\min}} = 5.863 \text{ kN}$



PROJECT: PROPOSED MOBILE HOUSE				Job Ref.	
STRUCTURAL CALCULATIONS: TIMBER ROOF BEAM DESIGN CHECK PER EN1995				Sheet no./rev.	
Calc. by	Date	Chk'd by	Date	App'd by	Date

Unfactored dead load reaction at support B; $R_{B_Dead} = 2.385 \text{ kN}$
 Unfactored snow load reaction at support B; $R_{B_Snow} = 0.937 \text{ kN}$
 Unfactored wind load reaction at support B; $R_{B_Wind} = 1.238 \text{ kN}$
 Reactions at support C; $R_{C_max} = 1.759 \text{ kN}; R_{C_min} = 1.759 \text{ kN}$
 Unfactored dead load reaction at support C; $R_{C_Dead} = 0.715 \text{ kN}$
 Unfactored snow load reaction at support C; $R_{C_Snow} = 0.281 \text{ kN}$
 Unfactored wind load reaction at support C; $R_{C_Wind} = 0.372 \text{ kN}$



Timber section details

Breadth of timber sections; $b = 66 \text{ mm}$
 Depth of timber sections; $h = 125 \text{ mm}$
 Number of timber sections in member; $N = 1$
 Overall breadth of timber member; $b_b = N \times b = 66 \text{ mm}$
 Timber strength class - EN 338:2016 Table 1; **C24**

Member details

Load duration - cl.2.3.1.2; **Short-term**
 Service class of timber - cl.2.3.1.3; **1**
 Length of span 1; $L_{s1} = 2975 \text{ mm}$
 Length of span 2; $L_{s2} = 2975 \text{ mm}$
 Length of bearing; $L_b = 66 \text{ mm}$

Section properties

Cross sectional area of member; $A = N \times b \times h = 8250 \text{ mm}^2$
 Section modulus; $W_y = N \times b \times h^2 / 6 = 171875 \text{ mm}^3$
 $W_z = h \times (N \times b)^2 / 6 = 90750 \text{ mm}^3$
 Second moment of area; $I_y = N \times b \times h^3 / 12 = 10742188 \text{ mm}^4$
 $I_z = h \times (N \times b)^3 / 12 = 2994750 \text{ mm}^4$
 Radius of gyration; $r_y = \sqrt{I_y / A} = 36.1 \text{ mm}$
 $r_z = \sqrt{I_z / A} = 19.1 \text{ mm}$

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3; $\gamma_M = 1.300$

Modification factors

Modification factor for load duration and moisture content - Table 3.1

$k_{mod} = 0.900$

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

Depth factor for bending - exp.3.1; $k_{h,m} = \min((150 \text{ mm} / h)^{0.2}, 1.3) = 1.037$

Depth factor for tension - exp.3.1; $k_{h,t} = \min((150 \text{ mm} / \max(b, h))^{0.2}, 1.3) = 1.037$

Bending stress re-distribution factor - cl.6.1.6(2); $k_m = 0.700$

Crack factor for shear resistance - cl.6.1.7(2); $k_{cr} = 0.670$



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PROJECT: PROPOSED MOBILE HOUSE				Job Ref.	
STRUCTURAL CALCULATIONS: TIMBER ROOF BEAM DESIGN CHECK PER EN1995				Sheet no./rev.	
Calc. by	Date	Chk'd by	Date	App'd by	Date

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;

$$\sigma_{c,90,d} = R_{B,max} / (N \times b \times L_b) = 1.346 \text{ N/mm}^2$$

Design compressive strength;

$$f_{c,90,d} = k_{mod} \times k_{sys} \times k_{c,90} \times f_{c,90,k} / \gamma_M = 1.731 \text{ N/mm}^2$$

$$\sigma_{c,90,d} / f_{c,90,d} = 0.778$$

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress;

$$\sigma_{m,d} = M / W_y = 10.149 \text{ N/mm}^2$$

Design bending strength;

$$f_{m,d} = k_{h,m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m,k} / \gamma_M = 17.232 \text{ N/mm}^2$$

$$\sigma_{m,d} / f_{m,d} = 0.589$$

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress;

$$\tau_d = 3 \times F / (2 \times k_{cr} \times A) = 0.796 \text{ N/mm}^2$$

Permissible shear stress;

$$f_{v,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.769 \text{ N/mm}^2$$

$$\tau_d / f_{v,d} = 0.287$$

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit;

$$\delta_{lim} = 0.003 \times L_{s1} = 8.925 \text{ mm}$$

Instantaneous deflection due to permanent load;

$$\delta_{instG} = 2.386 \text{ mm}$$

Final deflection due to permanent load;

$$\delta_{finG} = \delta_{instG} \times (1 + k_{def}) = 3.818 \text{ mm}$$

Instantaneous deflection due to variable load;

$$\delta_{instQ} = 2.177 \text{ mm}$$

Factor for quasi-permanent variable action;

$$\psi_2 = 0.3$$

Final deflection due to variable load;

$$\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def}) = 2.569 \text{ mm}$$

Total final deflection;

$$\delta_{fin} = \delta_{finG} + \delta_{finQ} = 6.387 \text{ mm}$$

$$\delta_{fin} / \delta_{lim} = 0.716$$

PASS - Total final deflection is less than the deflection limit

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PROJECT:

PROPOSED MOBILE HOUSE

Job Ref.

STRUCTURAL CALCULATIONS:

FLOOR JOIST DESIGN CHECK PER EN1995

Sheet no./rev.

