CHARTERED CONSULTING ENGINEERS

STRUCTURAL CALCULATIONS

STRUCTURAL DESIGN OF NEW GABLE WALL

2023 MAY

1.0 LOADING

1.1 ROOF LOADS

Roof Dead Loads

Roof tiles	=	*0.60*	*kN/m²*
Timber batten and felt	=	*0.05*	*kN/m²*
Timber rafter and insulation	=	*0.20*	*kN/m²*
Ceiling and Services	=	*0.25*	*kN/m²*
Σ	=	*1.10*	*kN/m²*

Roof ImposedLoads

Snow Loads	=	*0.75*	*kN/m²*
Snow loads (Dormer)	=	*1.50*	*kN/m²*

1.2 FLOOR LOADS

Floor Dead Loads

T&G Chipboard	=	*0.20*	*kN/m²*
Floor Joists	=	*0.20*	*kN/m²*
Ceiling and Insulation	=	*0.12*	*kN/m²*
Timber partition walls	=	*1.00*	*kN/m²*
Σ	=	*1.52*	*kN/m²*

Floor Loading - Imposed Loads

Domestic Imposed	=	*1.50*	*kN/m²*
Balcony/Staircase/Landings	=	*3.00*	*kN/m²*

1.3 EXISTING WALL LOADS

The exact wall thicknesses are not confirmed. For load calculation, the following SOLID wall thicknesses are considered.

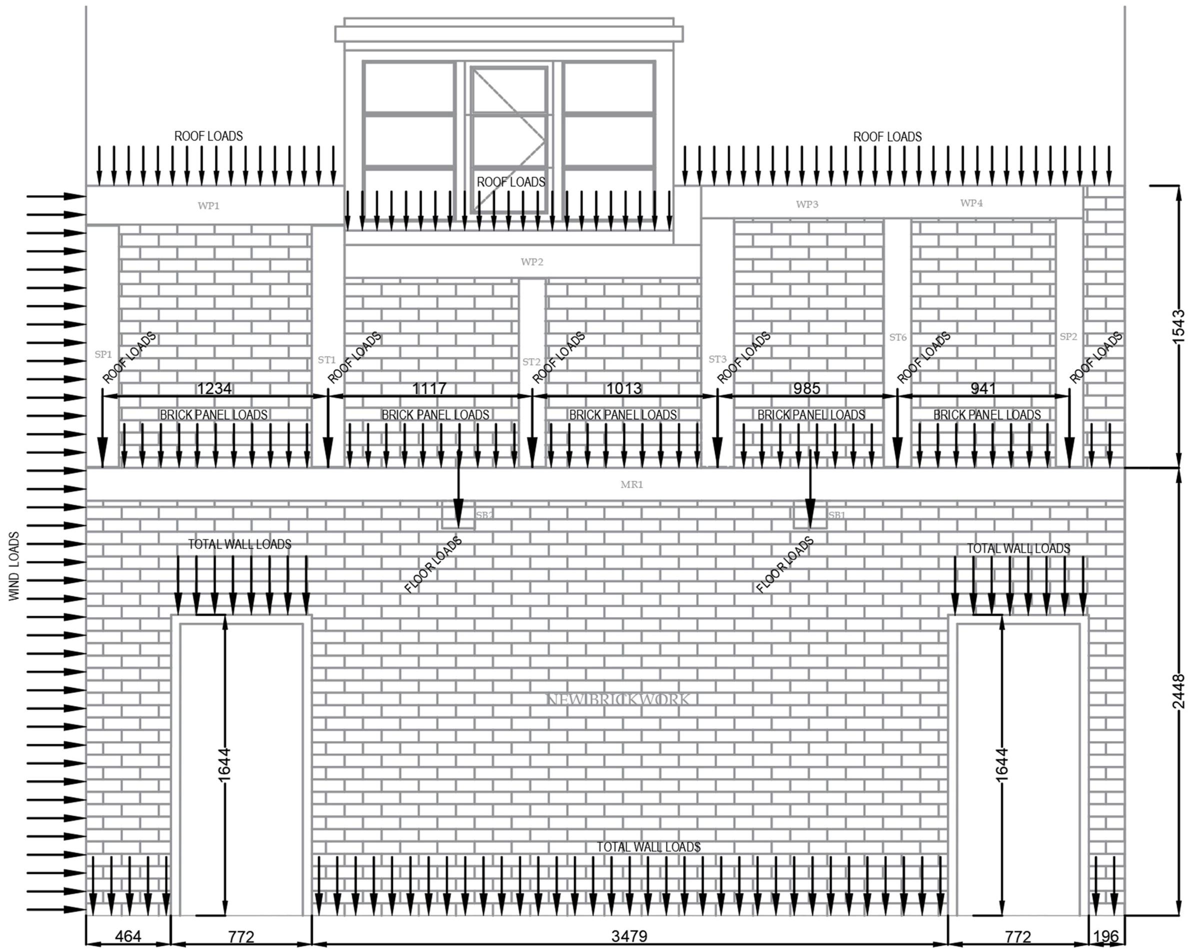
1st Floor Walls

220mm thick brick wall with height 2.75m	=	*13.31*	*kN/m*

Ground Floor Walls

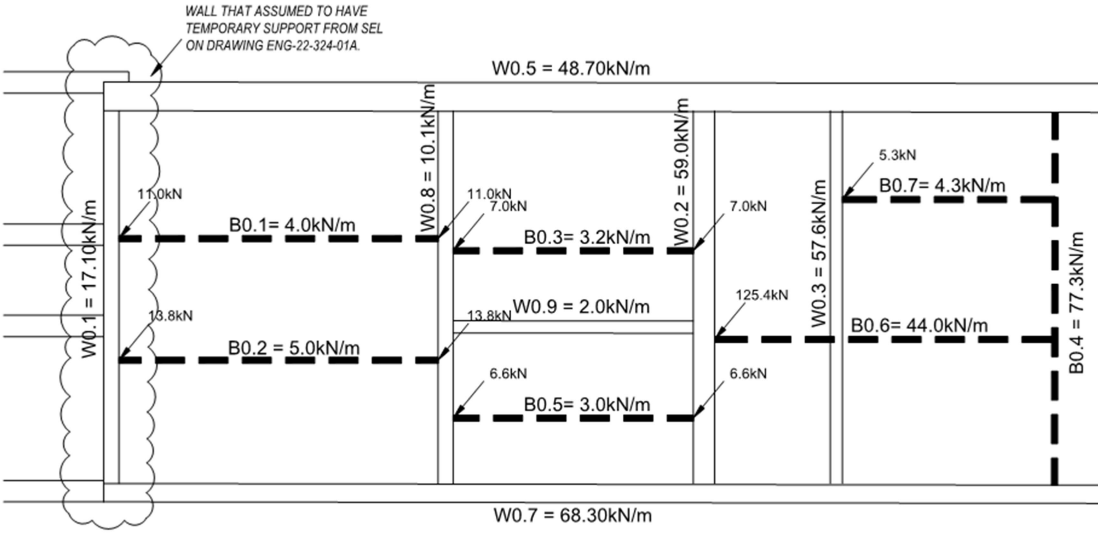
220mm thick brick wall with height 3.50m	=	*16.94*	*kN/m*

2.0 LOADS ON GABLE WALL

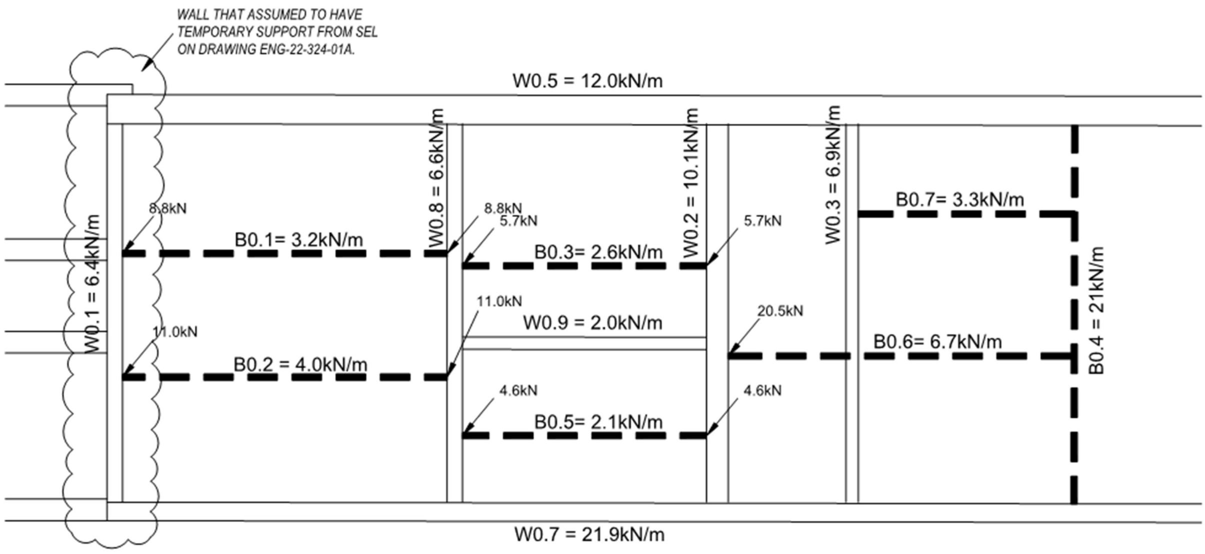


By inspection assumed that the gabel wall can resist shear forces resulting from the wind loads.

KEY PLAN



DEAD LOADS ON GROUND FLOOR WALLS AND BEAMS PLAN

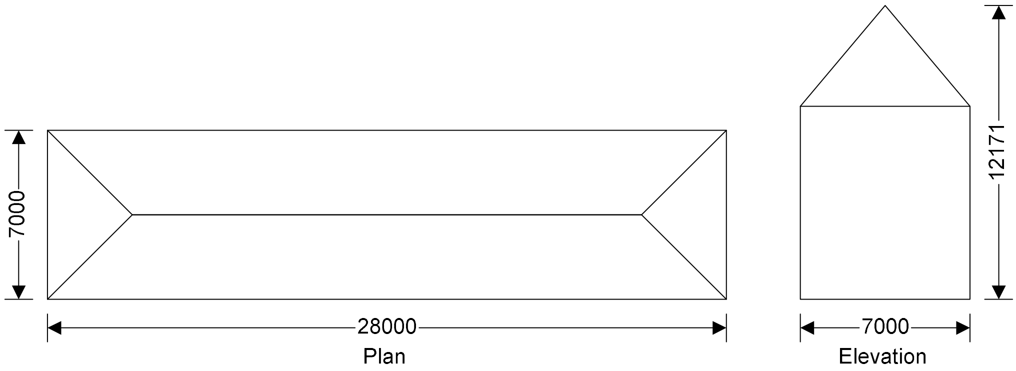


IMPOSED LOADS ON GROUND FLOOR WALLS AND BEAMS PLAN

3.0 WIND LOADING (EN1991)

WIND LOADING (EN1991-1-4)

In accordance with EN1991-1-3:2005+A1:2010 and the UK national annex



Building data

Type of roof: Hipped
Length of building: L = **28000** mm
Width of building: W = **7000** mm
Height to eaves: H = **8000** mm
Pitch of main slope: α_0 = **50.0** deg
Pitch of gable slope: α_{90} = **50.0** deg
Total height: h = **12171** mm

Basic values

Location: Birmingham
Wind speed velocity (FigureNA.1): $v_{b,map}$ = **21.7** m/s
Distance to shore: L_{shore} = **152.00** km
Altitude above sea level: A_{alt} = **120.0**m
Altitude factor: $c_{alt} = A_{alt} \times 0.001m^{-1} + 1 = \mathbf{1.120}$
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: $c_{prob} = [(1 - K \times \ln(-\ln(1-p)))/(1 - K \times \ln(-\ln(0.98)))]^n = \mathbf{1.00}$
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: $q_b = 0.5 \times \rho \times v_b^2$ = **0.362** kN/m^2

Orography

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

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

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

Peak velocity pressure - windward wall - Wind 0 deg

Reference height (at which q is sought) z = **8000**mm
Displacement height (sheltering effects excluded) h_{dis} = **0** mm
Exposure factor (Figure NA.7) c_e = **2.19**
Exposure correction factor (Figure NA.8) $c_{e,T}$ = **0.89**
Peak velocity pressure $q_p = c_e \times c_{e,T} \times q_b$ = **0.71** kN/m^2

Structural factor

Structural damping δ_s = **0.100**
Height of element h_{part} = **8000** mm
Size factor (Table NA.3) c_s = **0.813**
Dynamic factor (Figure NA.9) c_d = **1.012**
Structural factor $c_sc_d = c_s \times c_d$ = **0.823**

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

Reference height (at which q is sought) z = **7000**mm
Displacement height (sheltering effects excluded) h_{dis} = **0** mm
Exposure factor (Figure NA.7) c_e = **2.11**
Exposure correction factor (Figure NA.8) $c_{e,T}$ = **0.88**
Peak velocity pressure $q_p = c_e \times c_{e,T} \times q_b$ = **0.67** kN/m^2

Structural factor

Structural damping δ_s = **0.100**
Height of element h_{part} = **7000** mm
Size factor (Table NA.3) c_s = **0.870**
Dynamic factor (Figure NA.9) c_d = **1.044**
Structural factor $c_sc_d = c_s \times c_d$ = **0.908**

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

Reference height (at which q is sought) z = **8000**mm
Displacement height (sheltering effects excluded) h_{dis} = **0** mm
Exposure factor (Figure NA.7) c_e = **2.19**
Exposure correction factor (Figure NA.8) $c_{e,T}$ = **0.89**
Peak velocity pressure $q_p = c_e \times c_{e,T} \times q_b$ = **0.71** kN/m^2

Structural factor

Structural damping δ_s = **0.100**
Height of element h_{part} = **1000** mm
Size factor (Table NA.3) c_s = **0.934**
Dynamic factor (Figure NA.9) c_d = **1.044**
Structural factor $c_sc_d = c_s \times c_d$ = **0.975**

Structural factor

Structural damping δ_s = **0.100**
Height of element h_{part} = **8000** mm
Size factor (Table NA.3) c_s = **0.873**
Dynamic factor (Figure NA.9) c_d = **1.044**
Structural factor $c_sc_d = c_s \times c_d$ = **0.911**

