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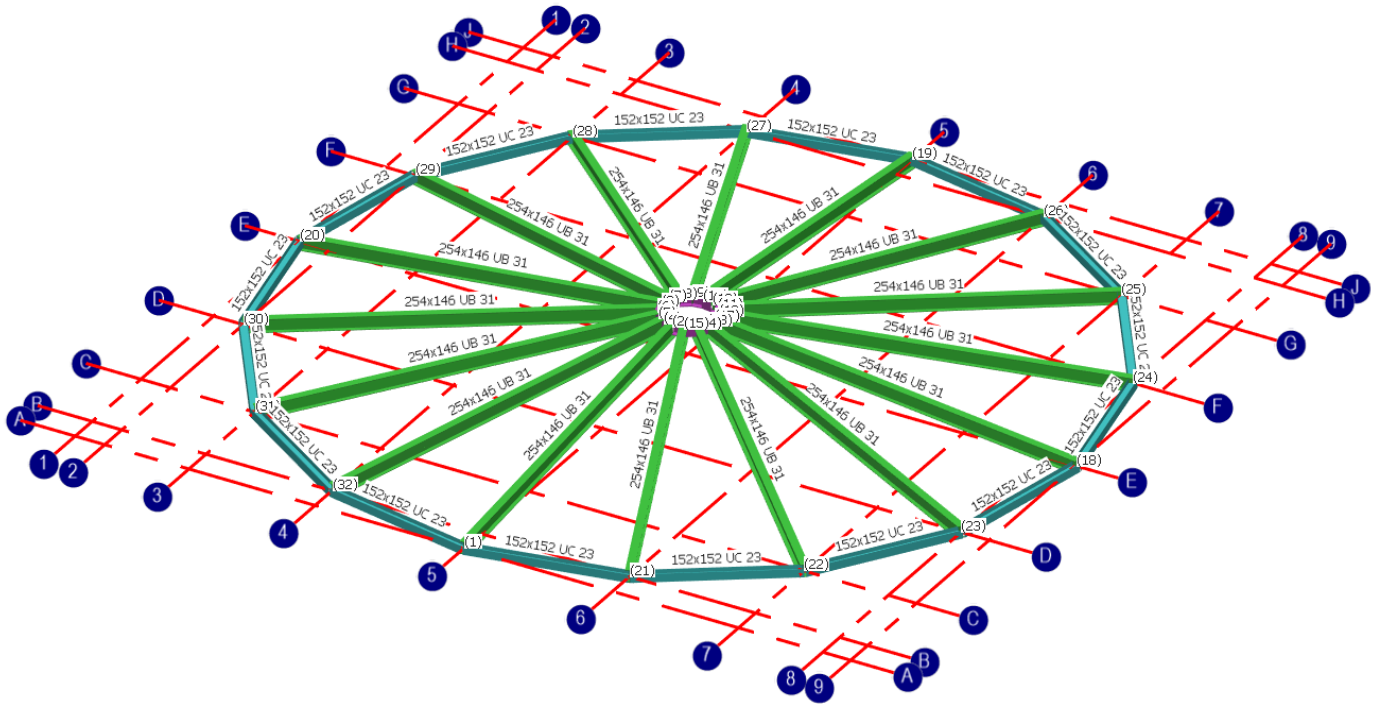
Brief

The existing roof is to be replaced due to timber decay.



Loadings

Roof Pitch 6°

Dead	Single Ply roof membrane	0.06	KN/m²
	20mm OSB Boarding	0.15	
	Insulation	0.05	
	20mm OSB Boarding	0.15	
	Timber rafters	0.10	
	Services	0.03	
	Ceiling + Skim	0.18	
		∑ 0.72	
	Plan Loading = 0.72/Cos 6°	0.72	KN/m²
Live	Snow	0.60	KN/m²

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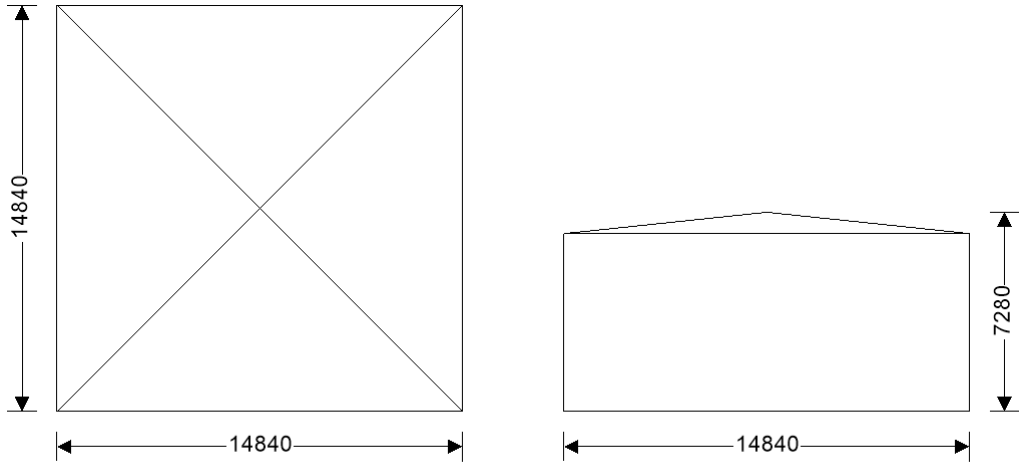
WIND LOADING SJ214421

WIND LOADING

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

Tedds calculation version 3.0.29



Plan Elevation

Building data

Type of roof	Hipped
Length of building	L = **14840** mm
Width of building	W = **14840** mm
Height to eaves	H = **6500** mm
Pitch of main slope	α_0 = **6.0** deg
Pitch of gable slope	α_{90} = **6.0** deg
Total height	h = **7280** mm

Basic values

Location	Llangollen
Wind speed velocity (FigureNA.1)	$v_{b,map}$ = **22.8** m/s
Distance to shore	L_{shore} = **44.50** km
Altitude above sea level	A_{alt} = **159.0**m
Altitude factor	$c_{alt} = A_{alt}/1m \times 0.001 + 1$ = **1.159**
Fundamental basic wind velocity	$v_{b,0} = v_{b,map} \times c_{alt}$ = **26.5** 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/m^3
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)	$v_b = c_{dir} \times c_{season} \times v_{b,0} \times c_{prob}$ = **26.5** m/s
Reference mean velocity pressure	$q_b = 0.5 \times \rho \times v_b^2$ = **0.430** kN/m^2

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Orography

Orography factor not significant	c_o = 1.0
Terrain category	Country
Displacement height (sheltering effect excluded)	h_{dis} = 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.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

Reference height (at which q is sought)	z = **6500**mm
Displacement height (sheltering effects excluded)	h_{dis} = **0** mm
Exposure factor (Figure NA.7)	c_e = **2.11**
Peak velocity pressure	$q_p = c_e \times q_b$ = **0.90** kN/m^2

Structural factor

Structural damping	δ_s = **0.100**
Height of element	h_{part} = **6500** mm
Size factor (Table NA.3)	c_s = **0.909**
Dynamic factor (Figure NA.9)	c_d = **1.015**
Structural factor	$c_{sCd} = c_s \times c_d$ = **0.922**

Peak velocity pressure - windward wall - Wind 90 deg

Reference height (at which q is sought)	z = **6500**mm
Displacement height (sheltering effects excluded)	h_{dis} = **0** mm
Exposure factor (Figure NA.7)	c_e = **2.11**
Peak velocity pressure	$q_p = c_e \times q_b$ = **0.90** kN/m^2

Structural factor

Structural damping	δ_s = **0.100**
Height of element	h_{part} = **6500** mm
Size factor (Table NA.3)	c_s = **0.909**
Dynamic factor (Figure NA.9)	c_d = **1.015**
Structural factor	$c_{sCd} = c_s \times c_d$ = **0.922**

Peak velocity pressure - roof

Reference height (at which q is sought)	z = **7280**mm
Displacement height (sheltering effects excluded)	h_{dis} = **0** mm
Exposure factor (Figure NA.7)	c_e = **2.18**
Peak velocity pressure	$q_p = c_e \times q_b$ = **0.93** kN/m^2

Structural factor - roof 0 deg

Structural damping	δ_s = **0.100**
Height of element	h_{part} = **7280** mm
Size factor (Table NA.3)	c_s = **0.910**
Dynamic factor (Figure NA.9)	c_d = **1.015**
Structural factor	$c_{sCd} = c_s \times c_d$ = **0.923**

Structural factor - roof 90 deg

Structural damping	δ_s = **0.100**

Height of element h_{part} = **7280** mm

Size factor (Table NA.3) c_s = **0.910**

Dynamic factor (Figure NA.9) c_d = **1.015**

Structural factor $c_{sCd} = c_s \times c_d$ = **0.923**

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) $q_{p,i}$ = **0.93** 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}

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)	-1.75	0.93	-1.70	8.53	-14.47
G (-ve)	-1.16	0.93	-1.19	11.07	-13.15
H (-ve)	-0.59	0.93	-0.70	35.77	-24.89
I (-ve)	-0.60	0.93	-0.70	35.23	-24.83
J (-ve)	-0.86	0.93	-0.93	19.59	-18.20
K (-ve)	-0.67	0.93	-0.76	0.53	-0.41
L (-ve)	-1.08	0.93	-1.12	20.66	-23.11
M (-ve)	-0.60	0.93	-0.70	90.06	-63.46

Total vertical net force $F_{w,v}$ = **-181.51** kN

Total horizontal net force $F_{w,h}$ = **-0.95** 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	0.90	-1.19	18.93	-22.48
B	-0.80	0.90	-0.85	75.71	-64.67
C	-0.50	0.90	-0.60	1.82	-1.10
D	0.73	0.90	0.42	96.46	40.86
E	-0.36	0.90	-0.49	96.46	-47.33

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wE}$ = **-47.3** kN

Net windward force for overall section $F_w = F_{w,wD}$ = **40.9** kN

Lack of correlation (cl.7.2.2(3) – Note) f_{corr} = **0.85** as h/W is 0.491

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **74.2** kN

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

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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)	0.02	0.93	0.30	8.53	2.54
G (+ve)	0.02	0.93	0.30	11.07	3.30
H (+ve)	0.02	0.93	0.30	35.77	10.65
I (+ve)	-0.60	0.93	-0.24	35.23	-8.36
J (+ve)	-0.86	0.93	-0.46	19.59	-9.04
K (+ve)	-0.67	0.93	-0.30	0.53	-0.16
L (+ve)	0.00	0.93	0.28	20.66	5.79
M (+ve)	0.00	0.93	0.28	90.06	25.26

Total vertical net force $F_{w,v}$ = **29.81** kN

Total horizontal net force $F_{w,h}$ = **3.56** kN

Walls 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)
A	-1.20	0.90	-0.72	18.93	-13.64
B	-0.80	0.90	-0.39	75.71	-29.29
C	-0.50	0.90	-0.14	1.82	-0.25
D	0.73	0.90	0.89	96.46	85.95
E	-0.36	0.90	-0.02	96.46	-2.24

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wE}$ = **-2.2** kN

Net windward force for overall section $F_w = F_{w,wD}$ = **86.0** kN

Lack of correlation (cl.7.2.2(3) – Note) f_{corr} = **0.85** as h/W is 0.491

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **78.0** 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)	-1.75	0.93	-1.70	8.53	-14.47
G (-ve)	-1.16	0.93	-1.19	11.07	-13.15
H (-ve)	-0.59	0.93	-0.70	35.77	-24.89
I (-ve)	-0.60	0.93	-0.70	35.77	-25.20
J (-ve)	-0.86	0.93	-0.93	19.59	-18.20
L (-ve)	-1.08	0.93	-1.12	21.73	-24.30
M (-ve)	-0.60	0.93	-0.70	34.10	-24.03
N (-ve)	-0.58	0.93	-0.69	54.89	-37.73

