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STRUCTURAL CALCULATIONS

Lutterworth Town Council Building

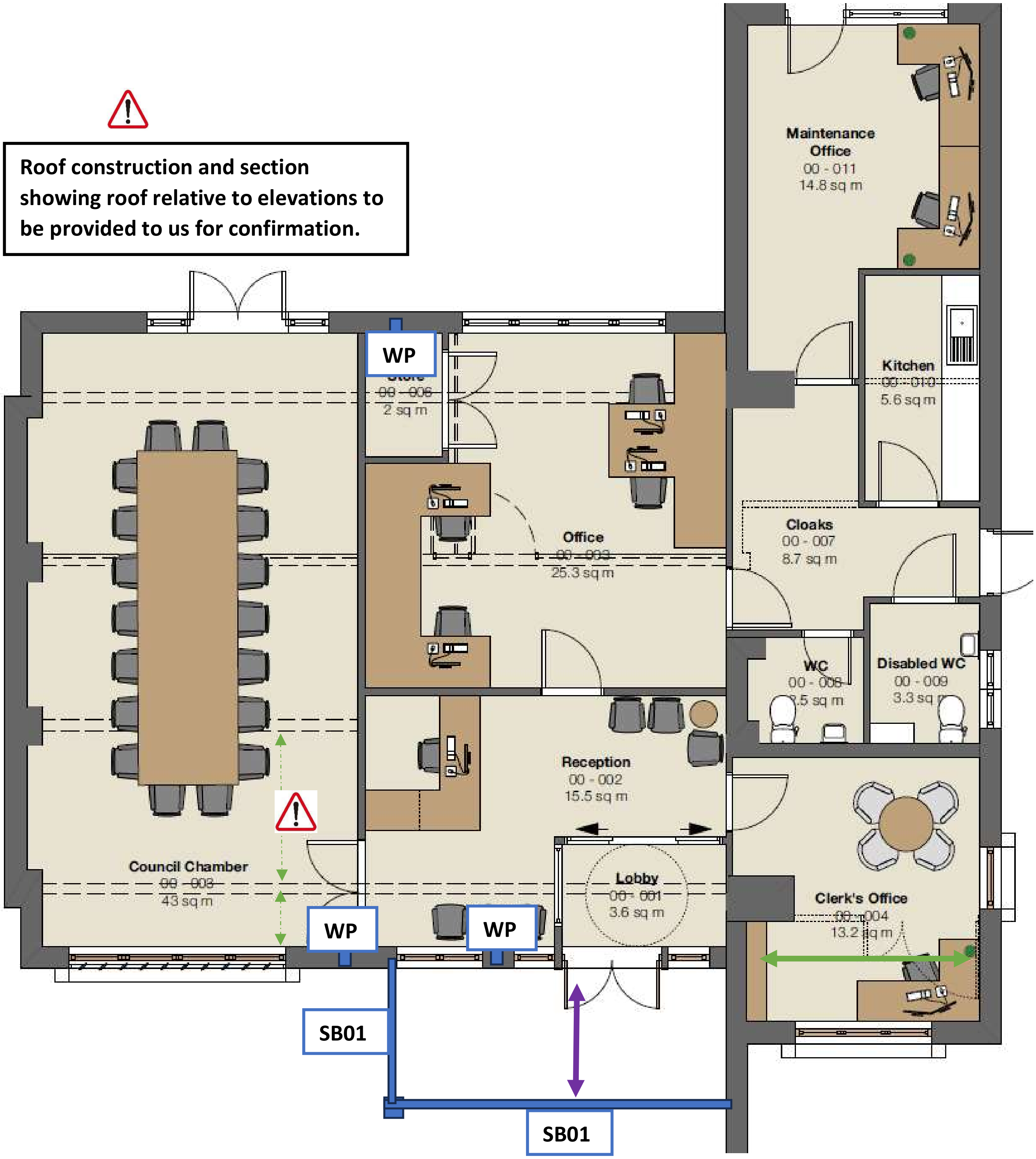
for

Lutterworth Town Council

Reference: **230646-TAA-CAL-001**

Revision History				
Version	**Description**	**By**	**Chk'd**	**Date**
A01	First issue	MF	MET	22.09.2023
A02	Second issue: Correction of existing foundation layout	MF	-	03.11.2023

Summary



Ground floor plan showing roof structure over

Timber Flat roof joists supporting new roof

Timber Flat roof joists supporting green roof over canopy. Assumed span = 2m

	Size	Bearing/connection details
Flat roof joists	47 x 175 C16 rafters @ 400mm c/c	Proprietary joist hangers and wall plates to support a safe working load of 1.1KN. For fixing into front elevation use M10 Rawlbolt shield anchors, staggered @ 225mm horizontal centres. Install to manufacturer's instructions.
SB01	178 x 102 x 19 UB Min.Grade S275 JR	An 8mm thk plate may be welded to the bottom flange to support any cladding. Beam-Beam connection: Weld a 10mm thk end plate to the web of the smaller SB01 using 6mm CFW and bolt into web of longer SB01 using 4No M16 8.8 bolts. Connection to SC01: Weld a 10mm thk capping plate to the top of SC01 using 6mm CFW and bolt bottom flange of SB01 to capping plate using 4No M12 8.8 bolts. See sheet 23. At wall ends: Bear 100mm onto a 215mm long x 100mm wide x 215mm deep 50N concrete padstone and bolt bottom flange of SB01 into concrete padstone using 2No M12 5.8 resin anchors, min. embedment of 150mm.
SC01	80 x 80 x 5 SHS Grade S355 JRH	10mm thk, 200mm x 200mm base plate to be welded to the bottom of SC01 using 6mm CFW and bolted into new pad foundation using 4No M16 5.6 resin anchors, min. embedment of 150mm.
External lintels	Standard duty Catnic lintels or equivalent to match thermal requirements and cavity width	Min. 150mm bearings at either end. Install to manufacturer's details.
Wind post, WP	100 x 50 x 5 RHS Grade S355 with the post the same width as the blockwork.	Bottom of wind post to be welded to a 10mm thk, 200mm long x 100mm wide base plate using 6mm CFW and bolted into the new strip footings using 2No M16 5.6 resin anchors. Fix into joist wall plate and strap back to at least 3 noggins.

Extension super structure Specifaction
All new external openings to have Ancon AMR/S/D3.0/W60 bed joint reinforcement provided in the first and second courses above and below all openings and should extend no less than 600mm either side of the opening unless otherwise specified. The North oriented front panel to consist of AMR/S/D3.0/W60 bed joint reinforcement in all courses of block and brick above openings. **<u>Position of existing flat roof joists relative to masonry panels to be confirmed back to us.</u>** Wind post to support front elevation and should be suitably strapped to new masonry @ max.225mm vertical centres. Ensure Joint between blockwork and wind post is to be tightly packed with mortar. External blockwork above DPC to consist of 3.6N blockwork in M4 mortar. External masonry below DPC to consist of 7.3N blockwork in M6 sulphate resistant mortar with blockwork below ground confirmed for use by the manufacturer.

Foundation Specification (See sheet 32)

The formation below all foundations is to comprise natural ground having a safe net bearing capacity **of at least 100 kN/m^2.** This is to be confirmed on site by Building Control, with Engineer to be informed in good time if any doubts arise so that the design of the foundations may be reassessed.

All new external footings to be min.600mm wide.

Pad footing under SC01 to be 600mm x 600mm x min. 300mm thk

Building control to advise on site if additional mesh reinforcement is required ie. Soft spots

Depth to bottom of all footings, to building control approval with all foundations to be founded to the same depth and bearing strata – subject to confirmation from Building Control.
Provide 50mm Cordek Claymaster board or similar approved to the internal face of trench fill foundations as a provision for heave protection.

Suitability of ground bearing concrete slab to building control approval. If more than 600mm of fill is required or the ground is susceptible to heave due to nearby trees, a suspended floor should be specified. This should be confirmed with Building Control. Should it be found ground bearing slabs are not suitable, TKA are to be consulted for further guidance.

NOTE:- All new unreinforced concrete foundations to be min.gen 3 grade concrete.

Contents

Notes

1. These calculations have been based on, and should be read in conjunction with the following drawings sent by Tim Adams Architecture:

- 143-TAA-XX-ZZ-DR-A-0200-P01-Existing Floor Plans
- 143-TAA-XX-ZZ-DR-A-0400-P01-Existing Elevations
- 143-TAA-XX-ZZ-DR-A-1100-P01-Proposed Site Plan
- 143-TAA-XX-ZZ-DR-A-1200-P01-Proposed Floor Plans
- 143-TAA-XX-ZZ-DR-A-1400-P01-Proposed Front Elevation
- 143-TAA-XX-ZZ-DR-A-1401-P01-Proposed Rear Elevation
- 143-TAA-XX-ZZ-DR-A-1402-P01-Proposed Side Elevation
- 143-TAA-XX-ZZ-DR-A-1403-P01-Proposed Side Elevation

If these are not the latest drawings, we should be informed.

2. All dimensions shown within these calculations are for design purposes only and are to be checked on site before ordering materials.

3. All details are subject to approval by all concerned parties which should be obtained before construction commences.

4. Any assumptions made by the designer have been highlighted within these calculations. These should be checked before ordering materials. If any findings differ from these assumptions, we are to be informed so that our design can be re-assessed.

5. In case of any doubts regarding these calculations please feel free to contact us.

Scope of Works and design philosophy

We have been asked to provide structural engineering services to allow for modifications to an existing single storey structure to accommodate an extended floor plate to the rear and front with an additional green roof canopy.

The extent of our design input is noted below:

- Design of:
  - Flat roof joists for extension and green roof canopy
  - Canopy eaves beams
  - Canopy support post
  - New foundations
- Specification of:
  - Beam bearings and padstones
  - Block strengths
  - Lintels
- Engineer's sketches to show structural requirements

Building control is to advice on the suitability of a ground bearing slab, however our calculations will assume beam and block floor will be required.

We have not been provided with a geotechnical report , therefore assumptions regarding ground conditions are taken from BS 8002. In the case of the new foundations we have assumed the bearing strata consists of a moderately stiff clay of a minimum safe net bearing capacity of 100KN/m^2 which is to be checked and confirmed by building control on site , with engineer to be informed in good time if doubts arise. Building control is also to decide the required depths to bottom of all footings considering the ground conditions and presence of any trees.