Peak velocity pressure - roof

Reference height (at which q is sought) z = **12171**mm

Displacement height (sheltering effects excluded) h_{dis} = **0** mm

Exposure factor (Figure NA.7) c_e = **2.45**

Exposure correction factor (Figure NA.8) $c_{e,T}$ = **0.94**

Peak velocity pressure $q_p = c_e \times c_{e,T} \times q_b$ = **0.84** kN/m^2

Structural factor - roof 0 deg

Structural damping δ_s = **0.100**

Height of element h_{part} = **12171** mm

Size factor (Table NA.3) c_s = **0.824**

Dynamic factor (Figure NA.9) c_d = **1.012**

Structural factor $c_sc_d = c_s \times c_d$ = **0.834**

Structural factor - roof 90 deg

Structural damping δ_s = **0.100**

Height of element h_{part} = **12171** mm

Size factor (Table NA.3) c_s = **0.876**

Dynamic factor (Figure NA.9) c_d = **1.044**

Structural factor $c_sc_d = c_s \times c_d$ = **0.914**

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) $q_{p,i}$ = **0.84** 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)	0.80	0.84	0.39	36.87	14.43
G (+ve)	0.67	0.84	0.30	59.94	17.87
H (+ve)	0.73	0.84	0.34	36.59	12.61
I (+ve)	-0.63	0.84	-0.61	74.04	-45.16
J (+ve)	-0.67	0.84	-0.63	17.29	-10.95
K (+ve)	-0.37	0.84	-0.42	42.07	-17.82
L (+ve)	-0.40	0.84	-0.45	21.90	-9.79
M (+ve)	-0.23	0.84	-0.33	16.22	-5.36

Total vertical net force $F_{w,v}$ = **-28.40** kN

Total horizontal net force $F_{w,h}$ = **91.04** 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	0.71	-0.87	38.95	-33.74
B	-0.80	0.71	-0.63	17.05	-10.80
D	0.80	0.71	0.30	224.00	66.79

E	-0.54	0.71	-0.48	224.00	-107.56

Overall loading

Equiv leeward net force for overall section: $F_l = F_{w,wE} = \mathbf{-107.6}$ kN

Net windward force for overall section: $F_w = F_{w,wD} = \mathbf{66.8}$ kN

Lack of correlation (cl.7.2.2(3) – Note): $f_{corr} = \mathbf{0.88}$ as h/W is 1.739

Overall loading overall section: $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h}) = \mathbf{232.9}$ kN

Roof load case 2 - 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)	0.80	0.84	0.45	3.05	1.36
G (+ve)	0.67	0.84	0.34	3.81	1.31
H (+ve)	0.73	0.84	0.39	12.20	4.81
I (+ve)	-0.63	0.84	-0.65	12.20	-7.96
J (+ve)	-0.67	0.84	-0.68	6.86	-4.65
L (+ve)	-0.40	0.84	-0.47	7.62	-3.61
M (+ve)	-0.23	0.84	-0.35	12.20	-4.22
N (+ve)	-0.20	0.84	-0.32	246.99	-79.21

Total vertical net force: $F_{w,v} = \mathbf{-59.26}$ kN

Total horizontal net force: $F_{w,h} = \mathbf{15.39}$ kN

Walls load case 2 - 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	0.71	-0.94	11.20	-10.54
B	-0.80	0.71	-0.68	44.80	-30.61
C	-0.50	0.71	-0.49	168.00	-82.29
D_b	0.72	0.67	0.27	49.00	13.36
D_u	0.72	0.71	0.33	7.00	2.33
E	-0.35	0.71	-0.39	56.00	-21.99

Overall loading

Equiv leeward net force for upper section: $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wu} = \mathbf{-2.7}$ kN

Net windward force for upper section: $F_w = F_{w,wu} = \mathbf{2.3}$ kN

Lack of correlation (cl.7.2.2(3) – Note): $f_{corr} = \mathbf{0.85}$ as h/L is 0.435

Overall loading upper section: $F_{w,u} = f_{corr} \times (F_w - F_l + F_{w,h}) = \mathbf{17.4}$ kN

Equiv leeward net force for bottom section: $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wb} = \mathbf{-19.2}$ kN

Net windward force for bottom section: $F_w = F_{w,wb} = \mathbf{13.4}$ kN

Lack of correlation (cl.7.2.2(3) – Note): $f_{corr} = \mathbf{0.85}$ as h/L is 0.435

Overall loading bottom section: $F_{w,b} = f_{corr} \times (F_w - F_l) = \mathbf{27.7}$ kN

Roof load case 3 - Wind 0, c_{pi} -0.30, + 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	0.84	0.81	36.87	29.87
G (+ve)	0.67	0.84	0.72	59.94	42.98
H (+ve)	0.73	0.84	0.76	36.59	27.94
I (+ve)	-0.63	0.84	-0.19	74.04	-14.14
J (+ve)	-0.67	0.84	-0.21	17.29	-3.71
K (+ve)	-0.37	0.84	0.00	42.07	-0.20
L (+ve)	-0.40	0.84	-0.03	21.90	-0.61
M (+ve)	-0.23	0.84	0.09	16.22	1.43

Total vertical net force $F_{w,v}$ = **53.71** kN

Total horizontal net force $F_{w,h}$ = **91.04** kN

Walls load case 3 - Wind 0, c_{pi} -0.30, + 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	0.71	-0.45	38.95	-17.42
B	-0.80	0.71	-0.21	17.05	-3.66
D	0.80	0.71	0.72	224.00	160.63
E	-0.54	0.71	-0.06	224.00	-13.72

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wE}$ = **-13.7** kN

Net windward force for overall section $F_w = F_{w,wD}$ = **160.6** kN

Lack of correlation (cl.7.2.2(3) – Note) f_{corr} = **0.88** as h/W is 1.739

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **232.9** kN

Roof load case 4 - Wind 90, c_{pi} -0.30, + 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	0.84	0.86	3.05	2.63
G (+ve)	0.67	0.84	0.76	3.81	2.90
H (+ve)	0.73	0.84	0.81	12.20	9.91
I (+ve)	-0.63	0.84	-0.23	12.20	-2.85
J (+ve)	-0.67	0.84	-0.26	6.86	-1.78
L (+ve)	-0.40	0.84	-0.05	7.62	-0.42
M (+ve)	-0.23	0.84	0.07	12.20	0.89
N (+ve)	-0.20	0.84	0.10	246.99	24.25

Total vertical net force $F_{w,v}$ = **22.85** kN

Total horizontal net force $F_{w,h}$ = **15.37** kN

Walls load case 4 - Wind 90, c_{pi} -0.30, + 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	0.71	-0.52	11.20	-5.85
B	-0.80	0.71	-0.26	44.80	-11.84
C	-0.50	0.71	-0.07	168.00	-11.91
D_b	0.72	0.67	0.69	49.00	33.89
D_u	0.72	0.71	0.75	7.00	5.26
E	-0.35	0.71	0.03	56.00	1.47

Overall loading

Equiv leeward net force for upper section $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wu} =$ **0.2** kN

Net windward force for upper section $F_w = F_{w,wu} =$ **5.3** kN

Lack of correlation (cl.7.2.2(3) – Note) $f_{corr} =$ **0.85** as h/L is 0.435

Overall loading upper section $F_{w,u} = f_{corr} \times (F_w - F_l + F_{w,h}) =$ **17.4** kN

Equiv leeward net force for bottom section $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wb} =$ **1.3** kN

Net windward force for bottom section $F_w = F_{w,wb} =$ **33.9** kN

Lack of correlation (cl.7.2.2(3) – Note) $f_{corr} =$ **0.85** as h/L is 0.435

Overall loading bottom section $F_{w,b} = f_{corr} \times (F_w - F_l) =$ **27.7** kN

Roof load case 5 - 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)	0.27	0.84	0.02	36.87	0.69
G (-ve)	0.27	0.84	0.02	59.94	1.12
H (-ve)	0.27	0.84	0.02	36.59	0.68
I (-ve)	-0.63	0.84	-0.61	74.04	-45.16
J (-ve)	-0.67	0.84	-0.63	17.29	-10.95
K (-ve)	-0.37	0.84	-0.42	42.07	-17.82
L (-ve)	-1.13	0.84	-0.96	21.90	-21.01
M (-ve)	-0.63	0.84	-0.61	16.22	-9.89

Total vertical net force $F_{w,v} =$ **-65.78** kN

Total horizontal net force $F_{w,h} =$ **58.54** kN

Walls load case 5 - 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	0.71	-0.87	38.95	-33.74
B	-0.80	0.71	-0.63	17.05	-10.80
D	0.80	0.71	0.30	224.00	66.79
E	-0.54	0.71	-0.48	224.00	-107.56

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wE} =$ **-107.6** kN

Net windward force for overall section $F_w = F_{w,wD} =$ **66.8** kN

Lack of correlation (cl.7.2.2(3) – Note) f_{corr} = **0.88** as h/W is 1.739

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **204.4** kN

Roof load case 6 - 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)	0.27	0.84	0.04	3.05	0.11
G (-ve)	0.27	0.84	0.04	3.81	0.14
H (-ve)	0.27	0.84	0.04	12.20	0.45
I (-ve)	-0.63	0.84	-0.65	12.20	-7.96
J (-ve)	-0.67	0.84	-0.68	6.86	-4.65
L (-ve)	-1.13	0.84	-1.04	7.62	-7.89
M (-ve)	-0.63	0.84	-0.65	12.20	-7.96
N (-ve)	-0.47	0.84	-0.52	246.99	-129.65

Total vertical net force $F_{w,v}$ = **-101.18** kN

Total horizontal net force $F_{w,h}$ = **10.20** kN

Walls load case 6 - 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	0.71	-0.94	11.20	-10.54
B	-0.80	0.71	-0.68	44.80	-30.61
C	-0.50	0.71	-0.49	168.00	-82.29
D_b	0.72	0.67	0.27	49.00	13.36
D_u	0.72	0.71	0.33	7.00	2.33
E	-0.35	0.71	-0.39	56.00	-21.99

Overall loading

Equiv leeward net force for upper section $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wu}$ = **-2.7** kN

Net windward force for upper section $F_w = F_{w,wu}$ = **2.3** kN

Lack of correlation (cl.7.2.2(3) – Note) f_{corr} = **0.85** as h/L is 0.435

Overall loading upper section $F_{w,u} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **13.0** kN

Equiv leeward net force for bottom section $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wb}$ = **-19.2** kN

Net windward force for bottom section $F_w = F_{w,wb}$ = **13.4** kN

Lack of correlation (cl.7.2.2(3) – Note) f_{corr} = **0.85** as h/L is 0.435

Overall loading bottom section $F_{w,b} = f_{corr} \times (F_w - F_l)$ = **27.7** kN

Roof load case 7 - Wind 0, c_{pi} -0.30, - 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)	0.27	0.84	0.44	36.87	16.14
G (-ve)	0.27	0.84	0.44	59.94	26.23
H (-ve)	0.27	0.84	0.44	36.59	16.01
I (-ve)	-0.63	0.84	-0.19	74.04	-14.14

J (-ve)	-0.67	0.84	-0.21	17.29	-3.71
K (-ve)	-0.37	0.84	0.00	42.07	-0.20
L (-ve)	-1.13	0.84	-0.54	21.90	-11.83
M (-ve)	-0.63	0.84	-0.19	16.22	-3.10

Total vertical net force $F_{w,v}$ = **16.33** kN

Total horizontal net force $F_{w,h}$ = **58.55** kN

Walls load case 7 - Wind 0, c_{pi} -0.30, - 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	0.71	-0.45	38.95	-17.42
B	-0.80	0.71	-0.21	17.05	-3.66
D	0.80	0.71	0.72	224.00	160.63
E	-0.54	0.71	-0.06	224.00	-13.72

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wE}$ = **-13.7** kN

Net windward force for overall section $F_w = F_{w,wD}$ = **160.6** kN

Lack of correlation (cl.7.2.2(3) – Note) f_{corr} = **0.88** as h/W is 1.739

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **204.4** kN

Roof load case 8 - Wind 90, c_{pi} -0.30, - 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)	0.27	0.84	0.46	3.05	1.39
G (-ve)	0.27	0.84	0.46	3.81	1.74
H (-ve)	0.27	0.84	0.46	12.20	5.56
I (-ve)	-0.63	0.84	-0.23	12.20	-2.85
J (-ve)	-0.67	0.84	-0.26	6.86	-1.78
L (-ve)	-1.13	0.84	-0.62	7.62	-4.70
M (-ve)	-0.63	0.84	-0.23	12.20	-2.85
N (-ve)	-0.47	0.84	-0.11	246.99	-26.18

Total vertical net force $F_{w,v}$ = **-19.07** kN

Total horizontal net force $F_{w,h}$ = **10.20** kN

Walls load case 8 - Wind 90, c_{pi} -0.30, - 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	0.71	-0.52	11.20	-5.85
B	-0.80	0.71	-0.26	44.80	-11.84
C	-0.50	0.71	-0.07	168.00	-11.91
D_b	0.72	0.67	0.69	49.00	33.89
D_u	0.72	0.71	0.75	7.00	5.26

E	-0.35	0.71	0.03	56.00	1.47

Overall loading

Equiv leeward net force for upper section $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wu} = \mathbf{0.2}$ kN

Net windward force for upper section $F_w = F_{w,wu} = \mathbf{5.3}$ kN

Lack of correlation (cl.7.2.2(3) – Note) $f_{corr} = \mathbf{0.85}$ as h/L is 0.435

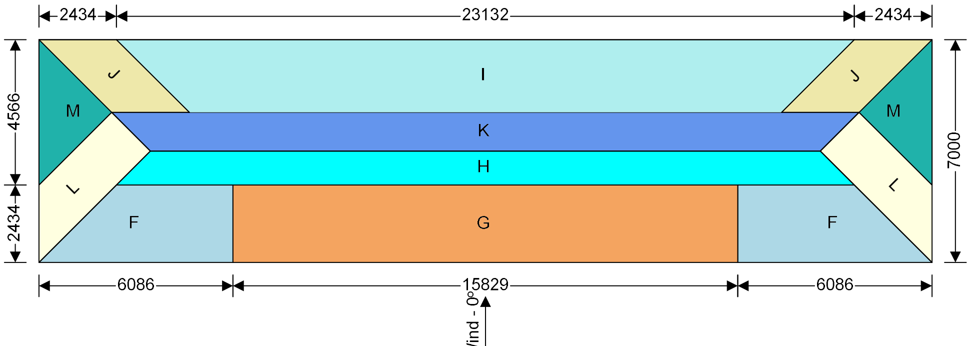
Overall loading upper section $F_{w,u} = f_{corr} \times (F_w - F_l + F_{w,h}) = \mathbf{13.0}$ kN

Equiv leeward net force for bottom section $F_l = F_{w,wE} / A_{ref,wE} \times A_{ref,wb} = \mathbf{1.3}$ kN

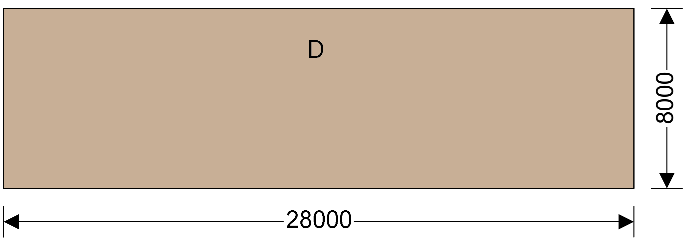
Net windward force for bottom section $F_w = F_{w,wb} = \mathbf{33.9}$ kN

Lack of correlation (cl.7.2.2(3) – Note) $f_{corr} = \mathbf{0.85}$ as h/L is 0.435