Total vertical net force $F_{w,v}$ = **-180.97** kN

Total horizontal net force $F_{w,h}$ = **-0.95** kN

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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	0.90	-1.19	18.93	-22.48
B	-0.80	0.90	-0.85	75.71	-64.67
C	-0.50	0.90	-0.60	1.82	-1.10
D	0.73	0.90	0.42	96.46	40.86
E	-0.36	0.90	-0.49	96.46	-47.33

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wE}$ = **-47.3** kN

Net windward force for overall section $F_w = F_{w,wD}$ = **40.9** kN

Lack of correlation (cl.7.2.2(3) – Note) f_{corr} = **0.85** as h/L is 0.491

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **74.2** kN

Roof 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)
F (+ve)	0.02	0.93	0.30	8.53	2.54
G (+ve)	0.02	0.93	0.30	11.07	3.30
H (+ve)	0.02	0.93	0.30	35.77	10.65
I (+ve)	-0.60	0.93	-0.24	35.77	-8.48
J (+ve)	-0.86	0.93	-0.46	19.59	-9.04
L (+ve)	0.00	0.93	0.28	21.73	6.09
M (+ve)	0.00	0.93	0.28	34.10	9.56
N (+ve)	0.00	0.93	0.28	54.89	15.39

Total vertical net force $F_{w,v}$ = **29.84** kN

Total horizontal net force $F_{w,h}$ = **3.56** 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	0.90	-0.72	18.93	-13.64
B	-0.80	0.90	-0.39	75.71	-29.29
C	-0.50	0.90	-0.14	1.82	-0.25
D	0.73	0.90	0.89	96.46	85.95
E	-0.36	0.90	-0.02	96.46	-2.24

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wE}$ = **-2.2** kN

Net windward force for overall section $F_w = F_{w,wD}$ = **86.0** kN

Lack of correlation (cl.7.2.2(3) – Note) f_{corr} = **0.85** as h/L is 0.491

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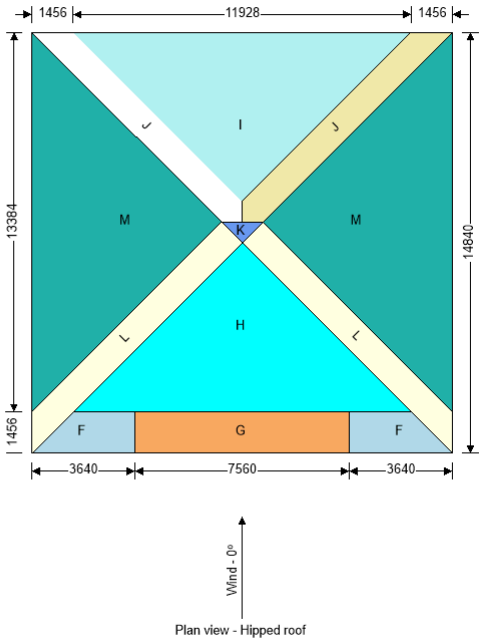
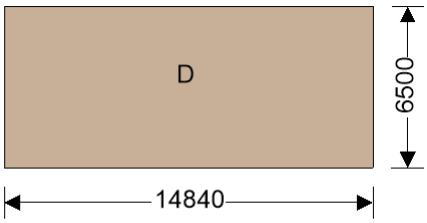
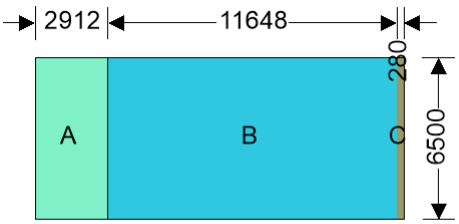
Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h}) =$ **78.0** kN



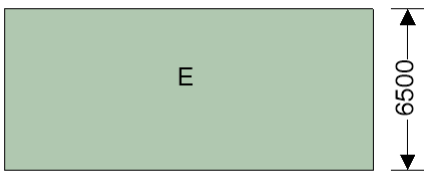
Plan view - Hipped roof



Windward face



Side face



Leeward face

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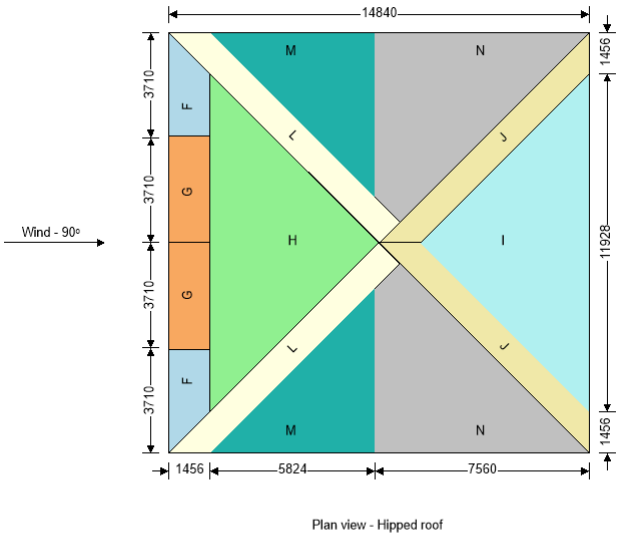
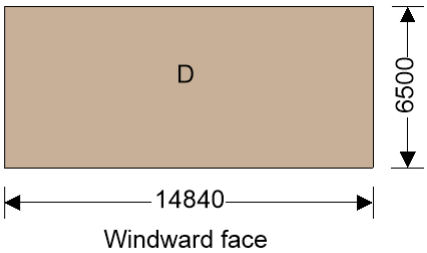
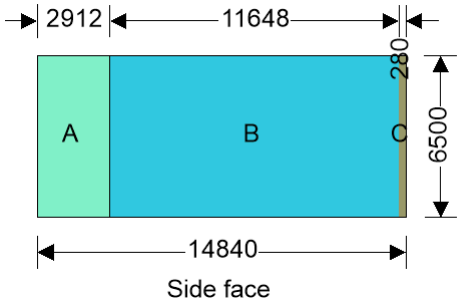


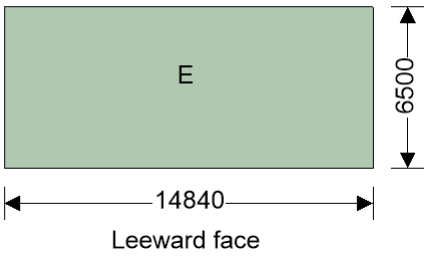
Plan view - Hipped roof



Windward face



Side face



Leeward face

Maximum Wind uplift Wind 0 ° Cpi – 0.2

Wind = -181.51KN

Wind/m² = $\frac{181.51}{14.84 \times 14.84}$ = 0.82KN/m²

Downward Loading

Roof Dead Load = 0.72KN/m^2

Selfweight of Steelwork

4 x 14.5 x 0.45 = 26.1KN

4 x 14.5 x 0.33 = 19.1KN

$\sum$ 45.2KN

Load/m^2 = $\frac{45.2}{14.84 \times 14.84}$ = 0.21KN/m^2

Total Roof + SW = 0.93KN/m^2 > 0.82KN/m^2 No resultant uplift

TIMBER RAFTERS

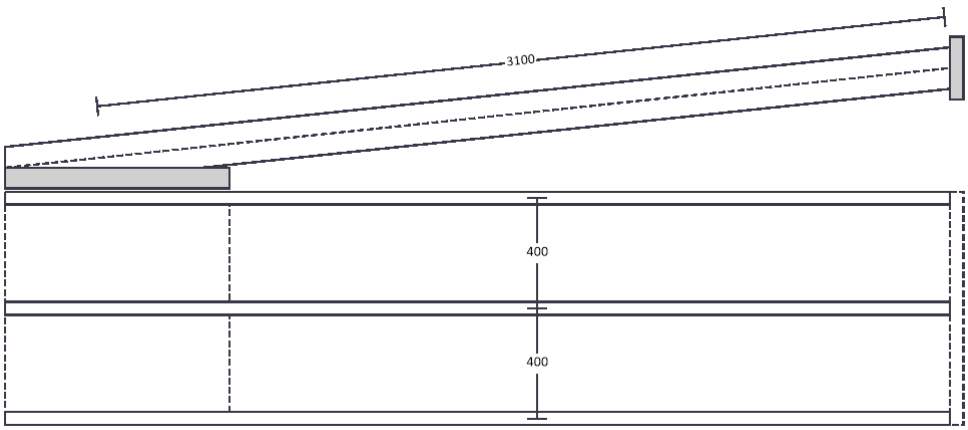
TIMBER RAFTER ANALYSIS & DESIGN (EN1995-1-1:2004)

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

Tedds calculation version 1.0.07

Rafter details

Rafter spacing s_{Rafter} = **400** mm Rafter inclination θ_{Rafter} = **6** deg



Forces input on Rafter

Permanent load on slope F_{G_Rafter}= **0.72** kN/m^2

Snow load on plan F_{S_Rafter}= **0.60** kN/m^2

Rafter loading details

Distributed loads

Permanent load on slope p_G= **0.29** kN/m

Snow load on slope p_S= **0.24** kN/m

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Member - Span 1

Partial factor for material properties and resistances

Partial factor γ_M = **1.300**

Member details

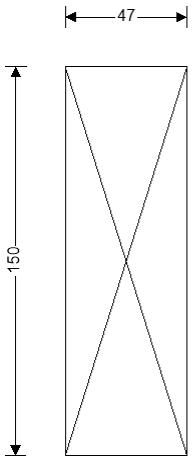
Load duration Medium-term Service class 2

Timber section details

Number of timber sections N = **1**

Breadth of sections b = **47** mm Depth of sections h = **150** mm

Timber strength class **C16**



47x150 timber section
Cross-sectional area, A, 7050 mm²
Section modulus, W_y, 176250 mm³
Section modulus, W_z, 55225 mm³
Second moment of area, I_y, 13218750 mm⁴
Second moment of area, I_z, 1297787 mm⁴
Radius of gyration, i_y, 43.3 mm
Radius of gyration, i_z, 13.6 mm
Timber strength class C16
Characteristic bending strength, $f_{m,k}$, 16 N/mm²
Characteristic shear strength, $f_{v,k}$, 3.2 N/mm²
Characteristic compression strength parallel to grain, $f_{c,0,k}$, 17 N/mm²
Characteristic compression strength perpendicular to grain, $f_{c,90,k}$, 2.2 N/mm²
Characteristic tension strength parallel to grain, $f_{t,0,k}$, 8.5 N/mm²
Mean modulus of elasticity, $E_{0,mean}$, 8000 N/mm²
Fifth percentile modulus of elasticity, $E_{0.05}$, 5400 N/mm²
Shear modulus of elasticity, G_{mean}, 500 N/mm²
Characteristic density, ρ_k, 310 kg/m³
Mean density, ρ_{mean}, 370 kg/m³

Span details

Bearing length L_b = **100** mm

Consider Combination 1 - 1.35G + 1.50Q (Strength)

Check compression parallel to the grain - cl.6.1.4

Design compressive stress $\sigma_{c,0,d}$ = **0.019** N/mm² Design compressive strength $f_{c,0,d}$ = **11.508** N/mm²

Utilisation = **0.002**

PASS - Design parallel compression strength exceeds design parallel compression stress

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Des.perp.comp.stress $\sigma_{c,y,90,d}$ = **0.107** N/mm² Des.perp.comp.strength $f_{c,y,90,d}$ = **1.489** N/mm²

Utilisation = **0.072**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear stress $\tau_{y,d}$ = **0.204** N/mm² Design shear strength $f_{v,y,d}$ = **2.166** N/mm²

