Robert E Fry & Associates Ltd
45 Bridgeman Terrace
Wigan
WN1 1TT

Project				Job Ref.	
Whalley Road, Barrow				19014	
Section				Sheet no./rev.	
Existing Wall to No 21				Ex- 1	
Calc. by	Date	Chk'd by	Date	App'd by	Date
SH	17/06/2020				

21 & 23 WHALLEY ROAD,

CLITHEROE

FOR

MULBERRY HOMES

IT'S PROPOSED TO DEMOLISH THE PROPERTY KNOW AS No 23 WHALLEY ROAD, CLITHEROE.
THESE CALCULATIONS HAVE BEEN PREPARED TO ASSESS THE SUITABILITY
OF THE PARTY WALL TO RESIST LATERAL WIND LOADING, AS WELL ASSESSING THE WALL BETWEEN GROUND AND BASEMENT LEVEL TO RESIST THE LATERAL LOADS APPLIED FROM THE BACKFILLED SUB-STRATA.

MASONRY HAS BEEN DESIGNED IN ACCORDANCE WITH BS 5628

These structural calculations must be read in conjunction with all architects and engineer's drawings and details for the project.

The calculations address only the proposed permanent works. All temporary works including temporary structural support **must** be addressed by a specialist's temporary works contractor/designer.

These works are notifiable under current Health & Safety Legislation. It is strongly recommended that reference be made to the Health and Safety Executive web site (http:/www.hse.gov.uk/construction) for guidance on the statutory responsibilities for all parties.

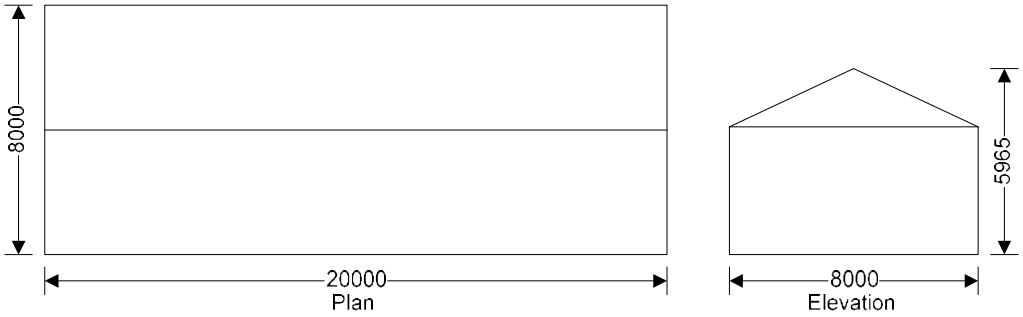
Where any queries or issues arise in relation to these calculations, reference should be made to REFA for guidance.

WIND LOADING ASSESSMENT

WIND LOADING (BS6399)

In accordance with BS6399

Tedds calculation version 3.0.17



Building data

Type of roof	Duopitch		
Length of building	L = **20000** mm	Width of building	W = **8000** mm
Pitch of roof	α_0 = **25.0** deg		
Reference height	H_r = **5965** mm		

Dynamic classification

Building type factor (table 1)	K_b = **1.0**	Dynamic augmentation factor (1.6.1)	C_r = **0.02**

Site wind speed

Location	Preston	Basic wind speed	V_b = **22.9** m/s
Site altitude	Δ_S = **75** m	Upwind dist from sea to site	d_{sea} = **40** km
Direction factor	S_d =**0.99**	Seasonal factor	S_s = **1.00**
Probability factor	S_p = **1.00**	Critical gap between buidlings	g = **5000** mm
Topography not significant			
Altitude factor	S_a = **1.08**	Site wind speed	V_s = **24.4** m/s
Terrain category	Country		
Displacement height	H_d = 0mm		

The velocity pressure for the windward face of the building with a 0 degree wind is to be considered as 1 part as the height h is less than b (cl.2.2.3.2)

The velocity pressure for the windward face of the building with a 90 degree wind is to be considered as 1 part as the height h is less than b (cl.2.2.3.2)

Dynamic pressure - windward wall - Wind 0 deg and roof

Reference height	H_e = **4100** mm		
Fetch factor (Table 22)	S_c = **0.872**	Turbulence factor (Table 22)	S_t = **0.199**
Gust peak factor	g_t = **3.44**	Terrain and building factor	S_b = **1.47**
Effective wind speed	V_e = **35.8** m/s	Dynamic pressure	q_s = **0.785** kN/m^2