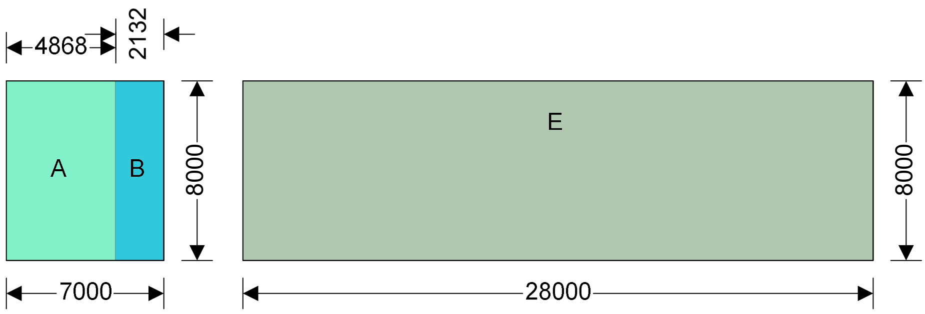
Overall loading bottom section $F_{w,b} = f_{corr} \times (F_w - F_l) = \mathbf{27.7}$ kN



Plan view - Hipped roof

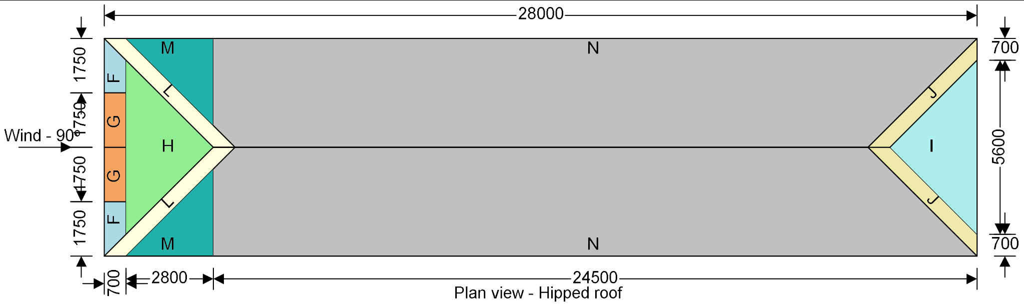


Windward face

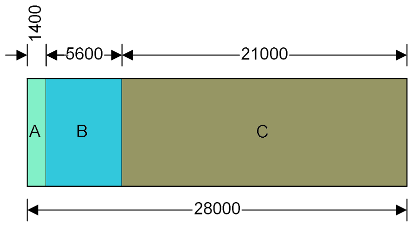


Side face

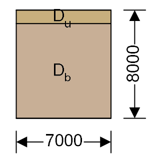
Leeward face



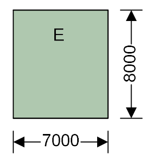
Plan view - Hipped roof



Side face



Windward face



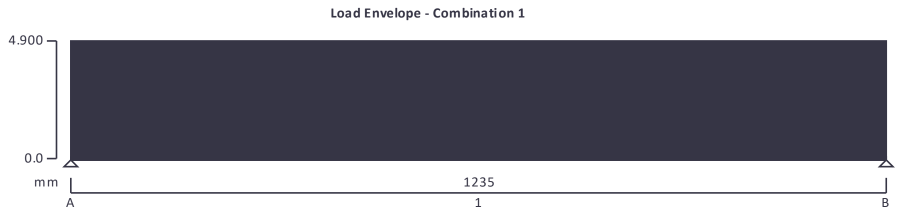
Leeward face

<u>SUMMERY OF ALL LOADS CASES</u>

Maximum wind pressure on gable end wall	= 0.95 kN/m²
Maximum wind pressure on roof above gable wall	= 0.86 kN/m²

4.0 TIMBER BEAM

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002



Applied loading

Beam loads

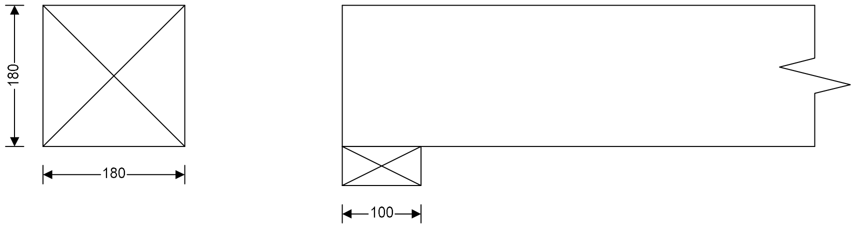
Dead full UDL 0.000 kN/m
Dead full UDL 2.090 kN/m
Imposed full UDL 2.810 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **0.934** kNm	M_{min} = **0.000** kNm
Design moment	$M = max(abs(M_{max}),abs(M_{min}))$ = **0.934** kNm	
Maximum shear	F_{max} = **3.026** kN	F_{min} = **-3.026** kN
Design shear	$F = max(abs(F_{max}),abs(F_{min}))$ = **3.026** kN	
Total load on beam	W_{tot} = **6.052** kN	
Reactions at support A	R_{A_max} = **3.026** kN	R_{A_min} = **3.026** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **1.291** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **1.735** kN	
Reactions at support B	R_{B_max} = **3.026** kN	R_{B_min} = **3.026** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **1.291** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **1.735** kN	



Timber section details

Breadth of sections	b = **180** mm
Depth of sections	h = **180** mm
Number of sections in member	N = **1**

Overall breadth of member $b_b = N \times b =$ **180** mm

Timber strength class **C24**

Member details

Service class of timber **1**

Load duration **Long term**

Length of span $L_{s1} =$ **1235** mm

Length of bearing $L_b =$ **100** mm

Section properties

Cross sectional area of member $A = N \times b \times h =$ **32400** mm^2

Section modulus $Z_x = N \times b \times h^2 / 6 =$ **972000** mm^3

$Z_y = h \times (N \times b)^2 / 6 =$ **972000** mm^3

Second moment of area $I_x = N \times b \times h^3 / 12 =$ **87480000** mm^4

$I_y = h \times (N \times b)^3 / 12 =$ **87480000** mm^4

Radius of gyration $i_x = \sqrt{(I_x / A)} =$ **52.0** mm

$i_y = \sqrt{(I_y / A)} =$ **52.0** mm

Modification factors

Duration of loading - Table 17 $K_3 =$ **1.00**

Bearing stress - Table 18 $K_4 =$ **1.00**

Total depth of member - cl.2.10.6 $K_7 = (300 \text{ mm} / h)^{0.11} =$ **1.06**

Load sharing - cl.2.9 $K_8 =$ **1.00**

Lateral support - cl.2.10.8

Ends held in position and members held in line, as by purlins or tie rods at centres not more than 30 times the breadth of the member

Permissible depth-to-breadth ratio - Table 19 **4.00**

Actual depth-to-breadth ratio $h / (N \times b) =$ **1.00**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 =$ **2.400** N/mm^2

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b) =$ **0.168** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm} =$ **0.070**

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 =$ **7.933** N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x =$ **0.961** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm} =$ **0.121**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8 =$ **0.710** N/mm^2

Applied shear stress $\tau_a = 3 \times F / (2 \times A) =$ **0.140** N/mm^2

$\tau_a / \tau_{adm} =$ **0.197**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min} =$ **7200** N/mm^2

Permissible deflection $\delta_{adm} = \min(0.118 \text{ in}, 0.003 \times L_{s1}) =$ **2.997** mm

Bending deflection $\delta_{b\_s1} =$ **0.236** mm

Shear deflection δ_{v_s1} = **0.077** mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **0.313** mm

δ_a / δ_{adm} = **0.104**

PASS - Total deflection is less than permissible deflection

5.0 MASONRY WALL PANEL DESIGN (EN1996)

MASONRY WALL PANEL DESIGN

In accordance with EN1996-1-1:2005 + A1:2012 incorporating Corrigenda February 2006 and July 2009 and the UK national annex

Masonry panel details

Wall between Timber frame - Unreinforced masonry wall without openings

Panel length — L = **1054** mm

Panel height — h = **1325** mm

Panel support conditions

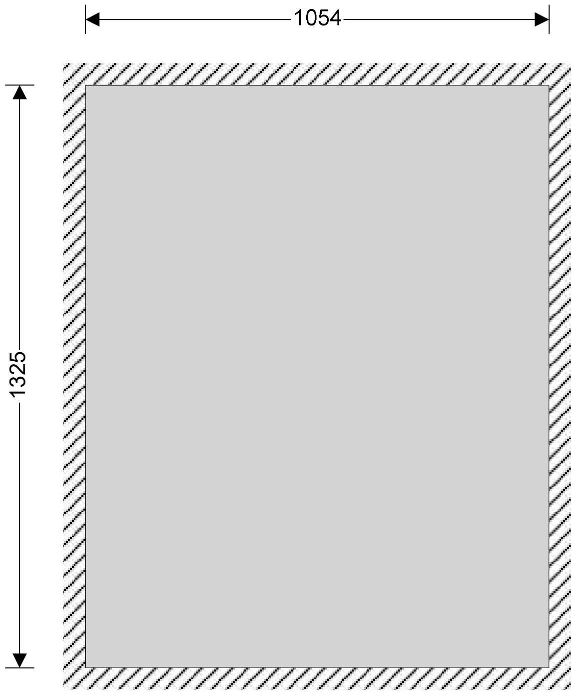
All edges supported

Effective height of masonry walls - Section 5.5.1.2

Reduction factor — ρ_2 = **1.000**

$\rho_4 = 0.5 \times L / h =$ **0.398**

Effective height of wall - eq 5.2 — $h_{ef} = \rho_4 \times h =$ **527** mm



Single-leaf wall construction details

Wall thickness — t = **100** mm

Effective thickness of masonry walls - Section 5.5.1.3

Effective thickness — $t_{ef} = t =$ **100** mm

Masonry details

Masonry type **Clay with water absorption between 7% and 12% - Group 1**

Compressive strength of masonry f_c = **7.3** N/mm^2

Height of unit h_u = **215** mm

Width of unit w_u = **100** mm

Conditioning factor k = **1.0**

- Conditioning to the air dry condition in accordance with cl.7.3.2

Shape factor - Table A.1 d_{sf} = **1.38**

Norm. mean compressive strength of masonry $f_b = f_c \times k \times d_{sf}$ = **10.074** N/mm^2

Density of masonry γ = **18** kN/m^3

Mortar type **M6 - General purpose mortar**

Compressive strength of masonry mortar f_m = **6** N/mm^2

Compressive strength factor - Table NA.4 K = **0.50**

Characteristic compressive strength of masonry - eq 3.1

$f_k = K \times f_b^{0.7} \times f_m^{0.3}$ = **4.312** N/mm^2

Characteristic flexural strength of masonry having a plane of failure parallel to the bed joints - Table NA.6

f_{xk1} = **0.4** N/mm^2

Characteristic flexural strength of masonry having a plane of failure perpendicular to the bed joints - Table NA.6

f_{xk2} = **1.1** N/mm^2

Lateral loading details

Characteristic wind load on panel W_k = **1.000** kN/m^2

Vertical loading details

Permanent load on top of wall G_k = **2.09** kN/m

Variable load on top of wall Q_k = **2.81** kN/m

Partial factors for material strength

Category of manufacturing control **Category II**

Class of execution control **Class 2**

Partial factor for masonry in compressive flexure γ_{Mc} = **3.00**

Partial factor for masonry in tensile flexure γ_{Mt} = **2.70**

Partial factor for masonry in shear γ_{Mv} = **2.50**

Slenderness ratio of masonry walls - Section 5.5.1.4

Allowable slenderness ratio SR_{all} = **27**

Slenderness ratio $SR = h_{ef} / t_{ef}$ = **5.3**

PASS - Slenderness ratio is less than maximum allowable

Unreinforced masonry walls subjected to mainly vertical loading - Section 6.1

Partial safety factors for design loads

Partial safety factor for permanent load γ_{fG} = **1.35**

Partial safety factor for variable imposed load γ_{fQ} = **1.5**

Partial safety factor for variable wind load γ_{fW} = **0.75**

Check vertical loads

Reduction factor for slenderness and eccentricity - Section 6.1.2.2

Vertical load at top of wall $N_{id} = \gamma_{fG} \times G_k + \gamma_{fQ} \times Q_k$ = **7.037** kN/m

Moment at top of wall due to vertical load $M_{id} = \gamma_{fG} \times G_k \times e_G + \gamma_{fQ} \times Q_k \times e_Q$ = **0** kNm/m

Initial eccentricity - cl.5.5.1.1 $e_{init} = h_{ef} / 450$ = **1.2** mm

Moment at top of wall due to horizontal load M_{Eid} = **0** kNm/m

Eccentricity at top of wall due to horizontal load e_h = **0** mm

Eccentricity at top of wall - eq.6.5 $e_i = \max(M_{id} / N_{id} + e_h + e_{init}, 0.05 \times t)$ = **5** mm

Reduction factor at top of wall - eq.6.4 $\Phi_i = \max(1 - 2 \times e_i / t, 0)$ = **0.9**

Vertical load at middle of wall $N_{md} = \gamma_{fG} \times (G_k + \gamma \times t \times h / 2) + \gamma_{fQ} \times Q_k$ = **8.646** kN/m

Moment at middle of wall due to vertical load $M_{md} = \gamma_{fG} \times G_k \times e_G + \gamma_{fQ} \times Q_k \times e_Q$ = **0** kNm/m

Moment at middle of wall due to horizontal load M_{Emd} = **0.019** kNm/m

Eccentricity at middle of wall due to horizontal load $e_{hm} = M_{Emd} / N_{md}$ = **2.2** mm

Eccentricity at middle of wall due to loads - eq.6.7 $e_m = M_{md} / N_{md} + e_{hm} + e_{init}$ = **3.4** mm

Eccentricity at middle of wall due to creep e_k = **0** mm

Eccentricity at middle of wall - eq.6.6 $e_{mk} = \max(e_m + e_k, 0.05 \times t)$ = **5** mm

From eq.G.2 $A_1 = 1 - 2 \times e_{mk} / t$ = **0.9**

Short term secant modulus of elasticity factor K_E = **1000**

Modulus of elasticity - cl.3.7.2 $E = K_E \times f_k$ = **4312** N/mm^2

Slenderness - eq.G.4 $\lambda = (h_{ef} / t_{ef}) \times \sqrt{(f_k / E)}$ = **0.167**

From eq.G.3 $u = (\lambda - 0.063) / (0.73 - 1.17 \times e_{mk} / t)$ = **0.154**

Reduction factor at middle of wall - eq.G.1 $\Phi_m = \max(A_1 \times e_e^{-(u \times u)/2}, 0)$ = **0.889**

Reduction factor for slenderness and eccentricity $\Phi = \min(\Phi_i, \Phi_m)$ = **0.889**

Verification of unreinforced masonry walls subjected to mainly vertical loading - Section 6.1.2

Design value of the vertical load $N_{Ed} = \max(N_{id}, N_{md})$ = **8.646** kN/m

Design compressive strength of masonry $f_d = f_k / \gamma_{Mc}$ = **1.437** N/mm^2

Vertical resistance of wall - eq.6.2 $N_{Rd} = \Phi \times t \times f_d$ = **127.822** kN/m