Calc. by

Date

Chk'd by

Date

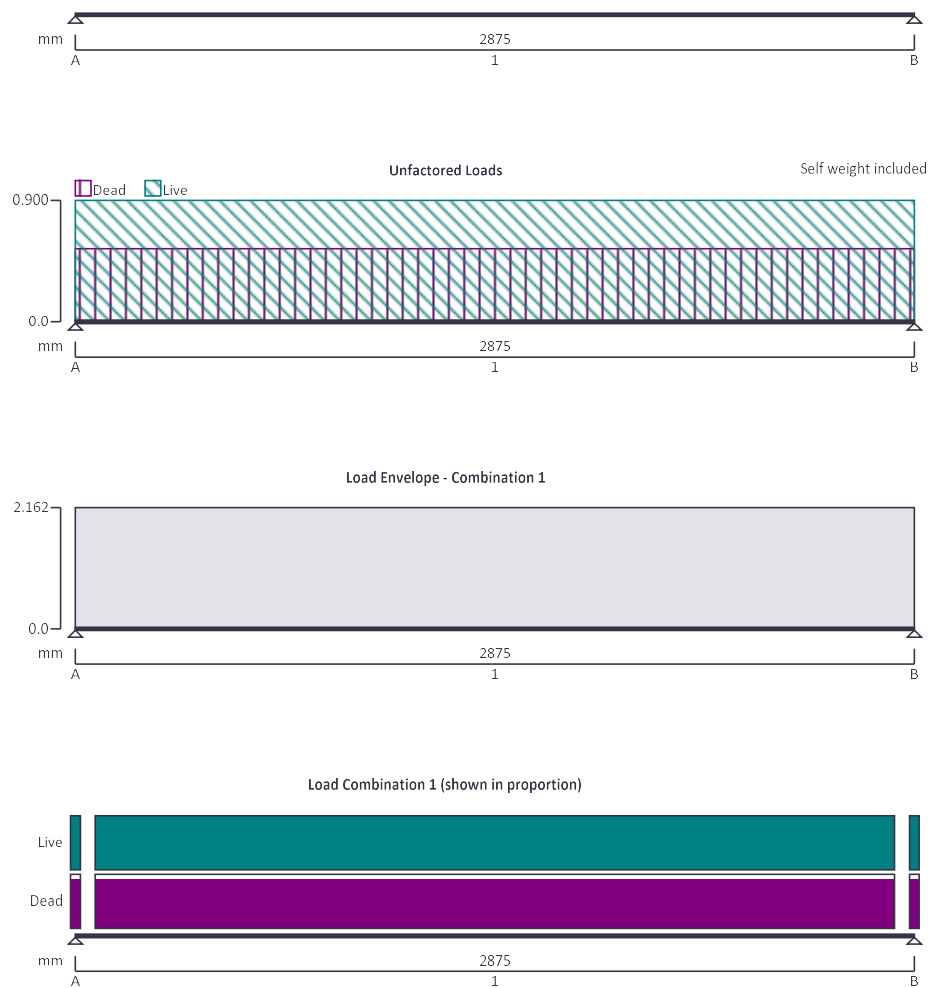
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Date

TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

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PROJECT:

PROPOSED MOBILE HOUSE

Job Ref.

STRUCTURAL CALCULATIONS:

FLOOR JOIST DESIGN CHECK PER EN1995

Sheet no./rev.

Calc. by

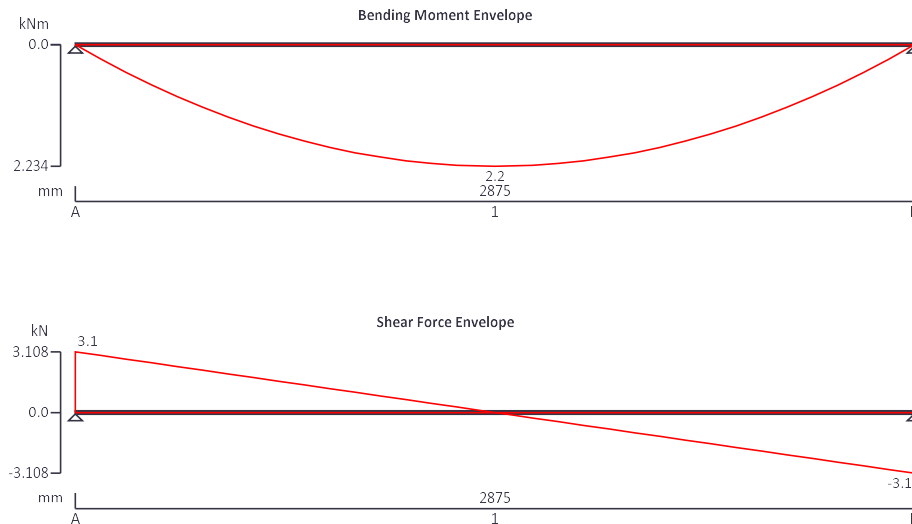
Date

Chk'd by

Date

App'd by

Date



Applied loading

Beam loads

0.903KPa x 0.60m Tributray Width
1.5KPa x 0.60m Tributray Width

Dead self weight of beam $\times 1$
Dead full UDL 0.542 kN/m
Live full UDL 0.900 kN/m

Load combinations

Load combination 1

Support A	Dead $\times 1.35$ Live $\times 1.50$
Span 1	Dead $\times 1.35$ Live $\times 1.50$
Support B	Dead $\times 1.35$ Live $\times 1.50$

Analysis results

Maximum moment;
Design moment;
Maximum shear;
Design shear;
Total load on beam;
Reactions at support A;
Unfactored dead load reaction at support A;
Unfactored live load reaction at support A;
Reactions at support B;
Unfactored dead load reaction at support B;
Unfactored live load reaction at support B;

$M_{\max} = 2.234$ kNm; $M_{\min} = 0.000$ kNm
 $M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 2.234$ kNm
 $F_{\max} = 3.108$ kN; $F_{\min} = -3.108$ kN
 $F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = 3.108$ kN
 $W_{\text{tot}} = 6.215$ kN
 $R_{A_{\max}} = 3.108$ kN; $R_{A_{\min}} = 3.108$ kN
 $R_{A_{\text{Dead}}} = 0.864$ kN
 $R_{A_{\text{Live}}} = 1.294$ kN
 $R_{B_{\max}} = 3.108$ kN; $R_{B_{\min}} = 3.108$ kN
 $R_{B_{\text{Dead}}} = 0.864$ kN
 $R_{B_{\text{Live}}} = 1.294$ kN



**Quick-
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PROJECT:

PROPOSED MOBILE HOUSE

Job Ref.

STRUCTURAL CALCULATIONS:

FLOOR JOIST DESIGN CHECK PER EN1995

Sheet no./rev.