Dynamic pressure - windward wall - Wind 90 deg and roof

Reference height	H_e = **5965** mm		
Fetch factor (Table 22)	S_c = **0.947**	Turbulence factor (Table 22)	S_t = **0.189**
Gust peak factor	g_t = **3.44**	Terrain and building factor	S_b = **1.56**
Effective wind speed	V_e = **38.1** m/s	Dynamic pressure	q_s = **0.889** kN/m^2

Size effect factors

Diag dim for gablewall	a_{eg} = **9.0** m	Exte size effect factor	C_{aeg} = **0.956**
Diag dim for side wall	a_{es} = **20.4** m	Exte size effect factor	C_{aes} = **0.894**
Diag dim for roof	a_{er} = **20.5** m	Exte size effect factor	C_{aer} = **0.894**
Volume for int size effect	V_i = **0.1** m^3	Diag dim for int size effect	a_i = **5.0** m
Internal size effect factor	C_{ai} = **1.000**		

Pressures and forces

Net pressure $p = q_s \times c_{pe} \times C_{ae} - q_s \times c_{pi} \times C_{ai}$

Net force $F_w = p \times A_{ref}$

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

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-0.70	0.89	0.894	-0.73	15.71	-11.53
B (-ve)	-0.60	0.89	0.894	-0.65	10.62	-6.95
C (-ve)	-0.27	0.89	0.894	-0.39	61.94	-24.14
E (-ve)	-1.03	0.89	0.894	-1.00	15.71	-15.69
F (-ve)	-0.63	0.89	0.894	-0.68	10.62	-7.24
G (-ve)	-0.50	0.89	0.894	-0.58	61.94	-35.63

Total vertical net force $F_{w,v}$ = **-91.70** kN Total horizontal net force $F_{w,h}$ = **6.74** kN

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

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A	-1.50	0.89	0.956	-1.46	11.11	-16.17
B	-0.87	0.89	0.956	-0.92	29.15	-26.68
w	0.77	0.78	0.894	0.38	82.00	31.46
l	-0.50	0.78	0.894	-0.51	82.00	-41.63

Overall loading

Leeward force overall F_l = **-41.6** kN Windward force overall F_w = **31.5** kN

Overall loading overall $F_{w,w}$ = **68.9** kN

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

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (+ve)	0.60	0.89	0.894	0.74	15.71	11.68
B (+ve)	0.40	0.89	0.894	0.58	10.62	6.21
C (+ve)	0.33	0.89	0.894	0.53	61.94	32.93
E (+ve)	-1.03	0.89	0.894	-0.55	15.71	-8.71
F (+ve)	-0.63	0.89	0.894	-0.24	10.62	-2.51
G (+ve)	-0.50	0.89	0.894	-0.13	61.94	-8.09

Total vertical net force $F_{w,v}$ = **28.57** kN Total horizontal net force $F_{w,h}$ = **29.64** kN

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

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A	-1.50	0.89	0.956	-1.01	11.11	-11.23
B	-0.87	0.89	0.956	-0.47	29.15	-13.72
w	0.77	0.78	0.894	0.78	82.00	63.63
l	-0.50	0.78	0.894	-0.12	82.00	-9.45

Overall loading

Leeward force overall F_l = **-9.5** kN Windward force overall F_w = **63.6** kN

Overall loading overall $F_{w,w}$ = **88.6** kN

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

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-1.33	0.89	0.894	-1.24	3.53	-4.37
B (-ve)	-1.23	0.89	0.894	-1.16	3.53	-4.09
C (-ve)	-0.60	0.89	0.894	-0.65	28.25	-18.49
D (-ve)	-0.47	0.89	0.894	-0.55	141.23	-77.50

Total vertical net force $F_{w,v}$ = **-94.66** 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}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A	-1.52	0.78	0.894	-1.23	6.56	-8.05
B	-0.87	0.78	0.894	-0.77	26.24	-20.22
C	-0.80	0.78	0.894	-0.72	49.20	-35.33
w	0.65	0.89	0.956	0.38	40.26	15.22
l	-0.50	0.89	0.956	-0.60	40.26	-24.27

Overall loading

Leeward force overall F_l = **-24.3** kN Windward force overall F_w = **15.2** kN

Overall loading overall $F_{w,w}$ = **34.1** kN

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

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-1.33	0.89	0.894	-0.79	3.53	-2.80
B (-ve)	-1.23	0.89	0.894	-0.71	3.53	-2.52
C (-ve)	-0.60	0.89	0.894	-0.21	28.25	-5.93
D (-ve)	-0.47	0.89	0.894	-0.10	141.23	-14.69