Utilisation = **0.094**

PASS - Design shear strength exceeds design shear stress

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

Effective length for y-axis $L_{e,y}$ = **2790** mm — Relative slenderness ratio $\lambda_{rel,y}$ = **1.151**

Effective length for z-axis $L_{e,z}$ = **0** mm — Relative slenderness ratio $\lambda_{rel,z}$ = **0**

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

Utilisation = **0.003**

PASS - Column stability is acceptable

Check design 1550 mm along span

Check bending moment - Section 6.1.6

Design bending stress $\sigma_{m,y,d}$ = **2.832** N/mm^2 — Design bending strength $f_{m,y,d}$ = **10.831** N/mm^2

Utilisation = **0.261**

PASS - Design bending strength exceeds design bending stress

Check combined bending and axial compression - Section 6.2.4

Utilisation = **0.261**

PASS - Combined bending and axial compression utilisation is acceptable

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

Effective length for y-axis $L_{e,y}$ = **2790** mm — Relative slenderness ratio $\lambda_{rel,y}$ = **1.151**

Effective length for z-axis $L_{e,z}$ = **0** mm — Relative slenderness ratio $\lambda_{rel,z}$ = **0**

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

Utilisation = **0.263**

PASS - Column stability is acceptable

Check beams subjected to either bending or combined bending and compression - cl.6.3.3

Utilisation = **0.069**

PASS - Beam stability is acceptable

Consider Combination 2 - 1.00G + 1.00Q (Service)

Check design 1550 mm along span

Check y-y axis deflection - Section 7.2

Final deflection with creep $\delta_{y,Final}$ = **6.5** mm — Allowable deflection $\delta_{y,Allowable}$ = **12.4** mm

Utilisation = **0.526**

PASS - Allowable deflection exceeds final deflection

TIMBER PURLINS

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 1.7.05

	Project			Job Ref.
	Llangollen Museum Parade Street LL20 8PW			24014
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	Proposed Replacement Roof			12
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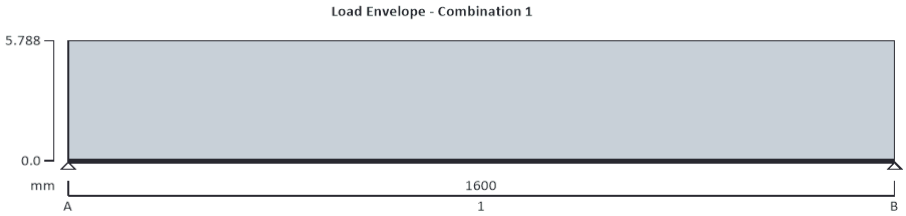
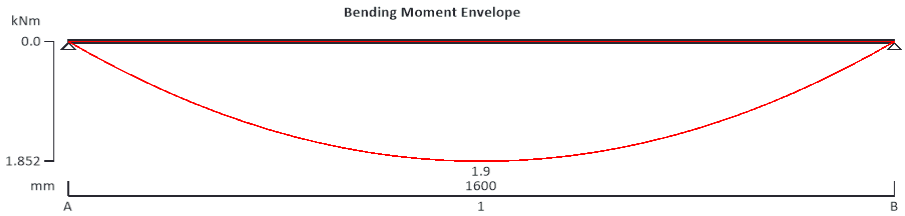




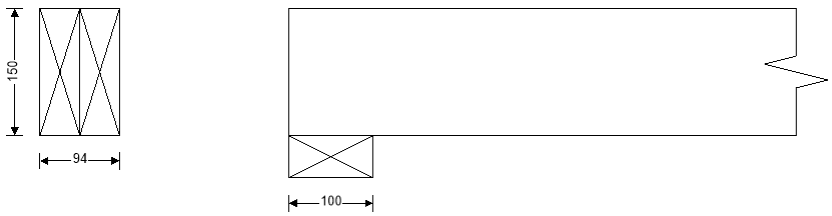
Applied loading

Beam loads

Dead	Permanent full UDL 2.200 kN/m
Live	Variable full UDL 1.840 kN/m
	Permanent self weight of beam × 1

Analysis results

Design moment	M = **1.852** kNm	Design shear	F = **4.630** kN
Total load on member	W_{tot} = **9.261** kN		
Reactions at support A	R_{A_max} = **4.630** kN	R_{A_min} = **4.630** kN	
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **1.794** kN		
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **1.472** kN		
Reactions at support B	R_{B_max} = **4.630** kN	R_{B_min} = **4.630** kN	
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **1.794** kN		
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **1.472** kN		



Timber section details

Breadth of section	b = **47** mm	Depth of section	h = **150** mm
Number of sections	N = **2**	Breadth of member	b_b = **94** mm
Timber strength class	**C16**		

Member details

Service class of timber	**1**	Load duration	**Long-term**
Length of span	L_{s1} = **1600** mm		

Length of bearing L_b = **100** mm

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c.90.d}$ = **0.493** N/mm^2 Design compressive strength $f_{c.90.d}$ = **1.185** N/mm^2

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d}$ = **5.254** N/mm^2 Design bending strength $f_{m.d}$ = **8.615** N/mm^2

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress τ_d = **0.735** N/mm^2 Permissible shear stress $f_{v.d}$ = **1.723** N/mm^2

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit δ_{lim} = **6.400** mm Total final deflection δ_{fin} = **2.638** mm

PASS - Total final deflection is less than the deflection limit

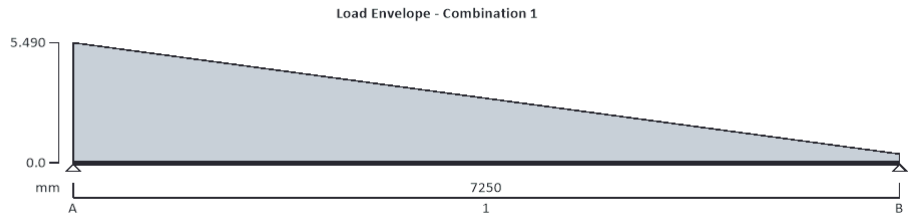
Area of roof loading the Main beams		9.80 m^2	
Roof Beam			
Loading		Dead	Live
Roof	(0.72 + 0.6) x (9.8 x 2)/2	7.1KN	5.9KN

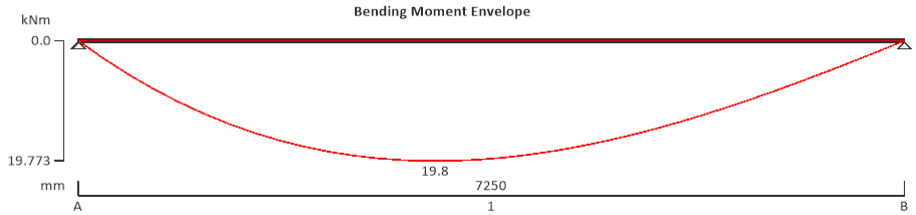
STEEL ROOF BEAM

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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Support conditions

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

Applied loading

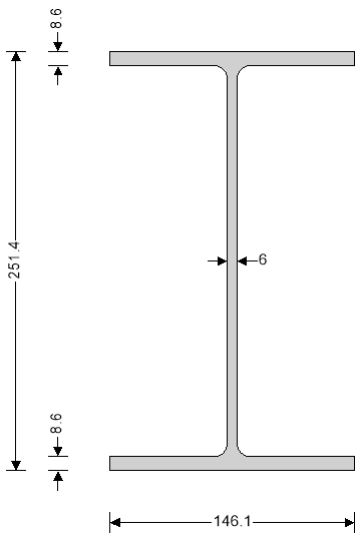
Beam loads	Roof - Permanent full VDL 1.95 kN/m to 0 kN/m
	Roof - Variable full VDL 1.63 kN/m to 0 kN/m
	Permanent self weight of beam × 1

Analysis results

Maximum moment	M_{max} = **19.8** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **13.8** kN	V_{min} = **-7.6** kN
Deflection	δ_{max} = **8.1** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **13.8** kN	R_{A_min} = **13.8** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **5.8** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **3.9** kN	
Maximum reaction at support B	R_{B_max} = **7.6** kN	R_{B_min} = **7.6** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **3.5** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **2** kN	

Section details

Section type **UKB 254x146x31 (Tata Steel Advance)** Steel grade **S275**



Section classification **Class 1**

Check shear - Section 6.2.6

Design shear force V_{Ed} = **14** kN Design shear resistance $V_{c,Rd}$ = **259.9** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = **19.8** kNm Des.bending resist.moment $M_{c,Rd}$ = **108.1** kNm

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	Hodgson	16/03/24	01/05/24	

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT}$ = **1.722** Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = **0.400**

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = **36.5** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check compression - Section 6.2.4

Design compression force N_{Ed} = **73** kN Design resistance of section $N_{c,Rd}$ = **1091.1** kN

Design resistance for buckling - Section 6.3.1.1

Design buckling resistance $N_{b,y,Rd}$ = **873.3** kN

PASS - Design buckling resistance exceeds design compression force

Design resistance for buckling - Section 6.3.1.1

Design buckling resistance $N_{b,z,Rd}$ = **153.9** kN

PASS - Design buckling resistance exceeds design compression force

Check torsional and torsional-flexural buckling

Torsional buckling force $N_{cr,T}$ = **776.1** kN Torsional-flexural buckling $N_{cr,TF}$ = **776.1** kN

Design resistance for buckling - Section 6.3.1.1

Design buckling resistance $N_{b,T,Rd}$ = **530.2** kN

PASS - Design buckling resistance exceeds design compression force

Combined bending and axial force - Section 6.2.9

Bending and axial force check $N_{Ed} <= \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0})$

No allowance on the plastic moment need to be accounted for due to the effect of axial force

Interaction factors k_{ij} for members not susceptible to torsional deformations - Table B.1

Interaction formulae $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{Ed} / (\chi_{LT} \times M_{Rk} / \gamma_{M1})$ = **0.624**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{Ed} / (\chi_{LT} \times M_{Rk} / \gamma_{M1})$ = **0.799**

PASS - Combined bending and compression checks are satisfied

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection δ_{lim} = **14.5** mm Maximum deflection δ = **8.144** mm

PASS - Maximum deflection does not exceed deflection limit

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Eaves Beam

Loading		Dead	Live
Roof	(0.72 + 0.6) x (3.06/2 + 0.55)	1.1	1.3

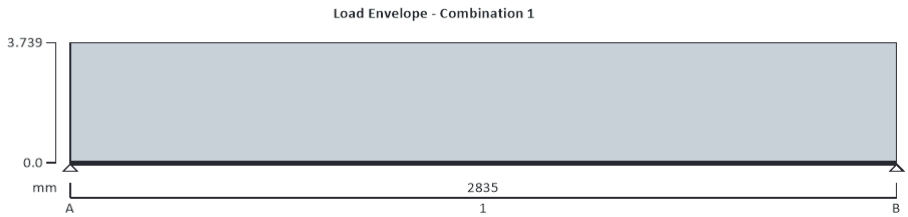
Axial Load $\frac{38.7}{\text{Sin } 11.25}$ = 198.4KN

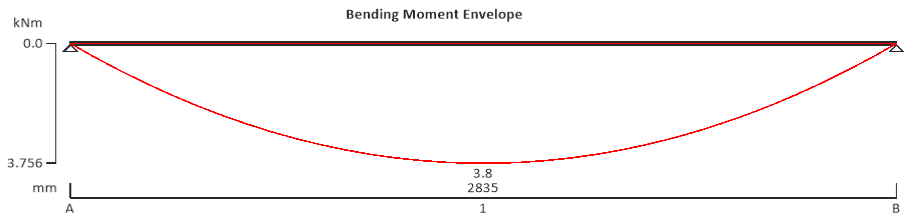
STEEL EAVES BEAM

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





Support conditions

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

Applied loading

Beam loads	Roof - Permanent full UDL 1.1 kN/m
	Roof - Variable full UDL 1.3 kN/m
	Permanent self weight of beam × 1