Calc. by

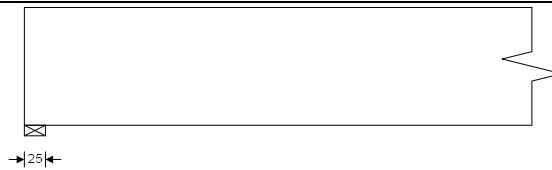
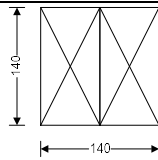
Date

Chk'd by

Date

App'd by

Date



Timber section details

Breadth of timber sections;

b = 70 mm

Depth of timber sections;

h = 140 mm

Number of timber sections in member;

N = 2

Overall breadth of timber member;

b_b = N × b = 140 mm

Timber strength class - EN 338:2016 Table 1;

C16

Member details

Load duration - cl.2.3.1.2;

Permanent

Service class of timber - cl.2.3.1.3;

1

Length of span;

L_{s1} = 2875 mm

Length of bearing;

L_b = 25 mm

Section properties

Cross sectional area of member;

A = N × b × h = 19600 mm²

Section modulus;

W_y = N × b × h² / 6 = 457333 mm³

W_z = h × (N × b)² / 6 = 457333 mm³

Second moment of area;

I_y = N × b × h³ / 12 = 32013333 mm⁴

I_z = h × (N × b)³ / 12 = 32013333 mm⁴

Radius of gyration;

r_y = √(I_y / A) = 40.4 mm

r_z = √(I_z / A) = 40.4 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} = 0.600

Deformation factor for service classes - Table 3.2;

k_{def} = 0.600

Depth factor for bending - exp.3.1;

k_{h,m} = min((150 mm / h)^{0.2}, 1.3) = 1.014

Depth factor for tension - exp.3.1;

k_{h,t} = min((150 mm / max(b, h))^{0.2}, 1.3) = 1.014

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;

σ_{c,90,d} = R_{A,max} / (N × b × L_b) = 0.888 N/mm²

Design compressive strength;

f_{c,90,d} = k_{mod} × k_{sys} × k_{c,90} × f_{c,90,k} / γ_M = 1.015 N/mm²

σ_{c,90,d} / f_{c,90,d} = 0.874

	PROJECT:				Job Ref.	
	PROPOSED MOBILE HOUSE					
	STRUCTURAL CALCULATIONS:				Sheet no./rev.	
FLOOR JOIST DESIGN CHECK PER EN1995						
Calc. by	Date	Chk'd by	Date	App'd by	Date	

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress;

$$\sigma_{m,d} = M / W_y = \mathbf{4.884 \text{ N/mm}^2}$$

Design bending strength;

$$f_{m,d} = k_{h,m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m,k} / \gamma_M = \mathbf{7.487 \text{ N/mm}^2}$$

$$\sigma_{m,d} / f_{m,d} = \mathbf{0.652}$$

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress;

$$\tau_d = 3 \times F / (2 \times k_{cr} \times A) = \mathbf{0.355 \text{ N/mm}^2}$$

Permissible shear stress;

$$f_{v,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = \mathbf{1.477 \text{ N/mm}^2}$$

$$\tau_d / f_{v,d} = \mathbf{0.240}$$

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit;

$$\delta_{lim} = 0.003 \times L_{s1} = \mathbf{8.625 \text{ mm}}$$

Instantaneous deflection due to permanent load;

$$\delta_{instG} = \mathbf{2.165 \text{ mm}}$$

Final deflection due to permanent load;

$$\delta_{finG} = \delta_{instG} \times (1 + k_{def}) = \mathbf{3.464 \text{ mm}}$$

Instantaneous deflection due to variable load;

$$\delta_{instQ} = \mathbf{3.240 \text{ mm}}$$

Factor for quasi-permanent variable action;

$$\psi_2 = \mathbf{0.3}$$

Final deflection due to variable load;

$$\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def}) = \mathbf{3.823 \text{ mm}}$$

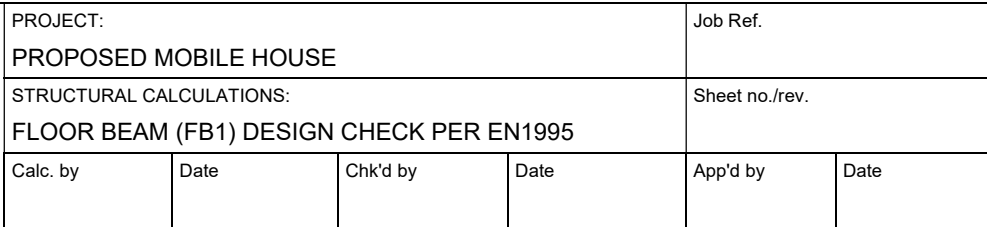
Total final deflection;

$$\delta_{fin} = \delta_{finG} + \delta_{finQ} = \mathbf{7.287 \text{ mm}}$$

$$\delta_{fin} / \delta_{lim} = \mathbf{0.845}$$

PASS - Total final deflection is less than the deflection limit

;



In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1



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PROJECT:

PROPOSED MOBILE HOUSE

Job Ref.

STRUCTURAL CALCULATIONS:

FLOOR BEAM (FB1) DESIGN CHECK PER EN1995

Sheet no./rev.

Calc. by

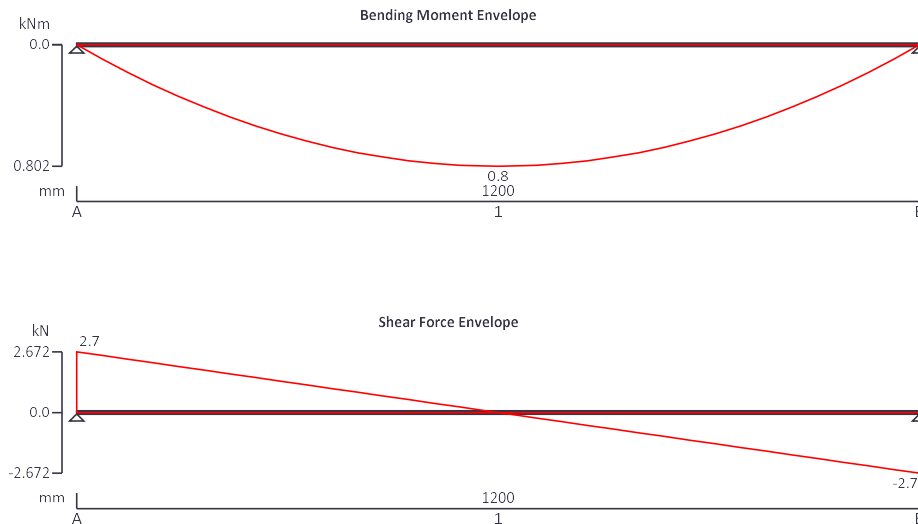
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Date

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Date



Applied loading

Beam loads

0.943KPa x 1.23m Tributray Width

1.5KPa x 1.23m Tributray Width

Dead self weight of beam $\times 1$

Dead full UDL 1.160 kN/m

Live full UDL 1.845 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.35$

Live $\times 1.50$

Span 1

Dead $\times 1.35$

Live $\times 1.50$

Support B

Dead $\times 1.35$

Live $\times 1.50$

Analysis results

Maximum moment;