Overview: The current structure, which TKA has not visited, is a load bearing masonry structure with a flat roof which is anticipated to be supported by joists spanning front to back and intermediated supported by several bulkhead beams spanning side to side across load bearing walls. The existing front elevation consists of several timber mullion members supporting a glazed facade. It is anticipated that the global lateral stability is provided by the bulkhead beams which transfer wind loadings into suitable shear walls. In plane wind loadings may be resisted by a portal frame, perhaps steel encased in concrete. The proposed structure will also rely on the existing global stability of the structure transferring any out of plane wind loadings from the masonry panels into the existing flat roof diaphragm. We have not been provided with a section showing the level of the existing diaphragm, therefore assumptions have been made when checking the masonry panels.

At the front a new canopy will be checked for the the combined saturated loading from an extensive green roof system, along with an uplift load case assuming a lightweight finish.

Proposed vertical loads will be transmitted to the ground via the foundations and masonry above.

The building is classed as a class 2A structure therefore effective ties need to be considered, in particular the canopy which is exposed externally.

Where proposed materials are unknown to us, permanent loadings have been approximated with BS-EN 1991-1-1:2002. In the case of the green roof loadings the loadings have been approximated from the tabel shown in appendix C.

All assumptions are to be checked before ordering materials. If any discrepancies are found, we are to be contacted so that we can re-assess our design.

The proposed calculations have been undertaken using the following Approved Documents, Eurocodes and National annexes.

BS 8004 – Code of practice for foundations
BS 8103 – Structural design of low-rise buildings
BS EN 1990 – Basis of structural design
BS EN 1991 – Actions on structures
BS EN 1993 – Design of steel structures
BS EN 1995 – Design of timber structures
BS EN 1996 – Design of Masonry structures

Designer's Risk Assessment

Checklist of potential operations and hazards		
Potential hazards	**Present**	**Key significant hazards to be addressed**
Client operations		
Adjacent activities		
Restricted site		
Traffic		
Interface with public		
Near to highways		
Near to railways		
Near to waterways		
Tidal working		
Ground instability		Unknow - No GI provided
Contamination		
Soil gas		
Ground water		
Vegetation		
Inundation		
Sewage		
Fuel tanks		
Services		
Overhead cables	X	BT cables overhead
Demolition		
Unstable structures		
Explosives		
Asbestos	X	Potential hazard given the age of the building
Bird droppings		
Dust		
Hazardous materials		
Radiation		
Hot working		
Confined spaces		
Working at height		
Manual handling	X	
Lifting operations	X	
Vibration		
Noise		
Other (state)		

Hazard	Action by Designer	Residual Hazard
Unknown ground conditions	To eliminate: Cannot be eliminated	
	To reduce: A net safe bearing capacity of 100KN/m² has been assumed which is considered an acceptable assumption for most clay strata.	Risk remains, all ground conditions are to be assessed on site by building control before any foundation works commence.

Structural Design

General Loadings

SECTION		OUTPUT
	Loadings - in accordance with BS EN 1991-1-1:2002 and NA	

Roof - flat

Dead		g_k	OUTPUT
Coverings & Serv	18mm plywood and finishes	0.38	
Joists etc.	47x175mm C16 timbers @ 400mm c/c	0.10	g_k =
Ceiling/insulation	12mm plasterboard and skim	0.18	0.66 kN/m^2
Variable		q_k	q_k =
Snow or roof (Cl. 3.3.2 (1))	Max. of snow or imposed load (Category H)	0.60	0.60 kN/m^2

Roof - flat, green canopy

Dead		g_k	OUTPUT
Coverings & Serv	High-end Extensive green roof., saturated	1.75	
Joists etc.	47x175mm C16 timbers @ 400mm c/c	0.10	g_k =
Ceiling/insulation	12mm plasterboard and skim	0.18	2.03 kN/m^2
Variable		q_k	q_k =
Snow or roof (Cl. 3.3.2 (1))	Max. of snow or imposed load (Category H)	0.60	0.60 kN/m^2

Walls

Dead		g_k
215 Solid brick		4.73
100 brick		2.20
100 block		1.50
Timber		0.40
Glazing		0.40
Plaster		0.18

NOTE: The section sizes above are to calculate approx. loadings only. Refer to architect's details for actual section sizes

*Note: We have not been provided with a green roof specification, therefore the maximum saturated weight of the green room system has been approximated through manufacturer's upper bounds for an extensive green roof (appropriate for green roof canopies). The weight of the additional water within the soil is taken as a dead load however may be combined with snow loadings during low temperatures.

Wind Loadings

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

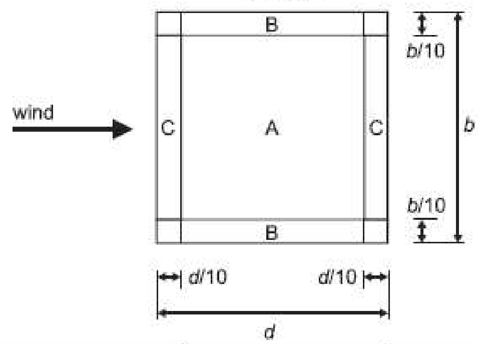
Tedds calculation version 3.0.28

Basic values	
Location;	Lutterworth
Wind speed velocity (FigureNA.1);	$v_{b,map}$ = **21.7** m/s
Distance to shore;	L_{shore} = **100.00** km
Altitude above sea level;	A_{alt} = **126.0**m
Altitude factor;	$c_{alt} = A_{alt}/1m \times 0.001 + 1$ = **1.126**
Fundamental basic wind velocity;	$v_{b,0} = v_{b,map} \times c_{alt}$ = **24.4** 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);	$v_b = c_{dir} \times c_{season} \times v_{b,0} \times c_{prob}$ = **24.4** m/s
Reference mean velocity pressure;	$q_b = 0.5 \times \rho \times v_b^2$ = **0.366** kN/m^2
Orography	
Orography factor not significant;	c_o = 1.0
Terrain category;	Country
Displacement height (sheltering effect excluded);	h_{dis} = 0mm
Reference height (at which q is sought);	z = **3700**mm
Displacement height (sheltering effects excluded);	h_{dis} = **0** mm
Exposure factor (Figure NA.7);	c_e = **1.73**
Peak velocity pressure;	$q_p = c_e \times q_b$ = **0.63** kN/m^2

Table 7.6 — $c_{p,net}$ and c_f values for monopitch canopies

Net Pressure coefficients $c_{p,net}$

Key plan



Roof angle α	Blockage φ	Overall Force Coefficients c_f	Zone A	Zone B	Zone C
0°	Maximum all φ	+ 0,2	+ 0,5	+ 1,8	+ 1,1
	Minimum $\varphi = 0$	- 0,5	- 0,6	- 1,3	- 1,4
	Minimum $\varphi = 1$	- 1,3	- 1,5	- 1,8	- 2,2

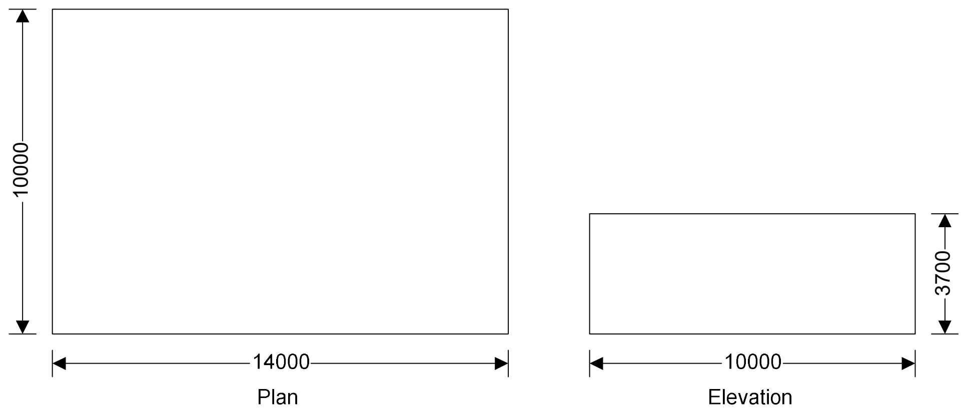
Wind acting on the canopy will create an uplift on the roof due to the blockage from the front elevation.

Uplift pressure = -1.3 x 0.63KN/m² = -0.82KN/m²

Lightest green roof dead load(not saturated) approximately taken as same dead load as a standard flat roof. Therefore max. net uplift = 0.82-0.66 = 0.16KN/m²

Connections required to resist uplift but design of joists and beams critical for combined snow and saturated green roof loadings.

Total potential uplift load acting on 6x2 canopy = 0.16KN/m² x 6m x 2m = 2KN



Building data

Type of roof;	Flat
Length of building;	L = **14000** mm
Width of building;	W = **10000** mm
Height to eaves;	H = **3700** mm
Eaves type;	Sharp
Total height;	h = **3700** mm

The velocity pressure for the windward face of the building with a 0 degree wind is to be considered as 1 part as the height h is less than b (cl.7.2.2)

The velocity pressure for the windward face of the building with a 90 degree wind is to be considered as 1 part as the height h is less than b (cl.7.2.2)

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

Structural factor

Structural damping;	δ_s = **0.100**
Height of element;	h_{part} = **3700** mm
Size factor (Table NA.3);	c_s = **0.917**
Dynamic factor (Figure NA.9);	c_d = **1.003**
Structural factor;	$c_sc_d = c_s \times c_d$ = **0.920**

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

Reference height (at which q is sought);	z = **3700**mm
Displacement height (sheltering effects excluded);	h_{dis} = **0** mm
Exposure factor (Figure NA.7);	c_e = **1.73**
Peak velocity pressure;	$q_p = c_e \times q_b$ = **0.63** kN/m^2

Structural factor

Structural damping;	δ_s = **0.100**
Height of element;	h_{part} = **3700** mm
Size factor (Table NA.3);	c_s = **0.929**
Dynamic factor (Figure NA.9);	c_d = **1.009**
Structural factor;	$c_sc_d = c_s \times c_d$ = **0.937**

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.);	$q_{p,i}$ = **0.63** kN/m^2

Pressures and forces

Net pressure;	$p = c_sc_d \times q_p \times c_{pe} - q_{p,i} \times c_{pi}$;
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)	-2.00	0.63	-1.30	2.74	-3.55
G (-ve)	-1.40	0.63	-0.94	7.62	-7.20
H (-ve)	-0.70	0.63	-0.54	41.44	-22.20
I (-ve)	-0.20	0.63	-0.24	88.20	-21.50