Total vertical net force $F_{w,v}$ = **-23.51** kN Total horizontal net force $F_{w,h}$ = **0.00** kN

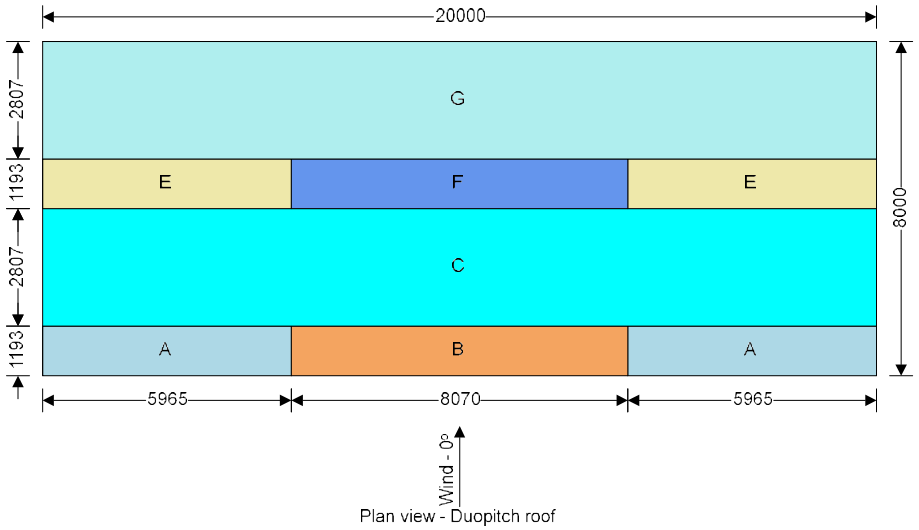
Walls load case 4 - Wind 90, c_{pi} -0.3, -c_{pe}

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A	-1.52	0.78	0.894	-0.83	6.56	-5.47
B	-0.87	0.78	0.894	-0.38	26.24	-9.93
C	-0.80	0.78	0.894	-0.33	49.20	-16.02
w	0.65	0.89	0.956	0.82	40.26	33.12
l	-0.50	0.89	0.956	-0.16	40.26	-6.37

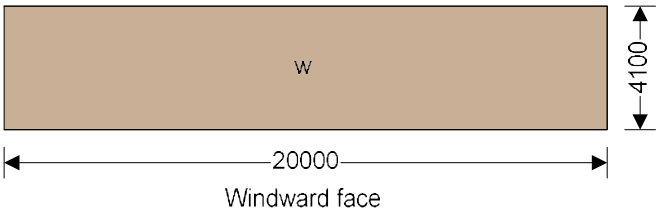
Overall loading

Leeward force overall F_l = **-6.4** kN Windward force overall F_w = **33.1** kN

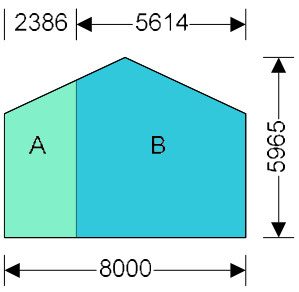
Overall loading overall $F_{w,w}$ = **34.1** kN