$M_{\max} = 0.802$ kNm;

$M_{\min} = 0.000$ kNm

Design moment;

$M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 0.802$ kNm

Maximum shear;

$F_{\max} = 2.672$ kN;

$F_{\min} = -2.672$ kN

Design shear;

$F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = 2.672$ kN

Total load on beam;

$W_{\text{tot}} = 5.345$ kN

Reactions at support A;

$R_{A_{\max}} = 2.672$ kN;

$R_{A_{\min}} = 2.672$ kN

Unfactored dead load reaction at support A;

$R_{A_{\text{Dead}}} = 0.750$ kN

Unfactored live load reaction at support A;

$R_{A_{\text{Live}}} = 1.107$ kN

Reactions at support B;

$R_{B_{\max}} = 2.672$ kN;

$R_{B_{\min}} = 2.672$ kN

Unfactored dead load reaction at support B;

$R_{B_{\text{Dead}}} = 0.750$ kN

Unfactored live load reaction at support B;

$R_{B_{\text{Live}}} = 1.107$ kN



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PROJECT:

PROPOSED MOBILE HOUSE

Job Ref.

STRUCTURAL CALCULATIONS:

FLOOR BEAM (FB1) DESIGN CHECK PER EN1995

Sheet no./rev.

Calc. by

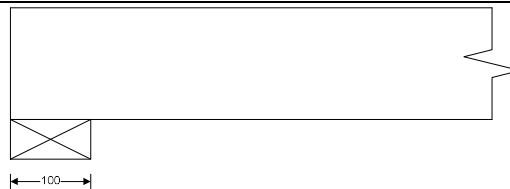
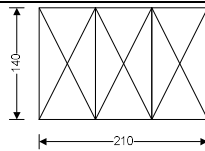
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Date



Timber section details

Breadth of timber sections;

b = 70 mm

Depth of timber sections;

h = 140 mm

Number of timber sections in member;

N = 3

Overall breadth of timber member;

b_b = N × b = 210 mm

Timber strength class - EN 338:2016 Table 1;

C16

Member details

Load duration - cl.2.3.1.2;

Permanent

Service class of timber - cl.2.3.1.3;

1

Length of span;

L_{st} = 1200 mm

Length of bearing;

L_b = 100 mm

Section properties

Cross sectional area of member;

A = N × b × h = 29400 mm²

Section modulus;

W_y = N × b × h² / 6 = 686000 mm³

W_z = h × (N × b)² / 6 = 1029000 mm³

Second moment of area;

I_y = N × b × h³ / 12 = 48020000 mm⁴

I_z = h × (N × b)³ / 12 = 108045000 mm⁴

Radius of gyration;

r_y = √(I_y / A) = 40.4 mm

r_z = √(I_z / A) = 60.6 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} = 0.600

Deformation factor for service classes - Table 3.2;

k_{def} = 0.600

Depth factor for bending - exp.3.1;

k_{h,m} = min((150 mm / h)^{0.2}, 1.3) = 1.014

Depth factor for tension - exp.3.1;

k_{h,t} = min((150 mm / max(b, h))^{0.2}, 1.3) = 1.014

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;

σ_{c,90,d} = R_{A,max} / (N × b × L_b) = 0.127 N/mm²

Design compressive strength;

f_{c,90,d} = k_{mod} × k_{sys} × k_{c,90} × f_{c,90,k} / γ_M = 1.015 N/mm²

σ_{c,90,d} / f_{c,90,d} = 0.125



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PROJECT:

PROPOSED MOBILE HOUSE

Job Ref.

STRUCTURAL CALCULATIONS:

FLOOR BEAM (FB1) DESIGN CHECK PER EN1995

Sheet no./rev.

Calc. by

Date

Chk'd by

Date

App'd by

Date

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress;

$$\sigma_{m,d} = M / W_y = \mathbf{1.169 \text{ N/mm}^2}$$

Design bending strength;

$$f_{m,d} = k_{h,m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m,k} / \gamma_M = \mathbf{7.487 \text{ N/mm}^2}$$

$$\sigma_{m,d} / f_{m,d} = \mathbf{0.156}$$

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress;

$$\tau_d = 3 \times F / (2 \times k_{cr} \times A) = \mathbf{0.204 \text{ N/mm}^2}$$

Permissible shear stress;

$$f_{v,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = \mathbf{1.477 \text{ N/mm}^2}$$

$$\tau_d / f_{v,d} = \mathbf{0.138}$$

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit;

$$\delta_{lim} = 0.003 \times L_{s1} = \mathbf{3.600 \text{ mm}}$$

Instantaneous deflection due to permanent load;

$$\delta_{instG} = \mathbf{0.106 \text{ mm}}$$

Final deflection due to permanent load;

$$\delta_{finG} = \delta_{instG} \times (1 + k_{def}) = \mathbf{0.170 \text{ mm}}$$

Instantaneous deflection due to variable load;

$$\delta_{instQ} = \mathbf{0.157 \text{ mm}}$$

Factor for quasi-permanent variable action;

$$\psi_2 = \mathbf{0.3}$$

Final deflection due to variable load;

$$\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def}) = \mathbf{0.185 \text{ mm}}$$

Total final deflection;

$$\delta_{fin} = \delta_{finG} + \delta_{finQ} = \mathbf{0.355 \text{ mm}}$$

$$\delta_{fin} / \delta_{lim} = \mathbf{0.099}$$

PASS - Total final deflection is less than the deflection limit

;



PROJECT: PROPOSED MOBILE HOUSE				Job Ref.	
STRUCTURAL CALCULATIONS: TIMBER COLUMN (C1) DESIGN CHECK PER EN1995				Sheet no./rev.	
Calc. by	Date	Chk'd by	Date	App'd by	Date

TIMBER MEMBER DESIGN (EN1995-1-1:2004)

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

Tedds calculation version 2.2.00

Design section 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3; $\gamma_M = 1.300$