Total vertical net force; $F_{w,v}$ = **-54.45** kN

Total horizontal net force; $F_{w,h}$ = **0.00** 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	0.63	-0.83	5.48	-4.53
B	-0.80	0.63	-0.59	21.90	-13.02
C	-0.50	0.63	-0.42	9.62	-4.03
D	0.72	0.63	0.29	51.80	15.08
E	-0.33	0.63	-0.32	51.80	-16.62

Overall loading

Equiv leeward net force for overall section; $F_l = F_{w,wE}$ = **-16.6** kN

Net windward force for overall section; $F_w = F_{w,wD}$ = **15.1** kN

Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.85**; as h/W is 0.370

Overall loading overall section; $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **27.0** 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)	-2.00	0.63	-0.98	2.74	-2.68
G (+ve)	-1.40	0.63	-0.63	7.62	-4.78
H (+ve)	-0.70	0.63	-0.22	41.44	-9.05
I (+ve)	0.20	0.63	0.31	88.20	27.10

Total vertical net force; $F_{w,v}$ = **10.60** kN

Total horizontal net force; $F_{w,h}$ = **0.00** 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	0.63	-0.51	5.48	-2.79
B	-0.80	0.63	-0.28	21.90	-6.06
C	-0.50	0.63	-0.10	9.62	-0.98
D	0.72	0.63	0.61	51.80	31.53
E	-0.33	0.63	0.00	51.80	-0.18

Overall loading

Equiv leeward net force for overall section; $F_l = F_{w,wE}$ = **-0.2** kN

Net windward force for overall section; $F_w = F_{w,wD}$ = **31.5** kN

Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.85**; as h/W is 0.370

Overall loading overall section; $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **27.0** 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)	-2.00	0.63	-1.32	2.74	-3.60
G (-ve)	-1.40	0.63	-0.96	4.66	-4.47
H (-ve)	-0.70	0.63	-0.54	29.60	-16.08
I (-ve)	-0.20	0.63	-0.25	103.00	-25.33

Total vertical net force; $F_{w,v}$ = **-49.50** 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	0.63	-0.84	5.48	-4.60
B	-0.80	0.63	-0.60	21.90	-13.20
C	-0.50	0.63	-0.42	24.42	-10.36
D	0.70	0.63	0.29	37.00	10.75
E	-0.30	0.63	-0.31	37.00	-11.38

Overall loading

Equiv leeward net force for overall section; $F_l = F_{w,wE}$ = **-11.4** kN

Net windward force for overall section; $F_w = F_{w,wD}$ = **10.7** kN

Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.85**; as h/L is 0.264

Overall loading overall section; $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **18.8** 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)	-2.00	0.63	-1.00	2.74	-2.74
G (+ve)	-1.40	0.63	-0.64	4.66	-2.99
H (+ve)	-0.70	0.63	-0.23	29.60	-6.69
I (+ve)	0.20	0.63	0.31	103.00	31.87

Total vertical net force; $F_{w,v}$ = **19.46** 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	0.63	-0.52	5.48	-2.87
B	-0.80	0.63	-0.29	21.90	-6.25
C	-0.50	0.63	-0.11	24.42	-2.61
D	0.70	0.63	0.61	37.00	22.50
E	-0.30	0.63	0.01	37.00	0.36

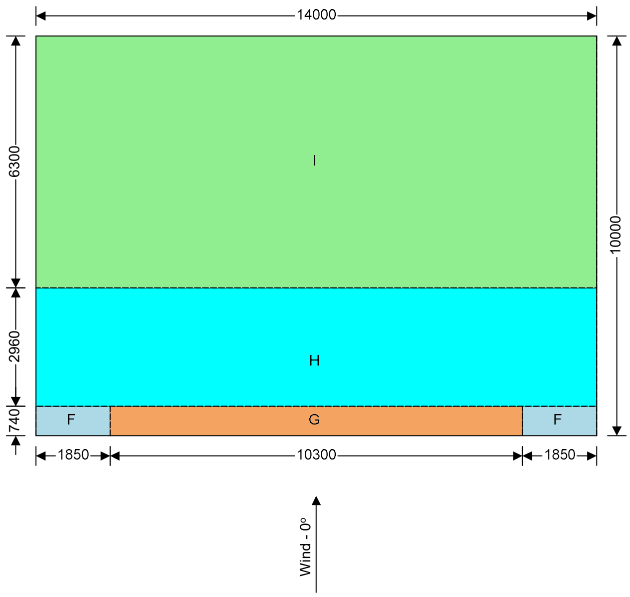
Overall loading

Equiv leeward net force for overall section; $F_l = F_{w,wE}$ = **0.4** kN

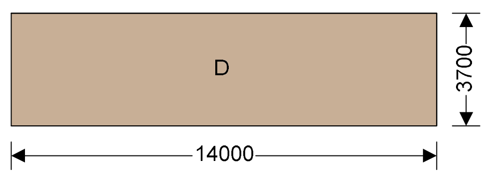
Net windward force for overall section; $F_w = F_{w,wD}$ = **22.5** kN

Lack of correlation (cl.7.2.2(3) – Note); f_{corr} = **0.85**; as h/L is 0.264

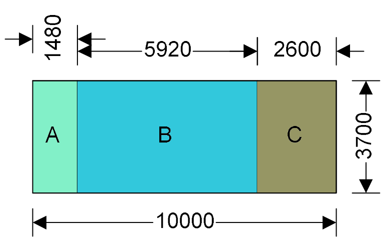
Overall loading overall section; $F_{w,D} = f_{corr} \times (F_w - F_I + F_{w,h})$ = **18.8** kN



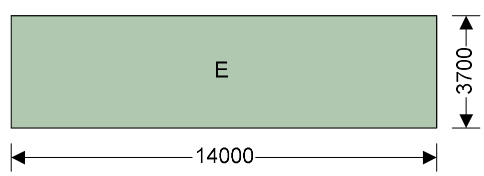
Plan view - Flat roof



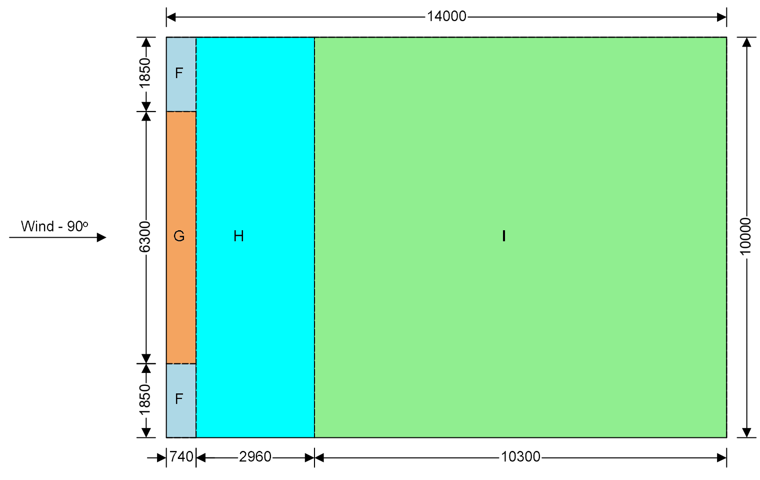
Windward face



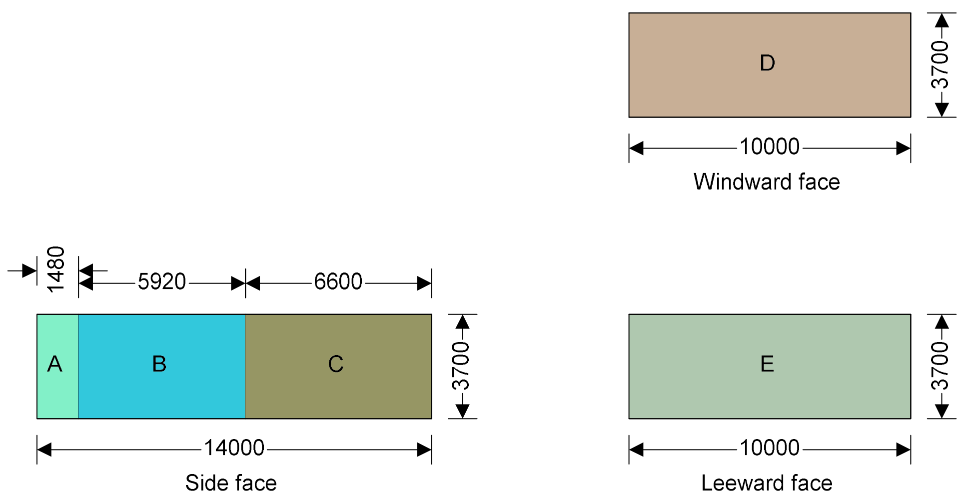
Side face



Leeward face



Plan view - Flat roof



Critical North facing front panel will require checking for out of plane wind loadings

Max pressure on North facing panel =
MAX [0.61KN/m^2 x 0.78(N-dir factor) , av(0.6|0.42) x 0.74(E-dir factor)] = 0.48KN/m^2

Sorth facing front panel will require checking for out of plane wind loadings

Max pressure on South facing panel =
MAX [0.61KN/m^2 x 0.85(S-dir factor) , av(0.6|0.42) x 0.74(E-dir factor)] = 0.52KN/m^2

Timber analysis and design

Timber flat roof joists

Flat roof joists span across the new extended roof and across the canopy. Design joists to equivalent sizes across the site.

Try 400mm c/c for joists.