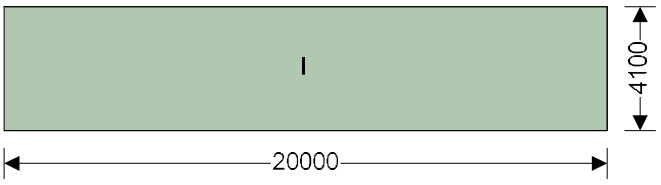
Plan view - Duopitch roof



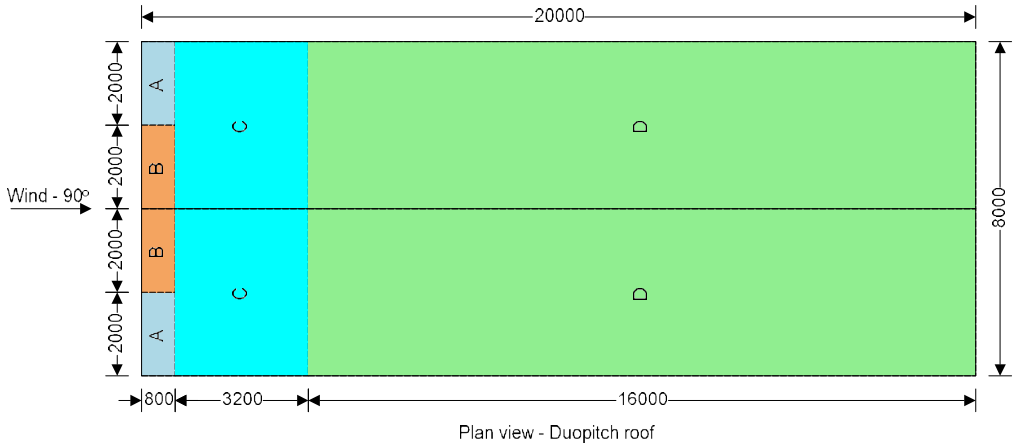
Windward face



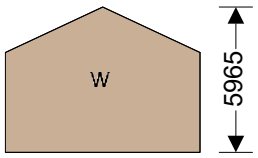
Side face



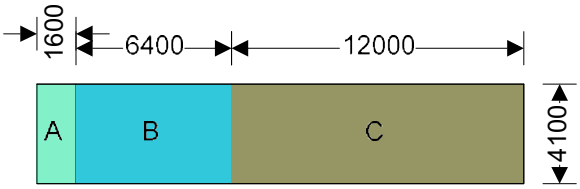
Leeward face



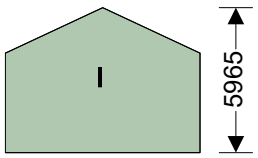
Plan view - Duopitch roof



Windward face



Side face



Leeward face

GABLE WALL ASSESSMENT FOR LATERAL WIND LOADING

Taking effective wall length = 7000mm Effective wall height = 4900mm

Ends of the wall panel will have an enhanced support, provided by the buttressing effect of the front and rear walls.

Simple support will be provided to the top and bottom of the wall panel.

The overall width of the existing party wall is circa 533mm, therefore for a conservative design, assess the wall assuming there are 2 leaves of 210mm wide stonework.

Characteristic wind loading:-

Wind direction parallel to the wall (0°)

Windward, wind pressure = 0.78 kN/m^2 Zone "B" = 0.92 kN/m^2

Wind direction perpendicular to the wall (90°)

Windward, wind pressure = 0.82 kN/m^2 Zone "B" = 0.77 kN/m^2

Masonry properties:-

Taking existing masonry to be random stone walling laid in M4 mortar.

Taking s/wt of stone = 20 kN/m^2

For lateral loading assess the existing wall based on the flexural properties of blockwork with a compressive strength of 3.5 N/mm^2, with a wall thickness of 250mm.

Therefore, from table 3 (BS 5628)

$f_{kx\ (par)}$ = 0.15 N/mm^2 $f_{kx\ (perp)}$ = 0.25 N/mm^2 γm = 3.0

Loading to Inner Leaf (21 Whalley road)

Taking roof dead load = 0.75 kN/m^2

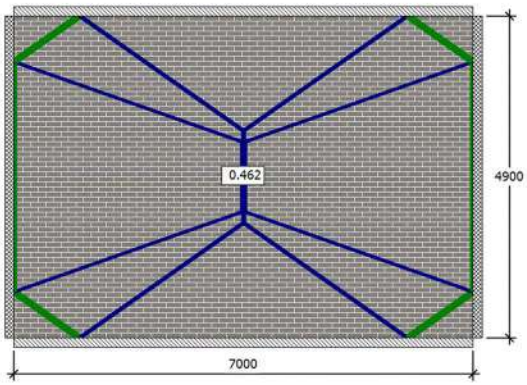
Roof load = 0.75 x 4.90/2 = 1.84 kN/m

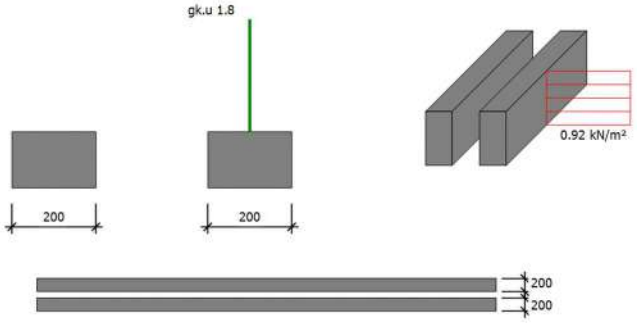
Taking stone density = 20 kN/m^2

Tekla Tedds	Project				Job Ref.	
Robert E Fry & Associates Ltd 45 Bridgeman Terrace Wigan WN1 1TT	Whalley Road, Barrow				19014	
	Section				Sheet no./rev.	
	Existing Wall to No 21				Ex- 9	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SH	17/06/2020				