PASS - Design vertical resistance exceeds applied design vertical load

Unreinforced masonry walls subjected to lateral loading - Section 6.3

Partial safety factors for design loads

Partial safety factor for permanent load γ_{fG} = **1**

Partial safety factor for variable imposed load γ_{fQ} = **0**

Partial safety factor for variable wind load γ_{fW} = **1.5**

Limiting height and length to thickness ratios for walls under the serviceability limit state - Annex F

Length to thickness ratio L / t = **10.54**

Limiting height to thickness ratio - Figure F.1 80

Height to thickness ratio h / t = **13.25**

PASS - Limiting height to thickness ratio is not exceeded

Design moments of resistance in panels

Self weight at middle of wall $S_{wt} = 0.5 \times h \times t \times \gamma$ = **1.193** kN/m

Design compressive strength of masonry $f_d = f_k / \gamma_{Mc}$ = **1.437** N/mm^2

Design vertical compressive stress $\sigma_d = \min(\gamma_{fG} \times (G_k + S_{wt}) / t, 0.15 \times \Phi \times f_d)$ = **0.033** N/mm^2

Design flexural strength of masonry parallel to bed joints

$f_{xd1} = f_{xk1} / \gamma_{Mt}$ = **0.148** N/mm^2

Apparent design flexural strength of masonry parallel to bed joints

$f_{xd1,app} = f_{xd1} + \sigma_d =$ **0.181** N/mm^2

Design flexural strength of masonry perpendicular to bed joints

$f_{xd2} = f_{xk2} / \gamma_{Mt} =$ **0.407** N/mm^2

Elastic section modulus of wall — $Z = t^2 / 6 =$ **1666667** mm^3/m

Moment of resistance parallel to bed joints - eq.6.15

$M_{Rd1} = f_{xd1,app} \times Z =$ **0.302** kNm/m

Moment of resistance perpendicular to bed joints - eq.6.15

$M_{Rd2} = f_{xd2} \times Z =$ **0.679** kNm/m

Design moment in panels

Orthogonal strength ratio — $\mu = f_{xd1,app} / f_{xd2} =$ **0.44**

Using yield line analysis to calculate bending moment coefficient

Bending moment coefficient — $\alpha =$ **0.068**

Design moment in wall — $M_{Ed} = \gamma_{fW} \times \alpha \times W_k \times L^2 =$ **0.114** kNm/m

PASS - Resistance moment exceeds design moment

6.0 MASONRY WALL PANEL DESIGN (EN1996)

MASONRY WALL PANEL DESIGN

In accordance with EN1996-1-1:2005 + A1:2012 incorporating Corrigenda February 2006 and July 2009 and the UK national annex

Masonry panel details

Wall between two doors - Unreinforced masonry wall with openings

Panel length — L = **5685** mm

Panel height — h = **2270** mm

Panel support conditions

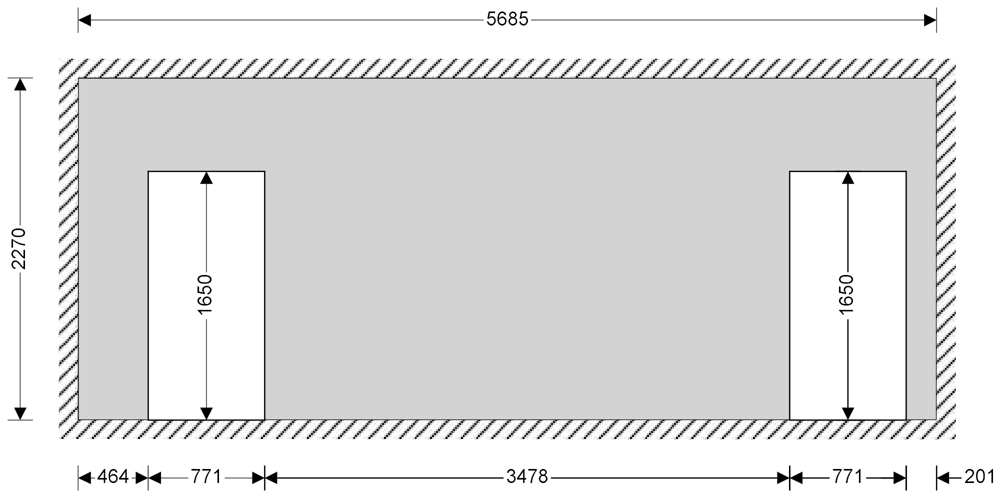
All edges supported, right and left continuous

Effective height of masonry walls - Section 5.5.1.2

Reduction factor — ρ_2 = **1.000**

$\rho_4 = \rho_2 / (1 + [\rho_2 \times h / L]^2)$ = **0.862**

Effective height of wall - eq 5.2 — $h_{ef} = \rho_4 \times h$ = **1958** mm



Panel opening details

Spacing length — L_1 = **464** mm

Opening width — w_1 = **771** mm

Height to underside of lintel — h_1 = **1650** mm

Height of opening — o_1 = **1650** mm

Spacing length — L_2 = **3478** mm

Opening width — w_2 = **771** mm

Height to underside of lintel — h_2 = **1650** mm

Height of opening — o_2 = **1650** mm

Single-leaf wall construction details

Wall thickness — t = **225** mm

Effective thickness of masonry walls - Section 5.5.1.3

Effective thickness — t_{ef} = t = **225** mm

Masonry details

Masonry type — **Clay with water absorption between 7% and 12% - Group 1**

Compressive strength of masonry — f_c = **7.3** N/mm^2

Height of unit — h_u = **215** mm

Width of unit — w_u = **225** mm

Conditioning factor — k = **1.0**

- Conditioning to the air dry condition in accordance with cl.7.3.2

Shape factor - Table A.1 — d_{sf} = **1.148**

Norm. mean compressive strength of masonry — $f_b = f_c \times k \times d_{sf}$ = **8.377** N/mm^2

Density of masonry — γ = **18** kN/m^3

Mortar type — **M6 - General purpose mortar**

Compressive strength of masonry mortar — f_m = **6** N/mm^2

Compressive strength factor - Table NA.4 — K = **0.50**

Characteristic compressive strength of masonry - eq 3.1

$f_k = K \times f_b^{0.7} \times f_m^{0.3}$ = **3.789** N/mm^2

Characteristic flexural strength of masonry having a plane of failure parallel to the bed joints - Table NA.6

f_{xk1} = **0.4** N/mm^2

Characteristic flexural strength of masonry having a plane of failure perpendicular to the bed joints - Table NA.6

f_{xk2} = **1.1** N/mm^2

Lateral loading details

Characteristic wind load on panel — W_k = **1.000** kN/m^2

Vertical loading details

Permanent load on top of wall — G_k = **7.5** kN/m

Variable load on top of wall — Q_k = **3** kN/m

Partial factors for material strength

Category of manufacturing control — **Category II**

Class of execution control — **Class 2**

Partial factor for masonry in compressive flexure — γ_{Mc} = **3.00**

Partial factor for masonry in tensile flexure — γ_{Mt} = **2.70**

Partial factor for masonry in shear — γ_{Mv} = **2.50**

Slenderness ratio of masonry walls - Section 5.5.1.4

Allowable slenderness ratio — SR_{all} = **27**

Slenderness ratio — $SR = h_{ef} / t_{ef}$ = **8.7**

PASS - Slenderness ratio is less than maximum allowable

Unreinforced masonry walls subjected to lateral loading - Section 6.3

Partial safety factors for design loads

Partial safety factor for permanent load γ_{fG} = **1**

Partial safety factor for variable imposed load γ_{fQ} = **0**

Partial safety factor for variable wind load γ_{fW} = **1.5**

Limiting height and length to thickness ratios for walls under the serviceability limit state - Annex F

Length to thickness ratio L / t = **25.267**

Limiting height to thickness ratio - Figure F.1 80

Height to thickness ratio h / t = **10.089**

PASS - Limiting height to thickness ratio is not exceeded

Design moments of resistance in panels

Self weight at top of wall S_{wt} = **0** kN/m

Design compressive strength of masonry $f_d = f_k / \gamma_{Mc}$ = **1.263** N/mm^2

Design vertical compressive stress $\sigma_d = \min(\gamma_{fG} \times (G_k + S_{wt}) / t, 0.15 \times f_d)$ = **0.033** N/mm^2

Design flexural strength of masonry parallel to bed joints

$f_{xd1} = f_{xk1} / \gamma_{Mt}$ = **0.148** N/mm^2

Apparent design flexural strength of masonry parallel to bed joints

$f_{xd1,app} = f_{xd1} + \sigma_d$ = **0.181** N/mm^2

Design flexural strength of masonry perpendicular to bed joints

$f_{xd2} = f_{xk2} / \gamma_{Mt}$ = **0.407** N/mm^2

Elastic section modulus of wall $Z = t^2 / 6$ = **8437500** mm^3/m

Moment of resistance parallel to bed joints - eq.6.15

$M_{Rd1} = f_{xd1,app} \times Z$ = **1.531** kNm/m

Moment of resistance perpendicular to bed joints - eq.6.15

$M_{Rd2} = f_{xd2} \times Z$ = **3.438** kNm/m

Design moment in panels

Orthogonal strength ratio $\mu = f_{xd1,app} / f_{xd2}$ = **0.45**

__**Sub panel no. 1 - Top, bottom and left supported**__

Ratio panel height to length h_{s1A} / L_{s1A} = **4.89**

Perpendicular design moment of resistance M_{Rd2} = **3.438** kNm/m

Using elastic analysis to determine bending moment coefficients for a horizontally spanning sub panel

Bending moment coefficient $\alpha_{s1A} = 0.5 \times (1 + 2 \times \beta_{s1A}) = \mathbf{1.331}$

Design moment in sub-panel $M_{Ed1A} = \gamma_{fW} \times \alpha_{s1A} \times W_k \times L_{s1A}^2 = \mathbf{0.430}$ kNm/m

PASS - Resistance moment exceeds design moment

WARNING! - The checking of sub-panels for vertical loading is currently beyond the scope of the calculation. This check can be performed by creating a new calculation for this sub-panel, modelled with the appropriate vertical and horizontal loading.

Sub panel no. 2 - Right, left and top supported

Ratio panel height to length $h_{s2A} / L_{s2A} = \mathbf{0.80}$

Perpendicular design moment of resistance $M_{Rd2} = \mathbf{3.438}$ kNm/m

Using yield line analysis to calculate bending moment coefficient

Bending moment coefficient $\alpha_{s2A} = \mathbf{0.213}$

Design moment in sub-panel $M_{Ed2A} = \gamma_{fW} \times \alpha_{s2A} \times W_k \times L_{s2A}^2 = \mathbf{0.190}$ kNm/m

PASS - Resistance moment exceeds design moment

WARNING! - The checking of sub-panels for vertical loading is currently beyond the scope of the calculation. This check can be performed by creating a new calculation for this sub-panel, modelled with the appropriate vertical and horizontal loading.

Sub panel no. 3 - Top and bottom supported

Ratio panel height to length $h_{s3A} / L_{s3A} = \mathbf{0.65}$

Parallel design moment of resistance $M_{Rd1} = \mathbf{1.531}$ kNm/m

Using elastic analysis to determine bending moment coefficients for a vertically spanning sub panel

Bending moment coefficient $\alpha_{s3A} = 0.125 \times (1 + 2 \times \beta_{s3A}) = \mathbf{0.180}$

Design moment in sub-panel $M_{Ed3A} = \gamma_{fW} \times \alpha_{s3A} \times W_k \times h_{s3A}^2 = \mathbf{1.395}$ kNm/m

PASS - Resistance moment exceeds design moment

WARNING! - The checking of sub-panels for vertical loading is currently beyond the scope of the calculation. This check can be performed by creating a new calculation for this sub-panel, modelled with the appropriate vertical and horizontal loading.

Sub panel no. 4 - Right, left and top supported

Ratio panel height to length $h_{s4A} / L_{s4A} = \mathbf{0.80}$

Perpendicular design moment of resistance $M_{Rd2} = \mathbf{3.438}$ kNm/m

Using yield line analysis to calculate bending moment coefficient

Bending moment coefficient $\alpha_{s4A} = \mathbf{0.213}$

Design moment in sub-panel $M_{Ed4A} = \gamma_{fW} \times \alpha_{s4A} \times W_k \times L_{s4A}^2 = \mathbf{0.190}$ kNm/m

PASS - Resistance moment exceeds design moment

WARNING! - The checking of sub-panels for vertical loading is currently beyond the scope of the calculation. This check can be performed by creating a new calculation for this sub-panel, modelled with the appropriate vertical and horizontal loading.

Sub panel no. 5 - Top, bottom and right supported

Ratio panel height to length $h_{s5A} / L_{s5A} = \mathbf{11.29}$

Perpendicular design moment of resistance $M_{Rd2} = \mathbf{3.438}$ kNm/m

Using elastic analysis to determine bending moment coefficients for a horizontally spanning sub panel

Bending moment coefficient $\alpha_{s5A} = 0.5 \times (1 + 2 \times \beta_{s5A}) = \mathbf{2.418}$

Design moment in sub-panel $M_{Ed5A} = \gamma_{fW} \times \alpha_{s5A} \times W_k \times L_{s5A}^2 = \mathbf{0.147}$ kNm/m

PASS - Resistance moment exceeds design moment

7.0 LOADS ON SINGLE SKIN WALL EXAMPLE

MASONRY BEARING DESIGN

In accordance with EN1996-1-1:2005 + A1:2012, incorporating Corrigenda February 2006 and July 2009 and the UK National Annex.