Analysis results

Maximum moment	M_{max} = **3.8** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **5.3** kN	V_{min} = **-5.3** kN

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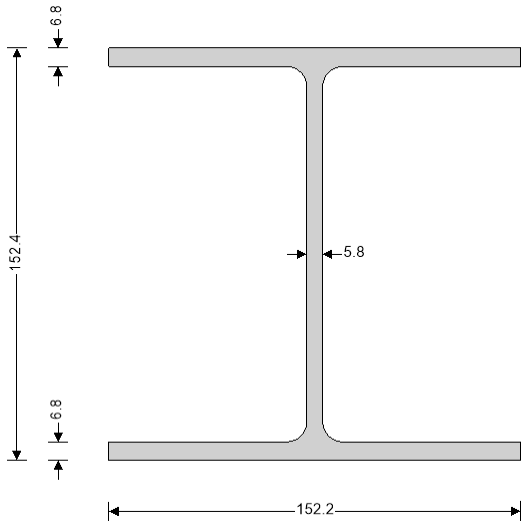
	Hodgson	16/03/24	01/05/24	

Deflection	δ_{max} = **0.8** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **5.3** kN	R_{A_min} = **5.3** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **1.9** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **1.8** kN	
Maximum reaction at support B	R_{B_max} = **5.3** kN	R_{B_min} = **5.3** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **1.9** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **1.8** kN	

Section details

Section type **UC 152x152x23 (British Steel Section Range 2022 (BS4-1))** Steel grade **S275**



Section classification **Class 3**

Check shear - Section 6.2.6

Design shear force V_{Ed} = **5** kN Design shear resistance $V_{c,Rd}$ = **?** kN

2D ANALYSIS 3 PINNED ARCH

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Materials

Name	Density (kg/m³)	Youngs Modulus kN/mm²	Shear Modulus kN/mm²	Thermal Coefficient °C⁻¹
Steel (BS5950)	7850	205	78.8	0.000012

Sections

Name	Area (cm²)	Moment of inertia		Shear area parallel to	
		Major (cm⁴)	Minor (cm⁴)	Minor (cm²)	Major (cm²)
UKB 254x146x31	39.7	4413.4	447.5	15.1	22.6

Nodes

Node	Co-ordinates		Freedom			Coordinate system		Spring		
	X (m)	Z (m)	X	Z	Rot.	Name	Angle (°)	X (kN/m)	Z (kN/m)	Rot. kNm/°
1	0	0	Fixed	Fixed	Free		0	0	0	0
2	3.573	0.446	Free	Free	Free		0	0	0	0
3	7.146	0.892	Free	Free	Free		0	0	0	0
4	10.719	0.446	Free	Free	Free		0	0	0	0
5	14.292	0	Fixed	Fixed	Free		0	0	0	0

Elements

Element	Length (m)	Nodes		Section	Material	Releases			Rotated
		Start	End			Start moment	End moment	Axial	
1	3.601	1	2	UKB 254x146x31	Steel (BS5950)	Fixed	Fixed	Fixed	
2	3.601	2	3	UKB 254x146x31	Steel (BS5950)	Fixed	Free	Fixed	
3	3.601	3	4	UKB 254x146x31	Steel (BS5950)	Fixed	Fixed	Fixed	
4	3.601	4	5	UKB 254x146x31	Steel (BS5950)	Fixed	Fixed	Fixed	

Loading

Self weight included

Load combination factors

Load combination	Self Weight	Permanent	Imposed
Dead + Live (Strength)	*1.40*	*1.40*	*1.60*
Dead + Live (Service)	*1.00*	*1.00*	*1.00*

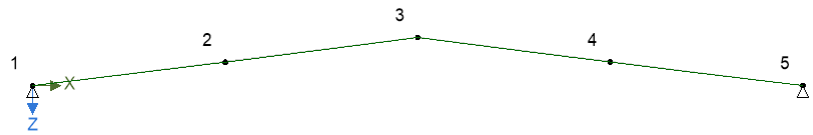
Element Loads

Element	Load case	Load Type	Orientation	Description
1	Permanent	UDL	GlobalZ	1 kN/m
4	Permanent	UDL	GlobalZ	1 kN/m
1	Permanent	Trapezoidal	GlobalZ	1 kN/m at 0 m to 0 m
2	Permanent	Trapezoidal	GlobalZ	1 kN/m at 0 m to 0 m
3	Permanent	Trapezoidal	GlobalZ	1 kN/m at 3.601 m to 3.601 m
4	Permanent	Trapezoidal	GlobalZ	1 kN/m at 3.601 m to 3.601 m
1	Imposed	UDL	GlobalZ	0.5 kN/m
4	Imposed	UDL	GlobalZ	0.5 kN/m
1	Imposed	Trapezoidal	GlobalZ	0.5 kN/m at 0 m to 0 m
2	Imposed	Trapezoidal	GlobalZ	0.5 kN/m at 0 m to 0 m
3	Imposed	Trapezoidal	GlobalZ	0.5 kN/m at 3.601 m to 3.601 m
4	Imposed	Trapezoidal	GlobalZ	0.5 kN/m at 3.601 m to 3.601 m

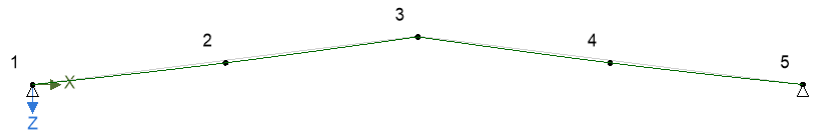
Results

Total deflection

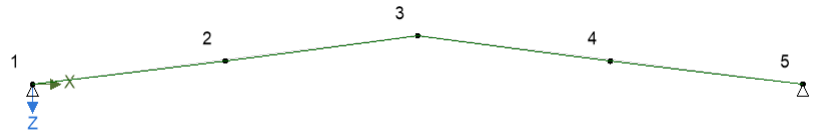
Self Weight - Total deflection



Permanent - Total deflection



Imposed - Total deflection



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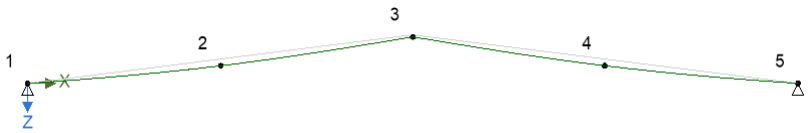
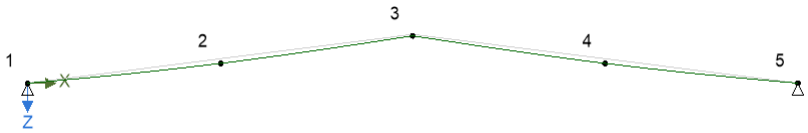
Dead + Live (Strength) - Total deflection



Dead + Live (Service) - Total deflection



Node deflections

Load case: Self Weight

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.03488	
2	0.1	1.5	0.00501	
3	0	0.6	0.02486	
4	-0.1	1.5	-0.00501	
5	0	0	-0.03488	

Load case: Permanent

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.11524	
2	0.5	4.6	0.00523	
3	0	1.4	0.08033	
4	-0.5	4.6	-0.00523	
5	0	0	-0.11524	

Load case: Imposed

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.05762	
2	0.2	2.3	0.00262	
3	0	0.7	0.04017	
4	-0.2	2.3	-0.00262	
5	0	0	-0.05762	

Load combination: Dead + Live (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.30237	
2	1.3	12.1	0.01853	
3	0	3.9	0.21153	
4	-1.3	12.1	-0.01853	
5	0	0	-0.30237	

Load combination: Dead + Live (Service)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.20775	
2	0.9	8.3	0.01286	
3	0	2.7	0.14536	
4	-0.9	8.3	-0.01286	
5	0	0	-0.20775	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
Self Weight	0	4.4
Permanent	0	14.4
Imposed	0	7.2
Dead + Live (Strength)	0	37.8
Dead + Live (Service)	0	26

Reactions

Load case: Self Weight

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	8.8	2.2	0
5	-8.8	2.2	0

Load case: Permanent

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	19.2	7.2	0
5	-19.2	7.2	0

Load case: Imposed

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	9.6	3.6	0
5	-9.6	3.6	0

Load combination: Dead + Live (Strength)

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	54.6	18.9	0
5	-54.6	18.9	0

Load combination: Dead + Live (Service)

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	37.7	13	0
5	-37.7	13	0

Element end forces

Load case: Self Weight

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.601	1	-9	-1.1	0
		2	8.9	0	2
2	3.601	2	-8.9	0	-2
		3	8.7	-1.1	0
3	3.601	3	-8.7	-1.1	0
		4	8.9	0	2
4	3.601	4	-8.9	0	-2
		5	9	-1.1	0

Load case: Permanent

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.601	1	-20	-4.8	0
		2	19.3	-0.6	6.4
2	3.601	2	-19.3	0.6	-6.4
		3	19.1	-2.4	0
3	3.601	3	-19.1	-2.4	0
		4	19.3	0.6	6.4
4	3.601	4	-19.3	-0.6	-6.4
		5	20	-4.8	0

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Load case: Imposed

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.601	1	-10	-2.4	0
		2	9.7	-0.3	3.2
2	3.601	2	-9.7	0.3	-3.2
		3	9.5	-1.2	0
3	3.601	3	-9.5	-1.2	0
		4	9.7	0.3	3.2
4	3.601	4	-9.7	-0.3	-3.2
		5	10	-2.4	0

Load combination: Dead + Live (Strength)

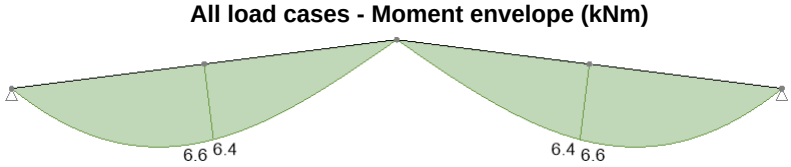
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.601	1	-56.6	-12	0
		2	54.9	-1.3	16.9
2	3.601	2	-54.9	1.3	-16.9
		3	54.2	-6.8	0
3	3.601	3	-54.2	-6.8	0
		4	54.9	1.3	16.9
4	3.601	4	-54.9	-1.3	-16.9
		5	56.6	-12	0

Load combination: Dead + Live (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.601	1	-39	-8.2	0
		2	37.8	-0.9	11.6
2	3.601	2	-37.8	0.9	-11.6
		3	37.4	-4.7	0
3	3.601	3	-37.4	-4.7	0
		4	37.8	0.9	11.6
4	3.601	4	-37.8	-0.9	-11.6
		5	39	-8.2	0

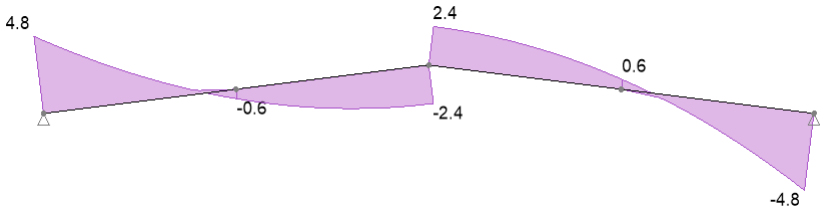
Forces

All load cases - Moment envelope (kNm)



	Project			Job Ref.
	Llangollen Museum Parade Street LL20 8PW			24014
	Section			Sheet no./rev.A
	Proposed Replacement Roof			24
	Calc. by	Date	Updated A	
	Hodgson	16/03/24	01/05/24	

All load cases - Shear envelope (kN)