Check joists spanning 3.4m over proposed Clerk's office:

SECTION		OUTPUT
	<u>Timber beam design - in accordance with BS EN 1995-1-1 and NA</u>	
	L = 3.4 m γg = 1.35 γq = 1.50	

	Dead	Vari.	LW		G_k	Q_k
Roof				m		
Flat roof	0.66	0.6	0.4	m	0.90	0.82
Floor				m		
Walls				m		
				kN	0.90	0.82

Tables NA1, NA2 & NA3

Load-duration class	Service class	γ_m
Short-term	2	1.30

BS EN 338 & Equation 3.1

Timber properties					
b	47 mm	h	175 mm	Grade	C16
$f_{m,k}$	16 N/mm^2	k_h	1.00	W_{yy}	239.9 x10^{3}mm^3
$f_{v,k}$	3.2 N/mm^2				
$E_{0,mean}$	8 x10^3 N/mm^2	A	8.23 x10^3 mm^2	I_{yy}	21.0 x10^6 mm^4

SECTION		OUTPUT
Table 3.1	k_{mod} = 0.9	
	Design bending strength fm.d = 0.9 x 1.00 x 16 / 1.30 =	11.1 N/mm^2
	Ultimate moment of resistance M_{ult} = 11.1 x 239.9 x 10^3 =	2.66 kNm
	Ultimate load = 1.35 x 0.90 + 1.50 x 0.82 =	2.44 kN
	M = 2.44 x 3.4 / 8 =	1.04 kNm
	M is less then M_{ult}	OK
	Instantaneous deflections:	
	$w_{inst,G} = \dfrac{5 \times 0.90 \times 10^3 \times 3400^3}{384 \times 8 \times 10^3 \times 21 \times 10^6} + \dfrac{19.2 \times 0.38 \times 10^6}{8 \times 10^3 \times 8.23 \times 10^3}$	2.85 mm
	$w_{inst,Q} = \dfrac{5 \times 0.82 \times 10^3 \times 3400^3}{384 \times 8 \times 10^3 \times 21 \times 10^6} + \dfrac{19.2 \times 0.35 \times 10^6}{8 \times 10^3 \times 8.23 \times 10^3}$	2.59 mm
	Total	**5.4 mm**
EC0 Table NA A1.1 & Cl. 2.2.6; Table 3.2	ψ_2 = 0.3 k_{def} = 0.8	
	$w_{fin,G}$ = 2.85 x (1 + 0.8)	5.12 mm
	$w_{fin,Q}$ = 2.59 x (1 + 0.3 x 0.8)	3.21 mm
	Total	**8.3 mm**
Table NA4	Deflection limit = span/250 = 13.6 mm 8.3 < 13.6	OK
	Bearings: kcr = 0.67	
	V = 2.44 / 2	1.22 kN
	Design shear strength $f_{v,d}$ = 0.9 x 3.2 / 1.30	2.22 N/mm^2
	Shear capacity = 2.22 x 31.5 x 175 / 1.5 / 1000	8.14 kN
	V is less than shear capacity	OK

Check joists spanning 2m over proposed canopy, support a green roof(External, therefore service class 3):

SECTION		OUTPUT
	Timber beam design - in accordance with BS EN 1995-1-1 and NA	

L = 2.0 m γg = 1.35 γq = 1.50

	Dead	Vari.	LW		G_k	Q_k
Roof				m		
Flat roof	2.03	0.6	0.4	m	1.62	0.48
Floor				m		
Walls				m		
				kN	1.62	0.48

Tables NA1, NA2 & NA3

Load-duration class	Service class	γ_m
Medium-term	3	1.30

BS EN 338 & Equation 3.1

Timber properties

b	47 mm	h	175 mm	Grade	C16		
$f_{m,k}$	16 N/mm^2	k_h	1.00	W_{yy}	239.9 x10^{3}mm^3	$f_{v,k}$	3.2 N/mm^2
$E_{0,mean}$	8 x10^3 N/mm^2	A	8.23 x10^3 mm^2	I_{yy}	21.0 x10^6 mm^4		

Table 3.1 k_{mod} = 0.65

Design bending strength fm.d = 0.65 x 1.00 x 16 / 1.30 = — **8 N/mm^2**

Ultimate moment of resistance M_{ult} = 8 x 239.9 x 10^3 = — **1.92 kNm**

Ultimate load = 1.35 x 1.62 + 1.50 x 0.48 = — **2.91 kN**

M = 2.91 x 2 / 8 = — **0.73 kNm**

M is less then M_{ult} — **OK**

Instantaneous deflections:

$$w_{inst,G} = \frac{5 \times 1.62 \times 10^3 \times 2000^3}{384 \times 8 \times 10^3 \times 21 \times 10^6} + \frac{19.2 \times 0.41 \times 10^6}{8 \times 10^3 \times 8.23 \times 10^3}$$ — **1.13 mm**

$$w_{inst,Q} = \frac{5 \times 0.48 \times 10^3 \times 2000^3}{384 \times 8 \times 10^3 \times 21 \times 10^6} + \frac{19.2 \times 0.12 \times 10^6}{8 \times 10^3 \times 8.23 \times 10^3}$$ — **0.33 mm**

Total — **1.5 mm**

ECO Table NA A1.1 & Cl. 2.2.6; Table 3.2

ψ_2 = 0.3 k_{def} = 3

$w_{fin,G}$ = 1.13 x (1 + 3) — **4.5 mm**

$w_{fin,Q}$ = 0.33 x (1 + 0.3 x 3) — **0.63 mm**

Total — **5.1 mm**

Table NA4 Deflection limit = span/250 = 8 mm 5.1 < 8.0 — **OK**

Bearings: kcr = 0.67

V = 2.91 / 2 — **1.46 kN**

Design shear strength $f_{v,d}$ = 0.65 x 3.2 / 1.30 — **1.6 N/mm^2**

Shear capacity = 1.6 x 31.5 x 175 / 1.5 / 1000 — **5.88 kN**

V is less than shear capacity — **OK**

Line working load from canopy on wall plate = [1.62KN + 0.48KN / 2] / 0.4m = 2.63KN/m

Fix wall plate into brickwork @ 225mm horizontal centres staggered.

Therefore load per fixing into face of wall = 2.63KN/m x 0.225 = 0.6KN

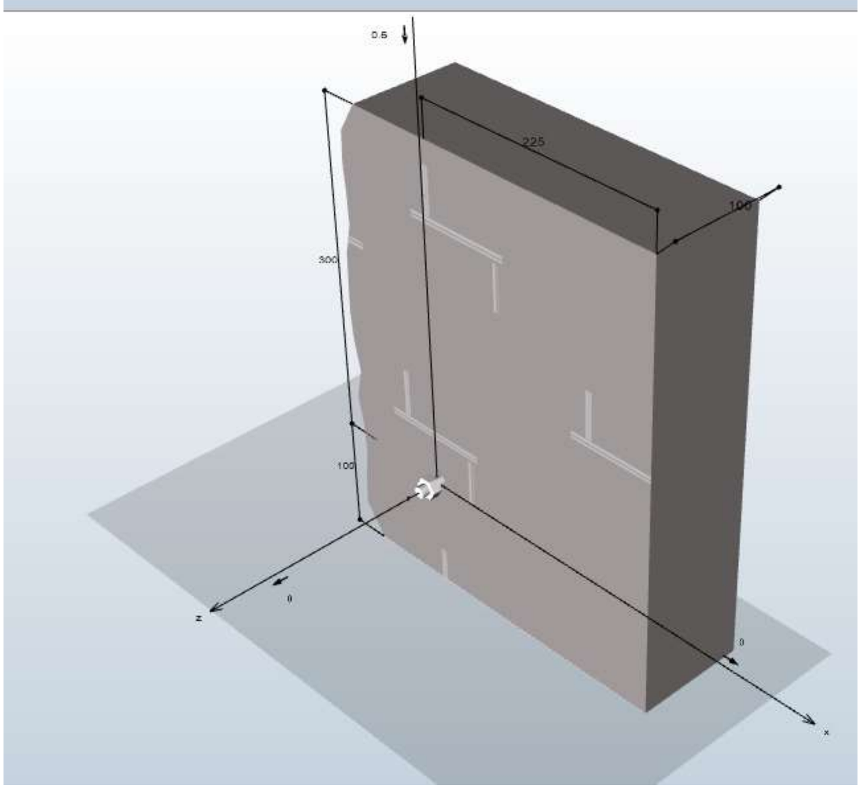
Specification Data

RAWLBOLT® Shield Anchor Loose Bolt Performance Data

SIZE	CONCRETE, $f_{ck,cube}$ = 30N/mm² (C20/25)									Brickwork = 20.5 N/mm²
	CHARACTERISTIC RESISTANCE (kN)		DESIGN RESISTANCE (Factored) *(kN)*		RECOMMENDED LOAD (Unfactored) (kN)		CHARACTERISTIC EDGE DISTANCE (mm)		CHARACTERISTIC SPACING (mm)	RECOMMENDED LOAD (Unfactored) (kN)
	TENSION (N_{Rk})	SHEAR (V_{Rk})	TENSION (N_{Rd})	SHEAR (V_{Rd})	TENSION (N_{rec})	SHEAR (V_{rec})	TENSION ($C_{cr,N}$)	SHEAR ($C_{cr,V}$)	TENSION & SHEAR ($S_{cr,N}$) ($S_{cr,V}$)	TENSION & SHEAR (N_{rec}) (V_{rec})
M6	9.6	8.2	4.5	4.5	3.8	3.8	80	100	120	1.8
M8	12.1	12.8	5.6	7.1	4.7	5.9	100	120	150	2.3
M10	16.7	20.9	7.7	11.6	6.4	9.7	120	160	180	2.9
M12	24.6	30.5	11.4	16.9	9.5	14.1	160	180	250	4.3
M16	57.4	55.3	26.6	30.7	22.2	25.6	190	260	290	Bolts above M12 are not recommended in brickwork.
M20	79.4	88.1	36.8	48.9	30.7	40.8	250	300	330	When calculating loads in brickwork, apply the published edge distance and spacing for concrete and assume these figures to be the absolute minimums. Concrete reduction factors must NOT be applied.
M24	99.0	122.8	45.8	68.2	38.2	56.8	280	350	420	

Try a M10 rawl fixing into the centre of a block unit, min. provided shear capacity of rawlbolt = 7.7KN

Load Type	N	Vx	Vy	Mx	My	Mz			
Design Load	0 kN	0 kN	-0.6 kN	0 kNm	0 kNm	0 kNm	0.0 %	37.5 %	0.0 %



Shear		
Steel failure without lever arm		
Utilisation	βV,s	5.00 %
Considered anchors		1
VRk,s	kN	15.00
γMsV	-	1.25
VRd,s	kN	12.00
V^h_{Sd}	kN	0.60
Local brick failure single anchor		
Utilisation	βV,bh	37.50 %
Considered anchors		1
VRk,b	kN	4.00
αj,V	-	1.00
γMm	-	2.50
VRd,b	kN	1.60
V^h_{Sd}	kN	0.60
Brick edge failure single anchor		
Utilisation	βV,c,h	7.16 %
Considered anchors		1
VRk,c,∥	kN	20.95
VRk,c,⊥	kN	11.64
αj,V	-	1.00
γMm	-	2.50
VRd,c,∥	kN	8.38
VRd,c,⊥	kN	4.66
VSd,c,∥	kN	0.60
VSd,c,⊥	kN	0.00

Steel analysis and design

Steel canopy beam: SB01

Design length of beam = 5.2m

Assume beam is tortionally unrestrained throughout length of beam.