Masonry panel details

Panel length — $L = \mathbf{3480}$ mm

Panel height — $h = \mathbf{2270}$ mm

Thickness of load bearing leaf — $t = \mathbf{225}$ mm

Effective height — $h_{ef} = \mathbf{2270}$ mm

Effective thickness — $t_{ef} = \mathbf{225}$ mm

Masonry material details

Unit type — **Clay - Group 2**

Compressive strength of masonry unit — $f_c = \mathbf{7.3}$ N/mm^2

Height of unit — $h_u = \mathbf{215}$ mm

Width of unit — $w_u = \mathbf{215}$ mm

Conditioning factor — $k = \mathbf{1.0}$

- Conditioning to the air dry condition in accordance with cl.7.3.2

Shape factor - Table A.1 — $d_{sf} = \mathbf{1.16}$

Mean compressive strength of masonry unit — $f_b = f_c \times k \times d_{sf} = \mathbf{8.47}$ N/mm^2

Specific weight of units — $\gamma = \mathbf{18}$ kN/m^3

Mortar type — **M4 - General Purpose**

Compressive strength of mortar — $f_m = \mathbf{4.0}$ N/mm^2

Compressive strength factor - Tbl. NA 4 — $K = \mathbf{0.40}$

Characteristic compressive strength of the masonry - eq. 3.1

$f_k = K \times f_b^{0.7} \times f_m^{0.3} = \mathbf{2.71}$ N/mm^2

Short term secant modulus of elasticity factor — $K_E = \mathbf{1000}$

Modulus of elasticity - cl.3.7.2 — $E_w = K_E \times f_k = \mathbf{2706}$ N/mm^2

Design compressive strength of masonry

Category of manufacturing control — **Category II**

Class of execution control — **Class 2**

Partial factor for material strength in direct or flexural compression

$\gamma_M = \mathbf{3.00}$

Cross-sectional area of wall — $A = L \times t = \mathbf{0.78}$ m^2

Design compressive strength of masonry — $f_d = f_k / \gamma_M = \mathbf{0.90}$ N/mm^2

Partial safety factors for design loads

Partial safety factor for permanent load — $\gamma_{fG} = \mathbf{1.35}$

Partial safety factor for variable load — $\gamma_{fQ} = \mathbf{1.50}$

Superimposed vertical loading details

Permanent UDL at top of wall — $g_k = \mathbf{0.00}$ kN/m

Variable UDL at top of wall — $q_k = \mathbf{0.00}$ kN/m

Eccentricity of permanent UDL load — $e_{gu} = \mathbf{0}$ mm

Eccentricity of variable UDL load — $e_{qu} = \mathbf{0}$ mm

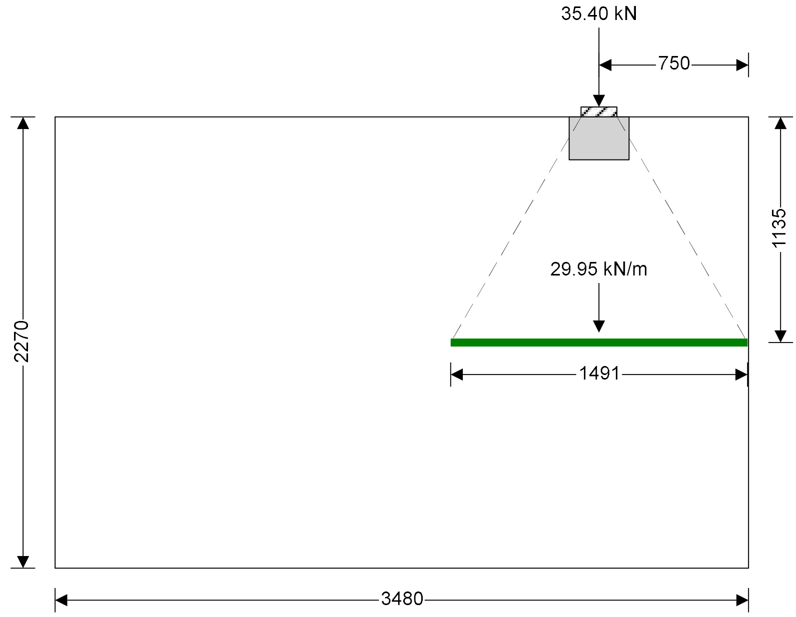
Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit λ_{lim} = **27**

Slenderness ratio $\lambda = h_{ef} / t_{ef}$ = **10.1**

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details - Timber main supporting beam reaction force



Permanent concentrated load G_{kc1} = **14.00** kN

Variable concentrated load Q_{kc1} = **11.00** kN

Eccentricity of concentrated load e_{c1} = **10** mm

Length of concentrated load L_{c1} = **180** mm

Width of concentrated load w_{c1} = **180** mm

Height of concentrated load h_{c1} = **2270** mm

Distance of load to right vertical edge r_{11} = **660** mm

Distance of load to nearest vertical edge a_{11} = **660** mm

Walls subjected to concentrated loads - Section 6.1.3

Eccentricity check $e_{c1} <= t / 4$

PASS - Eccentricity of load is less than t/4

Area of bearing $A_{b1} = L_{c1} \times w_{c1}$ = **32400** mm^2

Effective length of bearing at mid-height $l_{efm1} = L_{c1} + h_{c1} \times \tan(30)$ = **1491** mm

Effective bearing area $A_{ef1} = l_{efm1} \times t$ = **335381.65** mm^2

Bearing area ratio check $A_{ratio1} = \mathrm{Min}(A_{b1} / A_{ef1}, 0.45)$ = **0.10**

Enhancement factor - cl.6.1.3(3) β_1 = **1.00**

Design value of the concentrated load $N_{Edc1} = G_{kc1} \times \gamma_{fG} + Q_{kc1} \times \gamma_{fQ}$ = **35.40** kN

Design value concentrated load resistance $N_{Rdc1} = \beta_1 \times A_{b1} \times f_d$ = **29.22** kN

Applied concentrated load exceeds design resistance, spreader required!

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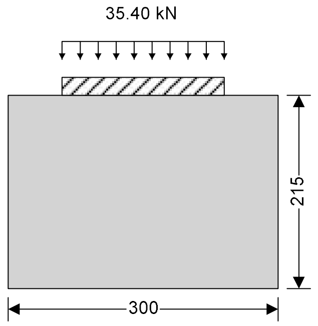
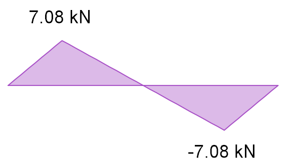
Design of spreader beam



Max. stress, 0.66 N/mm^2

0.53 kNm

Bending Moment



Shear Force

Type of spreader **Concrete padstone**

Type of bearing onto spreader **Uniformly distributed**

Point load as a UDL $N_{udl1} = N_{Edc1} / L_{c1}$ = **196.67** kN/m

Start of load from RHS of spreader P_{start1} = **240.00** mm

End of load from RHS of spreader P_{end1} = **60.00** mm

Length of spreader L_{sp1} = **300.00** mm

Height of spreader h_{sp1} = **215.00** mm

Width of spreader w_{sp1} = **225.00** mm

Eccentricity of load on spreader e_{sp1} = **10** mm

Modulus of elasticity E_{sp1} = **31476** N/mm^2

Second moment of area $I_{sp1} = 1/12 \times w_{sp1} \times h_{sp1}^3$ = **186344531** mm^4

Modulus of the wall $k_0 = E_w / h$ = **1.19** N/mm^2/mm

Winkler's constant $K_{c1} = k_0 \times w_{sp1}$ = **268.17** N/mm/mm

Characteristic of the system $\alpha_1 = (K_{c1} / (4 \times E_{sp1} \times I_{sp1}))^{1/4}$ = **0.00184** mm^{-1}

Classification of spreader $\alpha L_1 = \alpha_1 \times L_{sp1}$ = **0.55 Medium**

Krilov's functions for the spreader length $B_{\alpha l1} = 1/2 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) + \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi))$ = **0.55**

$C_{\alpha l1} = 1/2 \times \sinh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi)$ = **0.15**

$D_{\alpha l1} = 1/4 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) - \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi))$ = **0.03**

Krilov's functions at the start of the load — $B_{\alpha Pstart1} = 1/2 \times (\cosh(\alpha_1 \times P_{start1}) \times \sin(180 \times \alpha_1 \times P_{start1} / \pi) + \sinh(\alpha_1 \times P_{start1}) \times \cos(180 \times \alpha_1 \times P_{start1} / \pi)) = \mathbf{0.44}$

$C_{\alpha Pstart1} = 1/2 \times \sinh(\alpha_1 \times P_{start1}) \times \sin(180 \times \alpha_1 \times P_{start1} / \pi) = \mathbf{0.10}$

Krilov's functions at the end of the load — $B_{\alpha Pend1} = 1/2 \times (\cosh(\alpha_1 \times P_{end1}) \times \sin(180 \times \alpha_1 \times P_{end1} / \pi) + \sinh(\alpha_1 \times P_{end1}) \times \cos(180 \times \alpha_1 \times P_{end1} / \pi)) = \mathbf{0.11}$

$C_{\alpha Pend1} = 1/2 \times \sinh(\alpha_1 \times P_{end1}) \times \sin(180 \times \alpha_1 \times P_{end1} / \pi) = \mathbf{0.01}$

Using method of initial conditions

Initial moment of LH edge — $M_{01} = \mathbf{0}$ kNm

Initial shear of LH edge — $V_{01} = \mathbf{0}$ kN

Which gives — $(4 \times \alpha_1^2 \times C_{\alpha l1} \times \delta_{01} + 4 \times \alpha_1 \times D_{\alpha l1} \times \Phi_{01}) \times E_{sp1} \times I_{sp1} - N_{udl1} / \alpha_1^2 \times (C_{\alpha Pstart1} - C_{\alpha Pend1}) = \mathbf{0.00}$ kNm

and — $(4 \times \alpha_1^3 \times B_{\alpha l1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{\alpha l1} \times \Phi_{01}) \times E_{sp1} \times I_{sp1} - N_{udl1} / \alpha_1 \times (B_{\alpha Pstart1} - B_{\alpha Pend1}) = \mathbf{0.00}$ kN

Therefore,

Initial deflection of LH edge — $\delta_{01} = \mathbf{0.43955}$ mm

Initial rotationof LH edge — $\Phi_{01} = \mathbf{0.000007}$

Location of maximum deflection — $x_{def1} = \mathbf{150}$ mm

Krilov's functions at the spreader length — $A_{\alpha xdef1} = \cosh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi) = \mathbf{1.00}$

$B_{\alpha xdef1} = 1/2 \times (\cosh(\alpha_1 \times x_{def1}) \times \sin(180 \times \alpha_1 \times x_{def1} / \pi) + \sinh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi)) = \mathbf{0.28}$

Distance from start load right of location — $p_{startdef1} = \mathbf{90}$ mm

Krilov's functions at the spreader length — $A_{\alpha pstartdef1} = \cosh(\alpha_1 \times p_{startdef1}) \times \cos(180 \times \alpha_1 \times p_{startdef1} / \pi) = \mathbf{1.00}$

Distance from end load right of location — $p_{enddef1} = \mathbf{0}$ mm

Krilov's functions at the spreader length — $A_{\alpha penddef1} = \cosh(\alpha_1 \times p_{enddef1}) \times \cos(180 \times \alpha_1 \times p_{enddef1} / \pi) = \mathbf{1.00}$

Particular integral due to load — $\delta'_1 = (-N_{udl1} / (4 \times \alpha_1^4) \times (A_{\alpha pstartdef1} - A_{\alpha penddef1})) / (I_{sp1} \times E_{sp1}) = \mathbf{0.00009}$

mm

Maximum deflection — $\delta_{max1} = A_{\alpha xdef1} \times \delta_{01} + B_{\alpha xdef1} \times \Phi_{01} / \alpha_1 + \delta'_1 = \mathbf{0.44030}$ mm

Location of maximum moment — $x_{M1} = \mathbf{150}$ mm

Krilov's functions at the spreader length — $C_{\alpha xM1} = 1/2 \times \sinh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) = \mathbf{0.04}$

$D_{\alpha xM1} = 1/4 \times (\cosh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) - \sinh(\alpha_1 \times x_{M1}) \times \cos(180 \times \alpha_1 \times x_{M1} / \pi)) = \mathbf{0.00}$

Distance from start load right of location — $p_{startM1} = \mathbf{90}$ mm

Krilov's functions at the spreader length — $C_{\alpha pstartM1} = 1/2 \times \sinh(\alpha_1 \times p_{startM1}) \times \sin(180 \times \alpha_1 \times p_{startM1} / \pi) = \mathbf{0.01}$

Distance from end load right of location — $p_{endM1} = \mathbf{0}$ mm

Krilov's functions at the spreader length — $C_{\alpha pendM1} = 1/2 \times \sinh(\alpha_1 \times p_{endM1}) \times \sin(180 \times \alpha_1 \times p_{endM1} / \pi) = \mathbf{0.00}$

Particular integral due to load — $M'_1 = -N_{udl1} / \alpha_1^2 \times (C_{\alpha pstartM1} - C_{\alpha pendM1}) = \mathbf{-0.80}$ kNm

Maximum moment — $M_{Edsp1} = (4 \times \alpha_1^2 \times C_{\alpha xM1} \times \delta_{01} + 4 \times \alpha_1 \times D_{\alpha xM1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + M'_1 = \mathbf{0.53}$ kNm

Location of maximum shear — $x_{V1} = \mathbf{60}$ mm

Krilov's functions at the spreader length — $B_{\alpha xV1} = 1/2 \times (\cosh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) + \sinh(\alpha_1 \times x_{V1}) \times \cos(180 \times \alpha_1 \times x_{V1} / \pi)) = \mathbf{0.11}$

$C_{\alpha xV1} = 1/2 \times \sinh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) = \mathbf{0.01}$

Distance from start load right of location — $p_{startV1} = \mathbf{0}$ mm

Krilov's functions at the spreader length — $B_{\alpha pstartV1} = 1/2 \times (\cosh(\alpha_1 \times p_{startV1}) \times \sin(180 \times \alpha_1 \times p_{startV1} / \pi) + \sinh(\alpha_1 \times p_{startV1}) \times \cos(180 \times \alpha_1 \times p_{startV1} / \pi)) =$ **0.00**

Distance from end load right of location — p_{endV1} = **0** mm

Krilov's functions at the spreader length — $B_{\alpha pendV1} = 1/2 \times (\cosh(\alpha_1 \times p_{endV1}) \times \sin(180 \times \alpha_1 \times p_{endV1} / \pi) + \sinh(\alpha_1 \times p_{endV1}) \times \cos(180 \times \alpha_1 \times p_{endV1} / \pi)) =$ **0.00**

Particular integral due to load — $V'_1 = -N_{udl1} / \alpha_1 \times (B_{\alpha pstartV1} - B_{\alpha pendV1}) =$ **0.00** kN

Maximum shear — $V_{Edsp1} = (4 \times \alpha_1^3 \times B_{\alpha xV1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{\alpha xV1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + V'_1 =$ **7.08** kN

Maximum allowable stress under spreader — $\sigma_{Rdsp1} = \beta_1 \times f_d =$ **0.90** N/mm^2

Maximum reaction — $N_{Edsp1} = K_{c1} \times \delta_{max1} =$ **118.08** kN/m

Design stress — $\sigma_{Edsp1} = N_{Edsp1} / w_{sp1} \times (1 + 6 \times e_{sp1} / w_{sp1}) =$ **0.66** N/mm^2

PASS - Design stress under spreader is less than the allowable bearing stress

Walls subjected to mainly vertical loading - Section6.1.2

Eccentricity of permanent UDL at mid-height below concentrated load

$e_{gmu1} = e_{gu} \times h_{c1} / (2 \times h) =$ **0.0** mm

Eccentricity of variable UDL at mid-height below concentrated load

$e_{qmu1} = e_{qu} \times h_{c1} / (2 \times h) =$ **0.0** mm

Eccentricity of concentrated load at mid-height — $e_{mc1} = e_{c1} / 2 =$ **5.0** mm

Initial eccentricity - cl.5.5.1.1(4) — $e_{init} = h / 450 =$ **5.0** mm

Concentrated load at mid-height as UDL — $N_{mc1} = N_{Edc1} / l_{efm1} =$ **23.75** kN/m

Vertical load at mid-height — $N_{Ed1} = (g_k + \gamma \times t \times (h - h_{c1} / 2)) \times \gamma_{fG} + q_k \times \gamma_{fQ} + N_{mc1} =$ **29.95** kN/m