Element results

Envelope - All load cases

Element	Shear force		Moment			
	Pos (m)	Max abs (kN)	Pos (m)	Max (kNm)	Pos (m)	Min (kNm)
1	0	4.8 (max abs)	3.044	6.6 (max)	0	0 (min)
2	3.601	-2.4	0	6.4	3.601	0 (min)
3	0	2.4	3.601	6.4	0	0 (min)
4	3.601	-4.8	0.557	6.6 (max)	3.601	0 (min)

Element results

Load case: Self Weight

Element	Shear force		Moment			
	Pos (m)	Max abs (kN)	Pos (m)	Max (kNm)	Pos (m)	Min (kNm)
1	0	1.1 (max abs)	3.601	2 (max)	0	0 (min)
2	3.601	-1.1	0	2 (max)	3.601	0 (min)
3	0	1.1 (max abs)	3.601	2 (max)	0	0 (min)
4	3.601	-1.1	0	2 (max)	3.601	0 (min)

Element results

Load case: Permanent

Element	Shear force		Moment			
	Pos (m)	Max abs (kN)	Pos (m)	Max (kNm)	Pos (m)	Min (kNm)
1	0	4.8 (max abs)	3.044	6.6 (max)	0	0 (min)
2	3.601	-2.4	0	6.4	3.601	0 (min)
3	0	2.4	3.601	6.4	0	0 (min)
4	3.601	-4.8	0.557	6.6 (max)	3.601	0 (min)

Element results

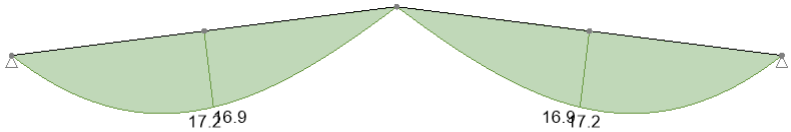
Load case: Imposed

Element	Shear force		Moment			
	Pos (m)	Max abs (kN)	Pos (m)	Max (kNm)	Pos (m)	Min (kNm)
1	0	2.4 (max abs)	3.044	3.3 (max)	0	0 (min)
2	3.601	-1.2	0	3.2	3.601	0 (min)

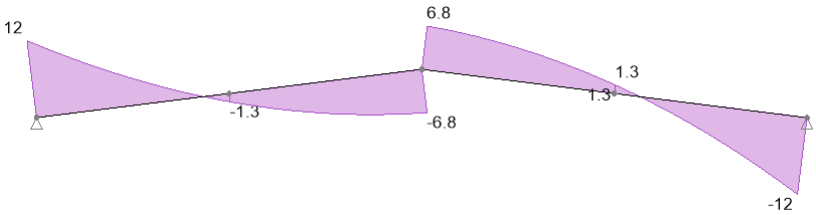
	Project			Job Ref.
	Llangollen Museum Parade Street LL20 8PW			24014
	Section			Sheet no./rev.A
	Proposed Replacement Roof			25
	Calc. by	Date	Updated A	
	Hodgson	16/03/24	01/05/24	

Element	Shear force		Moment			
	Pos (m)	Max abs (kN)	Pos (m)	Max (kNm)	Pos (m)	Min (kNm)
3	0	1.2	3.601	3.2	0	0 (min)
4	3.601	-2.4	0.557	3.3 (max)	3.601	0 (min)

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Element results

Envelope - Strength combinations

Element	Shear force		Moment			
	Pos (m)	Max abs (kN)	Pos (m)	Max (kNm)	Pos (m)	Min (kNm)
1	0	12 (max abs)	3.125	17.2 (max)	0	0 (min)
2	3.601	-6.8	0	16.9	3.601	0 (min)
3	0	6.8	3.601	16.9	0	0 (min)
4	3.601	-12	0.476	17.2 (max)	3.601	0 (min)

Element results

Load combination: Dead + Live (Strength)

Element	Shear force		Moment			
	Pos (m)	Max abs (kN)	Pos (m)	Max (kNm)	Pos (m)	Min (kNm)
1	0	12 (max abs)	3.125	17.2 (max)	0	0 (min)
2	3.601	-6.8	0	16.9	3.601	0 (min)
3	0	6.8	3.601	16.9	0	0 (min)
4	3.601	-12	0.476	17.2 (max)	3.601	0 (min)

	Project			Job Ref.
	Llangollen Museum Parade Street LL20 8PW			24014
	Section			Sheet no./rev.A
	Proposed Replacement Roof			26
	Calc. by	Date	Updated A	
	Hodgson	16/03/24	01/05/24	

MASTERFRAME DATA FILE

LOADING CASES AND LOAD COMBINATION

Load Group Labels

Load Group UT	Unity Load Factor (All Cases)
Load Group D1	Dead Load
Load Group L1	Live Load

Load Case 001 : Dead plus Live (Ultimate)

Load Combination + 1.00 UT + 1.25 D1 + 1.50 L1

Notional Loads Apply horizontal notional loads at 0.0 degrees from X axis equal to 0.5% of the factored vertical loads in case 1

Level	@ (m)	F (kN)	Level	@ (m)	F (kN)	Level	@ (m)	F (kN)
0	0.000	1.253	1	0.740	0.410			

Node	F (kN)	Node	F (kN)	Node	F (kN)	Node	F (kN)
1	0.078	2	0.024	3	0.024	4	0.026
5	0.027	6	0.026	7	0.026	8	0.027
9	0.026	10	0.026	11	0.027	12	0.026
13	0.026	14	0.027	15	0.026	16	0.024
17	0.024	18	0.078	19	0.078	20	0.078
21	0.078	22	0.079	23	0.078	24	0.078
25	0.079	26	0.078	27	0.078	28	0.079
29	0.078	30	0.078	31	0.079	32	0.078

Load Case 002 : Live Only (Serviceability)

Load Combination + 1.00 UT + 1.00 L1

Load Case 003 : Dead Plus Live (Serviceability)

Load Combination + 1.00 UT + 1.00 D1 + 1.00 L1

THE NODAL CO-ORDINATES

Node	X (m)	Y (m)	Z (m)	Node	X (m)	Y (m)	Z (m)
1	7.267	0.000	0.000	2	7.267	0.740	6.767
3	6.767	0.740	7.267	4	7.076	0.740	6.806
5	6.913	0.740	6.913	6	6.805	0.740	7.076
7	6.805	0.740	7.458	8	6.913	0.740	7.620
9	7.076	0.740	7.729	10	7.458	0.740	7.729
11	7.621	0.740	7.620	12	7.729	0.740	7.458
13	7.729	0.740	7.076	14	7.621	0.740	6.913
15	7.458	0.740	6.805	16	7.267	0.740	7.757
17	7.767	0.740	7.267	18	14.534	0.000	7.267
19	7.267	0.000	14.534	20	0.000	0.000	7.267
21	10.048	0.000	0.553	22	12.406	0.000	2.128
23	13.981	0.000	4.486	24	13.981	0.000	10.048
25	12.406	0.000	12.406	26	10.048	0.000	13.981
27	4.486	0.000	13.981	28	2.128	0.000	12.406
29	0.553	0.000	10.048	30	0.553	0.000	4.486
31	2.128	0.000	2.128	32	4.486	0.000	0.553

MEMBER PROPERTIES

Members 1-16

M	254x146 UB 31 [S 275]		
A 39.67E-4	Iy 4414E-8	Iz 448.4E-8	It 8.55E-8
E 210.0E6	G 80.77E6		

Members 17-32

```
   M  ...  ...          152x152 UC 23 [S 275]
    A 29.24E-4           Iy 1251E-8          Iz 400.8E-8         It 4.63E-8
    E 210.0E6            G 80.77E6
  Members 33-48
   M  ...  ...          260x90 PFC 34.8 [S 275]
    A 44.4E-4            Iy 4728E-8          Iz 353.0E-8         It 20.6E-8
    E 210.0E6            G 80.77E6
```

MEMBER LOADING

Member Self Weight Density Load Included in Load Group D1, defined by Modulus of Elasticity

E kN/mm²	Density kN/m³
>= 200.00	77.01
>= 20.00	24.00
>= 2.00	10.00

Member 1 - MasterFrame Pro Loads

```
     D1 PTRY -000.926 0.000 6.726 -000.069    (kN/m,m,m,kN/m)
     D1 PTY1 -000.002 6.767 6.807 (kN,m,m)
     L1 PTRY -000.772 0.000 6.726 -000.057    (kN/m,m,m,kN/m)
     L1 PTY1 -000.002 6.767 6.807 (kN,m,m)
     D1 PTRY -000.926 0.000 6.727 -000.069    (kN/m,m,m,kN/m)
     L1 PTRY -000.772 0.000 6.727 -000.057    (kN/m,m,m,kN/m)
     D1 D     077.010             ( kN/m³)
```

Members 2 & 4 - MasterFrame Pro Loads

```
     D1 PTY2 -000.002 0.000 0.041 (kN,m,m)
     D1 PTRY -000.138 0.041 6.767 -001.852    (kN/m,m,m,kN/m)
     L1 PTY2 -000.002 0.000 0.041 (kN,m,m)
     L1 PTRY -000.114 0.041 6.767 -001.544    (kN/m,m,m,kN/m)
     D1 D     077.010             ( kN/m³)
```

Member 3 - MasterFrame Pro Loads

```
     D1 PTY2 -000.004 0.000 0.051 (kN,m,m)
     D1 PTRY -000.138 0.050 6.777 -001.852    (kN/m,m,m,kN/m)
     L1 PTY2 -000.002 0.000 0.051 (kN,m,m)
     L1 PTRY -000.114 0.050 6.777 -001.544    (kN/m,m,m,kN/m)
     D1 D     077.010             ( kN/m³)
```

Members 5 & 14 - MasterFrame Pro Loads

```
     D1 PTY2 -000.001 0.000 0.042 (kN,m,m)
     D1 PTRY -000.069 0.042 6.767 -000.926    (kN/m,m,m,kN/m)
     L1 PTY2 -000.001 0.000 0.042 (kN,m,m)
     L1 PTRY -000.058 0.042 6.767 -000.772    (kN/m,m,m,kN/m)
     D1 PTRY -000.074 0.000 6.220 -001.001    (kN/m,m,m,kN/m)
     D1 PTY1 -000.274 6.257 6.808 (kN,m,m)
     L1 PTRY -000.062 0.000 6.220 -000.834    (kN/m,m,m,kN/m)
     L1 PTY1 -000.228 6.257 6.808 (kN,m,m)
     D1 D     077.010             ( kN/m³)
```

Members 6 & 15 - MasterFrame Pro Loads

```
     D1 PTRY -000.150 0.000 6.220 -002.002    (kN/m,m,m,kN/m)
     D1 PTY1 -000.548 6.257 6.807 (kN,m,m)
     L1 PTRY -000.124 0.000 6.220 -001.668    (kN/m,m,m,kN/m)
     L1 PTY1 -000.456 6.257 6.807 (kN,m,m)
     D1 D     077.010             ( kN/m³)
```

Member 7 - MasterFrame Pro Loads

```
     D1 PTRY -000.074 0.000 6.220 -001.001    (kN/m,m,m,kN/m)
     D1 PTY1 -000.274 6.257 6.808 (kN,m,m)
     L1 PTRY -000.062 0.000 6.220 -000.834    (kN/m,m,m,kN/m)
     L1 PTY1 -000.228 6.257 6.808 (kN,m,m)
     D1 PTY2 -000.001 0.000 0.042 (kN,m,m)
     D1 PTRY -000.069 0.042 6.767 -000.926    (kN/m,m,m,kN/m)
     L1 PTY2 -000.001 0.000 0.042 (kN,m,m)
     L1 PTRY -000.058 0.042 6.767 -000.772    (kN/m,m,m,kN/m)
     D1 D     077.010             ( kN/m³)
```