TWO WAY SPANNING, VERTICALLY AND LATERALLY LOADED, CAVITY WALL

DESIGN TO BS 5628 : 1992
Gable Wall Assessment





Summary of Design Data

Wall Dimensions	h=4.900 m, hef=4.900 m, L=7.000 m, Lef=7.000 m		
Support Conditions	Bottom Simple, Top Simple, Left Cont., Right Cont.		
Lateral Loads	Wx=0.92 kN/m²		
Cavity Wall (mm)	t1=200, t2=200, tef=266.7		
Limiting Dimensions	λ=18.4<=λ_{lim}=27, L.h<=2025 tef², L<=50 tef, h<=50 tef	0.680	OK

Outer-Leaf Design

Partial Safety Factor (γmc/γmf)	Construction Normal, Unit Manufacture Normal	3.5/3	Table4a/4b
Unit Material	Concrete Bricks, Standard bricks, γ=19.62 kN/m³ Normalised mean compressive streng10 N/mm²		
Mortar Material	M4/(iii)4 N/mm²		
Compressive Strength (fk)	Standard bricks	4.1 N/mm²	Table 2
Section Properties	Area=2000 cm²/m, Zp=6667 cm³/m		
Flexural Strength fkp (Perpendicular)		0.9 N/mm²	Table 3
Flexural Strength fkb (Parallel)	fkb=0.3, gd=0.038 N/mm², fkb=fkb+0.9 gd.γmf	0.404 N/mm²	Table 3
Critical axial compressive case	1.4(γ.tk.h)		
Max local stress @	X=0 m,Y=2.94 m < fk/γ_{mc}	0.13 N/mm²	OK
Critical axial buckling case	1.4(γ.tk.h)		
Max axial buckling force @	X=3.5 m, Y=2.94 m averaged over width of 2 m	26.92kN/m	
Capacity reduction factor top, β	ex=0.0 mm, hef=4900 mm, tef=266.7 mm, t=200.0 mm	0.824	
Fr=β.fk.tk/γmc	0.824x4.1x200/3.5	193.1 kN/m	
Fd/Fr	26.9/193.1	0.139	OK
Mro=fkp.Zp/γmf	0.9x6667/3	2.000 kN.m/m	
Mro=fkb.Zb/γmf	0.404x6667/3	0.897 kN.m/m	

Inner-Leaf Design

Partial Safety Factor (γmc/γmf)	Construction Normal, Unit Manufacture Normal	3.5/3	Table4a/4b
Unit Material	Concrete Bricks, Standard bricks, γ=19.62 kN/m³ Normalised mean compressive streng10 N/mm²		
Mortar Material	M4/(iii)4 N/mm²		
Compressive Strength (fk)	Standard bricks	4.1 N/mm²	Table 2
Loads from above	Dead Load=1.84 kN/m		
Section Properties	Area=2000 cm²/m, Zp=6667 cm³/m		
Flexural Strength fkp (Perpendicular)		0.9 N/mm²	Table 3
Flexural Strength fkb (Parallel)	fkb=0.3, gd=0.048 N/mm², fkb=fkb+0.9 gd.γmf	0.429 N/mm²	Table 3
Critical axial compressive case	1.4(γ.tk.h+gku)		
Max local stress @	X=0 m,Y=2.94 m < fk/γ_{mc}	0.15 N/mm²	OK
Critical axial buckling case	1.4(γ.tk.h+gku)		
Max axial buckling force @	X=3.5 m, Y=2.94 m averaged over width of 2 m	29.49kN/m	
Capacity reduction factor top, β	ex=0.0 mm, hef=4900 mm, tef=266.7 mm, t=200.0 mm	0.824	
Fr=β.fk.tk/γmc	0.824x4.1x200/3.5	193.1 kN/m	
Fd/Fr	29.5/193.1	0.153	OK
Mri=fkp.Zp/γmf	0.9x6667/3	2.000 kN.m/m	

Mri=fkb.Zb/γmf	0.429x6667/3	0.953 kN.m/m	

Design for Lateral Loads

Design Lateral Load Wd	1.4 Wx	1.288 kN/m²	
Yield Line Analysis	Load Factor, λ_p	2.164	
Ut=1/λ_p	1 / 2.164	0.462	OK