Member details

Load duration - cl.2.3.1.2; Short-term

Service class - cl.2.3.1.3; 1

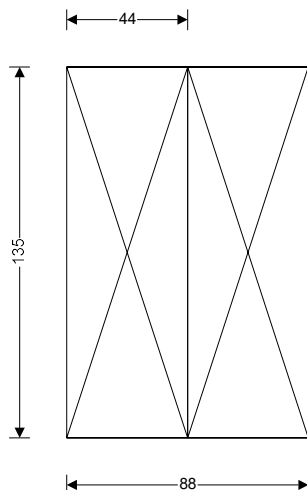
Timber section details

Number of timber sections in member; $N = 2$

Breadth of sections; $b = 44 \text{ mm}$

Depth of sections; $h = 135 \text{ mm}$

Timber strength class - EN 338:2016 Table 1; **C24**



2/44x135 timber sections

Cross-sectional area, A , 11880 mm²

Section modulus, W_y , 267300 mm³

Section modulus, W_z , 87120 mm³

Second moment of area, I_y , 18042750 mm⁴

Second moment of area, I_z , 1916640 mm⁴

Radius of gyration, i_y , 39 mm

Radius of gyration, i_z , 12.7 mm

Timber strength class **C24**

Characteristic bending strength, $f_{m,k}$, 24 N/mm²

Characteristic shear strength, $f_{v,k}$, 4 N/mm²

Characteristic compression strength parallel to grain, $f_{c,0,k}$, 21 N/mm²

Characteristic compression strength perpendicular to grain, $f_{c,90,k}$, 2.5 N/mm²

Characteristic tension strength parallel to grain, $f_{t,0,k}$, 14.5 N/mm²

Mean modulus of elasticity, $E_{0,mean}$, 11000 N/mm²

Fifth percentile modulus of elasticity, $E_{0.05}$, 7400 N/mm²

Shear modulus of elasticity, G_{mean} , 690 N/mm²

Characteristic density, ρ_k , 350 kg/m³

Mean density, ρ_{mean} , 420 kg/m³

Span details

Unbraced length - Major axis; $L_y = 2210 \text{ mm}$

Effective length - Major axis; $L_{e,y} = L_y = 2210 \text{ mm}$

Unbraced length - Minor axis; $L_z = 0 \text{ mm}$

Analysis results

Design axial compression force; $P_d = 6.59 \text{ kN}$

Section s1 results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm ²	16.0	0.6	0.035	PASS
Column stability check				0.048	PASS

Modification factors

Duration of load and moisture content - Table 3.1; $k_{mod} = 0.9$



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PROJECT: PROPOSED MOBILE HOUSE				Job Ref.	
STRUCTURAL CALCULATIONS: TIMBER COLUMN (C1) DESIGN CHECK PER EN1995				Sheet no./rev.	
Calc. by	Date	Chk'd by	Date	App'd by	Date

Deformation factor - Table 3.2;

$$k_{def} = 0.6$$

System strength factor - cl.6.6;

$$k_{sys} = 1.1$$

Check compression parallel to the grain - cl.6.1.4

Design axial compression;

$$P_d = 6.59 \text{ kN}$$

Design compressive stress;

$$\sigma_{c,0,d} = P_d / A = 0.555 \text{ N/mm}^2$$

Design compressive strength;

$$f_{c,0,d} = k_{mod} \times k_{sys} \times f_{c,0,k} / \gamma_M = 15.992 \text{ N/mm}^2$$

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

PASS - Design parallel compression strength exceeds design parallel compression stress

Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis bending;

$$L_{e,y} = 2210 \text{ mm}$$

Slenderness ratio;

$$\lambda_y = L_{e,y} / i_y = 56.709$$

Relative slenderness ratio - exp. 6.21;

$$\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{f_{c,0,k} / E_{0.05}} = 0.962$$

Effective length for z-axis bending;

$$L_{e,z} = 0 \text{ mm}$$

Slenderness ratio;

$$\lambda_z = L_{e,z} / i_z = 0$$

Relative slenderness ratio - exp. 6.22;

$$\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{f_{c,0,k} / E_{0.05}} = 0$$

$\lambda_{rel,y} > 0.3$ column stability check is required

Straightness factor;

$$\beta_c = 0.2$$

Instability factors - exp.6.25, 6.26, 6.27 & 6.28;

$$k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) = 1.028$$

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

$$k_{c,y} = 1 / (k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}) = 0.718$$

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

Column stability checks - exp.6.23 & 6.24;

$$\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) = 0.048$$

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

PASS - Column stability is acceptable

;

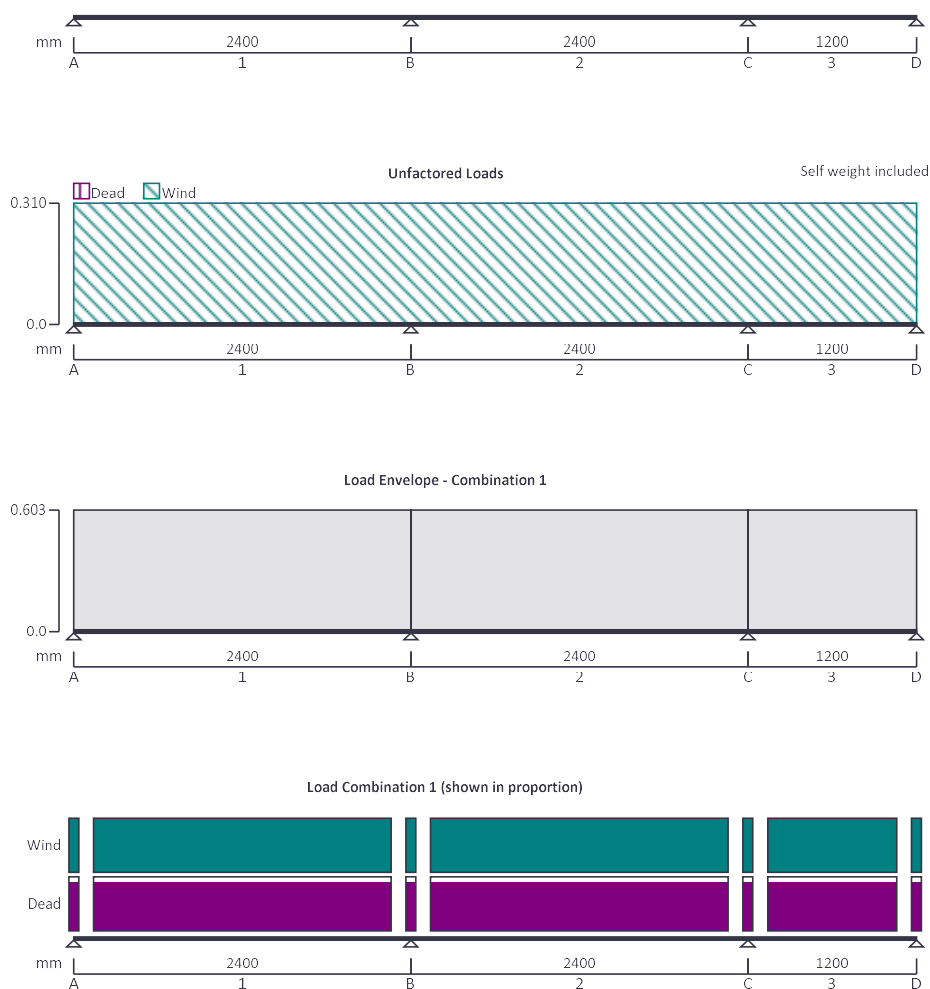


PROJECT: PROPOSED MOBILE HOUSE				Job Ref.	
STRUCTURAL CALCULATIONS: TIMBER WALL DESIGN CHECK PER EN1995				Sheet no./rev.	
Calc. by	Date	Chk'd by	Date	App'd by	Date