Lw of canopy roof = 2m/2 = 1m

SECTION		OUTPUT
SB01	Steel beam design - in accordance with BS EN 1993	
	L = 5.2 m γg = 1.35 γq = 1.50	

	Dead	Vari.	LW		G_k	Q_k
Roof				m		
Flat roof	2.03	0.6	1	m	10.56	3.12
Floor				m		
Walls				m		
Suspension				m		
SW	0.2	kN/m		kN	11.6	3.1

SECTION		OUTPUT
		F = 20.3 kN
	M = 20.3 x 5.2 / 8 = 13.2 kNm Beam type	
SCI P360 Ch. 3.3	Effective length k = 1.2L+2h = 6.6 m UB	
	C_1 = 1.13	
Tata 'Blue Book' & Table 7 BS EN 10025-2	178 x 102x 19 UB	
	$M_{b,Rd}$ 17.2 kNm M is less than $M_{b,Rd}$	OK
	A_v 9.9 cm^2 t_w 4.8 mm f_y 275 N/mm^2 I_y 1360 cm^4	
	E 210000 N/mm^2	
	V_{Ed} = 20.3 / 2 = 10.17 kN	
	$V_{pl,Rd}$ = 0.577 x 275 x 9.9 x 10^2 = 156936 N	= 157 kN
	V_{Ed} is less than $V_{pl,Rd}$	OK
	$wG_k = \dfrac{5 \times 11.6 \times 10^3 \times 5200^3}{384 \times 210 \times 10^3 \times 1360 \times 10^4}$	7.4 mm
	$wQ_k = \dfrac{5 \times 3.1 \times 10^3 \times 5200^3}{384 \times 210 \times 10^3 \times 1360 \times 10^4}$	2.0 mm
	Total deflection	9.4 mm
Cl. NA.2.23	Deflection limit = span/360 = 14.4 mm (Live loading only) 2.0 < 14.4	OK
	Therefore provide S275 178 x 102x 19 UB	

Steel canopy column: SC01

Design length of column = 2.5m

Column is partially restrained in position by incoming beams and unrestrained against rotation, therefore Leff = 1.5H = 1.5 X 2.5m = 3.8m

Factored vertical load acting on column = 10.2KN

Factored uplift = 1.5 x 2KN/4 = 0.75KN

Tying load, T_p = 0.4(gk + ψ qk) s L or 75 kN, whichever is the greater. In the event of a accidental failure of the masonry, the beam-column connection should be designed for 75KN of tensile loading.

Vertical load from beam will be supported via a capping plate ,

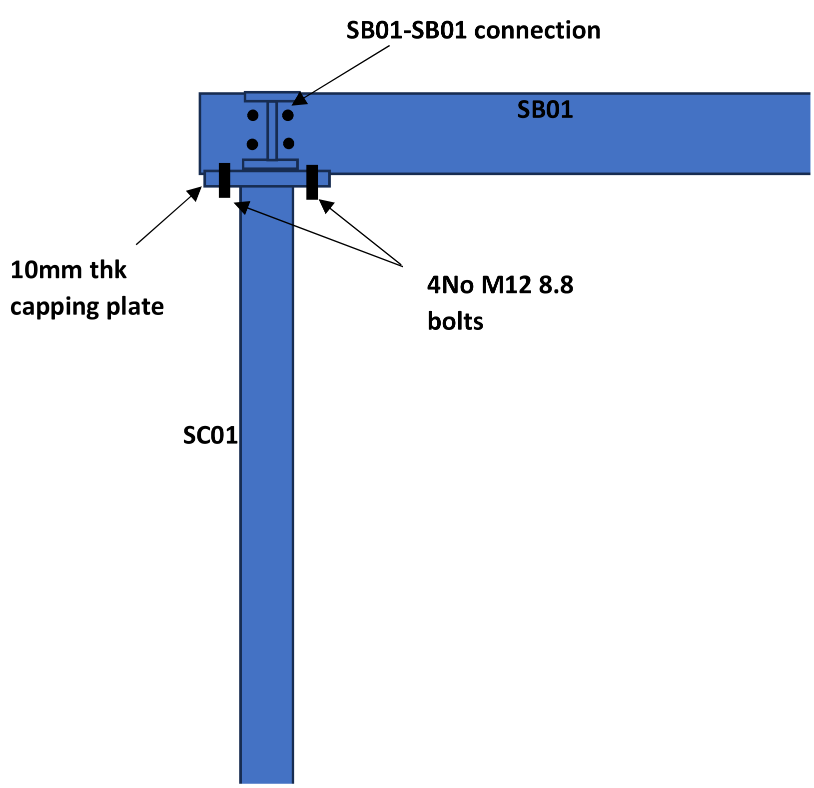
Check stability of 80 x 80 x 5 SHS column :

Eccentricity taken as 0.1 + d/2 = 0.1 + 0.08/2 = 0.14m

Therefore, eccentric moment applied to column = 10.2KN x 0.14m = 1.5KNm

10.2KN / 128KN + 1.5KNm / 14.1KNm = 0.2 < 1 , therefore ok.

<u>Beam – column connection design:</u>



Check shear and tensile capacity of bolts: Tensile resistance> Shear resistance of 4No M16 8.8 bolts = 4 x 60.3KN = 241KN > 75KN

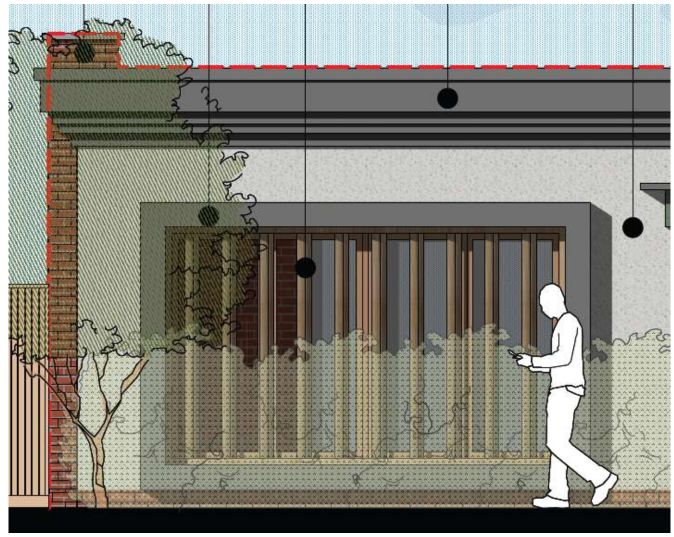
Lintel Analysis & Design

External wall lintels

Max clear span = 3.1m , Design span = 1.1 x 3.1m =3.4m

Critical lintel supports a small amount of flat roof loading and cavity wall over

No arching of the masonry over the lintel

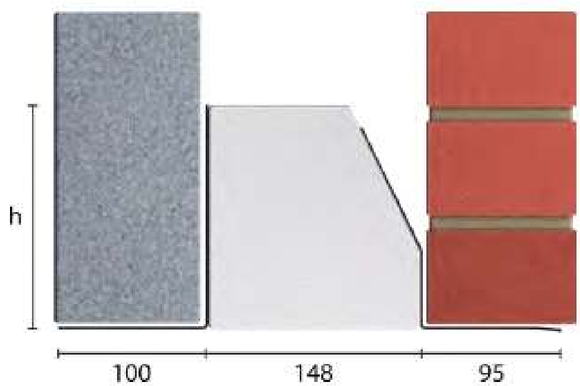


W(sls) = 1.26KN/m² x 1m/2 + 3.9KN/m² x 1.1m = 4.9KN/m

Total load = 4.9KN/m x 3.4m = 16.7KN

Ratio of loads = 1.26KN/m² x 1m/2 + 1.7KN/m² x 1.1m / 2.2KN/m² x 1.1m = 1 < 3

Standard lengths are available in 150mm increments up to 3000mm, 300mm at lengths from 3000mm to 3600mm.



TS150/100

Standard lengths (mm)	750-1500	1650-1800	1950-2100	2250-2400	2550-2700	2850-3600
SWL 1:1/3:1 (kN)	15	18	20	22	26	26
Weight (kg/m)	8.1	12.0	12.0	16.0	16.0	17.0
Nominal height 'h' (mm)	153	202	202	233	233	229**

Therefore, standard duty lintels are suitable across the entire site.

Masonry Analysis & Design

In the absence of any provided data of the proposed masonry: Block strengths assumed to be a minimum of 3.6N/mm² strength with a density of 900kg/m³ or 1800kg/m³ whichever is unfavourable.

Facing bricks are assumed to have a 20N/mm² strength.

All masonry classed as category 11 and execution class 2. Mortar strength assumed to be M4.