EXISTING WALL IS ACCEPTABLE FOR LATERAL WIND LOADING

CHECK WALL BELOW GROUND FLOOR LEVEL TO RESIST LATERAL PRESSURES FROM THE BACKFILL TO THE BASEMENT AREA OF No 23

Backfill properties:-

Sub-strata density = 18 kN/m^3 Internal angle of friction = 30^o

Active soil pressures

∴ k_a = 1 – sin 30/1 + sin 30 = 1/3

Loading:-

Surcharge loading = 2.50 kN/m^2

Load factors:-

Dead loads (γd) = 1.40 Live loads (γq) = 1.60

Storey height (gnd to basement level) ≈ 1950mm

Ground level to be at least 150mm below ground floor FFL

∴ Retained height = 1950 – 150 = 1800mm

The front basement wall to No 23 is to be left in place and will provide a buttress support to the party wall.

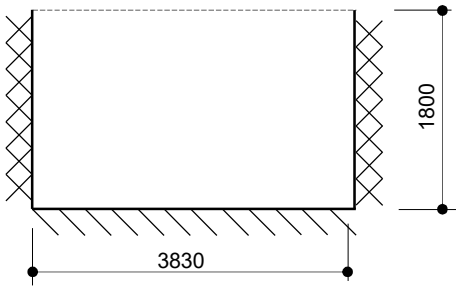
A 750mm length of the existing internal and rear basement walls are to be left in place, which will also provide buttress support to the party wall.

Design basement wall as a panel, with a free edge at ground floor level, simple support at it's base and enhanced support to the sides (due to the butttressing of the front, rear and internal walls.

Existing wall thickness at basement level ≈ 533mm.

However take wall thickness to be 500mm, and assess the wall based on a solid construction.

Worst case wall panel as below:-



Check wall panel for bending:-

By inspection worst case loading occurs to bottom of the wall, therefore assess the bottom 1000mm hight of wall.

As the backfill pressure varies with height, assess wall subject to average load applied 500mm above the basement floor level

ULS Pressures to Bottom 1000mm Section of Wall (average)

Backfill	(DL)	1/3 x 1.40 x 18 x (1.80 – 0.50)	=	10.92 kN
Surcharge	(LL)	1/3 x 1.60 x 2.50 x 1.80	=	2.40 kN

Referring to BS 5628, wall panel type = C

h/l = 1800/3830 = 0.47

For lateral loading assess the existing wall based on the flexural properties of concrete bricks with a compressive strength of 20 N/mm^2, with a wall thickness of 500mm. Assess wall with M2 mortar.

Therefore, from table 3 (BS 5628)

$f_{kx\,(par)}$ = 0.20 N/mm^2 $f_{kx\,(perp)}$ = 0.60 N/mm^2 γm = 3.0

$\mu = f_{kx\,(par)} / f_{kx\,(perp)}$

However as the wall extends upto roof level, additional vertical load will therefore increase the flexural strength in the parallel direction.

Modified flexural strength in the parallel direction = $f_{kx\,(par)} + \gamma m\, g_d$

$g_d = 0.90 \times 20 \times 0.50 \times 4.50 \times 10^3/1000 \times 500$ = 0.081 N/mm^2

$\therefore \mu = (0.20 + 3 \times 0.081)/0.60$ = 0.73

From BS 5628, μ is limited to a maximum value of 1.00

Therefore take value of α, based on a μ value of 1

From table 8, BS 5628 (extract below) :-

Table 8 — Bending moment coefficients in laterally loaded wall panels

NOTE 1 Linear interpolation of μ and h/L is permitted.

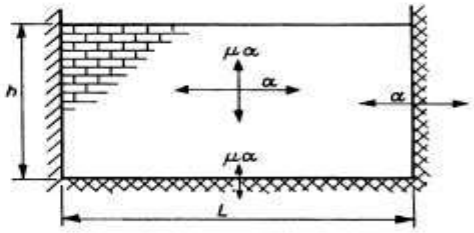
NOTE 2 When the dimensions of a wall are outside the range of h/L given in this table, it will usually be sufficient to calculate the moments on the basis of a simple span. For example, a panel of type A having h/L less than 0.3 will tend to act as a freestanding wall, whilst the same panel having h/L greater than 1.75 will tend to span horizontally.