Design moment at mid-height — $M_{Ed1} = g_k \times \gamma_{fG} \times e_{gmu1} + q_k \times \gamma_{fQ} \times e_{qmu1} + N_{mc1} \times e_{mc1} =$ **0.12** kNm/m

Eccentricities due to loads - eq. 6.7 — $e_{m1} = \mathrm{Abs}(M_{Ed1}) / N_{Ed1} + e_{init} =$ **9.0** mm

Slenderness ratio limit for creep eccentricity — λ_c = **27**

Eccentricity due to creep — e_{k1} = **0.0** mm

Eccentricity at mid-height - eq. 6.6 — $e_{mk1} = \mathrm{Max}(e_{m1} + e_{k1}, 0.05 \times t) =$ **11.3** mm

From eq. G2 — $A_{11} = 1 - 2 \times e_{mk1} / t =$ **0.90**

From eq. G3 — $u_1 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk1} / t) =$ **0.38**

Capacity reduction factor - eq. G1 — $\Phi_{m1} = A_{11} \times \exp(-(u_1^2) / 2) =$ **0.84**

Design vertical resistance of panel - eq.6.2 — $N_{Rd1} = \Phi_{m1} \times t \times f_d =$ **169.82** kN/m

PASS - Design value of vertical resistance exceeds applied vertical load

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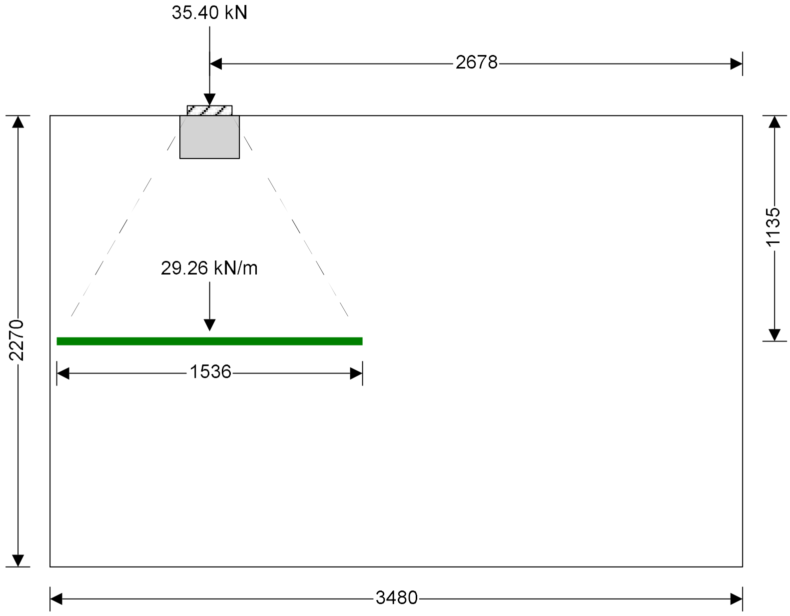
	Calc. by: IM	Date: 04/05/2023	Chk'd by:	Date:	App'd by:	Date:

Concentrated Load 2 details - Timber main beam supporting beam reaction force



Permanent concentrated load	G_{kc2} = **14.00** kN
Variable concentrated load	Q_{kc2} = **11.00** kN
Eccentricity of concentrated load	e_{c2} = **15** mm
Length of concentrated load	L_{c2} = **225** mm
Width of concentrated load	w_{c2} = **225** mm
Height of concentrated load	h_{c2} = **2270** mm
Distance of load to right vertical edge	r_{12} = **2566** mm
Distance of load to nearest vertical edge	a_{12} = **689** mm

Walls subjected to concentrated loads - Section 6.1.3

Eccentricity check $e_{c2} <= t / 4$

PASS - Eccentricity of load is less than t/4

Area of bearing	$A_{b2} = L_{c2} \times w_{c2}$ = **50625** mm^2
Effective length of bearing at mid-height	$l_{efm2} = L_{c2} + h_{c2} \times \tan(30)$ = **1536** mm
Effective bearing area	$A_{ef2} = l_{efm2} \times t$ = **345506.65** mm^2
Bearing area ratio check	$A_{ratio2} = \mathrm{Min}(A_{b2} / A_{ef2}, 0.45)$ = **0.15**
Enhancement factor - cl.6.1.3(3)	β_2 = **1.00**
Design value of the concentrated load	$N_{Edc2} = G_{kc2} \times \gamma_{fG} + Q_{kc2} \times \gamma_{fQ}$ = **35.40** kN
Design value concentrated load resistance	$N_{Rdc2} = \beta_2 \times A_{b2} \times f_d$ = **45.66** kN

PASS - Design resistance exceeds applied concentrated load

Design of spreader beam

Type of spreader	**Concrete padstone**
Type of bearing onto spreader	**Uniformly distributed**
Point load as a UDL	$N_{udl2} = N_{Edc2} / L_{c2}$ = **157.33** kN/m
Start of load from RHS of spreader	P_{start2} = **262.50** mm
End of load from RHS of spreader	P_{end2} = **37.50** mm
Length of spreader	L_{sp2} = **300.00** mm
Height of spreader	h_{sp2} = **215.00** mm
Width of spreader	w_{sp2} = **225.00** mm

Eccentricity of load on spreader — e_{sp2} = **15** mm

Modulus of elasticity — E_{sp2} = **29962** N/mm^2

Second moment of area — $I_{sp2} = 1/12 \times w_{sp2} \times h_{sp2}^3$ = **186344531** mm^4

Modulus of the wall — $k_0 = E_w / h$ = **1.19** N/mm^2/mm

Winkler's constant — $K_{c2} = k_0 \times w_{sp2}$ = **268.17** N/mm/mm

Characteristic of the system — $\alpha_2 = (K_{c2} / (4 \times E_{sp2} \times I_{sp2}))^{1/4}$ = **0.00186** mm^{-1}

Classification of spreader — $\alpha L_2 = \alpha_2 \times L_{sp2}$ = **0.56 Medium**

Krilov's functions for the spreader length — $B_{\alpha l2} = 1/2 \times (\cosh(\alpha L_2) \times \sin(180 \times \alpha L_2 / \pi) + \sinh(\alpha L_2) \times \cos(180 \times \alpha L_2 / \pi))$ = **0.56**

$C_{\alpha l2} = 1/2 \times \sinh(\alpha L_2) \times \sin(180 \times \alpha L_2 / \pi)$ = **0.16**

$D_{\alpha l2} = 1/4 \times (\cosh(\alpha L_2) \times \sin(180 \times \alpha L_2 / \pi) - \sinh(\alpha L_2) \times \cos(180 \times \alpha L_2 / \pi))$ = **0.03**

Krilov's functions at the start of the load — $B_{\alpha Pstart2} = 1/2 \times (\cosh(\alpha_2 \times P_{start2}) \times \sin(180 \times \alpha_2 \times P_{start2} / \pi) + \sinh(\alpha_2 \times P_{start2}) \times \cos(180 \times \alpha_2 \times P_{start2} / \pi))$ = **0.49**

$C_{\alpha Pstart2} = 1/2 \times \sinh(\alpha_2 \times P_{start2}) \times \sin(180 \times \alpha_2 \times P_{start2} / \pi)$ = **0.12**

Krilov's functions at the end of the load — $B_{\alpha Pend2} = 1/2 \times (\cosh(\alpha_2 \times P_{end2}) \times \sin(180 \times \alpha_2 \times P_{end2} / \pi) + \sinh(\alpha_2 \times P_{end2}) \times \cos(180 \times \alpha_2 \times P_{end2} / \pi))$ = **0.07**

$C_{\alpha Pend2} = 1/2 \times \sinh(\alpha_2 \times P_{end2}) \times \sin(180 \times \alpha_2 \times P_{end2} / \pi)$ = **0.00**

Using method of initial conditions

Initial moment of LH edge — M_{02} = **0** kNm

Initial shear of LH edge — V_{02} = **0** kN

Which gives — $(4 \times \alpha_2^2 \times C_{\alpha l2} \times \delta_{02} + 4 \times \alpha_2 \times D_{\alpha l2} \times \Phi_{02}) \times E_{sp2} \times I_{sp2} - N_{udl2} / \alpha_2^2 \times (C_{\alpha Pstart2} - C_{\alpha Pend2})$ = **0.00** kNm

and — $(4 \times \alpha_2^3 \times B_{\alpha l2} \times \delta_{02} + 4 \times \alpha_2^2 \times C_{\alpha l2} \times \Phi_{02}) \times E_{sp2} \times I_{sp2} - N_{udl2} / \alpha_2 \times (B_{\alpha Pstart2} - B_{\alpha Pend2})$ = **0.00** kN

Therefore,

Initial deflection of LH edge — δ_{02} = **0.43969** mm

Initial rotationof LH edge — Φ_{02} = **0.000005**

Location of maximum deflection — x_{def2} = **150** mm

Krilov's functions at the spreader length — $A_{\alpha xdef2} = \cosh(\alpha_2 \times x_{def2}) \times \cos(180 \times \alpha_2 \times x_{def2} / \pi)$ = **1.00**

$B_{\alpha xdef2} = 1/2 \times (\cosh(\alpha_2 \times x_{def2}) \times \sin(180 \times \alpha_2 \times x_{def2} / \pi) + \sinh(\alpha_2 \times x_{def2}) \times \cos(180 \times \alpha_2 \times x_{def2} / \pi))$ = **0.28**

Distance from start load right of location — $p_{startdef2}$ = **112.5** mm

Krilov's functions at the spreader length — $A_{\alpha pstartdef2} = \cosh(\alpha_2 \times p_{startdef2}) \times \cos(180 \times \alpha_2 \times p_{startdef2} / \pi)$ = **1.00**

Distance from end load right of location — $p_{enddef2}$ = **0** mm

Krilov's functions at the spreader length — $A_{\alpha penddef2} = \cosh(\alpha_2 \times p_{enddef2}) \times \cos(180 \times \alpha_2 \times p_{enddef2} / \pi)$ = **1.00**

Particular integral due to load — $\delta'_2 = (-N_{udl2} / (4 \times \alpha_2^4) \times (A_{\alpha pstartdef2} - A_{\alpha penddef2})) / (I_{sp2} \times E_{sp2})$ = **0.00019**

mm

Maximum deflection — $\delta_{max2} = A_{\alpha xdef2} \times \delta_{02} + B_{\alpha xdef2} \times \Phi_{02} / \alpha_2 + \delta'_2$ = **0.44021** mm

Location of maximum moment — x_{M2} = **150** mm

Krilov's functions at the spreader length — $C_{\alpha xM2} = 1/2 \times \sinh(\alpha_2 \times x_{M2}) \times \sin(180 \times \alpha_2 \times x_{M2} / \pi)$ = **0.04**

$D_{\alpha xM2} = 1/4 \times (\cosh(\alpha_2 \times x_{M2}) \times \sin(180 \times \alpha_2 \times x_{M2} / \pi) - \sinh(\alpha_2 \times x_{M2}) \times \cos(180 \times \alpha_2 \times x_{M2} / \pi))$ = **0.00**

Distance from start load right of location — $p_{startM2}$ = **112.5** mm

Krilov's functions at the spreader length $C_{\alpha pstartM2} = 1/2 \times \sinh(\alpha_2 \times p_{startM2}) \times \sin(180 \times \alpha_2 \times p_{startM2} / \pi) = \mathbf{0.02}$

Distance from end load right of location $p_{endM2} = \mathbf{0}$ mm

Krilov's functions at the spreader length $C_{\alpha pendM2} = 1/2 \times \sinh(\alpha_2 \times p_{endM2}) \times \sin(180 \times \alpha_2 \times p_{endM2} / \pi) = \mathbf{0.00}$

Particular integral due to load $M'_2 = -N_{udl2} / \alpha_2^2 \times (C_{\alpha pstartM2} - C_{\alpha pendM2}) = \mathbf{-1.00}$ kNm

Maximum moment $M_{Edsp2} = (4 \times \alpha_2^2 \times C_{\alpha xM2} \times \delta_{02} + 4 \times \alpha_2 \times D_{\alpha xM2} \times \Phi_{02}) \times (I_{sp2} \times E_{sp2}) + M'_2 = \mathbf{0.33}$ kNm

Location of maximum shear $x_{V2} = \mathbf{37.5}$ mm

Krilov's functions at the spreader length $B_{\alpha xV2} = 1/2 \times (\cosh(\alpha_2 \times x_{V2}) \times \sin(180 \times \alpha_2 \times x_{V2} / \pi) + \sinh(\alpha_2 \times x_{V2}) \times \cos(180 \times \alpha_2 \times x_{V2} / \pi)) = \mathbf{0.07}$

$C_{\alpha xV2} = 1/2 \times \sinh(\alpha_2 \times x_{V2}) \times \sin(180 \times \alpha_2 \times x_{V2} / \pi) = \mathbf{0.00}$

Distance from start load right of location $p_{startV2} = \mathbf{0}$ mm

Krilov's functions at the spreader length $B_{\alpha pstartV2} = 1/2 \times (\cosh(\alpha_2 \times p_{startV2}) \times \sin(180 \times \alpha_2 \times p_{startV2} / \pi) + \sinh(\alpha_2 \times p_{startV2}) \times \cos(180 \times \alpha_2 \times p_{startV2} / \pi)) = \mathbf{0.00}$

Distance from end load right of location $p_{endV2} = \mathbf{0}$ mm

Krilov's functions at the spreader length $B_{\alpha pendV2} = 1/2 \times (\cosh(\alpha_2 \times p_{endV2}) \times \sin(180 \times \alpha_2 \times p_{endV2} / \pi) + \sinh(\alpha_2 \times p_{endV2}) \times \cos(180 \times \alpha_2 \times p_{endV2} / \pi)) = \mathbf{0.00}$

Particular integral due to load $V'_2 = -N_{udl2} / \alpha_2 \times (B_{\alpha pstartV2} - B_{\alpha pendV2}) = \mathbf{0.00}$ kN

Maximum shear $V_{Edsp2} = (4 \times \alpha_2^3 \times B_{\alpha xV2} \times \delta_{02} + 4 \times \alpha_2^2 \times C_{\alpha xV2} \times \Phi_{02}) \times (I_{sp2} \times E_{sp2}) + V'_2 = \mathbf{4.42}$ kN

Maximum allowable stress under spreader $\sigma_{Rdsp2} = \beta_2 \times f_d = \mathbf{0.90}$ N/mm^2

Maximum reaction $N_{Edsp2} = K_{c2} \times \delta_{max2} = \mathbf{118.05}$ kN/m

Design stress $\sigma_{Edsp2} = N_{Edsp2} / w_{sp2} \times (1 + 6 \times e_{sp2} / w_{sp2}) = \mathbf{0.73}$ N/mm^2

PASS - Design stress under spreader is less than the allowable bearing stress

Walls subjected to mainly vertical loading - Section6.1.2

Eccentricity of permanent UDL at mid-height below concentrated load

$e_{gmu2} = e_{gu} \times h_{c2} / (2 \times h) = \mathbf{0.0}$ mm

Eccentricity of variable UDL at mid-height below concentrated load

$e_{qmu2} = e_{qu} \times h_{c2} / (2 \times h) = \mathbf{0.0}$ mm

Eccentricity of concentrated load at mid-height $e_{mc2} = e_{c2} / 2 = \mathbf{7.5}$ mm