Member 8 - MasterFrame Pro Loads

```
     D1 PTY2 -000.001 0.000 0.041 (kN,m,m)
     D1 PTRY -000.069 0.041 6.767 -000.926    (kN/m,m,m,kN/m)
     L1 PTY2 -000.001 0.000 0.041 (kN,m,m)
     L1 PTRY -000.057 0.041 6.767 -000.772    (kN/m,m,m,kN/m)
```

```
     D1 PTRY -000.074 0.000 6.220 -001.001    (kN/m,m,m,kN/m)
     D1 PTY1 -000.274 6.257 6.808 (kN,m,m)
     L1 PTRY -000.062 0.000 6.220 -000.834    (kN/m,m,m,kN/m)
     L1 PTY1 -000.228 6.257 6.808 (kN,m,m)
     D1 D     077.010             ( kN/m³)
Member 9 - MasterFrame Pro Loads
     D1 PTRY -000.075 0.000 6.221 -001.001    (kN/m,m,m,kN/m)
     D1 PTY1 -000.274 6.258 6.808 (kN,m,m)
     L1 PTRY -000.062 0.000 6.221 -000.834    (kN/m,m,m,kN/m)
     L1 PTY1 -000.228 6.258 6.808 (kN,m,m)
     D1 PTRY -000.074 0.000 6.220 -001.001    (kN/m,m,m,kN/m)
     D1 PTY1 -000.274 6.257 6.808 (kN,m,m)
     L1 PTRY -000.062 0.000 6.220 -000.834    (kN/m,m,m,kN/m)
     L1 PTY1 -000.228 6.257 6.808 (kN,m,m)
     D1 D     077.010             ( kN/m³)
Member 10 - MasterFrame Pro Loads
     D1 PTRY -000.074 0.000 6.220 -001.001    (kN/m,m,m,kN/m)
     D1 PTY1 -000.274 6.257 6.808 (kN,m,m)
     L1 PTRY -000.062 0.000 6.220 -000.834    (kN/m,m,m,kN/m)
     L1 PTY1 -000.228 6.257 6.808 (kN,m,m)
     D1 PTY2 -000.001 0.000 0.041 (kN,m,m)
     D1 PTRY -000.069 0.041 6.767 -000.926    (kN/m,m,m,kN/m)
     L1 PTY2 -000.001 0.000 0.041 (kN,m,m)
     L1 PTRY -000.058 0.041 6.767 -000.772    (kN/m,m,m,kN/m)
     D1 D     077.010             ( kN/m³)
Member 11 - MasterFrame Pro Loads
     D1 PTY2 -000.001 0.000 0.041 (kN,m,m)
     D1 PTRY -000.069 0.041 6.767 -000.926    (kN/m,m,m,kN/m)
     L1 PTY2 -000.001 0.000 0.041 (kN,m,m)
     L1 PTRY -000.058 0.041 6.767 -000.772    (kN/m,m,m,kN/m)
     D1 PTRY -000.074 0.000 6.220 -001.001    (kN/m,m,m,kN/m)
     D1 PTY1 -000.274 6.257 6.808 (kN,m,m)
     L1 PTRY -000.062 0.000 6.220 -000.834    (kN/m,m,m,kN/m)
     L1 PTY1 -000.228 6.257 6.808 (kN,m,m)
     D1 D     077.010             ( kN/m³)
Member 12 - MasterFrame Pro Loads
     D1 PTRY -000.074 0.000 6.220 -001.001    (kN/m,m,m,kN/m)
     D1 PTY1 -000.274 6.257 6.808 (kN,m,m)
     L1 PTRY -000.062 0.000 6.220 -000.834    (kN/m,m,m,kN/m)
     L1 PTY1 -000.228 6.257 6.808 (kN,m,m)
     D1 PTRY -000.075 0.000 6.221 -001.001    (kN/m,m,m,kN/m)
     D1 PTY1 -000.274 6.258 6.808 (kN,m,m)
     L1 PTRY -000.062 0.000 6.221 -000.834    (kN/m,m,m,kN/m)
     L1 PTY1 -000.228 6.258 6.808 (kN,m,m)
     D1 D     077.010             ( kN/m³)
Member 13 - MasterFrame Pro Loads
     D1 PTRY -000.074 0.000 6.220 -001.001    (kN/m,m,m,kN/m)
     D1 PTY1 -000.274 6.257 6.808 (kN,m,m)
     L1 PTRY -000.062 0.000 6.220 -000.834    (kN/m,m,m,kN/m)
     L1 PTY1 -000.228 6.257 6.808 (kN,m,m)
     D1 PTY2 -000.001 0.000 0.041 (kN,m,m)
     D1 PTRY -000.069 0.041 6.767 -000.926    (kN/m,m,m,kN/m)
     L1 PTY2 -000.001 0.000 0.041 (kN,m,m)
     L1 PTRY -000.057 0.041 6.767 -000.772    (kN/m,m,m,kN/m)
     D1 D     077.010             ( kN/m³)
Member 16 - MasterFrame Pro Loads
     D1 PTRY -000.074 0.000 6.221 -001.001    (kN/m,m,m,kN/m)
     D1 PTY1 -000.274 6.258 6.808 (kN,m,m)
     L1 PTRY -000.062 0.000 6.221 -000.834    (kN/m,m,m,kN/m)
     L1 PTY1 -000.228 6.258 6.808 (kN,m,m)
     D1 PTY2 -000.001 0.000 0.042 (kN,m,m)
     D1 PTRY -000.069 0.042 6.768 -000.926    (kN/m,m,m,kN/m)
     L1 PTY2 -000.001 0.000 0.042 (kN,m,m)
     L1 PTRY -000.058 0.042 6.768 -000.772    (kN/m,m,m,kN/m)
     D1 D     077.010             ( kN/m³)
   Members 17-32
     F1 De-activate        (Ignore Member)
        Ignore Self Weight
Members 17, 21, 25 & 29 - MasterFrame Pro Loads
```

```
     D1 TY2  -000.274              (  kN  )
     L1 TY2  -000.228              (  kN  )
```
Members 18-20, 22-24, 26-28 & 30-32 - MasterFrame Pro Loads
```
     D1 TY1  -000.274              (  kN  )
     L1 TY1  -000.228              (  kN  )
```
Members 33, 37, 41, 43-45 & 47-48 - MasterFrame Pro Loads
```
     D1 TY2  -000.001              (  kN  )
     L1 TY2  -000.001              (  kN  )
     D1 D     077.010              ( kN/m³)
```
Members 34, 38-40, 42 & 46 - MasterFrame Pro Loads
```
     D1 TY1  -000.001              (  kN  )
     L1 TY1  -000.001              (  kN  )
     D1 D     077.010              ( kN/m³)
```
Members 35-36 - MasterFrame Pro Loads
```
     D1 TY1  -000.002              (  kN  )
     L1 TY1  -000.001              (  kN  )
     D1 D     077.010              ( kN/m³)
```

MEMBER ORIENTATION

Members 17-32 ß -090.00

MEMBER END RELEASES

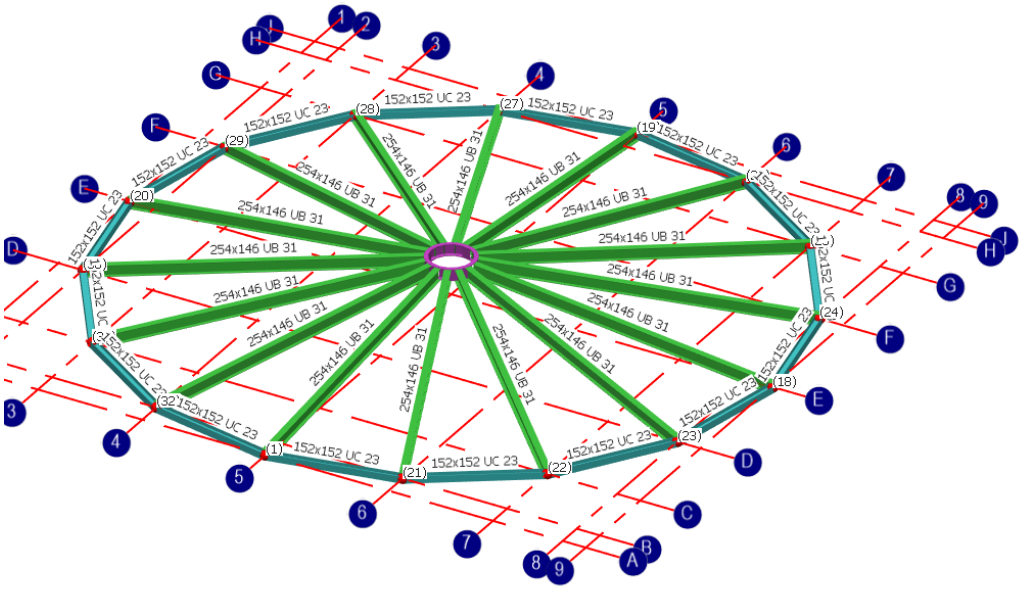
Symbols	End 1 torsion-minor-major	End 2 torsion-minor-major	O Released	® Fixed
1	®®® ®®O	2 ®®O ®®®	3 ®®O ®®®	4 ®®O ®®®
5	®®O ®®®	6 ®®O ®®®	7 ®®O ®®®	8 ®®O ®®®
9	®®O ®®®	10 ®®O ®®®	11 ®®O ®®®	12 ®®O ®®®
13	®®O ®®®	14 ®®O ®®®	15 ®®O ®®®	16 ®®O ®®®
17	®OO ®OO	18 ®OO ®OO	19 ®OO ®OO	20 ®OO ®OO
21	®OO ®OO	22 ®OO ®OO	23 ®OO ®OO	24 ®OO ®OO
25	®OO ®OO	26 ®OO ®OO	27 ®OO ®OO	28 ®OO ®OO
29	®OO ®OO	30 ®OO ®OO	31 ®OO ®OO	32 ®OO ®OO

NODAL LOADING AND SUPPORT CONDITIONS

NODE 1
```
      UT Rs 1 1 1 0 1 0 (Pinned with Y Moment Restraint)
```
NODES 18 & 20-32
```
      UT Rs 0 1 0 0 0 0 (Vertical Restraint)
```
NODE 19
```
      UT Rs 1 1 0 0 0 0 (Pinned, X and Y Restraint)
```

MasterFrame : Graphics

	Project			Job Ref.
	Llangollen Museum Parade Street LL20 8PW			24014
	Section			Sheet no./rev.A
	Proposed Replacement Roof			30
	Calc. by	Date	Updated A	
	Hodgson	16/03/24	01/05/24	