Key to support conditions

denotes free edge

simply supported edge

an edge over which full continuity exists



	μ	Values of α, h/L = 0.3	0.5	0.75	1.00	1.25	1.5	1.75
A	1.00	0.031	0.045	0.059	0.071	0.079	0.085	0.090
	0.90	0.032	0.047	0.061	0.073	0.081	0.087	0.092
	0.80	0.034	0.049	0.064	0.075	0.083	0.089	0.093
	0.70	0.035	0.051	0.066	0.077	0.085	0.091	0.095
	0.60	0.038	0.053	0.069	0.080	0.088	0.093	0.097
	0.50	0.040	0.056	0.073	0.083	0.090	0.095	0.099
	0.40	0.043	0.061	0.077	0.087	0.093	0.098	0.101
	0.35	0.045	0.064	0.080	0.089	0.095	0.100	0.103
	0.30	0.048	0.067	0.082	0.091	0.097	0.101	0.104
B	1.00	0.024	0.035	0.046	0.053	0.059	0.062	0.065
	0.90	0.025	0.036	0.047	0.055	0.060	0.063	0.066
	0.80	0.027	0.037	0.049	0.056	0.061	0.065	0.067
	0.70	0.028	0.039	0.051	0.058	0.062	0.066	0.068
	0.60	0.030	0.042	0.053	0.059	0.064	0.067	0.069
	0.50	0.031	0.044	0.055	0.061	0.066	0.069	0.071
	0.40	0.034	0.047	0.057	0.063	0.067	0.070	0.072
	0.35	0.035	0.049	0.059	0.065	0.068	0.071	0.073
	0.30	0.037	0.051	0.061	0.066	0.070	0.072	0.074
C	1.00	0.020	0.028	0.037	0.042	0.045	0.048	0.050
	0.90	0.021	0.029	0.038	0.043	0.046	0.048	0.050
	0.80	0.022	0.031	0.039	0.043	0.047	0.049	0.051
	0.70	0.023	0.032	0.040	0.044	0.048	0.050	0.051
	0.60	0.024	0.034	0.041	0.046	0.049	0.051	0.052
	0.50	0.025	0.035	0.043	0.047	0.050	0.052	0.053
	0.40	0.027	0.038	0.044	0.048	0.051	0.053	0.054
	0.35	0.029	0.039	0.045	0.049	0.052	0.053	0.054
	0.30	0.030	0.040	0.046	0.050	0.052	0.054	0.055
D	1.00	0.013	0.021	0.029	0.035	0.040	0.043	0.045
	0.90	0.014	0.022	0.031	0.036	0.040	0.043	0.046
	0.80	0.015	0.023	0.032	0.038	0.041	0.044	0.047
	0.70	0.016	0.025	0.033	0.039	0.043	0.045	0.047
	0.60	0.017	0.026	0.035	0.040	0.044	0.046	0.048
	0.50	0.018	0.028	0.037	0.042	0.045	0.048	0.050
	0.40	0.020	0.031	0.039	0.043	0.047	0.049	0.051
	0.35	0.022	0.032	0.040	0.044	0.048	0.050	0.051
	0.30	0.023	0.034	0.041	0.046	0.049	0.051	0.052

α @ 0.3 h/l = 0.022 & α @ 0.5 h/l = 0.031

Interpolating for h/l = 0.47

α = 0.031 – [(0.031-0.022)/0.2] x (0.50 – 0.47)] = 0.0297

Applied BM

Surcharge	0.0297 x 2.40 x 3.83^2	=	1.046 kNm
Backfill	0.0297 x 10.92 x 3.83^2	=	4.757 "
		Total =	5.803 kNm

Moment of resistance = ($f_{kx\ (perp)}$/ γm) z

z = $bd^2/6$ = 1000 x $500^2/6$ = 41.67 x $10^6 mm^3$

Mt of Rest. = (0.60/3) x 41.67 = 8.334 kNm > 5.803 kNm ∴ Okay

THEREFORE WALL IS ACCEPTABLE FOR BENDING

Check Shear

Surcharge	2.40 x 3.83/2	=	4.60 kN
Backfill	10.92 x 3.83/2	=	20.91 "
		Total =	25.51 kN

Shear stress = 25.51 x 10^3/500 x 1000 = 0.05 N/mm^2

Allowable design shear stress = 0.15 + 0.6ga

ga = 0.90 x 20 x 0.50 x 4.50 x 10^3/1000 x 500 = 0.081 N/mm^2

∴ Allowable characteristic shear stress = 0.15 + 0.6 x 0.081 = 0.198 N/mm^2

γmv = 2.50

∴ Design shear strength = 0.198/2.5 = 0.079 N/mm^2 > 0.05 N/mm^2 ∴ Okay

THEREFORE WALL IS ACCEPTABLE FOR BENDING

Check Wall Foundation

The basement wall is sat on firm/stiff clay strata, the bottom of the wall is propped by the basement floor.