Initial eccentricity - cl.5.5.1.1(4) $e_{init} = h / 450 = \mathbf{5.0}$ mm

Concentrated load at mid-height as UDL $N_{mc2} = N_{Edc2} / l_{efm2} = \mathbf{23.05}$ kN/m

Vertical load at mid-height $N_{Ed2} = (g_k + \gamma \times t \times (h - h_{c2} / 2)) \times \gamma_{fG} + q_k \times \gamma_{fQ} + N_{mc2} = \mathbf{29.26}$ kN/m

Design moment at mid-height $M_{Ed2} = g_k \times \gamma_{fG} \times e_{gmu2} + q_k \times \gamma_{fQ} \times e_{qmu2} + N_{mc2} \times e_{mc2} = \mathbf{0.17}$ kNm/m

Eccentricities due to loads - eq. 6.7 $e_{m2} = \mathrm{Abs}(M_{Ed2}) / N_{Ed2} + e_{init} = \mathbf{11.0}$ mm

Slenderness ratio limit for creep eccentricity $\lambda_c = \mathbf{27}$

Eccentricity due to creep $e_{k2} = \mathbf{0.0}$ mm

Eccentricity at mid-height - eq. 6.6 $e_{mk2} = \mathrm{Max}(e_{m2} + e_{k2}, 0.05 \times t) = \mathbf{11.3}$ mm

From eq. G2 $A_{12} = 1 - 2 \times e_{mk2} / t = \mathbf{0.90}$

From eq. G3 $u_2 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk2} / t) = \mathbf{0.38}$

Capacity reduction factor - eq. G1 $\Phi_{m2} = A_{12} \times \exp(-(u_2^2) / 2) = \mathbf{0.84}$

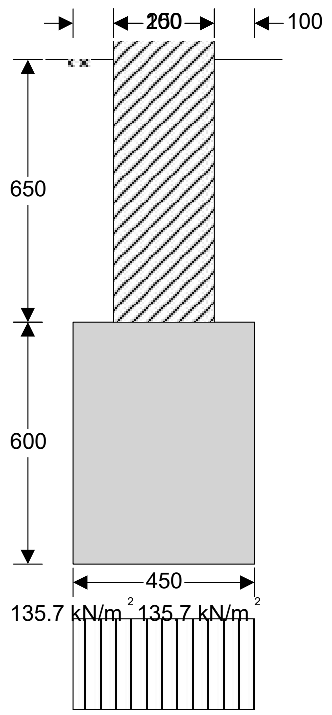
Design vertical resistance of panel - eq.6.2 $N_{Rd2} = \Phi_{m2} \times t \times f_d = \mathbf{169.82}$ kN/m

PASS - Design value of vertical resistance exceeds applied vertical load

8.0 STRIP FOOTING ANALYSIS & DESIGN (BS8110)

STRIP FOOTING ANALYSIS AND DESIGN (BS8110-1:1997)

Tedds calculation version 2.0.07



Strip footing details

Width of strip footing — B = **450** mm

Depth of strip footing — h = **600** mm

Depth of soil over strip footing — h_{soil} = **650** mm

Density of concrete — ρ_{conc} = **23.6** kN/m^3

Load details

Load width — b = **250** mm

Load eccentricity — e_P = **0** mm

Soil details

Density of soil — ρ_{soil} = **20.0** kN/m^3

Design shear strength — ϕ' = **25.0** deg

Design base friction — δ = **19.3** deg

Allowable bearing pressure (preloaded assumed) — $P_{bearing}$ = **150** kN/m^2

Axial loading on strip footing

Dead axial load — P_G = **36.4** kN/m

Imposed axial load — P_Q = **6.5** kN/m

Wind axial load — P_W = **1.5** kN/m

Total axial load — P = **44.4** kN/m

Foundation loads

Dead surcharge load — F_{Gsur} = **10.000** kN/m^2

Imposed surcharge load — F_{Qsur} = **0.000** kN/m^2

Strip footing self weight — $F_{swt} = h \times \rho_{conc}$ = **14.160** kN/m^2

Soil self weight — $F_{soil} = h_{soil} \times \rho_{soil}$ = **13.000** kN/m^2

Total foundation load $F = B \times (F_{Gsur} + F_{Qsur} + F_{swt} + F_{soil}) = \mathbf{16.7}$ kN/m

Calculate base reaction

Total base reaction $T = F + P = \mathbf{61.1}$ kN/m

Eccentricity of base reaction in x $e_T = (P \times e_P + M + H \times h) / T = \mathbf{0}$ mm

Base reaction acts within middle third of base

Calculate base pressures

$q_1 = (T / B) \times (1 - 6 \times e_T / B) = \mathbf{135.716}$ kN/m^2

$q_2 = (T / B) \times (1 + 6 \times e_T / B) = \mathbf{135.716}$ kN/m^2

Minimum base pressure $q_{min} = \min(q_1, q_2) = \mathbf{135.716}$ kN/m^2

Maximum base pressure $q_{max} = \max(q_1, q_2) = \mathbf{135.716}$ kN/m^2

PASS - Maximum base pressure is less than allowable bearing pressure

Material details

Characteristic strength of concrete $f_{cu} = \mathbf{30}$ N/mm^2

Calculate base lengths

Left hand length $B_L = B / 2 + e_P = \mathbf{225}$ mm

Right hand length $B_R = B / 2 - e_P = \mathbf{225}$ mm

Calculate rate of change of base pressure

Length of base reaction $B_x = B = \mathbf{450}$ mm

Rate of change of base pressure $C_x = (q_1 - q_2) / B_x = \mathbf{0.000}$ kN/m^2/m

Calculate minimum depth of unreinforced strip footing

Average pressure to left of strip footing $q_L = q_1 - C_x \times (B_L - b / 2) / 2 = \mathbf{135.716}$ kN/m^2

Minimum depth to left of strip footing $h_{Lmin} = (B_L\text{-}b/2)\times\max(0.15\times[(q_L/1\text{ kN/m}^2)^2/(f_{cu}/1\text{ N/mm}^2)]^{1/4},1) = \mathbf{100}$ mm

Average pressure to right of strip footing $q_R = q_2 + C_x \times (B_R - b / 2) / 2 = \mathbf{135.716}$ kN/m^2

Minimum depth to right of strip footing $h_{Rmin} = (B_R\text{-}b/2)\times\max(0.15\times[(q_R/1\text{kN/m}^2)^2/(f_{cu}/1\text{N/mm}^2)]^{1/4},1) = \mathbf{100}$ mm

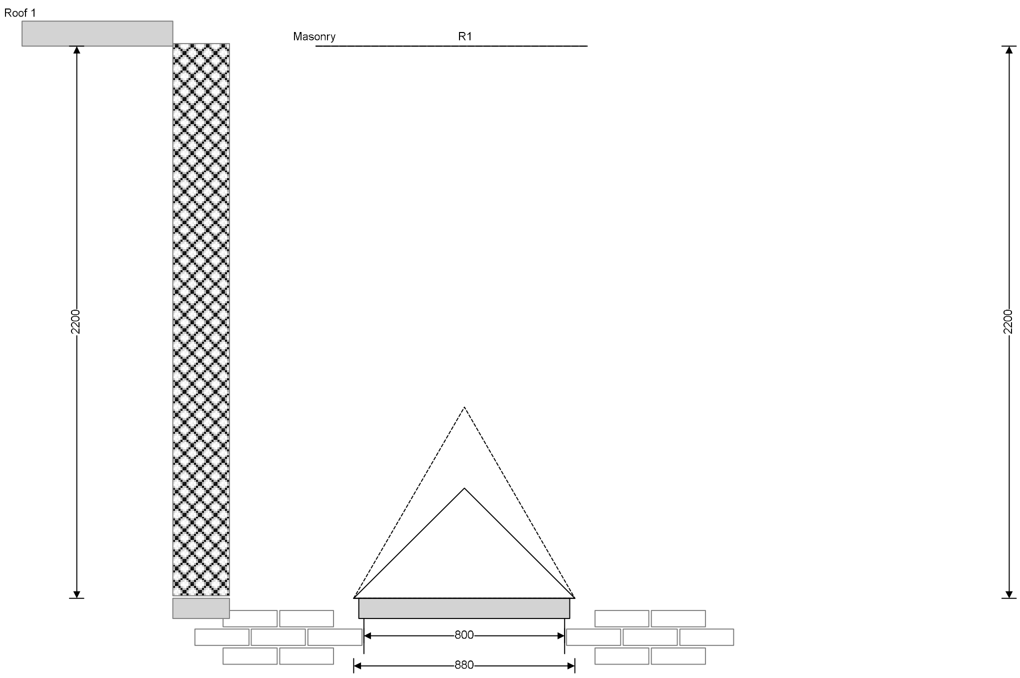
Minimum depth of unreinforced strip footing $h_{min} = \max(h_{Lmin}, h_{Rmin}, 300\text{ mm}) = \mathbf{300}$ mm

PASS - Unreinforced strip footing depth is greater than minimum

9.0 TWO STOREY WITH OPENINGS EXAMPLE

TWO STOREY WITH OPENINGS

Lintel analysis in accordance with BS5977-1:1981 incorporating Amendment No. 1



Basic lintel dimensions

Lintel clear span — L_{c1} = **800** mm

Lintel load application length — $L = L_{c1} \times 1.1$ = **880** mm

Load zone height — $h_{LZ} = \tan(45) \times L / 2$ = **440** mm

Interaction zone height — $h_{IZ} = \tan(60) \times L / 2$ = **762** mm

Load factors

Dead load factor — LF_d = **1.40**

Imposed load factor — LF_I = **1.60**

Masonry

Masonry height — h_m = **2200** mm

Leaf 1

Masonry density — γ_{mi} = **20.00** kN/m^3

Masonry thickness — t_{wi} = **225** mm

Load at midspan — $w_{mi} = h_{LZ} \times t_{wi} \times \gamma_{mi}$ = **1.98** kN/m

Roof loading side 1

Height of roof above lintel — h_{r1} = 2200 mm

Dead load — G_{kr1} = 2.100 kN/m

Imposed load — Q_{kr1} = 2.840 kN/m

Lintel self weight

Self weight of lintel — w_{lsw} = **0.150** kN/m

Masonry load zone

Height of load zone h_{LZ} = L / 2 = **440** mm

Total masonry area $A_{LZ} = h_{LZ} \times L / 2$ = **0.194** m^2

Total masonry load $W_{LZ} = A_{LZ} \times t_{wi} \times \gamma_{mi}$ = **0.871** kN

Equivalent UDL $w_{Equiv\_LZ} = W_{LZ} \times 1.33 / L$ = **1.317** kN/m

Load application summary

Load Description	UDL total length (mm)	Start of UDL on lintel (mm)	End of UDL on lintel (mm)	Equiv. dead load on lintel (kN/m)	Equiv. imposed load on lintel (kN/m)
Masonry from load triangle	880	0	880	1.317	0.000

Analysis results at ULS

Maximum moment M_{max} = **0.199** kNm

Maximum shear V_{max} = **0.702** kN

Maximum reaction at support A R_{A_max} = **0.702** kN

Maximum reaction at support B R_{B_max} = **0.702** kN

Support reactions at SLS

Dead loads

Reaction at support A R_{A_DL} = **0.502** kN

Reaction at support B R_{B_DL} = **0.502** kN

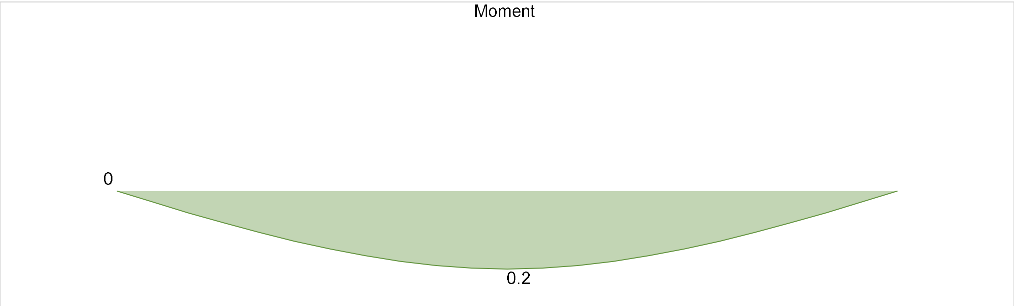
Imposed loads

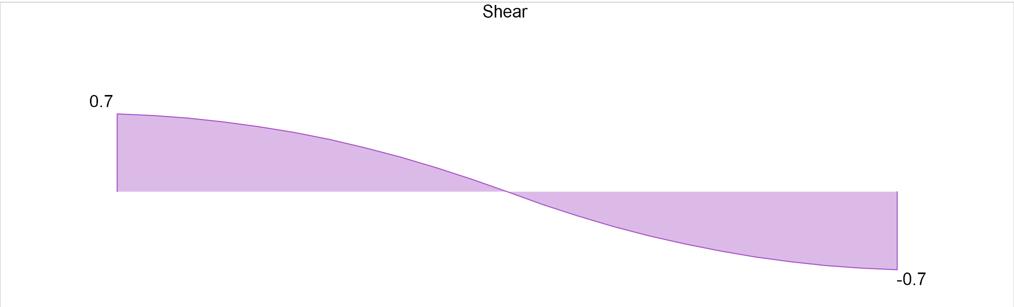
Reaction at support A R_{A_IL} = **0.000** kN

