ROOF LOADING APPRAISAL

ENERGYOASIS
Your Net Zero Partner

ROOF 1
CONNECT HOUSING ASSOCIATION
21 BOND ST, DEWSBURY WF13 1AX

Report No:	63326-1-SS				
Issue No.	-				
Date	16.10.2026				
Prepared By	DS				
Checked By	BMcG				
Signature					
Revisions:					

CALCULATIONS CONDUCTED IN ACCORDANCE WITH MCS SOLAR PV STANDARDS

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NOTE: This report has been conducted in accordance with BS EN 1991-1, BRE Digest 489 (2015) and (Where required) Section 5.9 of MIS 3002 The Solar PV Standard.

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1.0 SITE SURVEY INFORMATION

BMG Surveys Ltd have been commissioned to carry out a structural roof loading appraisal of the afore mentioned commercial property for its suitability for a Solar PV Installation. Our checks included an onsite inspection and following the collation of the required information a set of calculations have been carried out in respect of the existing roof loadings and the proposed loadings for a Solar PV installation.

Our on-site inspection was carried out on a cloudy day. Our inspections cover all areas of the property and particular attention was paid to the roof and its structure.

Our visual inspection was carried out of all visible aspects of the properties structure including walls & roof both externally and internally. It was found that the properties primary structure and roof structure look to be in good condition with no signs of failure or deflection.

We conclude that the onsite inspection of the property and our subsequent design checks within sections 2.0, 3.0 & 4.0 of this report confirm that the roof and supporting structure can safely accept the loadings from the proposed Solar PV installation and we have no concerns over the structures ability to accept the loadings.



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2.0 APPLIED LOADINGS

In considering the applied loading, we have assessed as noted below:

- Dead loads are based on the actual specified make up for the existing roof.
- Imposed floor loads are based on the loadings within BS 6399 & Eurocode- 1 (BS EN 1991-1) in line with the date of design/construction.
- Wind & Snow loadings are calculated on a site-specific basis in line with European Codes of Practice.
- The property has been built in accordance with relevant building codes.

Applied loads are as follows:

EXISTING ROOF MAKE UP:

Dead Load	**Load (kN/m2)**
Service Loading	0.15
Slate Roof Tiles	0.35
Felt & Battens	0.05
Rafters & Joists	0.10
Wool Insulation	0.02
Plasterboard Ceiling	0.08
Total	0.75

IMPOSED LOADS

CP3 Chapter V
(Building was constructed Prior to 1988.) **General Roof Load** **= 0.75kN/m2**

EXISTING ROOF WIND LOADINGS:

Calculated using TEDDS design software for both positive and negative internal pressure and for wind acting both perpendicular and parallel to the front elevation of the building.

The positive pressures on the building have already been considered at design stage and installing Solar PV does not increase or reduce these loadings, therefore positive wind pressures do not need to be considered in this loading assessment.

Negative wind pressures are used to calculate the wind uploads and determine the fixing resistance for any mounting frame designs.

EXISTING ROOF SNOW LOADINGS:

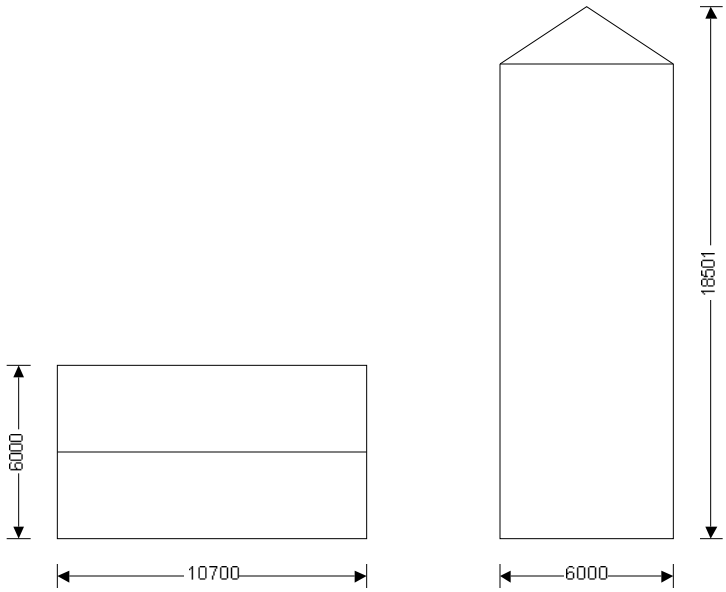
Calculated using TEDDS design software for both basic and where appropriate complex snow loadings.

WIND LOADING

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

Tedds calculation version 3.0.30

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Building data

Type of roof;	Duopitch
Length of building;	L = **10700** mm
Width of building;	W = **6000** mm
Height to eaves;	H = **16500** mm
Pitch of roof;	α_0 = **33.7** deg
Total height;	h = **18501** mm

Basic values

Location;	Dewsbury
Wind speed velocity (FigureNA.1);	$v_{b,map}$ = **22.5** m/s
Distance to shore;	L_{shore} = **61.30** km
Altitude above sea level;	A_{alt} = **79.0**m
Altitude factor;	$c_{alt} = A_{alt}/1m \times 0.001 + 1$ = **1.079**
Fundamental basic wind velocity;	$v_{b,0} = v_{b,map} \times c_{alt}$ = **24.3** 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);	$v_b = c_{dir} \times c_{season} \times v_{b,0} \times c_{prob}$ = **24.3** m/s
Reference mean velocity pressure;	$q_b = 0.5 \times \rho \times v_b^2$ = **0.361** kN/m^2

Orography

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

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The velocity pressure for the windward face of the building with a 0 degree wind is to be considered as 2 parts as the height h is greater than b but less than 2b (cl.7.2.2)

The velocity pressure for the windward face of the building with a 90 degree wind is to be considered as 3 parts as the height h is greater than 2b (cl.7.2.2)

Peak velocity pressure - windward wall (lower part) - Wind 0 deg

Reference height (at which q is sought);	z = **10700**mm
Displacement height (sheltering effects excluded);	h_{dis} = **0** mm
Exposure factor (Figure NA.7);	c_e = **2.40**
Peak velocity pressure;	$q_p = c_e \times q_b$ = **0.87** kN/m^2

Structural factor

Structural damping;	δ_s = **0.100**
Height of element;	h_{part} = **10700** mm
Size factor (Table NA.3);	c_s = **0.919**
Dynamic factor (Figure NA.9);	c_d = **1.036**
Structural factor;	$c_{sCd} = c_s \times c_d$ = **0.953**

Peak velocity pressure - windward wall (upper part) - Wind 0 deg and roof

Reference height (at which q is sought);	z = **16500**mm
Displacement height (sheltering effects excluded);	h_{dis} = **0** mm
Exposure factor (Figure NA.7);	c_e = **2.69**
Peak velocity pressure;	$q_p = c_e \times q_b$ = **0.97** kN/m^2

Structural factor

Structural damping;	δ_s = **0.100**
Height of element;	h_{part} = **5800** mm
Size factor (Table NA.3);	c_s = **0.936**
Dynamic factor (Figure NA.9);	c_d = **1.036**
Structural factor;	$c_{sCd} = c_s \times c_d$ = **0.970**

Structural factor

Structural damping;	δ_s = **0.100**
Height of element;	h_{part} = **16500** mm
Size factor (Table NA.3);	c_s = **0.919**
Dynamic factor (Figure NA.9);	c_d = **1.036**
Structural factor;	$c_{sCd} = c_s \times c_d$ = **0.953**

Peak velocity pressure - windward wall (lower part) - Wind 90 deg

Reference height (at which q is sought);	z = **6000**mm
Displacement height (sheltering effects excluded);	h_{dis} = **0** mm
Exposure factor (Figure NA.7);	c_e = **2.04**
Peak velocity pressure;	$q_p = c_e \times q_b$ = **0.74** kN/m^2

Structural factor

Structural damping;	δ_s = **0.100**
Height of element;	h_{part} = **6000** mm
Size factor (Table NA.3);	c_s = **0.934**
Dynamic factor (Figure NA.9);	c_d = **1.052**
Structural factor;	$c_{sCd} = c_s \times c_d$ = **0.983**

Peak velocity pressure - windward wall, (middle part) - Wind 90 deg

Reference height (at which q is sought);	z = **12501**mm
Displacement height (sheltering effects excluded);	h_{dis} = **0** mm

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Exposure factor (Figure NA.7);	c_e = **2.50**
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} = **6501** mm
Size factor (Table NA.3);	c_s = **0.944**
Dynamic factor (Figure NA.9);	c_d = **1.052**
Structural factor;	$c_{sCd} = c_s \times c_d$ = **0.993**

Peak velocity pressure - windward wall (upper part), other walls and roof - Wind 90 deg

Reference height (at which q is sought);	z = **18501**mm
Displacement height (sheltering effects excluded);	h_{dis} = **0** mm
Exposure factor (Figure NA.7);	c_e = **2.76**
Peak velocity pressure;	$q_p = c_e \times q_b$ = **1.00** kN/m^2

Structural factor

Structural damping;	δ_s = **0.100**
Height of element;	h_{part} = **6000** mm
Size factor (Table NA.3);	c_s = **0.949**
Dynamic factor (Figure NA.9);	c_d = **1.052**
Structural factor;	$c_{sCd} = c_s \times c_d$ = **0.999**

Structural factor

Structural damping;	δ_s = **0.100**
Height of element;	h_{part} = **18501** mm
Size factor (Table NA.3);	c_s = **0.924**
Dynamic factor (Figure NA.9);	c_d = **1.052**
Structural factor;	$c_{sCd} = c_s \times c_d$ = **0.972**

Structural factor - roof 0 deg

Structural damping;	δ_s = **0.100**
Height of element;	h_{part} = **18501** mm
Size factor (Table NA.3);	c_s = **0.919**
Dynamic factor (Figure NA.9);	c_d = **1.036**
Structural factor;	$c_{sCd} = c_s \times c_d$ = **0.953**

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.);	$q_{p,i}$ = **1.00** 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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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)	-0.38	1.00	-0.56	6.88	-3.84
G (-ve)	-0.38	1.00	-0.56	6.88	-3.84
H (-ve)	-0.15	1.00	-0.34	24.82	-8.51
I (-ve)	-0.50	1.00	-0.68	24.82	-16.76
J (-ve)	-0.88	1.00	-1.03	13.76	-14.21

Total vertical net force; $F_{w,v}$ = **-39.23** kN

Total horizontal net force; $F_{w,h}$ = **8.20** 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	1.00	-1.34	36.84	-49.40
B	-0.80	1.00	-0.96	68.17	-65.48
D_b	0.80	0.87	0.46	114.49	52.85
D_u	0.80	0.97	0.55	62.06	34.35
E	-0.60	0.97	-0.76	176.55	-133.91

Overall loading

Equiv leeward net force for upper section; $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wu}$ = **-47.1** kN

Net windward force for upper section; $F_w = F_{w,wu}$ = **34.4** kN

Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.93**; as h/W is 3.083

Overall loading upper section; $F_{w,u} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **83.2** kN

Equiv leeward net force for bottom section; $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wb}$ = **-86.8** kN

Net windward force for bottom section; $F_w = F_{w,wb}$ = **52.9** kN

Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.93**; as h/W is 3.083

Overall loading bottom section; $F_{w,b} = f_{corr} \times (F_w - F_l)$ = **129.6** 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)	0.80	1.00	1.06	6.88	7.30
G (+ve)	0.52	1.00	0.80	6.88	5.49
H (+ve)	0.47	1.00	0.75	24.82	18.63
I (+ve)	-0.50	1.00	-0.18	24.82	-4.37
J (+ve)	-0.88	1.00	-0.53	13.76	-7.34

Total vertical net force; $F_{w,v}$ = **16.40** kN

Total horizontal net force; $F_{w,h}$ = **23.93** kN

Walls 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)
A	-1.20	1.00	-0.84	36.84	-31.01
B	-0.80	1.00	-0.46	68.17	-31.45
D_b	0.80	0.87	0.96	114.49	110.01
D_u	0.80	0.97	1.05	62.06	65.33
E	-0.60	0.97	-0.26	176.55	-45.77

Overall loading

Equiv leeward net force for upper section; $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wu}$ = **-16.1** kN

Net windward force for upper section; $F_w = F_{w,wu}$ = **65.3** kN

Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.93**; as h/W is 3.083

Overall loading upper section; $F_{w,u} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **97.8** kN

Equiv leeward net force for bottom section; $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wb}$ = **-29.7** kN

Net windward force for bottom section; $F_w = F_{w,wb}$ = **110.0** kN

Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.93**; as h/W is 3.083

Overall loading bottom section; $F_{w,b} = f_{corr} \times (F_w - F_l)$ = **129.6** 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)	-1.20	1.00	-1.36	2.16	-2.95
G (-ve)	-1.12	1.00	-1.29	2.16	-2.79
H (-ve)	-0.60	1.00	-0.78	17.31	-13.54
I (-ve)	-0.48	1.00	-0.66	55.53	-36.71

Total vertical net force; $F_{w,v}$ = **-46.58** kN

Total horizontal net force; $F_{w,h}$ = **0.00** 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	0.97	-1.33	19.80	-26.38
B	-0.80	0.97	-0.95	79.20	-75.61
C	-0.50	0.97	-0.67	77.55	-52.08
D_b	0.80	0.74	0.38	36.00	13.66
D_m	0.80	0.90	0.52	39.00	20.23
D_u	0.80	1.00	0.60	30.00	17.93
E	-0.54	1.00	-0.72	105.00	-75.64

Overall loading

Equiv leeward net force for upper section; $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wu}$ = **-21.6** kN

Net windward force for upper section; $F_w = F_{w,wu}$ = **17.9** kN

Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.88**; as h/L is 1.729

Overall loading upper section; $F_{w,u} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **34.7** kN

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Equiv leeward net force for middle section; $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wm}$ = **-28.1** kN

Net windward force for middle section; $F_w = F_{w,wm}$ = **20.2** kN

Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.88**; as h/L is 1.729

Overall loading middle section; $F_{w,m} = f_{corr} \times (F_w - F_l)$ = **42.4** kN

Equiv leeward net force for bottom section; $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wb}$ = **-25.9** kN

Net windward force for bottom section; $F_w = F_{w,wb}$ = **13.7** kN

Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.88**; as h/L is 1.729

Overall loading bottom section; $F_{w,b} = f_{corr} \times (F_w - F_l)$ = **34.7** 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)	0.52	1.00	0.81	2.16	1.75
G (+ve)	0.42	1.00	0.71	2.16	1.54
H (+ve)	0.32	1.00	0.61	17.31	10.64
I (+ve)	0.22	1.00	0.52	55.53	28.74

Total vertical net force; $F_{w,v}$ = **35.50** 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	0.97	-0.83	19.80	-16.49
B	-0.80	0.97	-0.46	79.20	-36.07
C	-0.50	0.97	-0.17	77.55	-13.37
D_b	0.80	0.74	0.88	36.00	31.63
D_m	0.80	0.90	1.02	39.00	39.71
D_u	0.80	1.00	1.10	30.00	32.91
E	-0.54	1.00	-0.22	105.00	-23.22

Overall loading

Equiv leeward net force for upper section; $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wu}$ = **-6.6** kN

Net windward force for upper section; $F_w = F_{w,wu}$ = **32.9** kN

Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.88**; as h/L is 1.729

Overall loading upper section; $F_{w,u} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **34.7** kN

Equiv leeward net force for middle section; $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wm}$ = **-8.6** kN

Net windward force for middle section; $F_w = F_{w,wm}$ = **39.7** kN

Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.88**; as h/L is 1.729

Overall loading middle section; $F_{w,m} = f_{corr} \times (F_w - F_l)$ = **42.4** kN

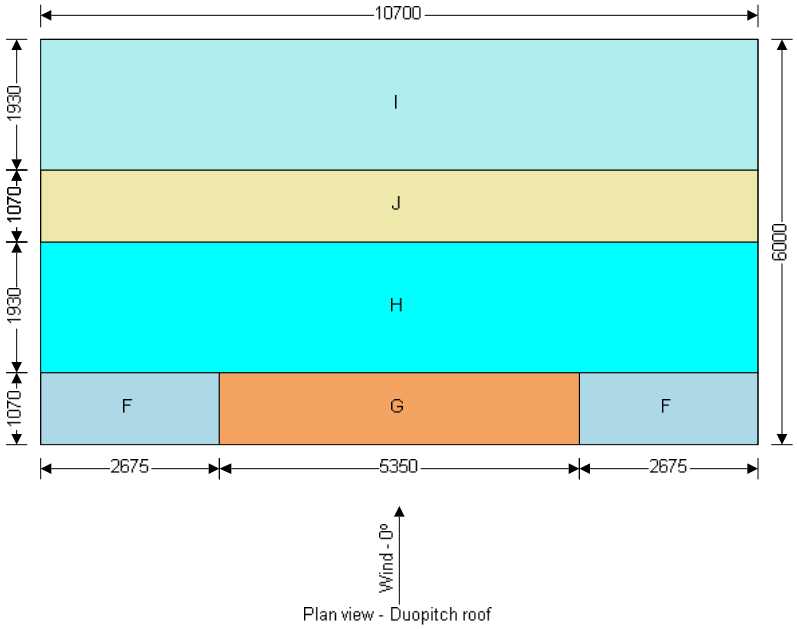
Equiv leeward net force for bottom section; $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wb}$ = **-8.0** kN

Net windward force for bottom section; $F_w = F_{w,wb}$ = **31.6** kN

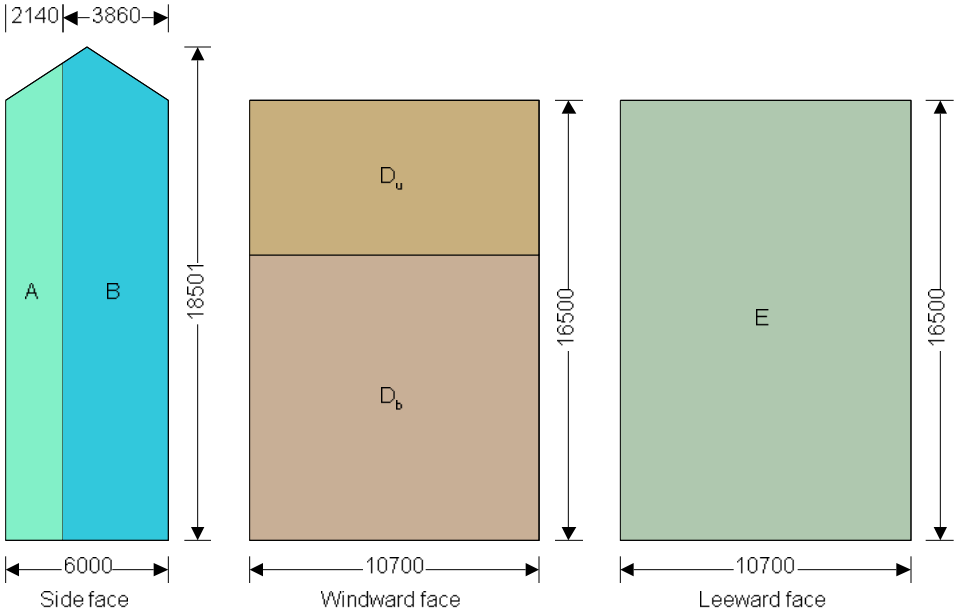
Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.88**; as h/L is 1.729

Overall loading bottom section; $F_{w,b} = f_{corr} \times (F_w - F_l)$ = **34.7** kN

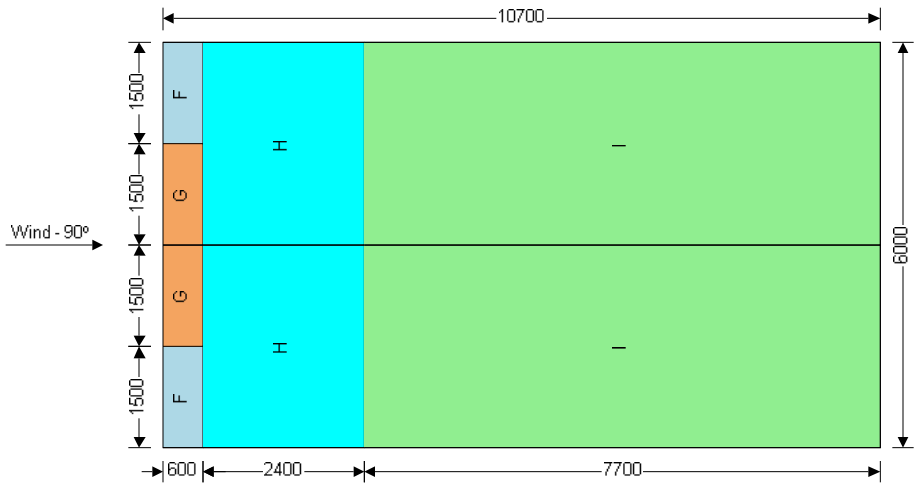
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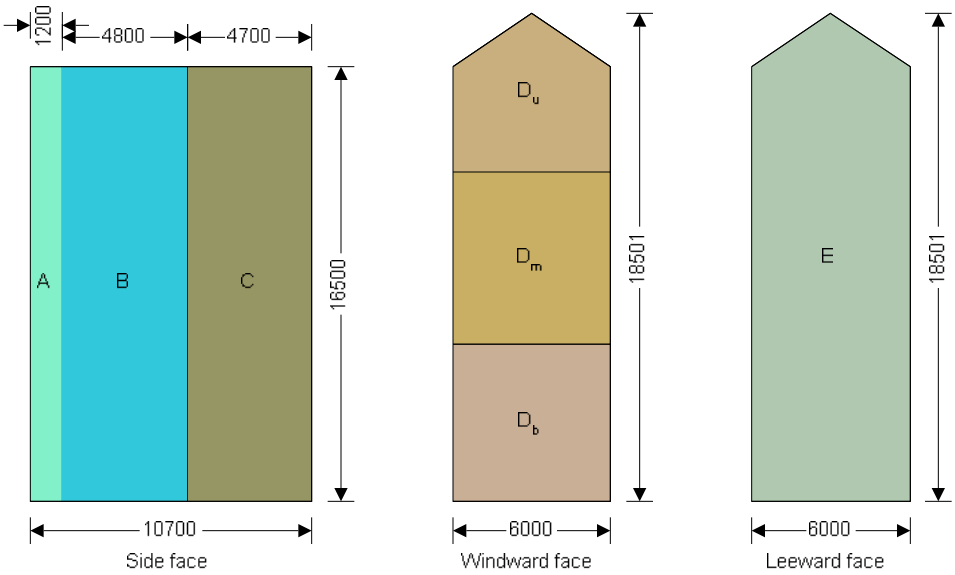
Plan view - Duopitch roof



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Plan view - Duopitch roof

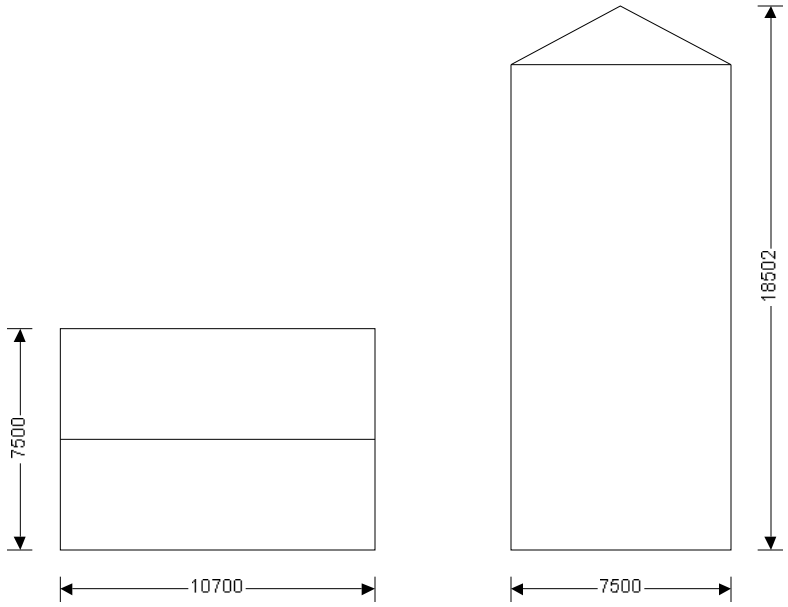


WIND LOADING

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

Tedds calculation version 3.0.30

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Building data

Type of roof;	Duopitch
Length of building;	L = **10700** mm
Width of building;	W = **7500** mm
Height to eaves;	H = **16500** mm
Pitch of roof;	α_0 = **28.1** deg
Total height;	h = **18502** mm

Basic values

Location;	Dewsbury
Wind speed velocity (FigureNA.1);	$v_{b,map}$ = **22.5** m/s
Distance to shore;	L_{shore} = **61.30** km
Altitude above sea level;	A_{alt} = **79.0**m
Altitude factor;	$c_{alt} = A_{alt}/1m \times 0.001 + 1$ = **1.079**
Fundamental basic wind velocity;	$v_{b,0} = v_{b,map} \times c_{alt}$ = **24.3** 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);	$v_b = c_{dir} \times c_{season} \times v_{b,0} \times c_{prob}$ = **24.3** m/s
Reference mean velocity pressure;	$q_b = 0.5 \times \rho \times v_b^2$ = **0.361** kN/m^2

Orography

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

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The velocity pressure for the windward face of the building with a 0 degree wind is to be considered as 2 parts as the height h is greater than b but less than 2b (cl.7.2.2)

The velocity pressure for the windward face of the building with a 90 degree wind is to be considered as 3 parts as the height h is greater than 2b (cl.7.2.2)

Peak velocity pressure - windward wall (lower part) - Wind 0 deg

Reference height (at which q is sought);	z = **10700**mm
Displacement height (sheltering effects excluded);	h_{dis} = **0** mm
Exposure factor (Figure NA.7);	c_e = **2.40**
Peak velocity pressure;	$q_p = c_e \times q_b$ = **0.87** kN/m^2

Structural factor

Structural damping;	δ_s = **0.100**
Height of element;	h_{part} = **10700** mm
Size factor (Table NA.3);	c_s = **0.919**
Dynamic factor (Figure NA.9);	c_d = **1.036**
Structural factor;	$c_{sCd} = c_s \times c_d$ = **0.953**

Peak velocity pressure - windward wall (upper part) - Wind 0 deg and roof

Reference height (at which q is sought);	z = **16500**mm
Displacement height (sheltering effects excluded);	h_{dis} = **0** mm
Exposure factor (Figure NA.7);	c_e = **2.69**
Peak velocity pressure;	$q_p = c_e \times q_b$ = **0.97** kN/m^2

Structural factor

Structural damping;	δ_s = **0.100**
Height of element;	h_{part} = **5800** mm
Size factor (Table NA.3);	c_s = **0.936**
Dynamic factor (Figure NA.9);	c_d = **1.036**
Structural factor;	$c_{sCd} = c_s \times c_d$ = **0.970**

Structural factor

Structural damping;	δ_s = **0.100**
Height of element;	h_{part} = **16500** mm
Size factor (Table NA.3);	c_s = **0.919**
Dynamic factor (Figure NA.9);	c_d = **1.036**
Structural factor;	$c_{sCd} = c_s \times c_d$ = **0.953**

Peak velocity pressure - windward wall (lower part) - Wind 90 deg

Reference height (at which q is sought);	z = **7500**mm
Displacement height (sheltering effects excluded);	h_{dis} = **0** mm
Exposure factor (Figure NA.7);	c_e = **2.18**
Peak velocity pressure;	$q_p = c_e \times q_b$ = **0.79** kN/m^2

Structural factor

Structural damping;	δ_s = **0.100**
Height of element;	h_{part} = **7500** mm
Size factor (Table NA.3);	c_s = **0.929**
Dynamic factor (Figure NA.9);	c_d = **1.046**
Structural factor;	$c_{sCd} = c_s \times c_d$ = **0.972**

Peak velocity pressure - windward wall, (middle part) - Wind 90 deg

Reference height (at which q is sought);	z = **11002**mm
Displacement height (sheltering effects excluded);	h_{dis} = **0** mm

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Exposure factor (Figure NA.7);	c_e = **2.42**
Peak velocity pressure;	$q_p = c_e \times q_b$ = **0.87** kN/m^2

Structural factor

Structural damping;	δ_s = **0.100**
Height of element;	h_{part} = **3502** mm
Size factor (Table NA.3);	c_s = **0.948**
Dynamic factor (Figure NA.9);	c_d = **1.046**
Structural factor;	$c_{sCd} = c_s \times c_d$ = **0.991**

Peak velocity pressure - windward wall (upper part), other walls and roof - Wind 90 deg

Reference height (at which q is sought);	z = **18502**mm
Displacement height (sheltering effects excluded);	h_{dis} = **0** mm
Exposure factor (Figure NA.7);	c_e = **2.76**
Peak velocity pressure;	$q_p = c_e \times q_b$ = **1.00** kN/m^2

Structural factor

Structural damping;	δ_s = **0.100**
Height of element;	h_{part} = **7500** mm
Size factor (Table NA.3);	c_s = **0.941**
Dynamic factor (Figure NA.9);	c_d = **1.046**
Structural factor;	$c_{sCd} = c_s \times c_d$ = **0.985**

Structural factor

Structural damping;	δ_s = **0.100**
Height of element;	h_{part} = **18502** mm
Size factor (Table NA.3);	c_s = **0.923**
Dynamic factor (Figure NA.9);	c_d = **1.046**
Structural factor;	$c_{sCd} = c_s \times c_d$ = **0.965**

Structural factor - roof 0 deg

Structural damping;	δ_s = **0.100**
Height of element;	h_{part} = **18502** mm
Size factor (Table NA.3);	c_s = **0.919**
Dynamic factor (Figure NA.9);	c_d = **1.036**
Structural factor;	$c_{sCd} = c_s \times c_d$ = **0.953**

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.);	$q_{p,i}$ = **1.00** 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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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)	-0.58	1.00	-0.75	6.49	-4.85
G (-ve)	-0.54	1.00	-0.71	6.49	-4.62
H (-ve)	-0.23	1.00	-0.41	32.51	-13.46
I (-ve)	-0.50	1.00	-0.68	32.51	-21.95
J (-ve)	-0.95	1.00	-1.10	12.98	-14.33

Total vertical net force; $F_{w,v}$ = **-52.23** kN

Total horizontal net force; $F_{w,h}$ = **6.29** 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	1.00	-1.34	36.53	-48.99
B	-0.80	1.00	-0.96	94.73	-91.00
D_b	0.80	0.87	0.46	114.49	52.85
D_u	0.80	0.97	0.55	62.06	34.35
E	-0.57	0.97	-0.73	176.55	-128.88

Overall loading

Equiv leeward net force for upper section; $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wu}$ = **-45.3** kN

Net windward force for upper section; $F_w = F_{w,wu}$ = **34.4** kN

Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.91**; as h/W is 2.467

Overall loading upper section; $F_{w,u} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **77.8** kN

Equiv leeward net force for bottom section; $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wb}$ = **-83.6** kN

Net windward force for bottom section; $F_w = F_{w,wb}$ = **52.9** kN

Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.91**; as h/W is 2.467

Overall loading bottom section; $F_{w,b} = f_{corr} \times (F_w - F_l)$ = **123.5** 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)	0.72	1.00	0.99	6.49	6.41
G (+ve)	0.46	1.00	0.74	6.49	4.80
H (+ve)	0.37	1.00	0.66	32.51	21.32
I (+ve)	-0.50	1.00	-0.18	32.51	-5.72
J (+ve)	-0.95	1.00	-0.60	12.98	-7.85

Total vertical net force; $F_{w,v}$ = **16.72** kN

Total horizontal net force; $F_{w,h}$ = **21.71** kN

Walls 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)
A	-1.20	1.00	-0.84	36.53	-30.76
B	-0.80	1.00	-0.46	94.73	-43.71
D_b	0.80	0.87	0.96	114.49	110.01
D_u	0.80	0.97	1.05	62.06	65.34
E	-0.57	0.97	-0.23	176.55	-40.74

Overall loading

Equiv leeward net force for upper section; $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wu}$ = **-14.3** kN

Net windward force for upper section; $F_w = F_{w,wu}$ = **65.3** kN

Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.91**; as h/W is 2.467

Overall loading upper section; $F_{w,u} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **91.7** kN

Equiv leeward net force for bottom section; $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wb}$ = **-26.4** kN

Net windward force for bottom section; $F_w = F_{w,wb}$ = **110.0** kN

Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.91**; as h/W is 2.467

Overall loading bottom section; $F_{w,b} = f_{corr} \times (F_w - F_l)$ = **123.5** 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)	-1.25	1.00	-1.40	3.19	-4.48
G (-ve)	-1.15	1.00	-1.31	3.19	-4.17
H (-ve)	-0.60	1.00	-0.78	25.51	-19.84
I (-ve)	-0.49	1.00	-0.67	59.09	-39.55

Total vertical net force; $F_{w,v}$ = **-60.01** kN

Total horizontal net force; $F_{w,h}$ = **0.00** 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	0.97	-1.32	24.75	-32.77
B	-0.80	0.97	-0.95	99.00	-93.97
C	-0.50	0.97	-0.67	52.80	-35.28
D_b	0.80	0.79	0.41	56.25	23.13
D_m	0.80	0.87	0.49	26.27	12.96
D_u	0.80	1.00	0.59	48.74	28.61
E	-0.54	1.00	-0.72	131.26	-94.06

Overall loading

Equiv leeward net force for upper section; $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wu}$ = **-34.9** kN

Net windward force for upper section; $F_w = F_{w,wu}$ = **28.6** kN

Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.88**; as h/L is 1.729

Overall loading upper section; $F_{w,u} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **55.7** kN

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Equiv leeward net force for middle section; $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wm} =$ **-18.8** kN

Net windward force for middle section; $F_w = F_{w,wm} =$ **13.0** kN

Lack of correlation (cl.7.2.2(3) – Note); $f_{corr} =$ **0.88**; as h/L is 1.729

Overall loading middle section; $F_{w,m} = f_{corr} \times (F_w - F_l) =$ **27.9** kN

Equiv leeward net force for bottom section; $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wb} =$ **-40.3** kN

Net windward force for bottom section; $F_w = F_{w,wb} =$ **23.1** kN

Lack of correlation (cl.7.2.2(3) – Note); $f_{corr} =$ **0.88**; as h/L is 1.729

Overall loading bottom section; $F_{w,b} = f_{corr} \times (F_w - F_l) =$ **55.7** 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)	0.46	1.00	0.74	3.19	2.37
G (+ve)	0.37	1.00	0.66	3.19	2.11
H (+ve)	0.29	1.00	0.58	25.51	14.70
I (+ve)	0.20	1.00	0.49	59.09	29.09

Total vertical net force; $F_{w,v} =$ **42.58** 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	0.97	-0.82	24.75	-20.41
B	-0.80	0.97	-0.45	99.00	-44.54
C	-0.50	0.97	-0.17	52.80	-8.92
D_b	0.80	0.79	0.91	56.25	51.22
D_m	0.80	0.87	0.99	26.27	26.08
D_u	0.80	1.00	1.09	48.74	52.94
E	-0.54	1.00	-0.22	131.26	-28.53

Overall loading

Equiv leeward net force for upper section; $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wu} =$ **-10.6** kN

Net windward force for upper section; $F_w = F_{w,wu} =$ **52.9** kN

Lack of correlation (cl.7.2.2(3) – Note); $f_{corr} =$ **0.88**; as h/L is 1.729

Overall loading upper section; $F_{w,u} = f_{corr} \times (F_w - F_l + F_{w,h}) =$ **55.7** kN

Equiv leeward net force for middle section; $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wm} =$ **-5.7** kN

Net windward force for middle section; $F_w = F_{w,wm} =$ **26.1** kN

Lack of correlation (cl.7.2.2(3) – Note); $f_{corr} =$ **0.88**; as h/L is 1.729

Overall loading middle section; $F_{w,m} = f_{corr} \times (F_w - F_l) =$ **27.9** kN

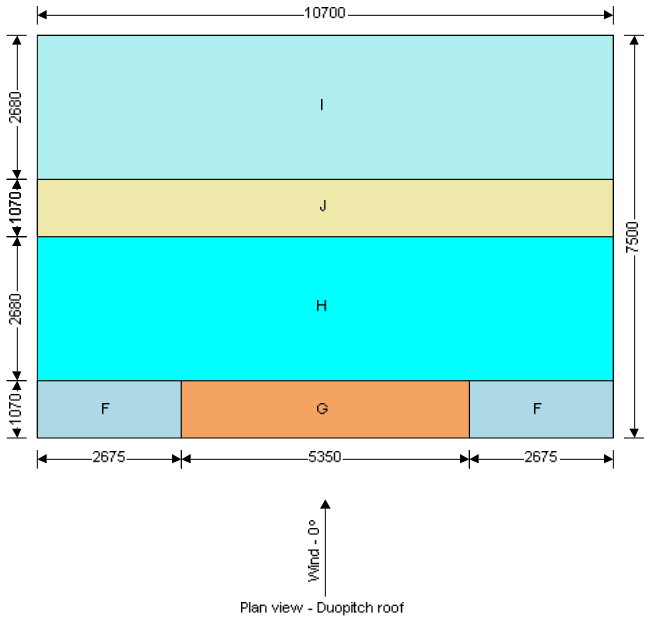
Equiv leeward net force for bottom section; $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wb} =$ **-12.2** kN

Net windward force for bottom section; $F_w = F_{w,wb} =$ **51.2** kN

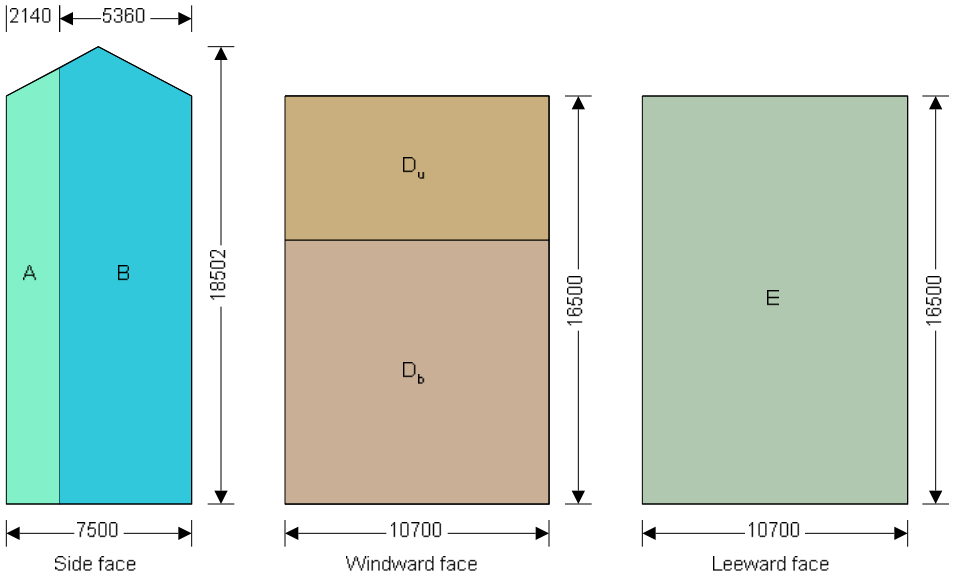
Lack of correlation (cl.7.2.2(3) – Note); $f_{corr} =$ **0.88**; as h/L is 1.729

Overall loading bottom section; $F_{w,b} = f_{corr} \times (F_w - F_l) =$ **55.7** kN

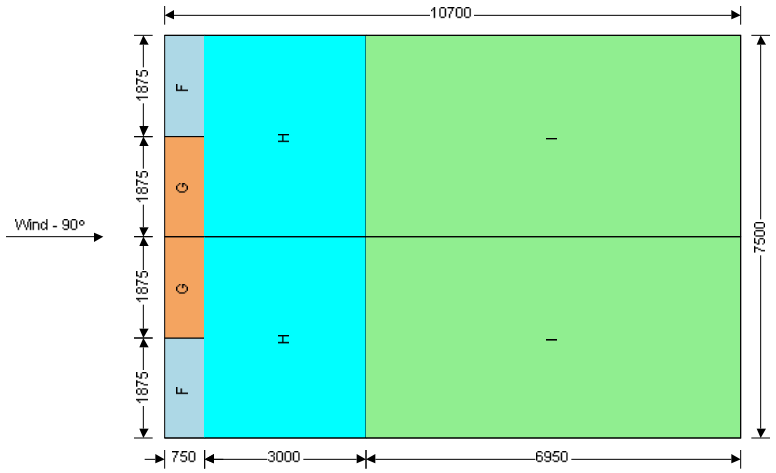
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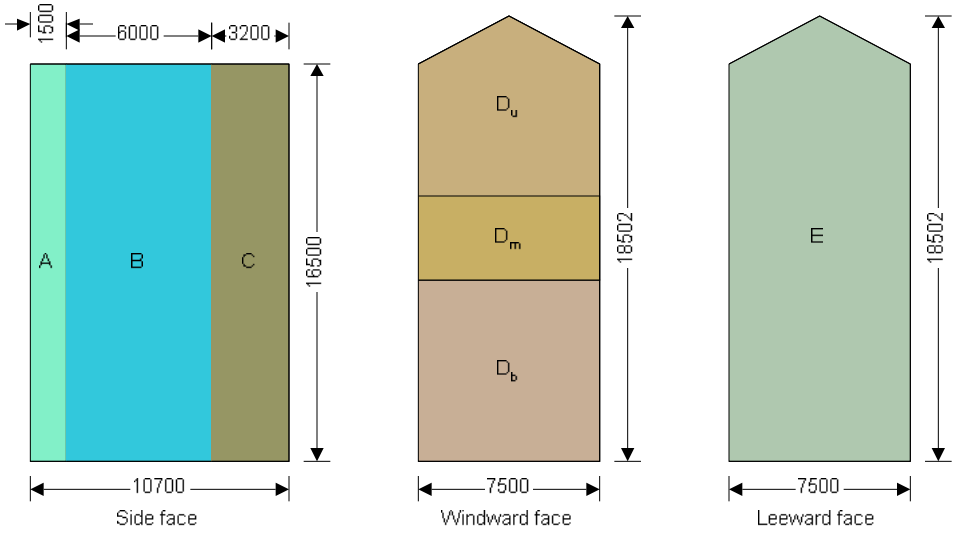
Plan view - Duopitch roof



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Plan view - Duopitch roof



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3.0 JUSTIFICATION OF PANELS FOR GRAVITY LOADINGS

3.1 Proposed PV Loadings

The main gravity loading within this assessment is the PV panel and frame loading which is details below:

Panel Size: 1762mm x 1134mm.
Panel Weight: 24kg
Support Frame: 2kg/m2

⇨ 24kg / (1.762m x 1.134m) + 2kg/m2 = 14kg/m2 or **0.14kN/m2.**

The allowable vertical imposed load is 0.75kN/m2 or 75kg/m2 which is far more than the weight of the panel being placed on the roof.

Once the panel is in situ this area of roof will not be trafficked and so there is no need to consider the actual weight of the panel as being an additional imposed load on the roof. Should anyone stand on the panel it will be destroyed, the owner of the property will therefore take strict steps to ensure that no one at any time stands on the panel. Therefore, this area of roof can be considered as carrying less than the design-imposed load indicated in CP3 CHAPTER V. Therefore, there is no requirement for strengthening as a result of combined imposed load and panel load.

3.2 Gravity Loadings During Installation

Stored Gravity Loadings

The total additional load permitted on the roof structure for the installation of the PV panels is 0.75kN/m2 or 75kg/m2. The maximum design concentrated load is 0.90kN (90kg).

In accordance with CP3 CHAPTER V, non-accessible roofs are subject to maintenance loading provisions comprising either a 0.75kN/m² uniformly distributed load or a 0.90kN concentrated load. These values provide the basis for managing temporary installation activities and material storage.

During installation, plywood sheets or other suitable load spreaders should be used where necessary to distribute stored materials across the roof structure. Where PV panels and associated equipment are temporarily stored on the roof, they shall be dispersed such that the applied loading does not exceed 0.75kN/m² over the storage area. Access to the roof shall not take place during periods of snow.

The roof structure is considered to be generally uniform across the assessed area and no significant localised strengthening or weakening has been identified. Accordingly, the above loading limitations are considered applicable across the roof unless otherwise stated.

Operative Gravity Loadings During Installation

The 0.75kN/m² uniformly distributed load and 0.90kN concentrated load are design load cases and should not be interpreted as limiting the combined weight of operatives carrying out the installation or transporting PV panels across the roof.

During installation, operatives and PV panels impose temporary, moving loads rather than permanent uniformly distributed loads over the roof area. These loads are transient in nature and are transferred to the supporting roof structure as the installation progresses.

Where two operatives are required to carry or position a PV panel, they should remain suitably separated where reasonably practicable and avoid standing simultaneously over the same structural member wherever possible. Materials shall not be stacked within designated access routes, and operatives shall avoid standing on or immediately adjacent to stored materials.

Provided these load management measures are followed, no excessive quantities of materials are stored on the roof,

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and installation is not undertaken during periods of snow or ice, the temporary loading associated with the installation process is considered acceptable.

This guidance assumes that the installation is undertaken in accordance with the installer's method statement and risk assessment, and that good site practice is adopted at all times to minimise temporary load concentrations on the roof structure. The responsibility for the safe execution of the installation, including the management of temporary construction loadings, remains with the installation contractor.

3.3 Snow Load Assessment

When carrying out a roof loading assessment for an installation of PV Panels we need to consider the combined loadings from both the PV Panels and the loadings from snow & snow drift.

With regards snow loading we see that the snow load is 0.45kN/m2. This load will be cumulative to the weight of the panel, frame and ballast loadings which are discussed within Section 3.1 of this report.

Snow Load Calculations

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 1.0.14

Characteristic ground snow load

Location;	Dewsbury
Site altitude above sea level (user modified value);	A = **79** m
Zone number (user modified value);	Z = **4.0**
Density of snow;	γ = **2.00** kN/m^3
Characteristic ground snow load;	s_k = ((0.15 + (0.1 × Z + 0.05)) + ((A - 100m) / 525m)) × 1kN/m^2 = **0.56** kN/m^2
Exposure coefficient (Normal);	C_e = **1.0**
Thermal coefficient;	C_t = **1.0**
Snow fence;	Not present

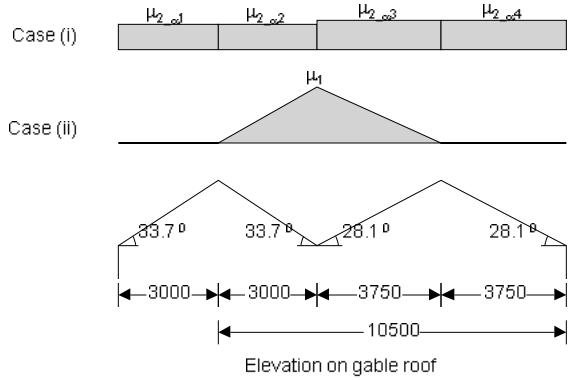
Building details

Roof type;	Multispan
Width of roof 1 (LHS on left building);	b_1 = **3.00** m
Width of roof 2 (RHS on left building);	b_2 = **3.00** m
Width of roof 3 (LHS on right building);	b_3 = **3.75** m
Width of roof 4 (RHS on right building);	b_4 = **3.75** m
Average roof height at valley;	h_{avg} = **2.00** m
Slope of roof 1 (LHS of left building);	α_1 = **33.70** deg
Slope of roof (RHS of left building);	α_2 = **33.70** deg
Slope of roof 3 (LHS of right building);	α_3 = **28.10** deg
Slope of roof 4 (RHS of right building);	α_4 = **28.10** deg

Shape coefficients

Shape coefficient roof(Table 5.2);	$\mu_{2_\alpha1}$ = **0.70**
Shape coefficient roof(Table 5.2);	$\mu_{2_\alpha2}$ = **0.70**
Shape coefficient roof(Table 5.2);	$\mu_{2_\alpha3}$ = **0.80**
Shape coefficient roof(Table 5.2);	$\mu_{2_\alpha4}$ = **0.80**
Shape coefficient roof (Annex B2);	μ_1 = min(2kN/m^3 × h_{avg} / s_k, 2 × (b_2 + b_3 + max(b_1, b_4)) / (b_2 + b_3), 5) = **3.11**

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Shape coef	Coef	Loading (kN/m²)
$\mu_{2_\alpha1}$	0.701	0.39
$\mu_{2_\alpha2}$	0.701	0.39
$\mu_{2_\alpha3}$	0.800	0.45
$\mu_{2_\alpha4}$	0.800	0.45
μ_1	3.111	1.74

Loadcase1 Table 5.2

Loading to roof 1; $s_{1_1} = \mu_{2_\alpha1} \times C_e \times C_t \times s_k =$ **0.39** kN/m²

Loading to roof 2; $s_{2_1} = \mu_{2_\alpha2} \times C_e \times C_t \times s_k =$ **0.39** kN/m²

Loading to roof 3; $s_{3_1} = \mu_{2_\alpha3} \times C_e \times C_t \times s_k =$ **0.45** kN/m²

Loading to roof 4; $s_{4_1} = \mu_{2_\alpha4} \times C_e \times C_t \times s_k =$ **0.45** kN/m²

Loadcase 2 Annex B2

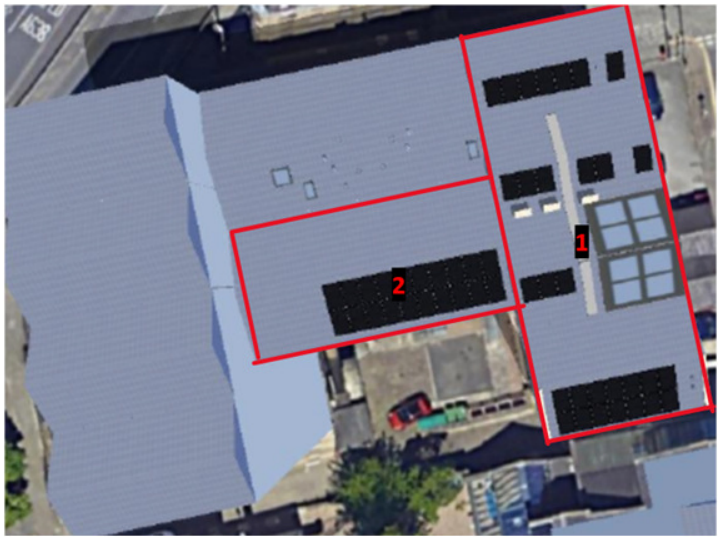
Loading to roof 1; $s_{1_2} = 0 \times s_k =$ **0.00** kN/m²

Loading to avg roof 2 & 3; $s_{2_2} = \mu_1 \times s_k =$ **1.74** kN/m²

Loading to roof 4; $s_{3_2} = 0 \times s_k =$ **0.00** kN/m²

3.4 Snow Drift

The valley area has been identified as a potential snow drift zone, with calculated snow drift loads of 1.74kN/m². This means that any PV panel would not be permitted within this snow drift zone. A structural assessment of the supporting rafters was conducted within this area (see Section 3.6 below). The full snow drift loads in this area have been assessed and found to be within acceptable capacity limits; therefore, snow drift is not considered as a potential issue on this project.



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3.5 Solar PV System Loadings to Resist Wind Uplift

From the calculations we see that the positive wind load is 1.06kN/m2.
(Please see Load Case 2 table and Plan View 0 Degrees above for Zone Areas and Dimensions.)

To calculate the actual wind uplift on the PV Array we refer to BRE Digest 489 (2015).

From our calculations above we know that the negative wind loading, q = 1.00kN/m2 and where a module is less than 0.3m from the roof surface the Wind Uplift Net Pressure Coefficients for the panels in the centre of the roof is -1.50

⇨ -1.00 x -1.50 = -1.50kN/m2 (All roof fixings have to be able to withstand this wind uplift load.)

Given that the panel fixings will transfer the load into the existing roof and the roof was originally designed for the positive & negative wind loadings, no strengthening works will be required to the roof structure.

3.6 Advanced Structure Check Valley Area

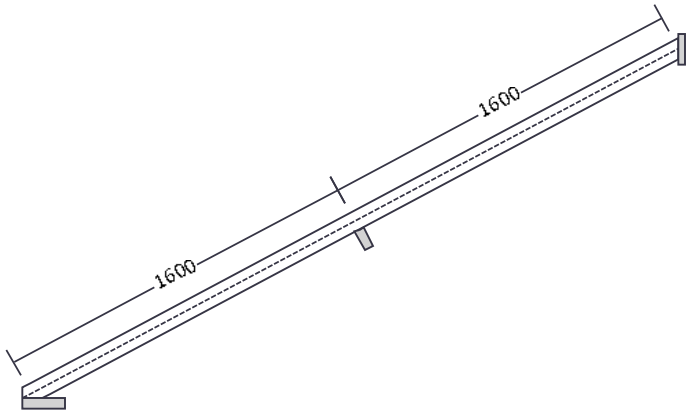
TIMBER RAFTER ANALYSIS & DESIGN (EN1995-1-1:2004)

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the recommended values

Tedds calculation version 1.0.09

Rafter details

Description;	60 x 80 C16+ timber rafters
Rafter spacing;	s_{Rafter} = **350** mm
Rafter inclination;	θ_{Rafter} = **28.1** deg



Forces input on Rafter

Permanent load on slope;	F_{G_Rafter}= **0.69** kN/m^2
Imposed load on plan;	F_{Q_Rafter}= **0.75** kN/m^2
Snow load on plan;	F_{S_Rafter}= **1.74** kN/m^2

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Rafter loading details

Distributed loads

Permanent load on slope; $p_G = F_{G_Rafter} \times s_{Rafter} =$ **0.24** kN/m

Imposed load on slope; $p_Q = F_{Q_Rafter} \times s_{Rafter} \times \cos(\theta_{Rafter}) =$ **0.23** kN/m

Snow load on slope; $p_S = F_{S_Rafter} \times s_{Rafter} \times \cos(\theta_{Rafter}) =$ **0.54** kN/m

Member span 1 results summary	**Unit**	**Capacity**	**Maximum**	**Utilisation**	**Result**
Compressive stress	N/mm^2	12.185	0.218	0.018	PASS
Bearing stress	N/mm^2	1.557	0.138	0.089	PASS
Bending stress	N/mm^2	14.201	3.026	0.213	PASS
Shear stress	N/mm^2	2.369	0.285	0.120	PASS
Bending and axial force				0.213	PASS
Column stability check				0.244	PASS
Deflection	mm	6.4	1.487	0.232	PASS
Member span 2 results summary	**Unit**	**Capacity**	**Maximum**	**Utilisation**	**Result**
Compressive stress	N/mm^2	12.185	0.109	0.009	PASS
Bearing stress	N/mm^2	1.557	0.127	0.082	PASS
Bending stress	N/mm^2	14.201	3.026	0.213	PASS
Shear stress	N/mm^2	2.369	0.285	0.120	PASS
Bending and axial force				0.213	PASS
Column stability check				0.228	PASS
Deflection	mm	6.4	1.487	0.232	PASS

ANALYSIS

Tedds calculation version 1.0.40

Loading

Self weight included (Permanent x 1)

Load combination factors

Load combination	**Permanent**	**Imposed**	**Snow**	**Wind**
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00

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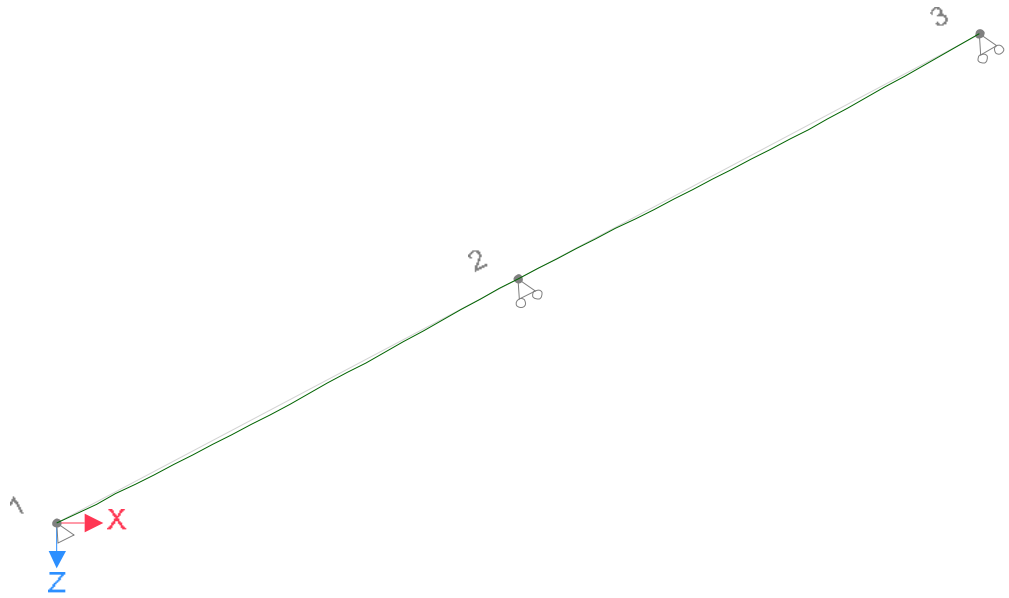
Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.24 kN/m at 0 m to 3.2 m
Member	Imposed	UDL	GlobalZ	0.23 kN/m at 0 m to 3.2 m
Member	Snow	UDL	GlobalZ	0.54 kN/m at 0 m to 3.2 m

Results

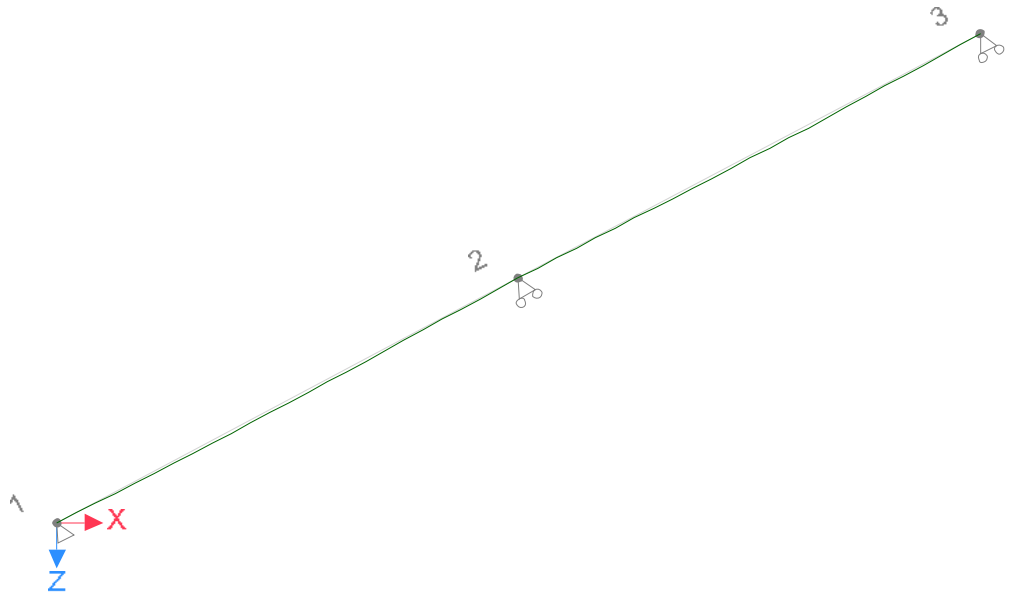
Total deflection

1.35G + 1.50Q (Strength) - Total deflection



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1.00G + 1.00Q (Service) - Total deflection



Node deflections

Load combination: 1.35G + 1.50Q (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.15142	Member
2	0	0	0	Member
3	0	0	-0.15142	Member

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.10655	Member
2	0	0	0	Member
3	0	0	-0.10655	Member

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	2.2

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Load case/combination	Force	
	FX (kN)	FZ (kN)
1.00G + 1.00Q (Service)	0	1.6

Element end forces

Load combination: 1.35G + 1.50Q (Strength)

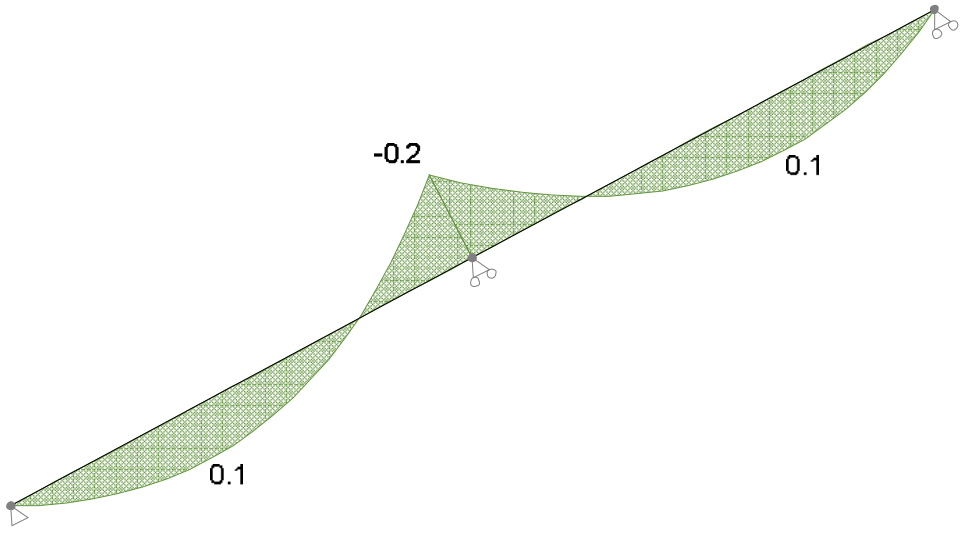
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	1.6	1	-1	-0.4	0
		2	0.5	-0.6	-0.2
2	1.6	2	-0.5	-0.6	0.2
		3	0	-0.4	0

Load combination: 1.00G + 1.00Q (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	1.6	1	-0.7	-0.3	0
		2	0.4	-0.4	-0.1
2	1.6	2	-0.4	-0.4	0.1
		3	0	-0.3	0

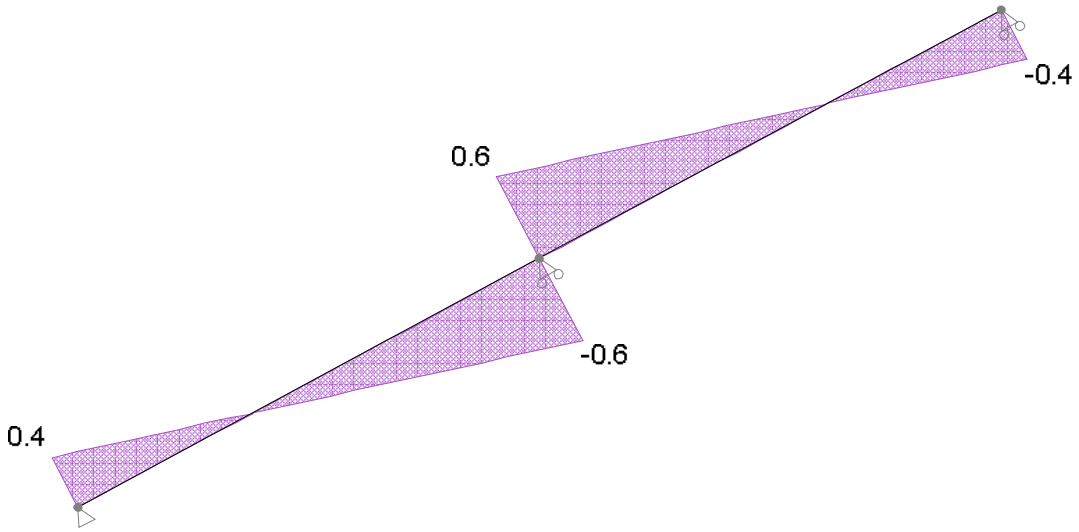
Forces

Strength combinations - Moment envelope (kNm)



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Strength combinations - Shear envelope (kN)



Member results

Envelope - Strength combinations

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Member	0.602	0		0.1 (max)	
	1.6	0.6 (max abs)	-0.6	-0.2 (min)	

;

Tedds calculation version 2.2.28

Member - Span 1

Partial factor for material properties and resistances

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

Member details

Load duration - cl.2.3.1.2; Medium-term

Service class - cl.2.3.1.3; 2

Timber section details

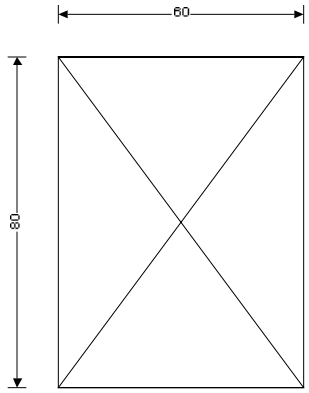
Number of timber sections in member; N = **1**

Breadth of sections; b = **60** mm

Depth of sections; h = **80** mm

Timber strength class - EN 338:2016 Table 1; **C16+**

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60x80 timber section
Cross-sectional area, A, 4800 mm^2
Section modulus, W_y, 64000 mm^3
Section modulus, W_z, 48000 mm^3
Second moment of area, I_y, 2560000 mm^4
Second moment of area, I_z, 1440000 mm^4
Radius of gyration, i_y, 23.1 mm
Radius of gyration, i_z, 17.3 mm
Timber strength class C16+
Characteristic bending strength, $f_{m,k}$, 18.5 N/mm^2
Characteristic shear strength, f_{vk}, 3.5 N/mm^2
Characteristic compression strength parallel to grain, $f_{c,0k}$, 18 N/mm^2
Characteristic compression strength perpendicular to grain, $f_{c,90k}$, 2.3 N/mm^2
Characteristic tension strength parallel to grain, $f_{t,0,k}$, 10.4 N/mm^2
Mean modulus of elasticity, $E_{0,mean}$, 8000 N/mm^2
Fifth percentile modulus of elasticity, $E_{0,05}$, 5400 N/mm^2
Shear modulus of elasticity, G_{mean}, 500 N/mm^2
Characteristic density, ρ_k, 330 kg/m^3
Mean density, ρ_{mean}, 400 kg/m^3

Span details

Bearing length; L_b = **100** mm

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

Modification factors

Duration of load and moisture content - Table 3.1; k_{mod} = **0.8**

Deformation factor - Table 3.2; k_{def} = **0.8**

Depth factor for bending - Major axis - exp.3.1; $k_{h,m,y} = \min((150\ mm / h)^{0.2}, 1.3)$ = **1.134**

Bending stress re-distribution factor - cl.6.1.6(2); k_m = **0.7**

Crack factor for shear resistance - cl.6.1.7(2); k_{cr} = **0.67**

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 = **1.047** kN

Design compressive stress; $\sigma_{c,0,d} = P_d / A$ = **0.218** N/mm^2

Design compressive strength; $f_{c,0,d} = k_{mod} \times k_{sys} \times f_{c.0.k} / \gamma_M$ = **12.185** N/mm^2

$\sigma_{c,0,d} / f_{c,0,d}$ = **0.018**

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

Design perpendicular compression - major axis; $F_{c,y,90,d}$ = **1.075** kN

Effective contact length; $L_{b,ef} = L_b + \min(L_b, 30\ mm)$ = **130** mm

Design perpendicular compressive stress - exp.6.4; $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef})$ = **0.138** N/mm^2

Design perpendicular compressive strength; $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c.90.k} / \gamma_M$ = **1.557** N/mm^2

$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d})$ = **0.089**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force; $F_{y,d}$ = **0.369** kN

Design shear stress - exp.6.60; $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.172** N/mm^2

Design shear strength; $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **2.369** N/mm^2

$\tau_{y,d} / f_{v,y,d}$ = **0.073**

PASS - Design shear strength exceeds design shear stress

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

Effective length for y-axis bending;	$L_{e,y} = 0.9 \times 1600$ mm $=$ **1440** mm
Slenderness ratio;	$\lambda_y = L_{e,y} / i_y =$ **62.354**
Relative slenderness ratio - exp. 6.21;	$\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})} =$ **1.146**
Effective length for z-axis bending;	$L_{e,z} =$ **0** 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;	$k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) =$ **1.241**
	$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) =$ **0.470**
	$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) =$ **0.582**
	$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) =$ **1.064**
Column stability checks - exp.6.23 & 6.24;	$\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) =$ **0.031**
	$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) =$ **0.017**

PASS - Column stability is acceptable

Check design at end of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis;	$F_{c,y,90,d} =$ **1.222** kN
Effective contact length;	$L_{b,ef} = \min(L_b, 30\text{ mm}) + L_b + \min(L_b, 30\text{ mm}) =$ **160** mm
Design perpendicular compressive stress - exp.6.4;	$\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef}) =$ **0.127** N/mm^2
Design perpendicular compressive strength;	$f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c.90.k} / \gamma_M =$ **1.557** N/mm^2
	$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d}) =$ **0.082**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force;	$F_{y,d} =$ **0.611** kN
Design shear stress - exp.6.60;	$\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) =$ **0.285** N/mm^2
Design shear strength;	$f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M =$ **2.369** N/mm^2
	$\tau_{y,d} / f_{v,y,d} =$ **0.120**

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending moment;	$M_{y,d} =$ **0.194** kNm
Design bending stress;	$\sigma_{m,y,d} = M_{y,d} / W_y =$ **3.026** N/mm^2
Design bending strength;	$f_{m,y,d} = k_{h,m,y} \times k_{mod} \times k_{sys} \times f_{m.k} / \gamma_M =$ **14.201** N/mm^2
	$\sigma_{m,y,d} / f_{m,y,d} =$ **0.213**

PASS - Design bending strength exceeds design bending stress

Check combined bending and axial compression - Section 6.2.4

Combined loading checks - exp.6.19 & 6.20;	$(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d} =$ **0.213**
	$(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \times \sigma_{m,y,d} / f_{m,y,d} =$ **0.149**

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 bending;	$L_{e,y} = 0.9 \times 1600$ mm $=$ **1440** mm
Slenderness ratio;	$\lambda_y = L_{e,y} / i_y =$ **62.354**
Relative slenderness ratio - exp. 6.21;	$\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})} =$ **1.146**

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Effective length for z-axis bending;	$L_{e,z}$ = **0** 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;	β_c = **0.2**
Instability factors - exp.6.25, 6.26, 6.27 & 6.28;	$k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2)$ = **1.241**
	$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2)$ = **0.470**
	$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)})$ = **0.582**
	$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)})$ = **1.064**
Column stability checks - exp.6.23 & 6.24;	$\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d}$ = **0.244**
	$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) + k_m \times \sigma_{m,y,d} / f_{m,y,d}$ = **0.166**
	PASS - Column stability is acceptable

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

Check design 684 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection;	δ_y = **0.8** mm
Quasi-permanent variable load factor;	ψ_2 = **0.3**
Final deflection with creep;	$\delta_{y,Final} = \delta_y \times (1 + k_{def})$ = **1.5** mm
Allowable deflection;	$\delta_{y,Allowable} = L_{m\,1_s1} / 250$ = **6.4** mm
	$\delta_{y,Final} / \delta_{y,Allowable}$ = **0.232**
	PASS - Allowable deflection exceeds final deflection

Member - Span 2

Partial factor for material properties and resistances

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

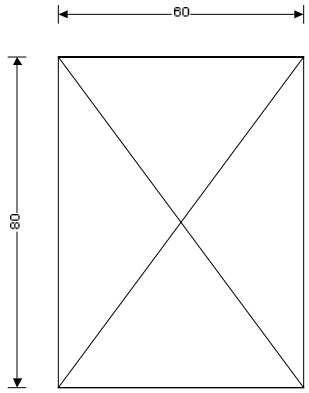
Member details

Load duration - cl.2.3.1.2;	Medium-term
Service class - cl.2.3.1.3;	2

Timber section details

Number of timber sections in member;	N = **1**
Breadth of sections;	b = **60** mm
Depth of sections;	h = **80** mm
Timber strength class - EN 338:2016 Table 1;	**C16+**

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60x80 timber section
Cross-sectional area, A, 4800 mm^2
Section modulus, W_y, 64000 mm^3
Section modulus, W_z, 48000 mm^3
Second moment of area, I_y, 2560000 mm^4
Second moment of area, I_z, 1440000 mm^4
Radius of gyration, i_y, 23.1 mm
Radius of gyration, i_z, 17.3 mm
Timber strength class C16+
Characteristic bending strength, $f_{m,k}$, 18.5 N/mm^2
Characteristic shear strength, f_{vk}, 3.5 N/mm^2
Characteristic compression strength parallel to grain, $f_{c,0k}$, 18 N/mm^2
Characteristic compression strength perpendicular to grain, $f_{c,90k}$, 2.3 N/mm^2
Characteristic tension strength parallel to grain, $f_{t,0,k}$, 10.4 N/mm^2
Mean modulus of elasticity, $E_{0,mean}$, 8000 N/mm^2
Fifth percentile modulus of elasticity, $E_{0,05}$, 5400 N/mm^2
Shear modulus of elasticity, G_{mean}, 500 N/mm^2
Characteristic density, ρ_k, 330 kg/m^3
Mean density, ρ_{mean}, 400 kg/m^3

Span details

Bearing length; L_b = **100** mm

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

Modification factors

Duration of load and moisture content - Table 3.1; k_{mod} = **0.8**

Deformation factor - Table 3.2; k_{def} = **0.8**

Depth factor for bending - Major axis - exp.3.1; $k_{h,m,y} = \min((150\ mm / h)^{0.2}, 1.3)$ = **1.134**

Bending stress re-distribution factor - cl.6.1.6(2); k_m = **0.7**

Crack factor for shear resistance - cl.6.1.7(2); k_{cr} = **0.67**

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 = **0.523** kN

Design compressive stress; $\sigma_{c,0,d} = P_d / A$ = **0.109** N/mm^2

Design compressive strength; $f_{c,0,d} = k_{mod} \times k_{sys} \times f_{c.0.k} / \gamma_M$ = **12.185** N/mm^2

$\sigma_{c,0,d} / f_{c,0,d}$ = **0.009**

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

Design perpendicular compression - major axis; $F_{c,y,90,d}$ = **1.222** kN

Effective contact length; $L_{b,ef} = \min(L_b, 30\ mm) + L_b + \min(L_b, 30\ mm)$ = **160** mm

Design perpendicular compressive stress - exp.6.4; $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef})$ = **0.127** N/mm^2

Design perpendicular compressive strength; $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c.90.k} / \gamma_M$ = **1.557** N/mm^2

$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d})$ = **0.082**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force; $F_{y,d}$ = **0.611** kN

Design shear stress - exp.6.60; $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.285** N/mm^2

Design shear strength; $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **2.369** N/mm^2

$\tau_{y,d} / f_{v,y,d}$ = **0.120**

PASS - Design shear strength exceeds design shear stress

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Check bending moment - Section 6.1.6

Design bending moment; $M_{y,d}$ = **0.194** kNm

Design bending stress; $\sigma_{m,y,d} = M_{y,d} / W_y$ = **3.026** N/mm^2

Design bending strength; $f_{m,y,d} = k_{h,m,y} \times k_{mod} \times k_{sys} \times f_{m,k} / \gamma_M$ = **14.201** N/mm^2

$\sigma_{m,y,d} / f_{m,y,d}$ = **0.213**

PASS - Design bending strength exceeds design bending stress

Check combined bending and axial compression - Section 6.2.4

Combined loading checks - exp.6.19 & 6.20; $(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d}$ = **0.213**

$(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \times \sigma_{m,y,d} / f_{m,y,d}$ = **0.149**

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 bending; $L_{e,y} = 0.9 \times 1600$ mm = **1440** mm

Slenderness ratio; $\lambda_y = L_{e,y} / i_y$ = **62.354**

Relative slenderness ratio - exp. 6.21; $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c.0.k} / E_{0.05})}$ = **1.146**

Effective length for z-axis bending; $L_{e,z}$ = **0** 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; β_c = **0.2**

Instability factors - exp.6.25, 6.26, 6.27 & 6.28; $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2)$ = **1.241**

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

$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)})$ = **0.582**

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

Column stability checks - exp.6.23 & 6.24; $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d}$ = **0.228**

$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) + k_m \times \sigma_{m,y,d} / f_{m,y,d}$ = **0.158**

PASS - Column stability is acceptable

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

Check design 916 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection; δ_y = **0.8** mm

Quasi-permanent variable load factor; ψ_2 = **0.3**

Final deflection with creep; $\delta_{y,Final} = \delta_y \times (1 + k_{def})$ = **1.5** mm

Allowable deflection; $\delta_{y,Allowable} = L_{m\,1_s2} / 250$ = **6.4** mm

$\delta_{y,Final} / \delta_{y,Allowable}$ = **0.232**

PASS - Allowable deflection exceeds final deflection

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4.0 CONCLUSIONS

PV System Loadings

From the calculations, we see that the combined system (0.14kN/m2) + snow load (0.45kN/m2) = (0.59kN/m2) is less than or equal to the design-imposed load of 0.75kN/m2.

We are able to confirm these loadings to be acceptable, therefore the proposed solar panels can safely be installed onto the existing roof structure with no strengthening works being required.

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Appendix A Photographs



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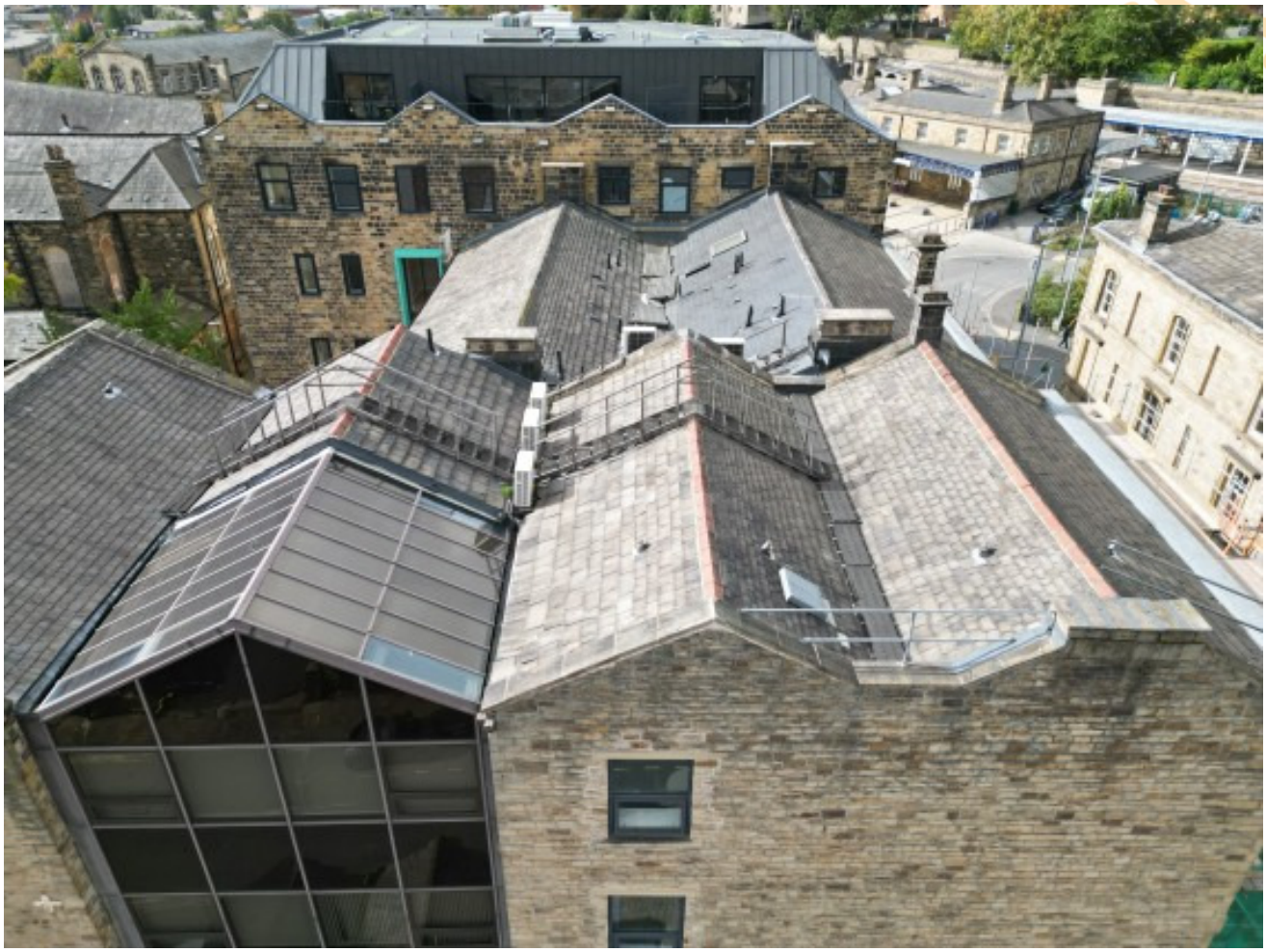
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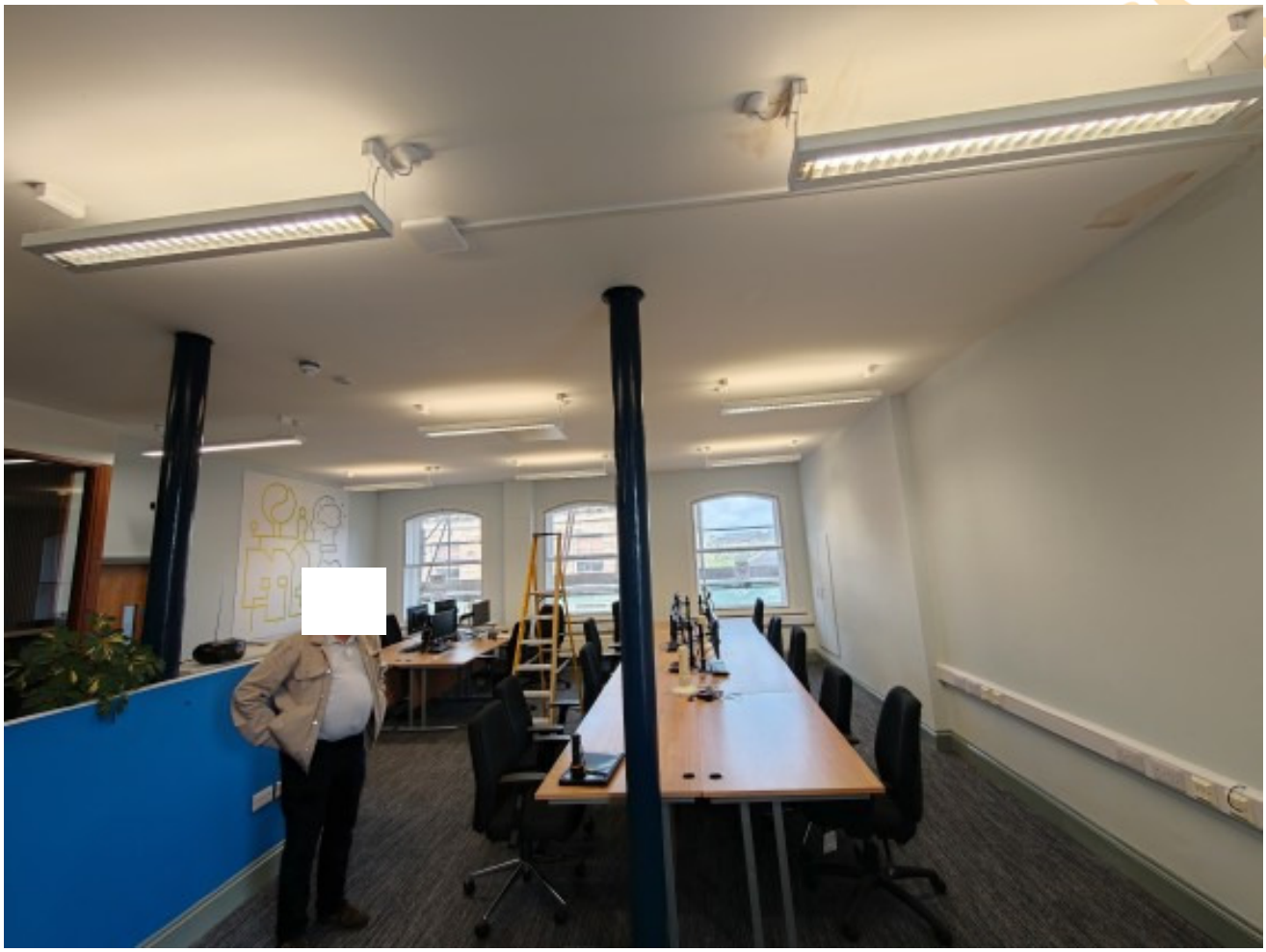
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