Front panel check

Applied vertical loadings from flat roof loading:
W(Gk) = 0.66KN/m² x 1m/2 = 0.3KN/m

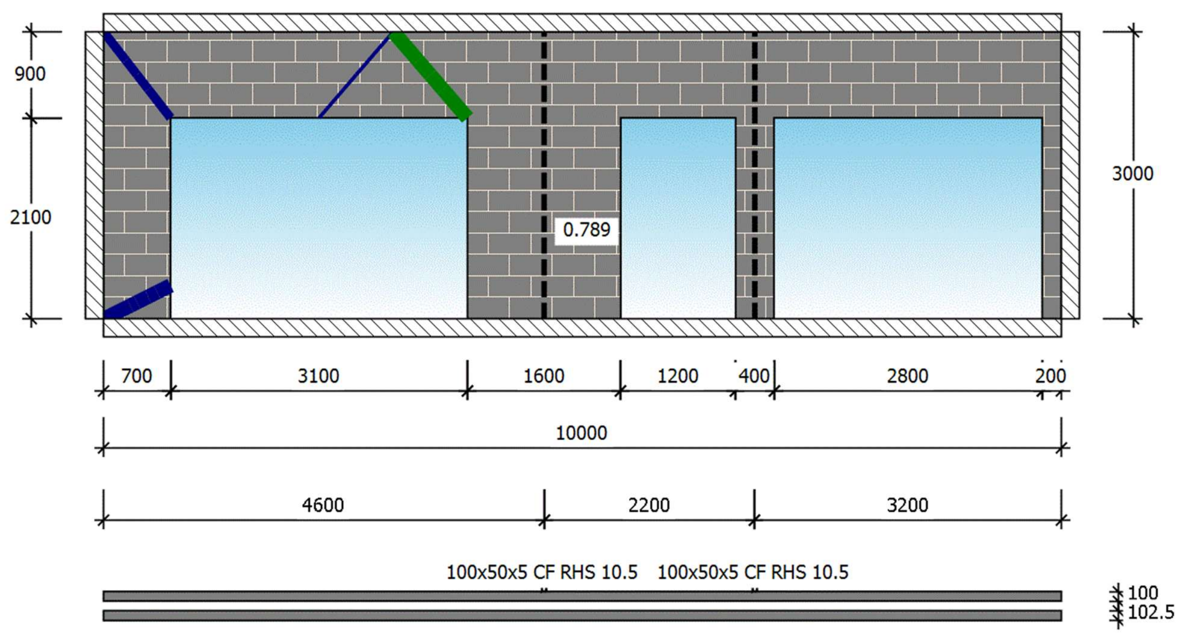
Applied wind loading = 0.48KN/m²

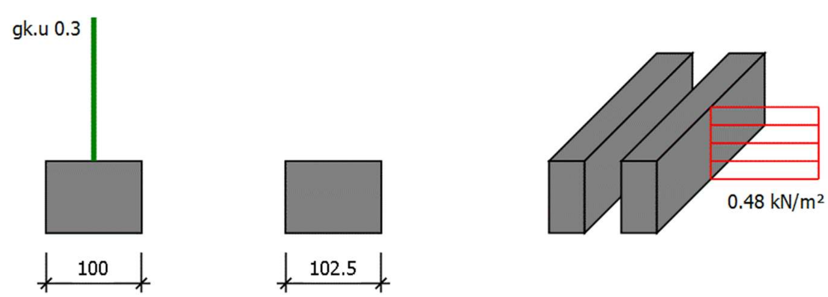
Consider all masonry lateral restraints to be a pinned support. Total height of panel to roof diaphragm restraint taken as 3m (TBC). It is assumed that both masonry leaves will be constructed in blockwork as the external face is rendered.

REINFORCED TWO WAY SPANNING, VERTICALLY AND LATERALLY LOADED, CAVITY WALL

DESIGN TO BS EN 1996-1-1:2005+A1:2012

Brief 1





Summary of Design Data

EuroCode National Annex	Using UK valuesA1 2012
Wall Dimensions	h=3.000 m, hef=2.105 m (Eqn. 5.8), L=10.000 m, Lef=10.000 m
Wind Post L reduction	Wind post assumed to act as stiffener, L = 4.600 m, Lef = 4.600 m
Support Conditions	Bottom Simple, Top Simple, Left Simple, Right Simple

Lateral Loads	Wx=0.48 kN/m²		
Opening load span direction	opening at X = 0.7, Y = 0 - Two way spanning opening at X = 5.4, Y = 0 - Two way spanning opening at X = 7, Y = 0 - Two way spanning		
Cavity Wall (mm)	t1=100, t2=102.5, tef=127.6		
Limiting Dimensions	λ=16.5<=λ_{lim}=27, L.h<=2400 tef², L<=60 tef, h<=60 tef (BS PD 6697 6.6.2.3)	0.611	OK
Reinforcement Design	BS PD 6697 6.6.5 Method 3		

Outer-Leaf Design

Partial Safety Factor (γmc/γmf)	Construction Class 2, Unit Manufacture II	3/2.7	Table NA.1
Unit Material	Concrete Blocks, Group 1, γ=8.83 kN/m³ Normalised mean compressive strengfb =3.6 N/mm²		
Mortar Material	M4 fm =4 N/mm²		
Unit Ratio	Unit height=215, Least horizontal dimensions=100	2.15	
Compressive Strength (f_k)	k = 0.75, α = 0.7, β = 0.3	2.7 N/mm²	Table NA.4
Section Properties	Area=1025 cm²/m, Zp=1751 cm³/m		
Flexural Strength f_{xk2} (Perpendicular)		0.447 N/mm²	Table NA.6
Flexural Strength f_{xk1} (Parallel)	f_{xk1}=0.248, gd=0.014 N/mm² f_{xk1}=f_{xk1}+min(gd, 0.15•f_k/γmc)γmf	0.287 N/mm²	Table NA.6
Critical axial compressive case	1.35(γ.tk.h)		
Max local stress @	X=9.8 m,Y=1.05 m < fk•0.76(small area red.)/γ_{mc}	0.1 N/mm²	OK
Critical axial buckling case	1.35(γ.tk.h)		
Max axial buckling force @	X=9.9 m, Y=1.5 m averaged over width of 0.2 m	10.66kN/m	
Moments from Lateral Load	$M_{wx,top}$=0.000 kN.m, $M_{wx,mid}$=0.000 kN.m		
Capacity reduction factor top, ~F	ex=0.0 mm, hef=300 mm, tef=127.6 mm, t=102.5 mm	0.900	
Capacity reduction factor mid, ~F_m	ehm = 0.000 mm, h_{ef} = 0.300	0.900	
Fr=~F.f_k.tk/γmc	0.900x2.06x102.5/3	63.2 kN/m	
Fd/Fr	10.7/63.2	0.169	OK
Mro=f_{xk2}.Zp/γmf	0.447x1751/2.7	0.290 kN.m/m	
Mro=f_{xk1}.Zb/γmf	0.287x1751/2.7	0.186 kN.m/m	
Bed Reinforcement	Proprietary product - Ancon AMR D3.0/W50 Product available in stainless steel or galvinised steel. Width 50 mm, No. Bars 2, fy 500 N/mm², E 200 kN/mm² https://www.ancon.co.uk/products/masonry-reinforcement/amr-masonry-reinforcement		
Placement	Vertically spaced at 450 mm, centred on leaf		
Minimum % Bed Reinforcement	0.015% per face, As/tk = 0.1x15.71/102.5	0.0153	OK
z =d(1 -0.5•As•fy•γmf/(b•d•fk•γms)	76.3(1-0.5•15.71•500•2.7/(1000•76.3•2.1•1.15))<0.95d	71.8 mm	
Mr=As.fy.z/γms	15.71x500x71.8/1.15	0.490 kN.m/m	
Md_{Limit}=0.4f_k.b.d²/γmf	0.4x2.1x1000x76.3²/2.7	1.771 kN.m/m	
Md = Min(Mr, Md_{Limit})	min(0.490, 1.771)	0.490 kN.m/m	

Inner-Leaf Design

Partial Safety Factor (γmc/γmf)	Construction Class 2, Unit Manufacture II	3/2.7	Table NA.1
Unit Material	Concrete Blocks, Group 1, γ=8.83 kN/m³ Normalised mean compressive strengfb =3.6 N/mm²		
Mortar Material	M4 fm =4 N/mm²		
Unit Ratio	Unit height=215, Least horizontal dimensions=100	2.15	
Compressive Strength (f_k)	k = 0.75, α = 0.7, β = 0.3	2.7 N/mm²	Table NA.4
Loads from above	Dead Load=0.3 kN/m		
Section Properties	Area=1000 cm²/m, Zp=1667 cm³/m		
Flexural Strength f_{xk2} (Perpendicular)		0.45 N/mm²	Table NA.6
Flexural Strength f_{xk1} (Parallel)	f_{xk1}=0.25, gd=0.018 N/mm² f_{xk1}=f_{xk1}+min(gd, 0.15•f_k/γmc)γmf	0.298 N/mm²	Table NA.6
Critical axial compressive case	1.35(γ.tk.h+gku)		
Max local stress @	X=9.8 m,Y=1.05 m < fk•0.76(small area red.)/γ_{mc}	0.13 N/mm²	OK
Critical axial buckling case	1.35(γ.tk.h+gku)		
Max axial buckling force @	X=9.9 m, Y=1.5 m averaged over width of 0.2 m	13.39kN/m	
Moments from Lateral Load	$M_{wx,top}$=0.000 kN.m, $M_{wx,mid}$=0.000 kN.m		
Capacity reduction factor top, ~F	ex=0.0 mm, hef=300 mm, tef=127.6 mm, t=100.0 mm	0.900	
Capacity reduction factor mid, ~F_m	ehm = 0.000 mm, h_{ef} = 0.300	0.900	
Fr=~F.f_k.tk/γmc	0.900x2.05x100/3	61.6 kN/m	
Fd/Fr	13.4/61.6	0.217	OK
Mri=f_{xk2}.Zp/γmf	0.45x1667/2.7	0.278 kN.m/m	
Mri=f_{xk1}.Zb/γmf	0.298x1667/2.7	0.184 kN.m/m	
Bed Reinforcement	Proprietary product - Ancon AMR D3.0/W50 Product available in stainless steel or galvinised steel. Width 50 mm, No. Bars 2, fy 500 N/mm², E 200 kN/mm² https://www.ancon.co.uk/products/masonry-reinforcement/amr-masonry-reinforcement		
Placement	Vertically spaced at 450 mm, centred on leaf		
Minimum % Bed Reinforcement	0.015% per face, As/tk = 0.1x15.71/100	0.0157	OK
z =d(1 -0.5•As•fy•γmf/(b•d•fk•γms)	75.0(1-0.5•15.71•500•2.7/(1000•75.0•2.1•1.15))<0.95d	70.5 mm	
Mr=As.fy.z/γms	15.71x500x70.5/1.15	0.482 kN.m/m	
Md_{Limit}=0.4f_k.b.d²/γmf	0.4x2.1x1000x75.0²/2.7	1.710 kN.m/m	

Md = Min(Mr, Md_{Limit})	min(0.482, 1.710)	0.482 kN.m/m	

Design for Lateral Loads

Design Lateral Load Wd	1.5 Wx	0.720 kN/m²	
Wind Post Data	Base pinned, Top pinned, Major axis bending		
Leaf Continuity at Wind Posts	Inner leaf continuous, outer leaf continuous		
Wind posts at 4.6m	100x50x5 CF RHS 10.5 (S 355), $M_{,el}$ 11.232 kN.m		
Wind posts at 6.8m	100x50x5 CF RHS 10.5 (S 355), $M_{,el}$ 11.232 kN.m		
Unreinforced Panel			
Yield Line Analysis	Load Factor, λ_p	0.895	
Ut=1/λ_p	1 / 0.895	1.118	=<1.5 OK
Reinforced Panel			
Yield Line Analysis	Load Factor, λ_p	1.267	
Ut=1/λ_p	1 / 1.267	0.789	OK
Wind Post Design	Full restrained moment capacity implicitly checked in yield line analysis		

Therefore, additional strengthening of masonry panels is required:

2No wind posts and bed joint reinforcement.