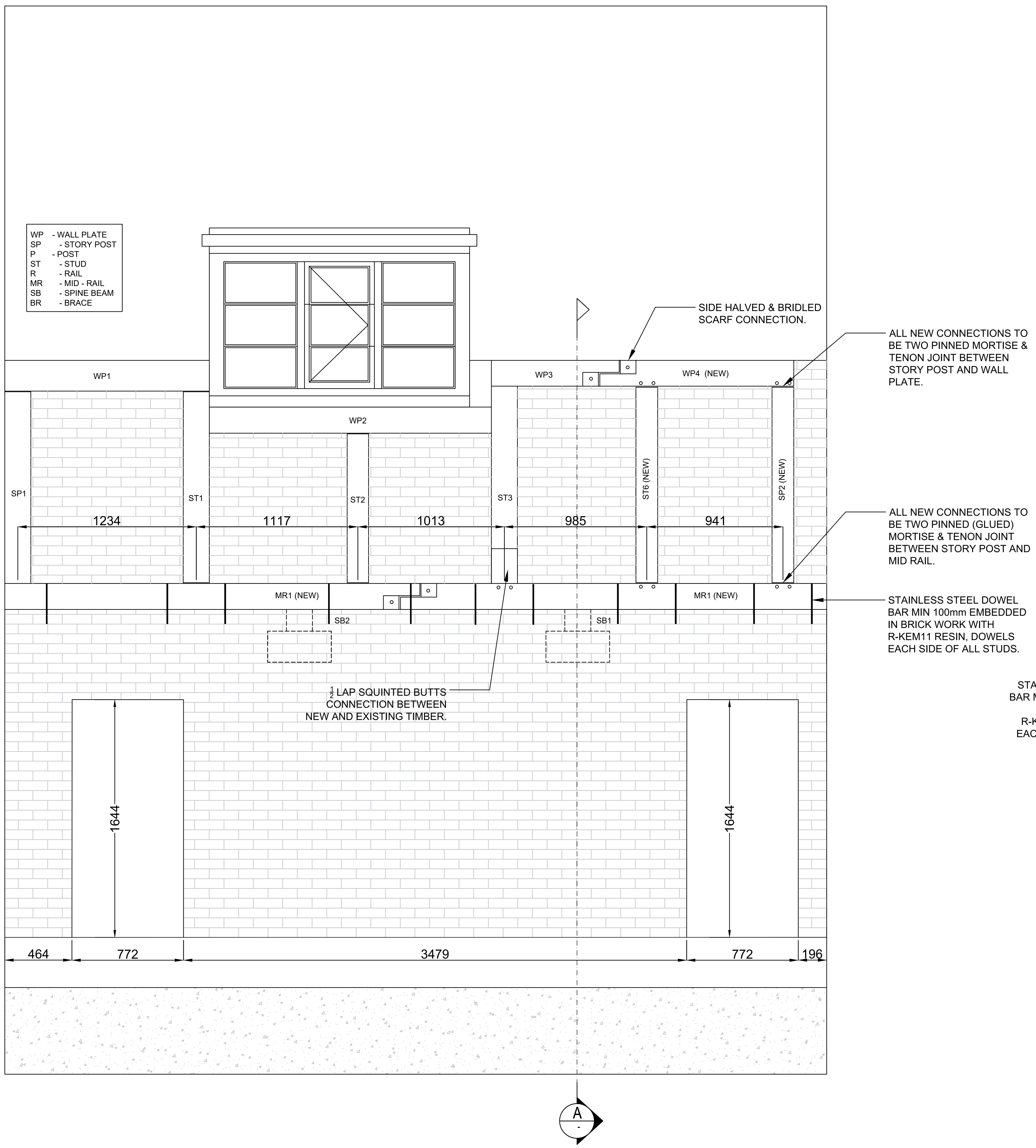
Reaction at support B R_{B_IL} = **0.000** kN

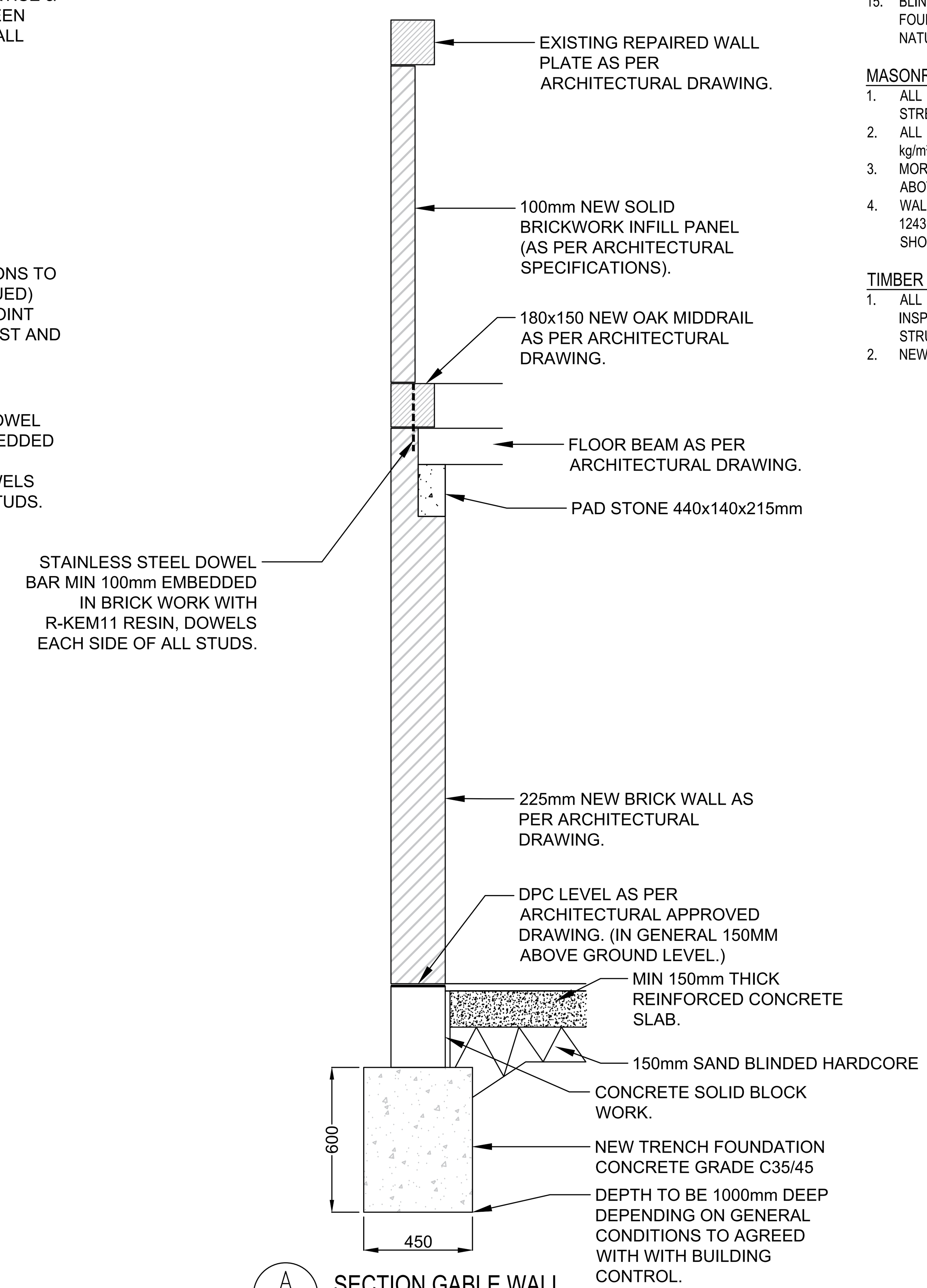
Equivalent UDL at SLS

Total equivalent UDL (inc. selfweight) w_e = **1.467** kN/m









A
SECTION GABLE WALL
NTS.

FOUNDATION WORK

1. TWP DRAWINGS ARE TO BE READ IN CONJUNCTION WITH THE EARTHWORK SPECIFICATION AND THE GENERAL CONCRETE SPECIFICATION.
2. UNLESS NOTED OTHERWISE, FOUNDATIONS ARE TO BE CENTERED UNDER THE WALLS.
3. THE FOUNDATIONS HAVE BEEN DESIGNED TO BEAR ON FIRM CLAY ON THE BASIS OF AN ALLOWABLE BEARING PRESSURE OF 150 kN/m² AND CONSIDERED EXISTING WALL FOUNDATION TO BE REMOVED.
4. ALL FORMED CONCRETE STRIP FOOTINGS HAVE BEEN DESIGNED ASSUMING MASS CONCRETE UPFILL FROM COMPACTED CLAY SUB STRATA, UNLESS OTHERWISE NOTED.
5. CONCRETE GRADES IN FOUNDATIONS TO BE: STRUCTURAL STRIP FOOTINGS - DESIGNED MIX C35/45 - UNREINFORCED.
6. MASS CONCRETE UPFILL - DESIGNATED GEN3 MIX C16/20 - UNREINFORCED CRUST RAFT SLAB - DESIGNED MIX C35/45 - REINFORCED.
7. FORMATION LEVEL TO TOP OF ALL PAD FOUNDATIONS TO BE IN ACCORDANCE WITH THE DRGS.
8. FOUNDATION LEVELS ARE TO BE APPROVED BY TWP STRUCTURAL ENGINEER AND BUILDING CONTROL AUTHORITY. UNSUITABLE MATERIAL IS TO BE REMOVED AS DIRECTED AND REPLACED WITH MASS CONCRETE.
9. ANY STEEL REINFORCEMENT BELOW GROUND TO BE ENCASED IN C35/45 GRADE CONCRETE WITH MINIMUM 75mm COVER OR AS DETAILED ON THE RELEVANT DRGS.
10. SERVICES PASSING THROUGH FOUNDATION TO BE ISOLATED AND PROTECTED IN ACCORDANCE WITH THE SERVICE ENGINEER'S SPECIFICATION.
11. ALL FOUNDATION WORK TO BE IN ACCORDANCE WITH BS EN 1997-1:2004, NA TO BS EN 1997-1:2004 AND BS EN 1997-2:2007. ALL CONCRETE WORK TO BE IN ACCORDANCE WITH BS EN 1992-1-1.
12. ALL EXCAVATIONS TO BE ADEQUATELY PROTECTED FROM THE INGRESS OF WATER. FOUNDATION EXCAVATIONS SHALL BE DRY AND SEALED AT THE EARLIEST AFTER EXCAVATION AND INSPECTION.
13. CARE IS TO BE TAKEN WHEN INSTALLING UNDERGROUND SERVICES TO ENSURE THAT EXCAVATED TRACKS LIE OUTSIDE A 45° SPREAD FROM THE BASE OF THE FOUNDATION.
14. FOUNDATION AND FLOOR SLAB FORMATION LEVELS SHALL BE WELL COMPACTED AND ALL SOFT SPOTS AND BACKFILLED, REMOVED AS AGREED TO IN ACCORDANCE WITH AGREEMENTS REACHED WITH ENGINEER.
15. BLINDING CONCRETE IS REQUIRED UNDER STRIP FOUNDATIONS SINCE THESE ARE TYPICALLY CAST AGAINST NATURAL SOIL FOUNDATIONS.

MASONRY WORK

1. ALL BLOCK WORK TO HAVE MINIMUM COMPRESSIVE STRENGTH OF 7.0 N/ mm² U.N.O.
2. ALL BLOCK WORK TO HAVE A MINIMUM DENSITY OF 1500 kg/m² U.N.O.
3. MORTAR BELOW D.P.C. LEVEL TO BE TYPE I (1:3) & MORTAR ABOVE D.P.C. LEVEL TO BE TYPE III (1:5)
4. WALL TIES AND DOWELS TO BE STAINLESS STEEL TO BS 1243, BS EN 845-1:2003 & BS EN ISO 1461:1999 & SPACED AS SHOWN.

TIMBER WORK

1. ALL EXISTING TIMBER BEAMS, FRAME MEMBER TO BE INSPECTED AND APPROVED BY THE ARCHITECT OR/AND STRUCTURAL ENGINEER BEFORE REUSED.
2. NEW TIMBER AS PER ARCHITECTURAL SPECIFICATIONS.

GENERAL NOTES:

1. ALL DIMENSIONS ARE IN MILLIMETERS (mm) & ALL LEVELS ARE IN METERS (m) UNLESS NOTED OTHERWISE.
2. DO NOT SCALE THIS DRAWING. FOR DISCREPANCIES OR OMISSIONS CONTACT THE STRUCTURAL ENGINEER PRIOR TO WORK COMMENCING.
3. MATERIALS AND WORKMANSHIP ARE TO COMPLY IN ALL RESPECTS WITH CURRENT BRITISH STANDARD SPECIFICATIONS, BRITISH STANDARD CODE OF PRACTICE, AND BUILDING REGULATIONS.
4. THE COPYRIGHT OF THIS DRAWING IS VESTED IN THE ENGINEER AND MUST NOT BE COPIED OR REPRODUCED WITHOUT WRITTEN CONSENT.
5. THE CONTRACTOR IS TO CHECK AND VERIFY ALL BUILDING AND SITE DIMENSIONS, LEVELS AND SEWER INVERT LEVELS AT CONNECTION POINTS BEFORE WORK COMMENCES.
6. THIS DRAWING IS TO READ IN CONJUNCTION WITH ALL RELEVANT SPECIFICATIONS AND DRAWINGS ISSUED BY THE ENGINEER, ARCHITECT AND OTHER SPECIALISTS.
7. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE DESIGN, FABRICATION, ERECTION AND REMOVAL OF ANY TEMPORARY WORKS AND SHALL PROVIDE ALL TEMPORARY BRACING AND PROPPING NECESSARY TO MAINTAIN STABILITY DURING CONSTRUCTION. THE DESIGN ENVISAGED CONSTRUCTION SEQUENCE IS DETAILED ON TWP'S DRGS ETC.
8. CONTRACTOR TO CHECK SERVICE ENGINEER'S DRAWINGS FOR DETAILS OF ANY CAST-IN FIXINGS REQUIRED THAT MAY NOT BE SHOWN ON THE ENGINEER'S STRUCTURAL DRAWINGS. CONTRACTOR TO CO-ORDINATE REQUIREMENTS.
9. IF ANY CAST-IN FIXINGS FOR BRICKWORK AND BLOCK WORK ARE REQUIRED, THE CONTRACTOR SHOULD TAKE COGNIZANCE OF THE ARCHITECTS MASONRY COURSING REQUIREMENTS.
10. DESIGN LOADING: (GRAVITY)
    IMPOSED LOADS:
    1ST. FLOOR 1.5 kN/m²
    STAIR CASE/LANDING 3.0 kN/m²
    ROOF (WITHOUT ACCESS) 0.6 kN/m²
    SNOW LOADS 0.75kN/m²
    SNOW LOADS (DORMER) 1.5 kN/m²
    DEAD LOADING:
    1st FLOOR 1.52kN/m²
    ROOF 1.10kN/m²
    WIND LOADING IN ACCORDANCE WITH BS EN 1991-1-3:2005
    BASIC WIND SPEED 21.70m/sec
11. LOADS TO BE CALCULATED IN ACCORDANCE WITH:
    BS EN 1991-1-1:2002: ACTION ON STRUCTURES. GENERAL ACTIONS-DENSITIES, SELF-WEIGHT, IMPOSED LOADS FOR BUILDINGS.
    BS EN 1991-1-4:2005 ACTIONS ON STRUCTURES - WIND ACTIONS.
12. THE CONTRACTOR SHOULD BE VIGILANT AGAINST THE POTENTIAL FOR PERCHED GROUNDWATER OR/AND DRAIN WATER WHICH MAY BE ENCOUNTERED DURING GROUND WORKS. ANY SOFT UNSTABLE AND MARSHY GROUND CONDITIONS AT FORMATION LEVEL ARE TO BE REMOVED.

REV.	AMENDMENTS	DRW.	CHK.	DATE

CLIENT

TOGETHER PROPERTY MANAGEMENT
MAINTENANCE OFFICE
PO BOX 1319
ENFIELD EN1 9ZJ
TEL.: 020 8366 7070
maintenance@togetherproperty.co.uk

PROJECT

1-6 HAFREN COURT
BEWDLEY DY12 2AR

TITLE

GABLE WALL DESIGN

DRAWN:	IM	DESIGN:	IM
CHECKED:	–	APPROVED:	–
DATE:	MAY 2023	SCALE:	AS SHOWN

CHARTERED CONSULTING ENGINEERS

DRAWING STATUS FOR TENDER

DRAWING NO.	REV.
23001-HC-001	T01