TIMBER WALL ANALYSIS & DESIGN TO EN1995-1-1:2004

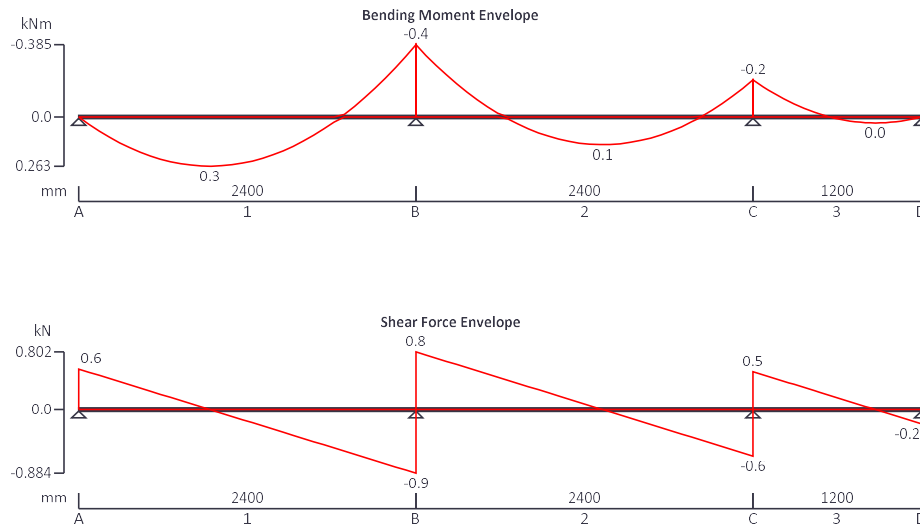
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Tedds calculation version 1.7.04





PROJECT: PROPOSED MOBILE HOUSE				Job Ref.	
STRUCTURAL CALCULATIONS: TIMBER WALL DESIGN CHECK PER EN1995				Sheet no./rev.	
Calc. by	Date	Chk'd by	Date	App'd by	Date



Applied loading

WALL loads

Dead self weight of WALL $\times 1$

0.46KPa $\times$ 0.675m Tributary Width (5No. $\times$ 0.135m) Wind full UDL 0.311 kN/m

Load combinations

Load combination 1	Support A	Dead $\times 1.35$ Wind $\times 1.50$
	Span 1	Dead $\times 1.35$ Wind $\times 1.50$
	Support B	Dead $\times 1.35$ Wind $\times 1.50$
	Span 2	Dead $\times 1.35$ Wind $\times 1.50$
	Support C	Dead $\times 1.35$ Wind $\times 1.50$
	Span 3	Dead $\times 1.35$ Wind $\times 1.50$
	Support D	Dead $\times 1.35$ Wind $\times 1.50$

Analysis results

Maximum moment;	$M_{\max} = 0.263$ kNm;	$M_{\min} = -0.385$ kNm
Design moment;	$M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 0.385$ kNm	
Maximum shear;	$F_{\max} = 0.802$ kN;	$F_{\min} = -0.884$ kN
Design shear;	$F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = 0.884$ kN	
Total load on WALL;	$W_{\text{tot}} = 3.620$ kN	
Reactions at support A;	$R_{A_{\max}} = 0.564$ kN;	$R_{A_{\min}} = 0.564$ kN



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PROJECT:

PROPOSED MOBILE HOUSE

Job Ref.

STRUCTURAL CALCULATIONS:

TIMBER WALL DESIGN CHECK PER EN1995

Sheet no./rev.

Calc. by

Date

Chk'd by

Date

App'd by

Date

Unfactored dead load reaction at support A;

$R_{A_Dead} = 0.095 \text{ kN}$

Unfactored wind load reaction at support A;

$R_{A_Wind} = 0.290 \text{ kN}$

Reactions at support B;

$R_{B_max} = 1.687 \text{ kN};$

$R_{B_min} = 1.687 \text{ kN}$

Unfactored dead load reaction at support B;

$R_{B_Dead} = 0.285 \text{ kN}$

Unfactored wind load reaction at support B;

$R_{B_Wind} = 0.868 \text{ kN}$

Reactions at support C;

$R_{C_max} = 1.172 \text{ kN};$

$R_{C_min} = 1.172 \text{ kN}$

Unfactored dead load reaction at support C;

$R_{C_Dead} = 0.198 \text{ kN}$

Unfactored wind load reaction at support C;

$R_{C_Wind} = 0.603 \text{ kN}$

Reactions at support D;

$R_{D_max} = 0.197 \text{ kN};$

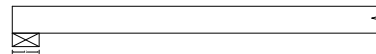
$R_{D_min} = 0.197 \text{ kN}$

Unfactored dead load reaction at support D;

$R_{D_Dead} = 0.033 \text{ kN}$

Unfactored wind load reaction at support D;

$R_{D_Wind} = 0.102 \text{ kN}$



Timber section details

Breadth of timber sections;

$b = 135 \text{ mm}$

Depth of timber sections;

$h = 44 \text{ mm}$

Number of timber sections in member;

$N = 5$

Overall breadth of timber member;

$b_b = N \times b = 675 \text{ mm}$

Timber strength class - EN 338:2016 Table 1;

C24

Member details

Load duration - cl.2.3.1.2;

Short-term

Service class of timber - cl.2.3.1.3;

1

Length of span 1;

$L_{s1} = 2400 \text{ mm}$

Length of span 2;

$L_{s2} = 2400 \text{ mm}$

Length of span 3;

$L_{s3} = 1200 \text{ mm}$

Length of bearing;