Frame Geometry - Full Frame - 3D Front View

Support Reactions - Maximum Values

Maximum and Minimum Elastic Member Stresses (Compression +)|001 : Dead plus Live (Ultimate)										
M (Case)	Section Size	F kN	Mx kN.m	My kN.m	fa N/mm²	fx N/mm²	fy N/mm²	fmax N/mm²	fmin N/mm²	Length m
1(1)	254x146 UB 31	73.1	16.6	0.0	18.4	47.3	0.4	66.1	-29.2	6.807
2(1)	254x146 UB 31	73.1	16.6	0.0	18.4	47.3	0.3	66.0	-29.2	6.807
3(1)	254x146 UB 31	73.1	16.6	0.0	18.4	47.4	0.4	66.2	-29.4	6.817
4(1)	254x146 UB 31	73.1	16.6	0.0	18.4	47.3	0.3	66.0	-29.2	6.807
5(1)	254x146 UB 31	73.2	17.8	0.0	18.5	50.6	0.1	69.1	-32.2	6.808
6(1)	254x146 UB 31	73.3	18.9	0.0	18.5	53.8	0.1	72.4	-35.4	6.807
7(1)	254x146 UB 31	73.1	17.8	0.0	18.4	50.6	0.2	69.2	-32.3	6.808
8(1)	254x146 UB 31	73.1	17.8	0.0	18.4	50.6	0.3	69.3	-32.4	6.808
9(1)	254x146 UB 31	73.3	18.9	0.0	18.5	53.8	0.0	72.3	-35.4	6.808
10(1)	254x146 UB 31	73.2	17.8	0.0	18.5	50.6	0.5	69.5	-32.6	6.808
11(1)	254x146 UB 31	73.0	17.8	0.1	18.4	50.6	1.2	70.2	-33.4	6.808
12(1)	254x146 UB 31	73.2	18.9	0.0	18.4	53.8	0.5	72.8	-35.9	6.808
13(1)	254x146 UB 31	73.1	17.8	0.0	18.4	50.6	0.0	69.0	-32.1	6.808
14(1)	254x146 UB 31	73.1	17.8	0.0	18.4	50.6	0.4	69.4	-32.6	6.808
15(1)	254x146 UB 31	73.2	18.9	0.0	18.4	53.8	0.6	72.9	-36.0	6.807
16(1)	254x146 UB 31	73.0	17.8	0.0	18.4	50.6	0.6	69.5	-32.7	6.808
17(1)	152x152 UC 23	-182.7	0.0	0.2	-62.5	0.0	4.7	-57.7	-67.2	2.836
18(1)	152x152 UC 23	-182.6	0.0	0.2	-62.5	0.0	4.7	-57.7	-67.2	2.836
19(1)	152x152 UC 23	-182.5	0.0	0.2	-62.4	0.0	4.7	-57.7	-67.2	2.835
20(1)	152x152 UC 23	-183.3	0.0	0.2	-62.7	0.0	4.7	-58.0	-67.4	2.835
21(1)	152x152 UC 23	-183.2	0.0	0.2	-62.7	0.0	4.7	-57.9	-67.4	2.836
22(1)	152x152 UC 23	-183.2	0.0	0.2	-62.6	0.0	4.7	-57.9	-67.4	2.836
23(1)	152x152 UC 23	-183.2	0.0	0.2	-62.6	0.0	4.7	-57.9	-67.4	2.835
24(1)	152x152 UC 23	-183.1	0.0	0.2	-62.6	0.0	4.7	-57.9	-67.4	2.835
25(1)	152x152 UC 23	-183.2	0.0	0.2	-62.6	0.0	4.7	-57.9	-67.4	2.836
26(1)	152x152 UC 23	-183.2	0.0	0.2	-62.7	0.0	4.7	-57.9	-67.4	2.836
27(1)	152x152 UC 23	-183.3	0.0	0.2	-62.7	0.0	4.7	-58.0	-67.4	2.835
28(1)	152x152 UC 23	-182.5	0.0	0.2	-62.4	0.0	4.7	-57.7	-67.2	2.835

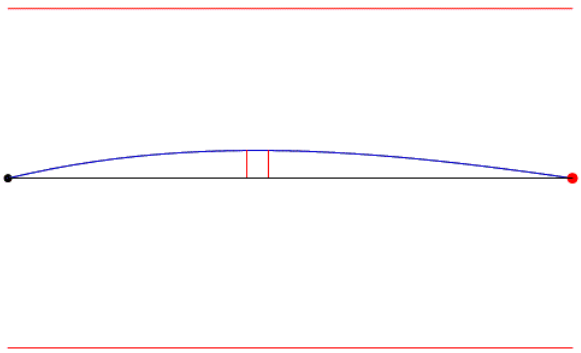
29(1)	152x152 UC 23	-182.6	0.0	0.2	-62.5	0.0	4.7	-57.7	-67.2	2.836
30(1)	152x152 UC 23	-182.7	0.0	0.2	-62.5	0.0	4.7	-57.7	-67.2	2.836
31(1)	152x152 UC 23	-182.7	0.0	0.2	-62.5	0.0	4.7	-57.8	-67.2	2.835
32(1)	152x152 UC 23	-182.7	0.0	0.2	-62.5	0.0	4.7	-57.8	-67.2	2.835
33(1)	260x90 PFC 34.8	183.2	0.0	0.2	41.3	0.1	4.3	45.7	36.8	0.195
34(1)	260x90 PFC 34.8	183.4	0.0	0.3	41.3	0.1	5.9	47.3	35.3	0.196
35(1)	260x90 PFC 34.8	183.2	0.0	1.4	41.3	0.1	25.0	66.4	16.1	0.193
36(1)	260x90 PFC 34.8	183.1	0.0	1.4	41.2	0.1	24.6	65.9	16.6	0.193
37(1)	260x90 PFC 34.8	183.2	0.0	0.3	41.3	0.1	5.8	47.2	35.4	0.196
38(1)	260x90 PFC 34.8	183.1	0.0	0.2	41.2	0.1	4.3	45.7	36.8	0.195
39(1)	260x90 PFC 34.8	183.0	0.1	0.1	41.2	0.2	2.6	44.0	38.4	0.195
40(1)	260x90 PFC 34.8	182.8	0.1	0.1	41.2	0.1	1.4	42.7	39.6	0.195
41(1)	260x90 PFC 34.8	182.6	0.0	0.1	41.1	0.1	1.0	42.3	40.0	0.196
42(1)	260x90 PFC 34.8	182.5	0.0	0.1	41.1	0.1	1.9	43.1	39.1	0.196
43(1)	260x90 PFC 34.8	182.4	0.0	0.1	41.1	0.1	1.9	43.1	39.1	0.195
44(1)	260x90 PFC 34.8	182.5	0.0	0.1	41.1	0.1	2.6	43.8	38.4	0.195
45(1)	260x90 PFC 34.8	182.6	0.1	0.2	41.1	0.1	3.2	44.5	37.8	0.195
46(1)	260x90 PFC 34.8	182.7	0.0	0.1	41.2	0.1	1.1	42.3	40.0	0.196
47(1)	260x90 PFC 34.8	182.9	0.1	0.1	41.2	0.1	1.3	42.6	39.7	0.195
48(1)	260x90 PFC 34.8	183.1	0.1	0.1	41.2	0.2	2.5	43.9	38.6	0.195

AXIAL WITH MOMENTS (MEMBER)

Member SB D-E\5-2L1Id 45 @ Level 1 in Load Case 1

Member Loading and Member Forces
Loading Combination : 1 UT + 1.25 D1 + 1.5 L1

D1 PTY2 -000.001 0.000 0.042 (kN,m,m)
D1 PTRY -000.069 0.042 6.767 -000.926
L1 PTY2 -000.001 0.000 0.042 (kN,m,m)
L1 PTRY -000.058 0.042 6.767 -000.772
D1 PTRY -000.074 0.000 6.220 -001.001
D1 PTY1 -000.274 6.257 6.808 (kN,m,m)
L1 PTRY -000.062 0.000 6.220 -000.834
L1 PTY1 -000.228 6.257 6.808 (kN,m,m)
D1 D 077.010 (kN/m³)



Member Forces in Load Case 1 and Maximum Deflection from Load Case 3										
Member No.	Node End 1 End 2	Axial Force (kN)	Torque Moment (kNm)	Shear Force (kN)		Bending Moment (kNm)		Maximum Moment (kNm @ m)		Max Def (mm @ m)
				y-y	z-z	y-y	z-z	y-y	z-z	
14	6	70.93C	0.00	-7.67	0.00	0.00	0.03	-17.76		6.76
	30	73.10C	0.00	12.22	0.00	0.00	0.00	@ 3.834		@ 3.521

Classification and Effective Area (EN 1993: 2006)

Section (31.14 kg/m)	254x146 UB 31 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	8.49, 36.5, 275, 73.1, 17.76, 0.03	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Shear Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	12.217 / 259.813 =	0.047	OK

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	0.476 / 259.813 =	0.002	Low Shear
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 393.1/1	108.103 kN.m	
$V_{z.Ed}/V_{pl.z.Rd}$	0.004 / 398.98 =	0	Low Shear
$M_{c.z.Rd} = f_y.W_{pl.z}/\gamma_{M0}$	275 x 94.1/1	25.878 kN.m	
$N_{pl.Rd} = A_g.f_y/\gamma_{M0}$	39.67 x 275/1 =	1090.925 kN	

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	Hodgson	16/03/24	01/05/24	

$n = N_{Ed}/N_{pl.Rd}$	73.101 / 1090.925 =	0.067	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy},)$	393.1, 16.364, 0.067	393.1 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/ \gamma_{M0}$	393.1 x 275/1	108.103 kN.m	
$W_{pl.N.z} = Fn(W_{pl.z}, A_{vz},)$	94.1, 25.129, 0.067	94.1 cm³	
$M_{N.z.Rd} = W_{pl.N.z}.f_y/ \gamma_{M0}$	94.1 x 275/1	25.878 kN.m	
$(M_{y.Ed}/M\_(N.y.Rd+(M_{z.Ed}/M_{N.z.Rd})$	$(17.73/108.103)^2+(0.012/25.878)^1=$	0.027	OK

Compression Resistance N.b.Rd

Ley = Ky.Ly	1x6.807 =	6.807	
$\lambda y = \sqrt{A.fy/Ncr}$	√39.67x275/1974.36	0.743	
$N_{b.y.Rd} = Area.c.f_y/ \gamma_{M1}$	39.67x0.826x275/10/1 =	901.645 kN	Curve a
Lez = Kz.Lz	1x6.807 =	6.807	
$\lambda z = \sqrt{A.fy/Ncrz}$	√39.67x275/200.57	2.334	
$N_{b.z.Rd} = Area.c.f_y/ \gamma_{M1}$	39.67x0.159x275/10/1 =	172.951 kN	Curve b
Let = Kt.Lx	1x6.807 =	6.807	
$\lambda T = \sqrt{A.fy/NcrT}$	√39.67x275/803.76	1.165	
$N_{b.T.Rd}= Area.c.f_y/ \gamma_{M1}$	39.67x0.497x275/10/1 =	542.700 kN	Curve b

Equivalent Uniform Moment Factors C1, C.mLT, C.mz, and C.my

$C_1= fn(M_1, M_2, M_o, \sim y, \sim m)$	0.0, 0.0, 17.5, 0.098, 300.000	1.127	Uniform
$C_{mLT}=0.95+0.05\alpha_h$	M_h = 0.04, M_s = 17.49, ~y = 0.098, α_s = 0.002	0.95	Table B.3
$C_{mz}=Max(0.6+0.4\sim y, 0.4)$	M = 0.03, ~y = 0.000	0.6	Table B.3
$C_{my}=0.95+0.05\alpha_h$	M_h = 0, M_s = 17.49, ~y = 1.000, α_s = 0.000	0.95	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.00 L	1 x 6.807 =	6.807 m	
$M_{cr} = Fn(C_1,Le,I_z,I_t,I_w,E)$	1.127, 6.807, 448.4, 8.552, 0.06588, 210000	50.103 kN.m	
$\lambda_{LT} = \sqrt{} W.f_y/M_{cr}$	√ 393.1 x 275 / 50.103	1.469	
$c_{LT} = Fn(\lambda_{LT}, \Phi_{LT}, \beta, \lambda_{LT}0)$	1.469, 1.491, 0.750, 0.400	0.441	Curve b
$c_{LT}.mod = Fn(c_{LT},\lambda_{LT},k_c,f)$	0.441, 1.469, 0.942, 0.997	0.442	6.3.2.3
$M_{b.Rd} = c\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.442 x 393.1 x 275 ≤ 108.103 =	47.806 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(c_y.N_{Rk}/\gamma_{M1})$	73.101 / 901.645	0.081	OK
$U_{N.z} = N_{Ed}/(c_z.N_{Rk}/\gamma_{M1})$	73.101 / 172.951	0.423	OK
$U_{M.y} = M_{y.Ed}/(c_{LT}.M_{y.Rk}/\gamma_{M1})$	17.743 / 47.806	0.371	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0.011 / 25.878	0.000	OK
$k_yy=C_{my}\{1+(\lambda_y-0.2)U_{N.y}\}$		0.992	
$k_zz=C_{mz}\{1+1.4U_{N.z}\}$		0.955	
$k_yz=0.6\ k_zz$		0.573	
$k_zy= 1- \{0.1\lambda_z/(C_{mLT}-0.25)\}U_{N.z}$		0.859	
$U_{Ny}+k_yy.U_{M.y}+k_yz.U_{M.z}$	0.081+0.992x0.371+0.573x0.000	0.449	OK
$U_{Nz}+k_zy.U_{M.y}+k_zz.U_{M.z}$	0.423+0.859x0.371+0.955x0.000	0.742	OK