Therefore sliding of the wall foundation isn't critical

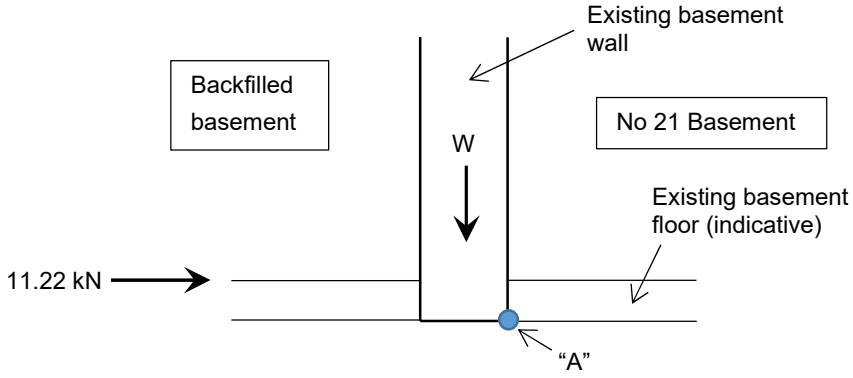
As the wall is designed with a simple support at its base, assess the wall for overturning and bearing pressures with the wall subject to horizontal loads from the backfill and surcharge applied at basement floor level.

For the foundation assessment, assume the basement floor is 150mm thick

Retained height = 1.80m

Characteristic Force at Basement Level

Surcharge	1/3 x 2.50 x 1.80	=	1.50	kN
Backfill	1/3 x 18 x 1.80^2/2	=	9.72	"
	Total	=	11.22	kN



Indicative Foundation Section Thro Basement Wall

Vertical load "W"

Roof	(DL)	0.75 x 4.90/2	=	1.84	kN/m
	(LL)	0.75 x 4.90/2	=	1.84	"
First floor	(DL)	0.50 x 0.40	=	0.20	"
	(LL)	2.00 x 0.40	=	0.80	"
Gnd floor	(DL)	0.50 x 0.40	=	0.20	"
	(LL)	2.00 x 0.40	=	0.80	"
Stonework	(DL)	20 x 0.50 x 7.0	=	70.00	"
		Σ DL	=	72.24	kN/m
		Σ LL	=	3.44	kN/m

Check Overturning (take moments about "A")

Overturning Mt = 11.22 x 0.15 = 1.68 kNm

Restoring moment = (72.24 – 0.2 – 0.2) x 0.50/2 = 17.96 kNm

Factor of safety = 17.96 / 1.68 = 10.69 > 1.67 ∴ Okay

THEREFORE FOUNDATION IS ACCEPTABLE FOR OVERTURNING

Check Bearing Pressures

Existing bearing pressure ≈ (72.24 + 3.44)/0.50 = 152 kN/m^2

∴ Assume original foundations where designed to transmit a safe bearing pressure of 175 kN/m^2

Eccentricity of loading = 17.96 – 1.68/71.84 = 0.227m from "A"

Eccentricity = 0.50/2 – 0.227 = 0.023m within middle third of footing

∴ Bearing pressures = [(72.24+3.44) / 1 x 0.5] +/- [(72.24+3.44) x 6 x 0.023 / 1 x 0.5^2]

= 151.36 +/- 41.7

Maximum bearing pressure = 193 kN/m^2

Minimum bearing pressure = 109 kN/m^2

Maximum bearing pressure exceeds 175 kN/m^2

Percentage increase in BP = 193 / 175 = 1.10

As percentage increase in bearing pressure is approximatey 10%,

THEREFORE FOUNDATION IS ACCEPTABLE FOR BEARING PRESSURES

Foundation Assessment to New Stone Facings to the Party Wall

Foundation load

Stonework	20 x 0.15 x 7.0	=	21.00 kN/m
S/wt footing	24 x 0.35 x 0.225	=	1.89 "
	Σ Total	=	22.89 kN/m

Bearing pressure (45° spread thro the footing) = 22.89/0.35 = 66 kN/m^2 > 100 kN/m^2 ∴ Okay

LAST PAGE OF CALCULATIONS