$L_b = 44 \text{ mm}$

Section properties

Cross sectional area of member;

$A = N \times b \times h = 29700 \text{ mm}^2$

Section modulus;

$W_y = N \times b \times h^2 / 6 = 217800 \text{ mm}^3$

$W_z = h \times (N \times b)^2 / 6 = 3341250 \text{ mm}^3$

Second moment of area;

$I_y = N \times b \times h^3 / 12 = 4791600 \text{ mm}^4$

$I_z = h \times (N \times b)^3 / 12 = 1127671875 \text{ mm}^4$

Radius of gyration;

$r_y = \sqrt{I_y / A} = 12.7 \text{ mm}$

$r_z = \sqrt{I_z / A} = 194.9 \text{ mm}$

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3;

$\gamma_M = 1.300$

Modification factors

Modification factor for load duration and moisture content - Table 3.1

$k_{mod} = 0.900$

Deformation factor for service classes - Table 3.2;

$k_{def} = 0.600$

Depth factor for bending - exp.3.1;

$k_{h,m} = \min((150 \text{ mm} / h)^{0.2}, 1.3) = 1.278$

Depth factor for tension - exp.3.1;

$k_{h,t} = \min((150 \text{ mm} / \max(b, h))^{0.2}, 1.3) = 1.021$



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PROJECT:

PROPOSED MOBILE HOUSE

Job Ref.

STRUCTURAL CALCULATIONS:

TIMBER WALL DESIGN CHECK PER EN1995

Sheet no./rev.

Calc. by

Date

Chk'd by

Date

App'd by

Date

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; $\sigma_{c,90,d} = R_{B,max} / (N \times b \times L_b) = 0.057 \text{ N/mm}^2$

Design compressive strength; $f_{c,90,d} = k_{mod} \times k_{sys} \times k_{c,90} \times f_{c,90,k} / \gamma_M = 1.731 \text{ N/mm}^2$

$\sigma_{c,90,d} / f_{c,90,d} = 0.033$

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress; $\sigma_{m,d} = M / W_y = 1.768 \text{ N/mm}^2$

Design bending strength; $f_{m,d} = k_{h,m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m,k} / \gamma_M = 21.234 \text{ N/mm}^2$

$\sigma_{m,d} / f_{m,d} = 0.083$

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress; $\tau_d = 3 \times F / (2 \times k_{cr} \times A) = 0.067 \text{ N/mm}^2$

Permissible shear stress; $f_{v,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.769 \text{ N/mm}^2$

$\tau_d / f_{v,d} = 0.024$

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit; $\delta_{lim} = 0.003 \times L_{s1} = 7.200 \text{ mm}$

Instantaneous deflection due to permanent load; $\delta_{instG} = 0.403 \text{ mm}$

Final deflection due to permanent load; $\delta_{finG} = \delta_{instG} \times (1 + k_{def}) = 0.645 \text{ mm}$

Instantaneous deflection due to variable load; $\delta_{instQ} = 1.228 \text{ mm}$

Factor for quasi-permanent variable action; $\psi_2 = 0.3$

Final deflection due to variable load; $\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def}) = 1.450 \text{ mm}$

Total final deflection; $\delta_{fin} = \delta_{finG} + \delta_{finQ} = 2.095 \text{ mm}$

$\delta_{fin} / \delta_{lim} = 0.291$

PASS - Total final deflection is less than the deflection limit

;



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PROJECT:
PROPOSED MOBILE HOUSE

Job Ref.

STRUCTURAL CALCULATIONS:
LIFTING STEEL FRAME SUPPORT BEAM DESIGN CHECK

Sheet no./rev.

Calc. by

Date

Chk'd by

Date

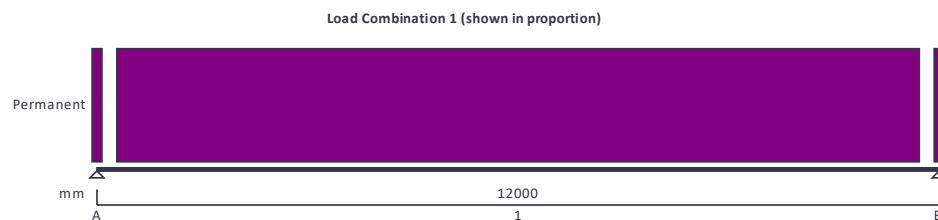
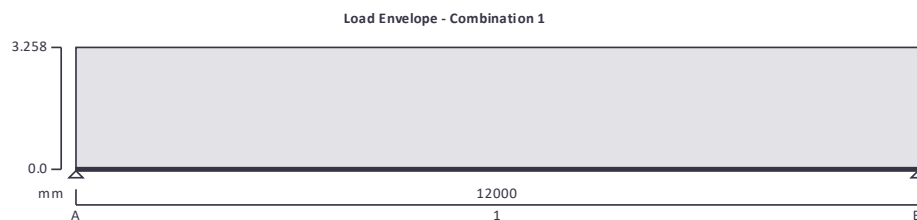
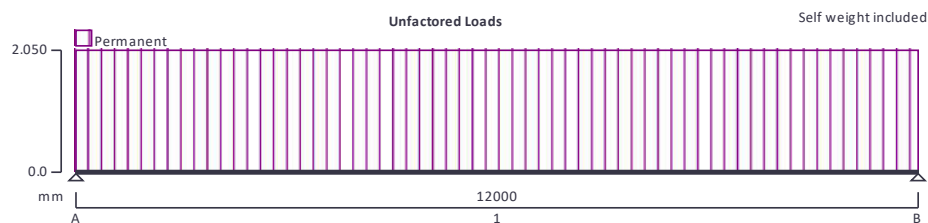
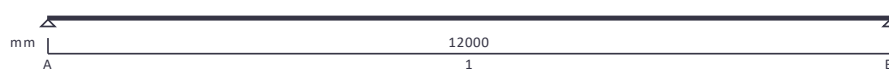
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STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14





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PROJECT:
PROPOSED MOBILE HOUSE

STRUCTURAL CALCULATIONS:
LIFTING STEEL FRAME SUPPORT BEAM DESIGN CHECK

Job Ref.

Sheet no./rev.