Deflection Check - Load Case 3

In-span δ ≤ Span/360	6.76 ≤ 6807 / 360	6.76 mm	OK

STRUT AND TIE (MEMBER)

Member SB C\3-4L1Id 29 @ Level 1

Classification and Effective Area (EN 1993: 2006)

Section (22.95 kg/m)	152x152 UC 23 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	11.19, 21.31, 275, -182.61, 0, 0.25	(Axial: Non-Slender)	Class 3
Effective Properties	Area=29.24 cm², $W_{pl.y}$=179.39(182) cm³, $W_{pl.z}$=76.18(80.2) cm³		
Auto Design Load Cases	1		

Axially Loaded Member in Tension : 6.2.3 (Case 1)

$Tc = A_g.fy/\gamma_{M0}$	29.24x275/1 (No bearing / block tearing design)	804.1 kN	
F (Tie)/Tc	182.614 / 804.1	0.227	OK

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	Llangollen Museum Parade Street LL20 8PW			24014
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	Hodgson	16/03/24	01/05/24	

STRUT AND TIE (MEMBER)

Applied Major and Minor Axes Bending Moments have been ignored

Member SB 5-4\EL1Id 48 @ Level 1

Classification and Effective Area (EN 1993: 2006)

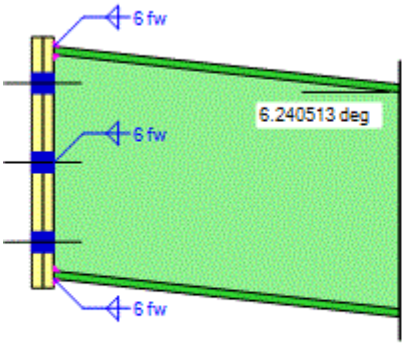
Section (34.8 kg/m)	260x90 PFC 34.8 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	6.43, 26, 275, 0, 0.04, 0.25	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Strut by One Row of Bolts or Equivalent Weld (Case 1)

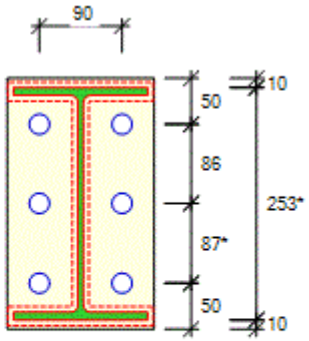
λ = Fn(Lx,Ly,rx,ry)	0.195,0.195,10.32,2.82	6.9	OK
$N_{b.Rd}$ = Area.f_c	44.4x275/10 =	1221 kN	Curve c
F (Strut)/$N_{b.Rd}$	183.25 / 1221	0.150	OK

254X146UB TO 260X90PFC

APEX JOINT AT : END 1 OF MEMBER SB F-E\5-8L1ID 39 - LEVEL 1



Plates S 275
Beam 254x146 UB 31 [S275]



End-Plate 273 x 160 x 12 mm (4 kg)
6 No. M20 Grade 8.8 Bolts in 22 mm holes

Bea m to Beam End-Plated Connection to EC 3 (UK NAD)

LOADING CASE 001 : DEAD PLUS LIVE (ULTIMATE)

Basic Data

Integrated Applied Forces at End-plate Interface

Right Rafter Forces M, Fvr, Fr	0.0 kNm, 7.7 kN, 71.0 kN	
Resultant Forces M, Fv, F	0.0 kNm, -0.1 kN, 71.4 kN	Load directionsRafter moving Up and in Compression.
Design to	EC 3: Part 1-8: 2005 Design of Connections SCI Green Book P398: Joints in steel construction: Moment-Resisting Joints to Eurocode 3	
Weld Grades	All weld grades provided to suit minimum connected steel grade	

Basic Dimensions

Rafter-254x146UB31 [S 275]	D=251.4, B=146.1, T=8.6, t=6.0, r=7.6, py=275		
Bolts 20 mm Ø in 22 mm holes	Grade 8.8 Bolts		
Plates S 275	All weld grades provided to suit minimum connected steel grade		
Rafter Capacities Mc, Fvc, Fc	108.9 kN.m, 261.2 kN, 1093.4 kN	Fc = 1093.4 kN	OK

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	Llangollen Museum Parade Street LL20 8PW			24014
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	Proposed Replacement Roof			34
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	Hodgson	16/03/24	01/05/24	

Summary of Results (Unity Ratios)

Compression Checks Bottom	0.08	0.08	OK
Web Welds	0.00	0.00	OK
Compression Checks Top	0.08	0.08	OK
Shear Capacity		0.00	OK

Step 2: Bottom Compression Zone

Compression Force		35.47 kN	

Beam Compression

$F_{c,fb,Rd}= M_{c,Rd}/(h_b-t_{fb})$	108.1•1000/(242.80)	445.2 kN	OK

Step 7: Bottom Welds

Beam $f_{vw,d}=f_u / (\sqrt{3}•ß_w) / \gamma_{M2}$	410.0 / √3 / 0.85 / 1.25	222.8 N/mm²	

Flange Bottom Compression Weld

Direct Bearing assumed. No check required

Web Welds in Shear Zone

$Lws=D-(T_t+T_b)-r_t -r_b -Lwt$	252.9 - 17.2 - 7.6 - 7.6 - 0	220.5 mm	
$F_{wCap} =2•0.7•ts•Lws•f_{vw,d}$	2•0.7•6•220.5•223	416.8 kN	OK

Step 2: Top Compression Zone

Compression Force		35.91 kN	

Beam Compression

$F_{c,fb,Rd}= M_{c,Rd}/(h_b-t_{fb})$	108.1•1000/(242.80)	445.2 kN	OK

Step 7: Top Welds

Beam $f_{vw,d}=f_u / (\sqrt{3}•ß_w) / \gamma_{M2}$	410.0 / √3 / 0.85 / 1.25	222.8 N/mm²	

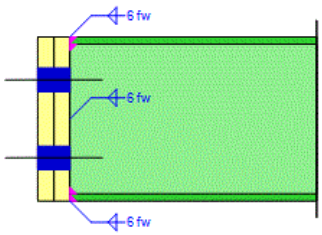
FlangeTop Compression Weld

Direct Bearing assumed. No check required

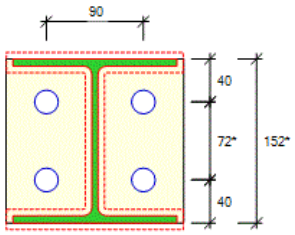
Step 5: Shear Bolts

Bolt Shear Capacity	BSC=91.875, tg=24	94.1 kN	
Bearing Capacity-End Plate	pb=460, edge=50.0, Ø=20, tk=12, kbs=1.00	110.4 kN	
Bearing Capacity-Bolts	pb=1000, Ø=20, tk=12	240.0 kN	
Pss=Min(bearing...,shear)	Min(110.4, 240.0, 94.1)	94.1 kN	
Pts Min(bearing...,0.4•shear)	Min(110.4, 240.0, 37.6)	37.6 kN	
V=Ns•Pss+Nt•Pts	6•94.1 + 0•37.6	564 kN	OK

152UC TIE BEAM



Plates S 275
Beam 152x152 UC 23 [S275]



End-Plate 152 x 165 x 15 mm (3 kg)
4 No. M20 Grade 8.8 Bolts in 22 mm holes

Beam to Beam End-Plated Connection to EC 3 (UK NAD)

LOADING CASE 001 DEAD PLUS LIVE

Basic Data

User Defined Applied Forces at End-plate Interface

Resultant Forces M, Fv, F	5.0 kNm, 0.0 kN, 180.0 kN
Load directions	Top of Connection in Tension, Rafter in Compression.
Design to	EC 3: Part 1-8: 2005 Design of Connections
	SCI Green Book

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	Llangollen Museum Parade Street LL20 8PW			24014
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	Proposed Replacement Roof			35
	Calc. by	Date	Updated A	
	Hodgson	16/03/24	01/05/24	

P398: Joints in steel construction: Moment-Resisting Joints to Eurocode 3

Weld Grades — All weld grades provided to suit minimum connected steel grade

Basic Dimensions

Beam-152x152UC23 [S 275]	D=152.4, B=152.2, T=6.8, t=5.8, r=7.6, py=275		
Bolts 20 mm Ø in 22 mm holes	Grade 8.8 Bolts		
Plates S 275	All weld grades provided to suit minimum connected steel grade		
Rafter Capacities Mc, Fvc, Fc	45.1 kN.m, 158.3 kN, 804.1 kN	Fc = 804.1 kN	OK

Summary of Results (Unity Ratios)

Compression Checks Bottom	0.40	0.40	OK
Web Welds	0.00	0.00	OK
Compression Checks Top	0.18	0.18	OK
Shear Capacity		0.00	OK

Step 2: Bottom Compression Zone

Compression Force — 123.55 kN

Beam Compression

$F_{c,fb,Rd}= M_{c,Rd}/(h_b-t_{fb})$	45.1•1000/(145.60)	310.0 kN	OK

Step 7: Bottom Welds

Beam $f_{vw,d}=f_u / (\sqrt{3}•ß_w) / \gamma_{M2}$	410.0 / √3 / 0.85 / 1.25	222.8 N/mm²	

Flange Bottom Compression Weld

Direct Bearing assumed. No check required

Web Welds in Shear Zone

$Lws=D-(T_t+T_b)-r_t-r_b-Lwt$	152.4 - 13.6 - 7.6 - 7.6 - 0	123.6 mm	
$F_{wCap}=2•0.7•ts•Lws•f_{vw,d}$	2•0.7•6•123.6•223	233.6 kN	OK

Step 2: Top Compression Zone

Compression Force — 56.45 kN

Beam Compression

$F_{c,fb,Rd}= M_{c,Rd}/(h_b-t_{fb})$	45.1•1000/(145.60)	310.0 kN	OK

Step 7: Top Welds

Beam $f_{vw,d}=f_u / (\sqrt{3}•ß_w) / \gamma_{M2}$	410.0 / √3 / 0.85 / 1.25	222.8 N/mm²	

FlangeTop Compression Weld

Direct Bearing assumed. No check required

Step 5: Shear Bolts

Bolt Shear Capacity	BSC=91.875, tg=30	94.1 kN	
Bearing Capacity-End Plate	pb=460, edge=40.0, Ø=20, tk=15, kbs=1.00	138.0 kN	
Bearing Capacity-Bolts	pb=1000, Ø=20, tk=15	300.0 kN	
Pss=Min(bearing...,shear)	Min(138.0, 300.0, 94.1)	94.1 kN	
Pts Min(bearing...,0.4•shear)	Min(138.0, 300.0, 37.6)	37.6 kN	
V=Ns•Pss+Nt•Pts	4•94.1 + 0•37.6	376 kN	OK