Rear panel check

Applied vertical loadings from flat roof loading:

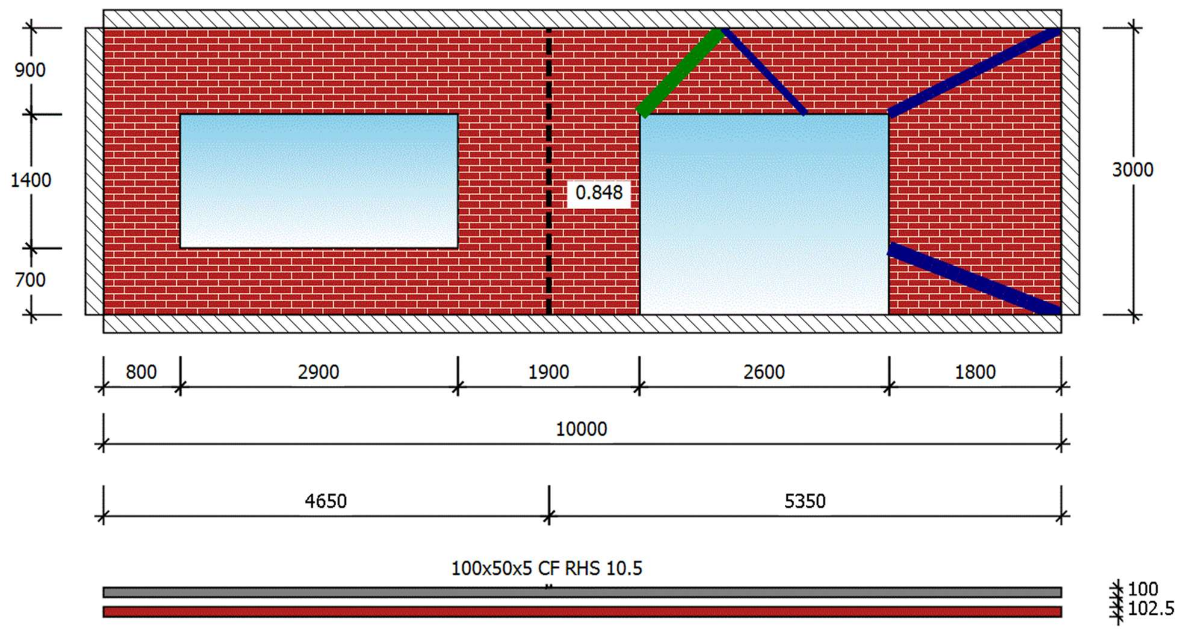
W(Gk) = 0.66KN/m² x 1m/2 = 0.3KN/m

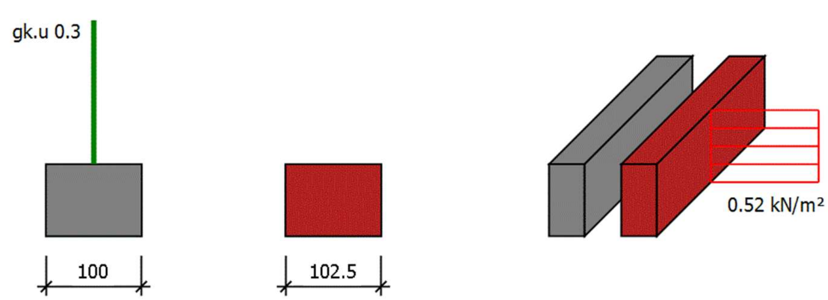
Applied wind loading = 0.52KN/m²

Consider all masonry lateral restraints to be a pinned support. Total height of panel to roof diaphragm restraint taken as 3m (TBC).

TWO WAY SPANNING, VERTICALLY AND LATERALLY LOADED, CAVITY WALL

DESIGN TO BS EN 1996-1-1:2005+A1:2012





Summary of Design Data

EuroCode National Annex	Using UK valuesA1 2012
Wall Dimensions	h=3.000 m, hef=2.282 m (Eqn. 5.8), L=10.000 m, Lef=10.000 m
Wind Post L reduction	Wind post assumed to act as stiffener, L = 5.350 m, Lef = 5.350 m
Support Conditions	Bottom Simple, Top Simple, Left Simple, Right Simple
Lateral Loads	Wx=0.52 kN/m²
Opening load span direction	opening at X = 5.6, Y = 0 - Two way spanning
	opening at X = 0.8, Y = 0.7 - Two way spanning

Cavity Wall (mm)	t1=100, t2=102.5, tef=127.6		
Limiting Dimensions	λ=17.9<=λ_{lim}=27, L/t_{ef}=41.9, H/t_{ef}=23.5, Hence H/t_{ef}<=49.5	0.662	OK

Outer-Leaf Design

Partial Safety Factor (γmc/γmf)	Construction Class 2, Unit Manufacture II	3/2.7	Table NA.1
Unit Material	Clay water absorption7% to 12%, Group 1, γ=20 kN/m³ Normalised mean compressive strengfb =20 N/mm²		
Mortar Material	M4 fm =4 N/mm²		
Compressive Strength (f_k)	k = 0.5, α = 0.7, β = 0.3	6.17 N/mm²	Table NA.4
Section Properties	Area=1025 cm²/m, Zp=1751 cm³/m		
Flexural Strength f_{xk2} (Perpendicular)		1.1 N/mm²	Table NA.6
Flexural Strength f_{xk1} (Parallel)	f_{xk1}=0.4, gd=0.03 N/mm²		
	f_{xk1}=f_{xk1}+min(gd, 0.15•f_k/γmc)γmf	0.481 N/mm²	Table NA.6
Critical axial compressive case	1.35(γ.tk.h)		
Max local stress @	X=0.249 m,Y=1.5 m < fk•0.95(small area red.)/γ_{mc}	0.14 N/mm²	OK
Critical axial buckling case	1.35(γ.tk.h)+0.75(Wx)		
Max axial buckling force @	X=9.487 m, Y=1.5 m averaged over width of 1.025 m	11.46kN/m	
Moments from Lateral Load	$M_{wx,top}$=0.000 kN.m, $M_{wx,mid}$=0.148 kN.m		
Capacity reduction factor top, ~F	ex=0.0 mm, hef=2292 mm, tef=127.6 mm, t=102.5 mm	0.900	
Capacity reduction factor mid, ~F_m	ehm = 0.250 mm, h_{ef} = 2.292	0.674	
Fr=~F.f_k.tk/γmc	0.674x6.17x102.5/3	142.0 kN/m	
Fd/Fr	11.5/142.0	0.081	OK
Mro=f_{xk2}.Zp/γmf	1.1x1751/2.7	0.713 kN.m/m	
Mro=f_{xk1}.Zb/γmf	0.481x1751/2.7	0.312 kN.m/m	

Inner-Leaf Design

Partial Safety Factor (γmc/γmf)	Construction Class 2, Unit Manufacture II	3/2.7	Table NA.1
Unit Material	Concrete Blocks, Group 1, γ=8.83 kN/m³ Normalised mean compressive strengfb =3.6 N/mm²		
Mortar Material	M4 fm =4 N/mm²		
Unit Ratio	Unit height=215, Least horizontal dimensions=100	2.15	
Compressive Strength (f_k)	k = 0.75, α = 0.7, β = 0.3	2.7 N/mm²	Table NA.4
Loads from above	Dead Load=0.3 kN/m		
Section Properties	Area=1000 cm²/m, Zp=1667 cm³/m		
Flexural Strength f_{xk2} (Perpendicular)		0.45 N/mm²	Table NA.6
Flexural Strength f_{xk1} (Parallel)	f_{xk1}=0.25, gd=0.016 N/mm²		
	f_{xk1}=f_{xk1}+min(gd, 0.15•f_k/γmc)γmf	0.294 N/mm²	Table NA.6
Critical axial compressive case	1.35(γ.tk.h+gku)		
Max local stress @	X=0.254 m,Y=1.5 m < fk•0.94(small area red.)/γ_{mc}	0.08 N/mm²	OK
Critical axial buckling case	1.35(γ.tk.h+gku)+0.75(Wx)		
Max axial buckling force @	X=9.499 m, Y=1.5 m averaged over width of 1 m	5.9kN/m	
Moments from Lateral Load	$M_{wx,top}$=0.000 kN.m, $M_{wx,mid}$=0.060 kN.m		
Capacity reduction factor top, ~F	ex=0.0 mm, hef=2292 mm, tef=127.6 mm, t=100.0 mm	0.898	
Capacity reduction factor mid, ~F_m	ehm = 0.250 mm, h_{ef} = 2.292	0.671	
Fr=~F.f_k.tk/γmc	0.671x2.7x100/3	60.4 kN/m	
Fd/Fr	5.9/60.4	0.098	OK
Mri=f_{xk2}.Zp/γmf	0.45x1667/2.7	0.278 kN.m/m	
Mri=f_{xk1}.Zb/γmf	0.294x1667/2.7	0.181 kN.m/m	

Design for Lateral Loads

Design Lateral Load Wd	1.5 Wx	0.780 kN/m²	
Wind Post Data	Base pinned, Top pinned, Major axis bending		
Leaf Continuity at Wind Posts	Inner leaf continuous, outer leaf continuous		
Wind posts at 4.65m	100x50x5 CF RHS 10.5 (S 355), $M_{,el}$ 11.232 kN.m		
Yield Line Analysis	Load Factor, λ_p	1.179	
Ut=1/λ_p	1 / 1.179	0.848	OK
Wind Post Design	Full restrained moment capacity implicitly checked in yield line analysis		

Therefore, additional strengthening of masonry panels is required:

1No wind post.

Foundations Analysis & Design

Ground parameters

No ground investigation work has been carried out and we have assumed a suitable net bearing capacity of the soil of at least 100KN/m² which is to be confirmed on site.

We shall assume with regard to concrete grade to be used will be based on the best case situation of a Design Sulphate Class of DS-1 and an aggressive chemical environment for concrete(ACEC) class of AC-1.

Concrete specification

The thickness of footings is such that the strip footings are assumed to be unreinforced, therefore Gen 3 concrete is deemed acceptable.

Pad footing under SC01

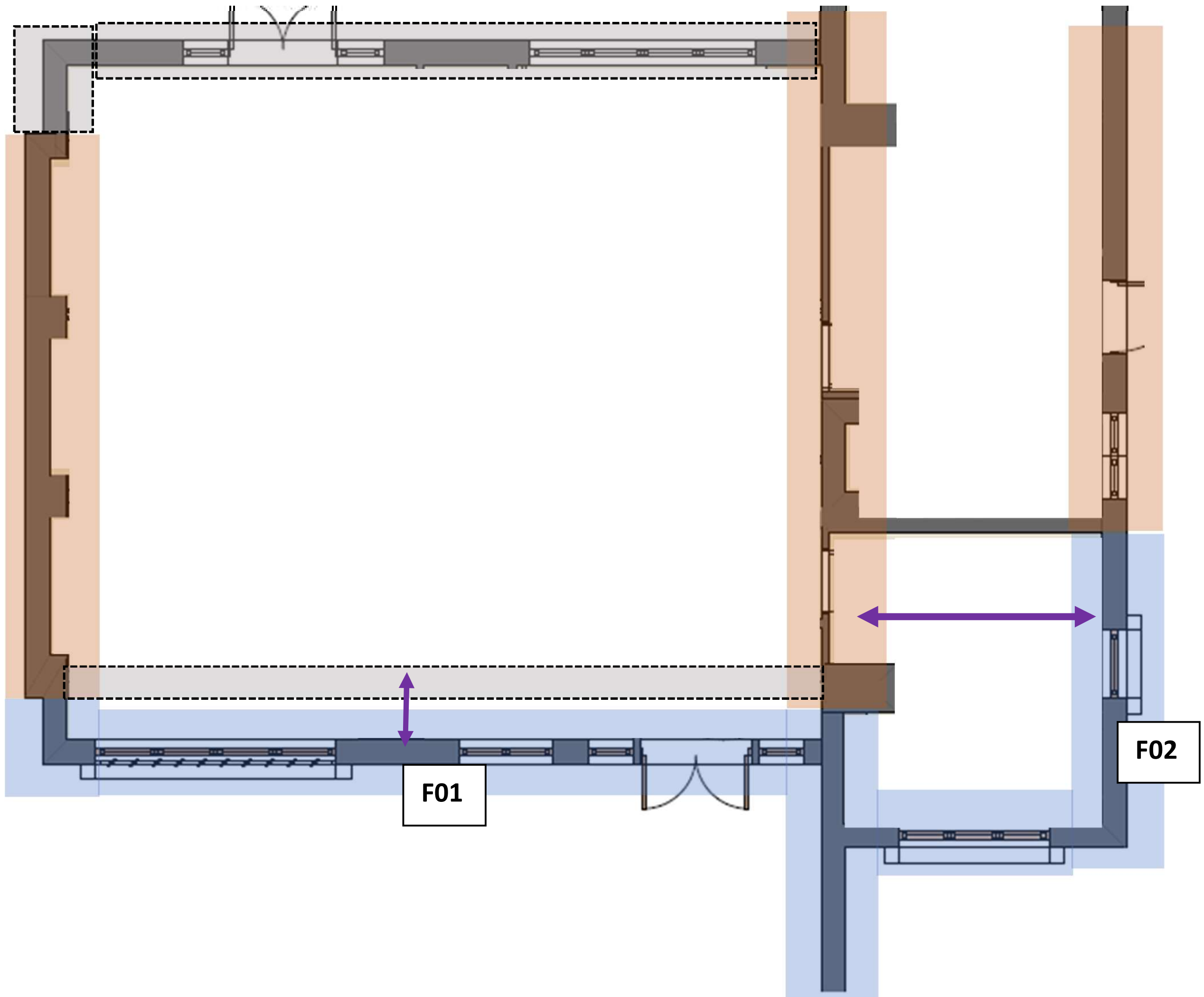
Unfactored vertical load acting at bottom of column = 7.4KN + [0.1KN/m x 2.5m] = 7.6KN

Applied pressure under 600 x 600 pad x 600 = (24KN/m³ x 0.6m³ + 7.6) / 0.6m² = 36KN/m²

New foundations under walls (not retaining)

For the new walls at the front, back and side a new footing will be required to support the new walls.

Depths to the bottom of footings are dependent upon found ground conditions, however, will likely match the existing footings. All walls are assumed to go 500mm below DPM level.



Assumed B&B spans

600mm wide
New external wall
foundations

Existing footings, Unknown

Foundation Design

Ref: **Front and rear wall,F01**

Position		Load kN/m²	Cover m	Dead(gk) kN/m	mposed(qk) kN/m
Wall, Inner	DL	1.70	4.00	6.80	
Wall, Outer	DL	2.25	4.00	9.00	
Roof	DL	0.66	1.00	0.66	
	IL	0.60	1.00		0.60
2nd Floor	DL			0.00	
	IL				0.00
1st Floor	DL			0.00	
	IL				0.00
Grd floor	DL	3.65	1.00	3.65	
	IL	2.00	1.00		2.00
				20.11	2.60

Allows for 0.5m of below ground construction. Ground floor to be concrete B&B

Total	22.71	kN/m	Strip width	450
Required width	0.2271			
Allowable GBP	100	kN/m²		
Pressures @ x kN/m²	600	37.9	450	50.5

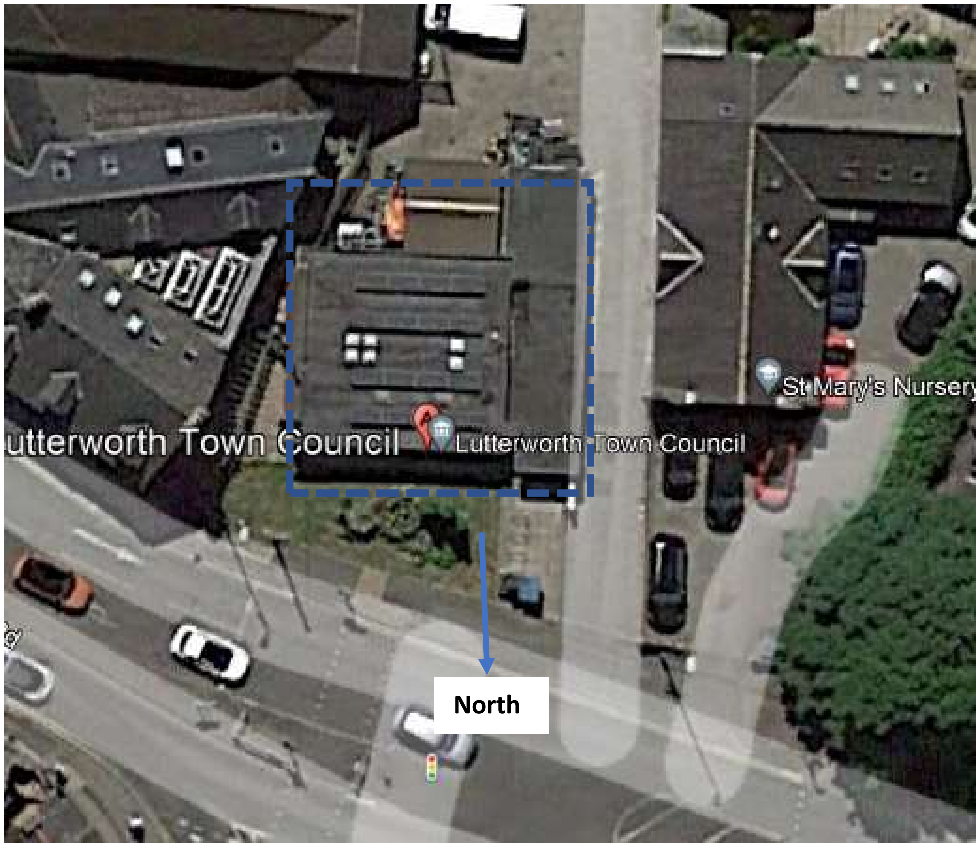
Ref: **Side wall, F02**

Position		Load kN/m²	Cover m	Dead(gk) kN/m	mposed(qk) kN/m
Wall, Inner	DL	1.70	4.00	6.80	
Wall, Outer	DL	2.25	4.00	9.00	
Roof	DL	0.66	1.80	1.19	
	IL	0.60	1.80		1.08
2nd Floor	DL				
	IL				
1st Floor	DL			0.00	
	IL				0.00
Grd floor	DL	3.65	1.80	6.57	
	IL	2.00	1.80		3.60
				23.56	4.68

Allows for 0.5m of below ground construction. Ground floor to be concrete B&B

Total	28.24	kN/m	Strip width	450
Required width	0.28238			
Allowable GBP	100	kN/m²		
Pressures @ x kN/m²	600	47.1	450	62.8

Appendix A: Existing Site



Elv = 126m

Appendix B: Wind directional factors

NA.2.6 Directional factor, c_{dir} [BS EN 1991-1-4:2005, 4.2 (2)P Note 2]

The directional factor c_{dir} is given in Table NA.1.

Table NA.1 **Directional factor c_{dir}**

Direction	0°	30°	60°	90°	120°	150°	180°	210°	240°	270°	300°	330°
c_{dir}	0,78	0,73	0,73	0,74	0,73	0,80	0,85	0,93	1,00	0,99	0,91	0,82

NOTE 1 Interpolation may be used within Table NA.1.

NOTE 2 The directions are defined by angles from due North in a clockwise direction.

NOTE 3 Where the wind loading on a building is assessed only for orthogonal load cases, the maximum value of the factor for the directions that lie ± 45° either side of the normal to the face of the building is to be used.

NOTE 4 Conservatively, c_{dir} may be taken as 1,0 for all directions.

Appendix C : Green roof loadings

Table 2.1 Major types of green roofs and their characteristics (Hui, 2006)

Characteristics	Extensive	Semi-intensive	Intensive
Depth of material	150 mm or less	Above and below 150 mm	More than 150 mm
Accessibility	Often inaccessible	May be partially accessible	Usually accessible
Fully saturated weight	Low (70–170 kg/m²)	Varies (170–290 kg/m²)	High (290–970 kg/m²)
Plant diversity	Low	Greater	Greatest
Plant communities	Moss-sedum-herbs and grasses	Grass-herbs and shrubs	Lawn or perennials, shrubs and trees
Use	Ecological protection layer	Designed green roof	Park-like garden
Cost	Low	Varies	Highest
Maintenance	Minimal	Varies	Highest