Calc. by

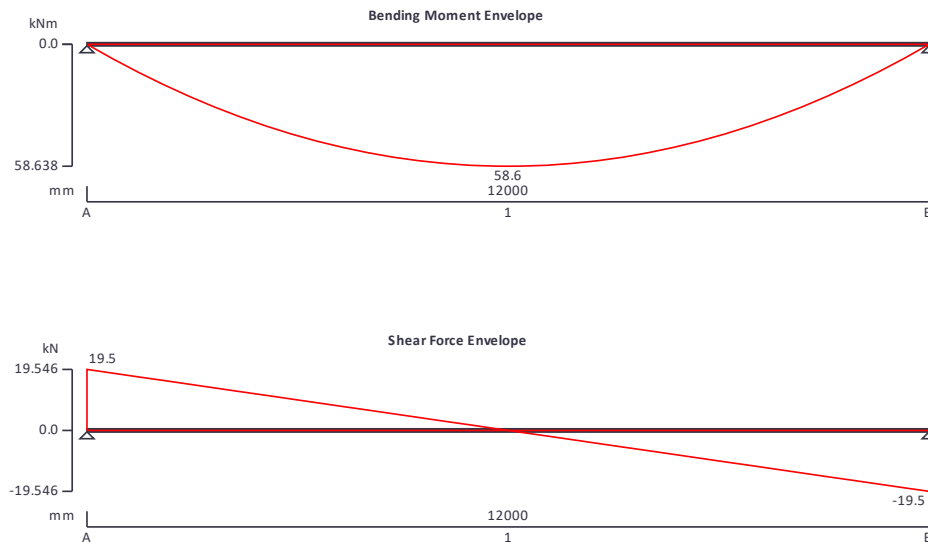
Date

Chk'd by

Date

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Date



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 2.05 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Permanent $\times 1.35$

Support B

Permanent $\times 1.35$

Analysis results

Maximum moment

$M_{\max} = 58.6$ kNm

$M_{\min} = 0$ kNm

Maximum moment span 1 segment 1

$M_{s1_seg1_max} = 58.6$ kNm

$M_{s1_seg1_min} = 0$ kNm

Maximum moment span 1 segment 2

$M_{s1_seg2_max} = 58.6$ kNm

$M_{s1_seg2_min} = 0$ kNm

Maximum shear

$V_{\max} = 19.5$ kN

$V_{\min} = -19.5$ kN

Maximum shear span 1 segment 1

$V_{s1_seg1_max} = 19.5$ kN

$V_{s1_seg1_min} = 0$ kN

Maximum shear span 1 segment 2

$V_{s1_seg2_max} = 0$ kN

$V_{s1_seg2_min} = -19.5$ kN

Deflection segment 3

$\delta_{\max} = 56$ mm

$\delta_{\min} = 0$ mm

Maximum reaction at support A

$R_{A_max} = 19.5$ kN

$R_{A_min} = 19.5$ kN

Unfactored permanent load reaction at support A

$R_{A_Permanent} = 14.5$ kN

Maximum reaction at support B

$R_{B_max} = 19.5$ kN

$R_{B_min} = 19.5$ kN

Unfactored permanent load reaction at support B

$R_{B_Permanent} = 14.5$ kN

Section details

Section type

UKB 254x146x37 (Tata Steel Advance)



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PROJECT:
PROPOSED MOBILE HOUSE

Job Ref.

STRUCTURAL CALCULATIONS:
LIFTING STEEL FRAME SUPPORT BEAM DESIGN CHECK

Sheet no./rev.

Calc. by

Date

Chk'd by

Date

App'd by

Date

Steel grade

S275

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element

$$t = \max(t_f, t_w) = 10.9 \text{ mm}$$

Nominal yield strength

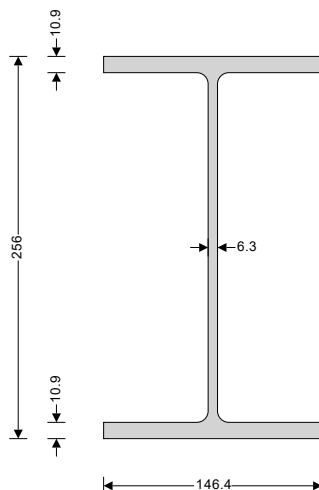
$$f_y = 275 \text{ N/mm}^2$$

Nominal ultimate tensile strength

$$f_u = 410 \text{ N/mm}^2$$

Modulus of elasticity

$$E = 210000 \text{ N/mm}^2$$



Partial factors - Section 6.1

Resistance of cross-sections

$$\gamma_{M0} = 1.00$$

Resistance of members to instability

$$\gamma_{M1} = 1.00$$

Resistance of tensile members to fracture

$$\gamma_{M2} = 1.10$$

Lateral restraint

Span 1 has lateral restraint at supports plus midspan

Effective length factors

Effective length factor in major axis

$$K_y = 1.000$$

Effective length factor in minor axis

$$K_z = 1.000$$

Effective length factor for torsion

$$K_{LT,A} = 1.000$$

$$K_{LT,B} = 1.000$$

Classification of cross sections - Section 5.5

$$\varepsilon = \sqrt{235 \text{ N/mm}^2 / f_y} = 0.92$$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section

$$c = d = 219 \text{ mm}$$

$$c / t_w = 37.6 \times \varepsilon \leq 72 \times \varepsilon$$

Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = 62.5 \text{ mm}$$

$$c / t_f = 6.2 \times \varepsilon \leq 9 \times \varepsilon$$

Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 234.2 \text{ mm}$$



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Shear area factor

$$\eta = 1.000$$

$$h_w / t_w < 72 \times \varepsilon / \eta$$

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 19.5 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1759 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 279.3 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Check bending moment at span 1 segment 1 major (y-y) axis - Section 6.2.5

Design bending moment

$$M_{Ed} = \max(\text{abs}(M_{s1_seg1_max}), \text{abs}(M_{s1_seg1_min})) = 58.6 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 132.9 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = 0.873$$

$$C_1 = 1 / k_c^2 = 1.311$$

Curvature factor

$$g = \sqrt{1 - (I_z / I_y)} = 0.947$$

Poissons ratio

$$\nu = 0.3$$

Shear modulus

$$G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$$

Unrestrained length

$$L = 1.0 \times L_{s1_seg1} = 6000 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 104.4 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 1.128$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = 0.4$$

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5

$$b$$

Imperfection factor - Table 6.3

$$\alpha_{LT} = 0.34$$

Correction factor for rolled sections

$$\beta = 0.75$$

LTB reduction determination factor

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 1.101$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.622$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.950$$

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.654$$

Design buckling resistance moment - eq 6.55

$$M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = 86.9 \text{ kNm}$$

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 180 = 66.7 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 56.035 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit