



Project Title:	New Double Garage: Walls and Foundations. Pepper Hill Farm Cottage, North Drove Bank, SPALDING, Lincolnshire PE11 3JX
Client:	Mt & Mrs MacGregor
Report Type:	Structural Design Report
Date:	March 2022
Project No.	PW-22-01-04

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Calculations

REF		OUTPUT

A. INTRODUCTION AND DESIGN DATA

The client wishes to construct a new detached double garage to replace the existing detached single garage. The new garage will have masonry walls. The tiled roof will be supported on timber trusses.

The north elevation is within an estimated 8m of a line of of trees on the adjacent property. These include mature birches and hawthorns. The north-east corner of the new garage will be within 4m of a mature walnut tree which is on the clients property.

NHBC guidelines would lead to foundations a minimum depth of over 2m below ground level. Trenches for such a deep foundation would cut through the roots of the walnut tree and possibly through the roots of the trees on the adjacent property. This would in turn weaken the roots systems of the trees and increase their risk of collapse in high winds.

Hence a reinforced concrete raft foundation will be used to support the new garage.

The following calculations are for the design of the raft foundation.

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Calculations

REF | OUTPUT

A.2 DESIGN DATA

A.2.1 Codes and Standards etc.

Steelwork :	BS5950 - Part 1: 2000
Masonry	BS5628 - Part 1: 2005
Reinforced Concrete	BS8110 - Part 1: 1997
Loading (Dead and Imposed)	BS6399 - Part 1: 1996
Loading (Roof Imposed)	BS6399 - Part 3: 1988

A.2.2 Material Unit Weights in kN/m2

Concrete Roof Tiles	=	0.55 kN/m^2
Felt, Roof Boards and Firrings	=	0.18 kN/m^2
Insulation	=	0.01 kN/m^2
Trusses/Rafters	=	0.19 kN/m^2
Ceilings	=	0.15 kN/m^2
Floor Joists	=	0.20 kN/m^2
Floor Boards	=	0.15 kN/m^2
Floor Finishes	=	0.10 kN/m^2
102 Brickwork	=	1.80 kN/m^2
100 Blockwork	=	1.80 kN/m^2
Wall Finishes	=	0.12 kN/m^2
Stud Partition Walls	=	0.35 kN/m^2

A.2.3 Floor Imposed Loads

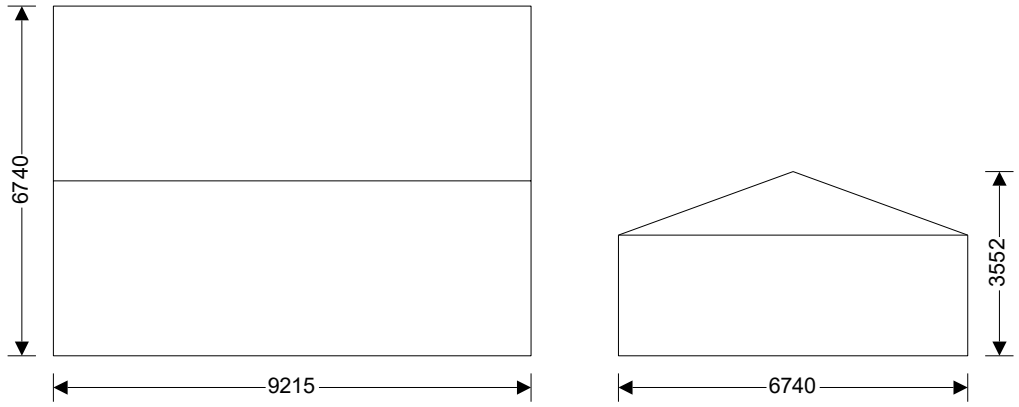
Floor Imposed - Domestic	=	1.50 kN/m^2
Floor Imposed - Office	=	2.50 kN/m^2

REF				OUTPUT
	A.2.4 Wind Loads			
	Use TEDDS to determine the wind load on the building which will be used for the design.			
BS6399-2 Table 1	Building Classification	=	0.5	
	Site Altitude	=	3.00 m AOD	
2.2.2.2	Terrain	=	Insignificant	
1.7.2	Ground Roughness	=	Country	
2.2.2.3	Roof orientation east of north	=	245.0 degrees	
Table 3	Directional factor	=	1.00	
Fig 6	Basic wind Speed	=	23.0 m/s	
	Building Length	=	9.215 m	
	Building Width	=	6.740 m	
	Building Height to Eaves	=	2.325 m	
	Roof Height above Eaves	=	1.227 m	
	The wind loading is given on pages A-A/1 to A-A/6 below.			
	A.2.5 Roof Imposed Loads			
	Roof Area on plan = 9.215 x 6.74 + 0.00 x 0.50	=	31.1 m^2	< 200.0 m^2
BS6399-3 4.3.2	Roof Imposed Load	=	0.75 kN/m^2	
	A.2.6 Floor Imposed Loads			
BS6399-1 Table 1 F	Floor Imposed Load	=	2.50 kN/m^2	Concentrated 9kN
	A.3 COMPUTER SOFTWARE.			
	This calculation will use TEDDS (by Tekla/Trimble) for some of the design works.			

WIND LOADING (BS6399)

In accordance with BS6399

Tedds calculation version 3.0.18



Building data

Type of roof	Duopitch
Length of building	L = **9215** mm
Width of building	W = **6740** mm
Height to eaves	H = **2325** mm
Pitch of roof	α_0 = **20.0** deg
Reference height	H_r = **3552** mm

Dynamic classification

Building type factor (Table 1)	K_b = **1.0**
Dynamic augmentation factor (1.6.1)	$C_r = [K_b \times (H_r / (0.1\ m))^{0.75}] / (800 \times \log(H_r / (0.1\ m))) =$ **0.01**

Site wind speed

Location	Cuckoo Bridge
Basic wind speed (Figure 6 BS6399:Pt 2)	V_b = **23.0** m/s
Site altitude	Δ_S = **2** m
Upwind distance from sea to site	d_{sea} = **25** km
Direction factor	S_d =**1.00**
Seasonal factor	S_s = **1.00**
Probability factor	S_p = **1.00**
Critical gap between buildings	g = **5000** mm
Topography not significant	
Altitude factor	$S_a = 1 + 0.001 \times \Delta_S / 1m =$ **1.00**
Site wind speed	$V_s = V_b \times S_a \times S_d \times S_s \times S_p =$ **23.0** m/s
Terrain category	Country
Displacement height (sheltering effect excluded)	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 (at which q is sought)	H_{ref} = **2325**mm
Effective height	$H_e = \max(H_{ref} - H_d, 0.4 \times H_{ref})$ = **2325**mm
Fetch factor (Table 22)	S_c = **0.782**
Turbulence factor (Table 22)	S_t = **0.213**
Gust peak factor	g_t = **3.44**
Terrain and building factor	$S_b = S_c \times (1 + (g_t \times S_t) + S_h)$ = **1.35**
Effective wind speed	$V_e = V_s \times S_b$ = **31.2** m/s
Dynamic pressure	q_s = 0.613 kg/m^3 × V_e^2 = **0.597** kN/m^2

Dynamic pressure - windward wall - Wind 90 deg and roof

Reference height (at which q is sought)	H_{ref} = **3552**mm
Effective height	$H_e = \max(H_{ref} - H_d, 0.4 \times H_{ref})$ = **3552**mm
Fetch factor (Table 22)	S_c = **0.851**
Turbulence factor (Table 22)	S_t = **0.203**
Gust peak factor	g_t = **3.44**
Terrain and building factor	$S_b = S_c \times (1 + (g_t \times S_t) + S_h)$ = **1.45**
Effective wind speed	$V_e = V_s \times S_b$ = **33.3** m/s
Dynamic pressure	q_s = 0.613 kg/m^3 × V_e^2 = **0.680** kN/m^2

Size effect factors

Diagonal dimension for gablewall	a_{eg} = **7.1** m
External size effect factor gablewall	C_{aeg} = **0.973**
Diagonal dimension for side wall	a_{es} = **9.5** m
External size effect factor side wall	C_{aes} = **0.952**
Diagonal dimension for roof	a_{er} = **9.9** m
External size effect factor roof	C_{aer} = **0.949**
Room/storey volume for internal size effect factor	V_i = **0.125** m^3
Diagonal dimension for internal size effect factors	$a_i = 10 \times (V_i)^{1/3}$ = **5.000** 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)	-0.90	0.68	0.949	-0.72	5.37	-3.85
B (-ve)	-0.70	0.68	0.949	-0.59	1.60	-0.94
C (-ve)	-0.33	0.68	0.949	-0.35	26.08	-9.15
E (-ve)	-1.17	0.68	0.949	-0.89	5.37	-4.77
F (-ve)	-0.77	0.68	0.949	-0.63	1.60	-1.01
G (-ve)	-0.50	0.68	0.949	-0.46	26.08	-11.96

Total vertical net force	$F_{w,v}$ = **-29.76** kN
Total horizontal net force	$F_{w,h}$ = **1.30** 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	-1.48	0.68	0.973	-1.11	3.67	-4.09
B	-0.86	0.68	0.973	-0.70	16.13	-11.37
w	0.69	0.60	0.952	0.27	21.42	5.86
l	-0.50	0.60	0.952	-0.40	21.42	-8.64

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-8.6** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **5.9** kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **13.6** 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)	0.40	0.68	0.949	0.46	5.37	2.48
B (+ve)	0.30	0.68	0.949	0.40	1.60	0.63
C (+ve)	0.27	0.68	0.949	0.38	26.08	9.81
E (+ve)	-1.17	0.68	0.949	-0.55	5.37	-2.94
F (+ve)	-0.77	0.68	0.949	-0.29	1.60	-0.46
G (+ve)	-0.50	0.68	0.949	-0.12	26.08	-3.09

Total vertical net force $F_{w,v}$ = **6.03** kN

Total horizontal net force $F_{w,h}$ = **6.64** 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	-1.48	0.68	0.973	-0.77	3.67	-2.84
B	-0.86	0.68	0.973	-0.36	16.13	-5.88
w	0.69	0.60	0.952	0.57	21.42	12.25
l	-0.50	0.60	0.952	-0.10	21.42	-2.25

Overall loading

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

Net windward force for overall section $F_w = F_{w,ww}$ = **12.3** kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **18.2** 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)	-1.47	0.68	0.949	-1.08	2.42	-2.61
B (-ve)	-1.37	0.68	0.949	-1.02	2.42	-2.46
C (-ve)	-0.60	0.68	0.949	-0.52	19.34	-10.11
D (-ve)	-0.43	0.68	0.949	-0.42	41.92	-17.42

Total vertical net force $F_{w,v}$ = **-30.64** 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	-1.30	0.60	0.952	-0.86	2.16	-1.85
B	-0.80	0.60	0.952	-0.57	8.65	-4.96
C	-0.50	0.60	0.952	-0.40	10.61	-4.28
w	0.72	0.68	0.973	0.34	19.80	6.70
l	-0.50	0.68	0.973	-0.47	19.80	-9.25

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-9.2** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **6.7** kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **13.7** 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)	-1.47	0.68	0.949	-0.74	2.42	-1.79
B (-ve)	-1.37	0.68	0.949	-0.68	2.42	-1.64
C (-ve)	-0.60	0.68	0.949	-0.18	19.34	-3.54
D (-ve)	-0.43	0.68	0.949	-0.08	41.92	-3.16

Total vertical net force $F_{w,v}$ = **-9.52** kN

Total horizontal net force $F_{w,h}$ = **0.00** kN

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

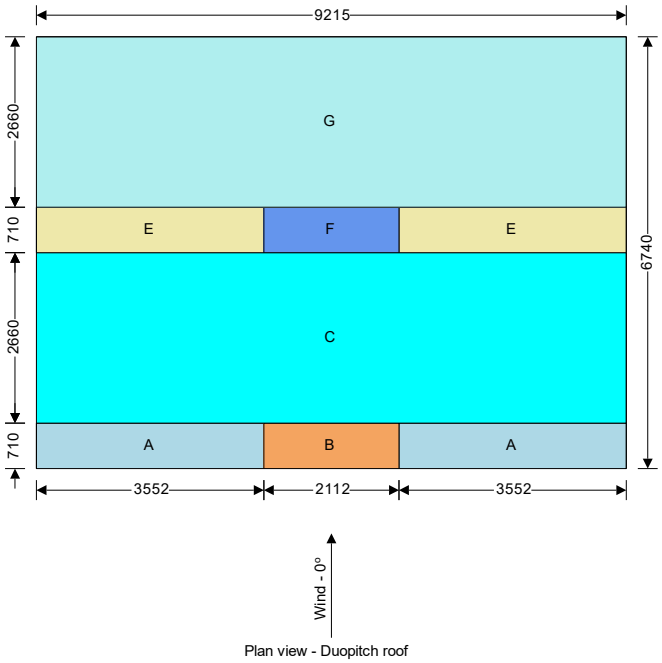
Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A	-1.30	0.60	0.952	-0.56	2.16	-1.21
B	-0.80	0.60	0.952	-0.28	8.65	-2.38
C	-0.50	0.60	0.952	-0.10	10.61	-1.11
w	0.72	0.68	0.973	0.68	19.80	13.44
l	-0.50	0.68	0.973	-0.13	19.80	-2.51

Overall loading

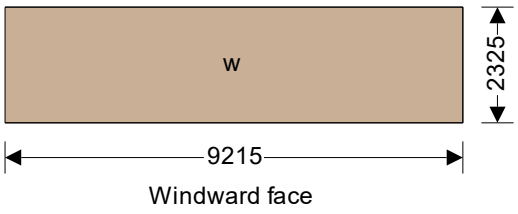
Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-2.5** kN

Net windward force for overall section $F_w = F_{w,ww} = \mathbf{13.4}$ kN

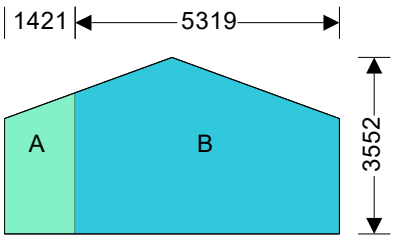
Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h}) = \mathbf{13.7}$ kN



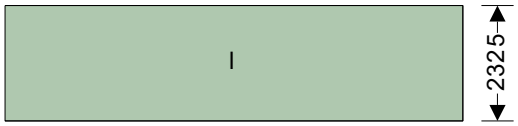
Plan view - Duopitch roof



Windward face

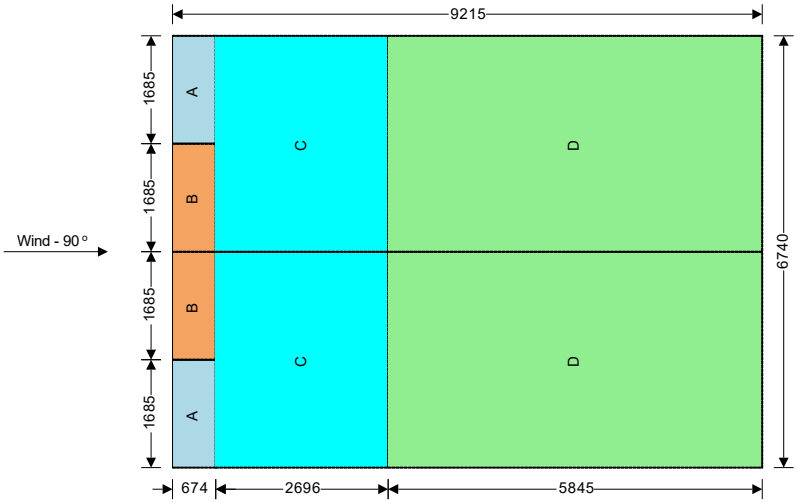


Side face

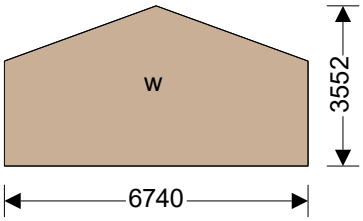


Leeward face

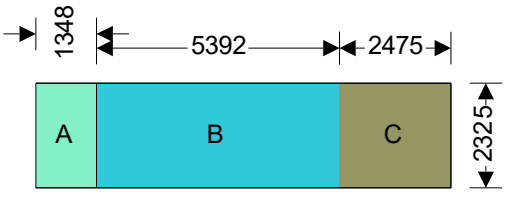
PW Structural Solutions Ltd 19 Coleridge Place, Bourne Lincolnshire, PE10 9WA	Project Pepper Hill Farm Cottage - New Double Garage			Job no. PW-22-01-04	
	Calcs for Wind Loading			Start page no./Revision A/ 6	
	Calcs by P D Wood	Calcs date 14/02/2022	Checked by	Checked date	Approved by	Approved date



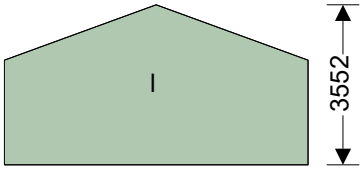
Plan view - Duopitch roof



Windward face



Side face



Leeward face

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Calculations

REF		OUTPUT

B ROOF

The weight of the existing gable end wall and any loads applied to it need to be determined to enable the support structure to be calculated.

B.1 Existing Roof

Roof Construction:

	Roof Height		=	1.227 m
	Roof Span		=	6.740 m
	Roof Pitch = Tan-1	1.227 /(6.740 / 2)	=	20.000 deg
A/ 2	Roof Unit Weight:	Concrete Roof Tiles	=	0.55 kN/m^2
A/ 2		Insulation	=	0.01 kN/m^2
A/ 2		Trusses/Rafters	=	0.19 kN/m^2
A/ 2		Ceilings	=	0.15 kN/m^2
		Total	=	0.90 kN/m^2
	Load on Plan =	0.90 / Cos 20.000	=	0.96 kN/m^2
	Dead Load on Walls =	6.740 x 0.96 / 2	=	**3.23 kN/m**
A/ 3	Live Load on Walls =	6.740 x 0.75 / 2	=	**2.53 kN/m**

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Calculations

REF		OUTPUT
	C WALLS	
	C.1 WALLS GENERALLY.	
	The architect has specified a masonry wall with piers. Initially calculate thickness and pier size for a solid wall. Then re-assess the panels with penetrations.	
	C.1.1 Long Walls	
	Use TEDDS to claculate the wall capacity. Initially try with the architects arrangement.	
	Wall Height (Nett - Raft Foundation assumed) = 2.175 m	
	Wall Length = 9.215 - 0.105 - 0.05 = 9.060 m	
	Average Pier Spacing = 9.060 / 4 = 2.265 m	
B/ 1	Dead Load on Wall = 3.23 kN/m^2	
B/ 1	Live Load on wall = 2.53 kN/m	
A-A/ 3	Wind Load on wall = 0.57 kN/m	
	Wall Thickness = 100 mm	
	Pier Width = 440 mm	
	Pier Depth = 220 mm	
	Blockwork Characteristic Strength = 10.40 N/mm^2	
	From the TEDDS output given on pages A-C/1 to A-C/3 below, the wall fails in	
A-C/ 3	bending. Wall utilization ratio = 0.472 / 0.265 = 1.781	> 1.00
	Try increasing wall thickness and the number of piers.	
	Wall Thickness = 140 mm	
	Pier Width = 440 mm	
	Pier Depth = 280 mm	

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Calculations

REF		OUTPUT
	Average Pier Spacing = 9.060 / 6 = 1.510 m	
	Maximum Pier Spacing = 1.248 + 0.440 = 1.688 m	
	From the TEDDS output given on pages A-C/1 to A-C/3 below	
A-C/ 5	Wall utilization ratio = 9.060 / 10.100 = 0.897	< 1.00
	For Side Walls **Block Strength** = **10.4** **N/mm²**	
	Block Thickness = **140** **mm**	
	Pier Size = **440** **mm x** **280** **mm**	
	Pier Spacing (max) = **1688** **mm**	
	C.1.2 Rear Gable End Wall	
	Wall Height (Average) = 2.815 m	
	Wall Length = 6.740 - 0.140 - 0.00 = 6.600 m	
	Average Pier Spacing = 6.600 / 3 = 2.200 m	
A-A/ 3	Basic Wind Load on wall	
A-A/ 3	Zone A Nett wind load = 1.110 - 0.700 = 0.41 kN/m²	
	Zone A Load = 0.41 x 1.421 = 0.58 kN/m	
A-A/ 5	Zone A Load Position = 5.319 +(1.421 / 2) = 6.030 m	
A-A/ 5	Maximum Reaction = 0.58 x 6.030 / 6.740 = 0.52 kN/m	
	Maximum Moment = 0.58 x 6.030 x 0.711 / 6.740 = 0.37 kNm/m	
	Equivalent uniform load = 0.58 x 8 /(6.740 x 6.740)= 0.10 kN/m²	
	Gable Wall Design Wind Load = 0.70 + 0.10 = 0.803 kN/m²	
	From the TEDDS output given on pages A-C/6 to A-C/7 below, the wall fails in	
A-C/ 8 & 9	bending. Wall utilization ratio = 0.823 / 0.333 = 2.471	> 1.00
	Try increasing wall thickness and the number of piers.	

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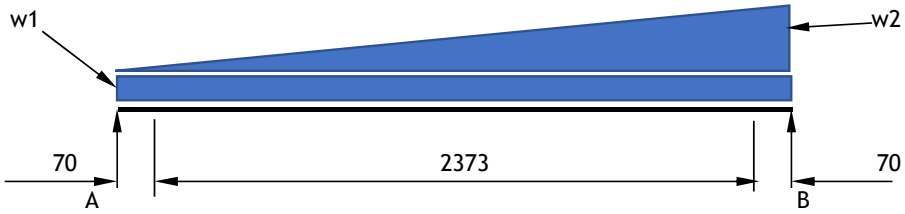
Calculations

REF		OUTPUT
	Wall Thickness = 140 mm	
	Pier Width = 440 mm	
	Pier Depth = 280 mm	
	Average Pier Spacing = 6.600 / 5 = 1.320 m	
	From the TEDDS output given on pages A-C/1 to A-C/3 below	
A-C/ 12	Wall utilization ratio = 0.764 / 0.817 = 0.935	< 1.00
	For Rear Gable Wall **Block Strength** = **10.4** **N/mm²**	
	Block Thickness = **140** **mm**	
	Pier Size = **440** **mm x** **280** **mm**	
	Pier Spacing (max) = **1320** **mm**	
	C.1.3 Front Gable End Wall	
	Consider this as a wall panel supported on three masonry columns. Each of the columns will need to support. Initially check a 1305 x 6740 panel.	
	C.1.3.1 Panel Over Doors	
C/ 2	Design wind load = 0.803 kN/m²	
	From the TEDDS output given on pages A-C/12 to A-C/15 below:	
	For Front Gable Wall **Block Strength** = **10.4** **N/mm²**	
	Block Thickness = **140** **mm**	
	Bed Joint Reinforcement = **5.0** **mm diameter**	
	Ladder Width = **60** **mm**	
	Vertical Spacing = **225** **mm**	
	C.1.3.2 Lintels Over Doors Span = 2.373 m	Clear
	Consider using pre-cast concrete lintels	

PW STRUCTURAL SOLUTIONS LTD

Calculations

REF		OUTPUT
A/ 2	Load at apex = 1.305 x 1.800 x 140.0 / 100.0 = 3.289 kN/m	
	Height at Eaves = 1.305 - 2.373 TAN 20.00 = 0.441 m	
A/ 2	Load at Eaves = 0.441 x 1.800 x 140.0 / 100.0 = 1.112 kN/m	



w1 = 1.112 kN/m

w2 = 3.289 - 1.112 = 2.177 kN/m

R_B =(1.112 x 2.513 x 2)+(2.177 x 2.513 / 3) = 3.221 kN

R_A =(1.112 x 2.513)+(2.177 x 2.513 / 2) - 3.221 = 2.309 kN

For lintel selection, let load on lintel = 3.221 x 2 = 6.441 kN

Hi-Spec Range		R3	S5	R8	R11
Load Table Units suitable for 140mm wide walls		100 / 140	140 / 140	215 / 140	290 / 140
Fire Resistance Available (mins)		30	30	30	30
Suitable for Foundation Use		yes	yes	yes	yes
Maximum Stock Length Available		3000mm	3600mm	3600mm	3600mm
				Longer lengths available on request - up to 4800mm	
Available Range Finish		Faced	Faced	Faced	Faced
		UNFACTORED LOADS IN kN/m			
Length	Clear Span	140x100	140x140	140x215	140x290
900mm	700mm	49.17	37.80	100.57	128.00
1100mm	900mm	33.04	29.63	80.31	102.20
1200mm	1000mm	27.08	26.30	72.95	98.82
1500mm	1200mm	19.11	19.70	62.22	77.58
1800mm	1500mm	12.38	14.90	41.60	63.30
2100mm	1800mm	8.62	11.50	29.53	51.38
2400mm	2100mm	6.31	8.90	21.97	38.35
2700mm	2400mm	4.79	6.80	16.93	29.64
3000mm	2700mm	n/a	5.10	13.40	23.53
3300mm	3000mm	n/a	3.70	10.83	19.09
3600mm	3200mm	n/a	2.40	9.49	16.25
Lintel Weight kg/m		34	47	72	97

From Table Above

Use NAYLOR S5 or NAYLOR R8 Lintels

PW STRUCTURAL SOLUTIONS LTD

Calculations

REF		OUTPUT
	C.1.3.3 Lintel Bearing Minimum Bearing Length = 0.150 m	
	Use TEDDS to check bearing requirements	
	Thickness of Wall = 140 mm	
	Strength of Wall = 10.40 N/mm^2	
	Width of bearing = 140 mm	
	Length of Bearing = 150 mm	
	Eccentricity of Bearing = 0 mm	
C/ 4	Loads from beam Dead = 3.221 kN	
	Live = 0.00 kN	
	Loads from wall Above Dead = 0.000 kN/m	
	Live = 0.00 kN/m	
	From the output given on pages A-C/16 to A-C/17 below:	
	NO Spreader Required	
	C.1.3.4 Central Masonry Column Height = 2.100 m	
	Width Supported by Column = 2.373 + 0.665 = 3.038 m	
	Column Axial Load = 3.221 x 2 = 6.441 kN	
	Let wind load on column:	
C/ 3	Panel Over Load = 3.038 x 1.305 x 0.803 / 2 = 1.591 kN	
	Moment = 1.591 x 2.100 x 1.000 / 1 = 3.341 kNm	
C/ 3	Panel Below Load = 3.038 x 1.000 x 0.803 / 1 = 2.438 kN/m	
	Moment = 2.438 x 2.100 x 2.100 / 2 = 5.376 kNm	
	Total Moment in Column = 3.341 + 5.376 = 8.718 kNm	
	Vertical Load Eccentricity = 8.718 / 6.441 = 1.353 m	

REF		OUTPUT
	Eccentricity is outside of the width of the column. Hence a timber strut will be required between the top of the column and the point on the apex where the roof bracing meets.	
	Now consider the moment on the simply-supported column	
	Moment = 2.438 x 2.100 x 2.100 / 8 = 1.344 kNm	
	Vertical Load Eccentricity = 1.344 / 6.441 = 0.209 m	
	Try a 670 wide by 440 column	
	Eccentricity of applied vertical load = 0.220 - 0.070 = 0.150 m	
	Total Eccentricity = 0.359 m	
	From the output given on pages A-C/18 to A-C/19 below:	
	Column Fails - Try a deeper column	
	Try a 670 wide by 670 deep column	
	Eccentricity of applied vertical load = 0.335 - 0.070 = 0.265 m	
	Total Eccentricity = 0.474 m	
	From the output given on pages A-C/20 to A-C/21 below:	
	Column Fails - Try Reinforced Column	
	Consider a column formed of hollow blocks with concrete infill and reinforcement.	
C/ 5	Design Moment = 8.718 x 1.600 = 13.95 kNm	
C/ 5	Design Axial = 6.441 x 1.400 = 9.02 kN	
	Design Shear = 13.95 / 2.100 = 6.64 kN	
	Use the TEDDS reinforced concrete column routine to determine the reinforcement required. In the output ignore the results in respect of conatinment links and bras in the side faces.	

REF					OUTPUT
	Let concrete strength		=	10.00 N/mm^2	
	From the output given on pages A-C/22 to A-C/24 below:				
	Column width = **670 mm**				
	Column Depth = **440 mm**				
	Reinforcement = **3H16 Each Face**				
	C.1.4 Lintels Over Windows and Doors	Span	=	0.910 m	Clear
B/ 1	Roof Loads:	Dead	=	3.228 kN/m^2	
B/ 1		Live	=	2.528 kN/m^2	
		Total	=	5.755 kN/m^2	
	Load on Lintel = 5.755 x 0.910		=	5.237 kN	
	Window and door head height		=	2.100 m	
	Top of wall Level (estimated)		=	2.250 m	
	Depth of Lintel		=	0.150 m	
	From the table on page C/4, and by inspection:				
	Use NAYLOR S5 Lintels over personnel door and windows				
	N.B By comparison with the lintel bearing check in section 1.3.3 above				
	No Spreader Required.				

MASONRY WALL PANEL DESIGN

In accordance with BS5628-1:2005

Tedds calculation version 1.2.15

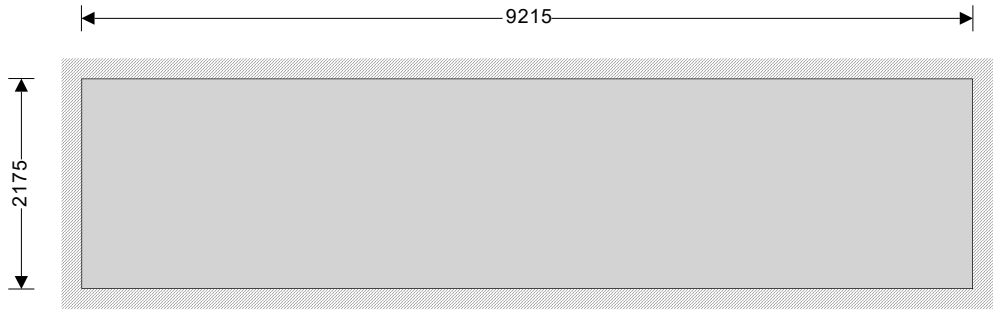
Masonry panel details

Garage Long wall - Unreinforced masonry wall without openings

Panel length	L = **9215** mm
Panel height	h = **2175** mm

Panel support conditions

	All edges supported, right and left continuous
Effective panel length	L_{ef} = 1.0 × L = **9215** mm
Effective panel height	h_{ef} = 1.0 × h = **2175** mm



Pier details

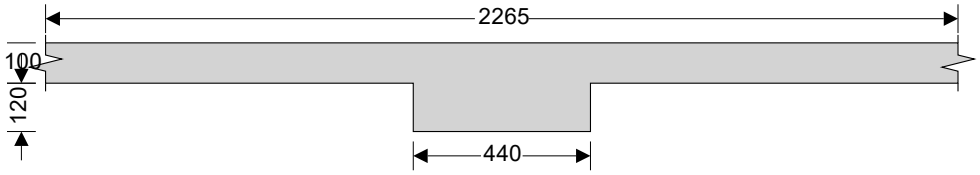
Thickness of pier	t_p = **220** mm
Breadth of pier	b_p = **440** mm
Spacing between piers	s_p = **2265** mm

From BS5628-1 Table 5 - Stiffness coefficient for walls stiffened by piers

Stiffness coefficient	K = **1.52**

Single-leaf wall construction details

Wall thickness	t = **100** mm
Effective wall thickness	t_{ef} = t × K = **152** mm



Masonry details

Masonry type	**Aggregate concrete blocks with no voids**
Compressive strength of unit	p_{unit} = **7.3** N/mm^2
Mortar strength Class/Designation	**M6 / (ii)**
Height of masonry units	h_b = **215** mm
Density of masonry	γ = **18.0** kN/m^3
Least horizontal dimension of masonry units	t_{min} = **100** mm
Ratio of height to least horizontal dimension	h_b / t_{min} = **2.15**

From BS5628-1 Table 2d - Characteristic compressive strength of masonry

Characteristic compressive strength f_k = **6.40** N/mm^2

From BS5628-1 Table 3 - Characteristic flexural strength of masonry

Plane of failure parallel to bed joints f_{kx_para} = **0.25** N/mm^2

Plane of failure perpendicular to bed joints f_{kx_perp} = **0.60** N/mm^2

Lateral loading details

Characteristic wind load on panel W_k = **0.570** kN/m^2

Vertical loading details

Dead load on top of wall G_k = **3.23** kN/m

Imposed load on top of wall Q_k = **2.53** kN/m

Partial safety factors for material strength

Category of manufacturing control **Category II**

Category of construction control **Normal**

Partial safety factor for masonry in compression γ_{mc} = **3.50**

Partial safety factor for masonry in flexure γ_{mf} = **3.00**

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

Slenderness ratio (cl 24.1)

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

Slenderness ratio $SR = \min(h_{ef}, L_{ef}) / t_{ef}$ = **14.3**

PASS - Slenderness ratio is less than maximum allowable

Vertical loading (cl 28)

Partial safety factors for design loads

Partial safety factor for design dead load γ_{fG} = **1.40**

Partial safety factor for design imposed load γ_{fQ} = **1.60**

Check vertical loads at top of wall

Design vertical load on wall $F_v = G_k \times \gamma_{fG} + Q_k \times \gamma_{fQ}$ = **8.6** kN/m

Design vertical load stress on wall $f_v = F_v / t$ = **0.086** N/mm^2

Design bending moment $M_v = G_k \times \gamma_{fG} \times e_G + Q_k \times \gamma_{fQ} \times e_Q$ = **0.0** kNm/m

Resultant eccentricity at the top of the wall $e_x = M_v / F_v$ = **0** mm

From BS5628-1 Table 7 - Capacity reduction factor

Capacity reduction factor β = **0.95**

Allowable stress capacity $f_{cap} = \beta \times f_k / \gamma_{mc}$ = **1.729** N/mm^2

PASS - Allowable stress capacity exceeds design vertical load stress on wall

Horizontal loading (cl 32)

Limiting dimensions (cl 32.3)

Area of panel $A_p = h \times L$ = **20.0** m^2

Limiting area of panel $A_{max} = 2025 \times t_{ef}^2$ = **46.8** m^2

PASS - Area of panel does not exceed limiting area of panel

Limiting panel dimension $L_{max} = 50 \times t_{ef}$ = **7600** mm

FAIL - Panel length exceeds limiting panel dimension

Partial safety factors for design loads

Partial safety factor for design wind load γ_{fW} = **1.40**

Partial safety factor for design dead load γ_{fG} = **0.90**

Design moments of resistance in panels (cl 32.4.2)

Self weight of wall	$S_{wt} = 0.375 \times h \times t \times \gamma =$ **1.47** kN/m
Design vertical compressive stress	$g_d = \gamma_{fG} \times (G_k + S_{wt}) / t =$ **0.04** N/mm^2
Enhanced flexural strength of masonry	$f_{ka_para} = f_{kx_para} + \gamma_{mf} \times g_d =$ **0.38** N/mm^2
Flange width	$b_f = \min(s_p, b_p + 12 \times t) =$ **1640** mm
Neutral axis depth	$y_2 = [(b_f \times t^2/2) + (b_p \times (t_p\text{-}t) \times (t_p+t)/2)] / (b_f \times t + b_p \times (t_p\text{-}t)) =$ **77** mm
	$y_1 = t_p - y_2 =$ **143** mm
Moment of inertia	$I = b_f \times t^3/12 + b_f \times t \times (y_2\text{-}t/2)^2 + b_p \times (t_p\text{-}t)^3/12 + b_p \times (t_p\text{-}t) \times (y_1\text{-}(t_p\text{-}t)/2)^2$
	$I =$ **683312276** mm^4
Section modulus of pier and flange	$Z = I / (\max(y_1, y_2) \times s_p) =$ **2106574** mm^3/m
Parallel design moment of resistance	$M_{para} = f_{ka_para} \times Z / \gamma_{mf} =$ **0.265** kNm/m
Section modulus of wall	$Z = t^2 / 6 =$ **1666667** mm^3/m
Perpendicular design moment of resistance	$M_{perp} = f_{kx_perp} \times Z / \gamma_{mf} =$ **0.333** kNm/m
Elastic design moment of resistance	$M_d = M_{para} =$ **0.265** kNm/m

Design moment in panels (cl 32.4.2)

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

Bending moment coefficient	$\alpha =$ **0.125**
Design moment in wall	$M = \alpha \times W_k \times \gamma_{fW} \times h^2 =$ **0.472** kNm/m

FAIL - Resistance moment is exceeded by design moment

MASONRY WALL PANEL DESIGN

In accordance with BS5628-1:2005

Tedds calculation version 1.2.15

Masonry panel details

Garage Long wall - Unreinforced masonry wall without openings

Panel length L = **9215** mm

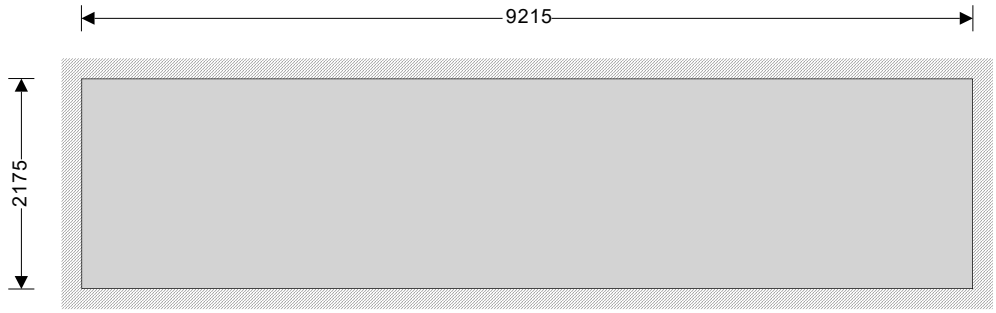
Panel height h = **2175** mm

Panel support conditions

All edges supported, right and left continuous

Effective panel length L_{ef} = 1.0 × L = **9215** mm

Effective panel height h_{ef} = 1.0 × h = **2175** mm



Pier details

Thickness of pier t_p = **280** mm

Breadth of pier b_p = **440** mm

Spacing between piers s_p = **1688** mm

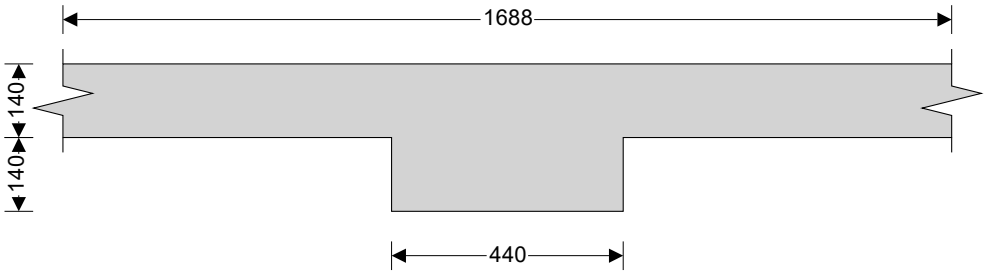
From BS5628-1 Table 5 - Stiffness coefficient for walls stiffened by piers

Stiffness coefficient K = **1.40**

Single-leaf wall construction details

Wall thickness t = **140** mm

Effective wall thickness t_{ef} = t × K = **196** mm



Masonry details

Masonry type **Aggregate concrete blocks with no voids**

Compressive strength of unit p_{unit} = **10.4** N/mm^2

Mortar strength Class/Designation **M6 / (ii)**

Height of masonry units h_b = **215** mm

Density of masonry	γ = **18.0** kN/m^3
Least horizontal dimension of masonry units	t_{min} = **100** mm
Ratio of height to least horizontal dimension	h_b / t_{min} = **2.15**

From BS5628-1 Table 2d - Characteristic compressive strength of masonry

Characteristic compressive strength	f_k = **8.40** N/mm^2

From BS5628-1 Table 3 - Characteristic flexural strength of masonry

Plane of failure parallel to bed joints	f_{kx_para} = **0.25** N/mm^2
Plane of failure perpendicular to bed joints	f_{kx_perp} = **0.75** N/mm^2

Lateral loading details

Characteristic wind load on panel	W_k = **0.570** kN/m^2

Vertical loading details

Dead load on top of wall	G_k = **3.23** kN/m
Imposed load on top of wall	Q_k = **2.53** kN/m

Partial safety factors for material strength

Category of manufacturing control	**Category II**
Category of construction control	**Normal**
Partial safety factor for masonry in compression	γ_{mc} = **3.50**
Partial safety factor for masonry in flexure	γ_{mf} = **3.00**
Partial safety factor for masonry in shear	γ_{mv} = **2.50**

Slenderness ratio (cl 24.1)

Allowable slenderness ratio	SR_{all} = **27**
Slenderness ratio	SR = min(h_{ef}, L_{ef}) / t_{ef} = **11.1**

PASS - Slenderness ratio is less than maximum allowable

Vertical loading (cl 28)

Partial safety factors for design loads

Partial safety factor for design dead load	γ_{fG} = **1.40**
Partial safety factor for design imposed load	γ_{fQ} = **1.60**

Check vertical loads at top of wall

Design vertical load on wall	$F_v = G_k \times \gamma_{fG} + Q_k \times \gamma_{fQ}$ = **8.6** kN/m
Design vertical load stress on wall	$f_v = F_v / t$ = **0.061** N/mm^2
Design bending moment	$M_v = G_k \times \gamma_{fG} \times e_G + Q_k \times \gamma_{fQ} \times e_Q$ = **0.0** kNm/m
Resultant eccentricity at the top of the wall	$e_x = M_v / F_v$ = **0** mm

From BS5628-1 Table 7 - Capacity reduction factor

Capacity reduction factor	β = **1.00**
Allowable stress capacity	$f_{cap} = \beta \times f_k / \gamma_{mc}$= **2.400** N/mm^2

PASS - Allowable stress capacity exceeds design vertical load stress on wall

Horizontal loading (cl 32)

Limiting dimensions (cl 32.3)

Area of panel	$A_p = h \times L$ = **20.0** m^2
Limiting area of panel	$A_{max} = 2025 \times t_{ef}^2$ = **77.8** m^2

PASS - Area of panel does not exceed limiting area of panel

Limiting panel dimension	$L_{max} = 50 \times t_{ef}$ = **9800** mm

PASS - Limiting panel dimension is not exceeded

Partial safety factors for design loads

Partial safety factor for design wind load γ_{fW} = **1.40**

Partial safety factor for design dead load γ_{fG} = **0.90**

Design moments of resistance in panels (cl 32.4.2)

Self weight of wall $S_{wt} = 0.375 \times h \times t \times \gamma$ = **2.06** kN/m

Design vertical compressive stress $g_d = \gamma_{fG} \times (G_k + S_{wt}) / t$ = **0.03** N/mm^2

Enhanced flexural strength of masonry $f_{ka_para} = f_{kx_para} + \gamma_{mf} \times g_d$ = **0.35** N/mm^2

Flange width $b_f = \min(s_p, b_p + 12 \times t)$ = **1688** mm

Neutral axis depth $y_2 = [(b_f \times t^2/2) + (b_p \times (t_p-t) \times (t_p+t)/2)] / (b_f \times t + b_p \times (t_p-t))$ = **99** mm

$y_1 = t_p - y_2$ = **181** mm

Moment of inertia $I = b_f \times t^3/12 + b_f \times t \times (y_2-t/2)^2 + b_p \times (t_p-t)^3/12 + b_p \times (t_p-t) \times (y_1-(t_p-t)/2)^2$

I = **1444320561** mm^4

Section modulus of pier and flange $Z = I / (\max(y_1, y_2) \times s_p)$ = **4725919** mm^3/m

Parallel design moment of resistance $M_{para} = f_{ka_para} \times Z / \gamma_{mf}$ = **0.554** kNm/m

Section modulus of wall $Z = t^2 / 6$ = **3266667** mm^3/m

Perpendicular design moment of resistance $M_{perp} = f_{kx_perp} \times Z / \gamma_{mf}$ = **0.817** kNm/m

Elastic design moment of resistance $M_d = M_{para}$ = **0.554** kNm/m

Design moment in panels (cl 32.4.2)

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

Bending moment coefficient α = **0.125**

Design moment in wall $M = \alpha \times W_k \times \gamma_{fW} \times h^2$ = **0.472** kNm/m

PASS - Resistance moment exceeds design moment

MASONRY WALL PANEL DESIGN

In accordance with BS5628-1:2005

Tedds calculation version 1.2.15

Masonry panel details

Garage Long wall - Unreinforced masonry wall without openings

Panel length — L = **9060** mm

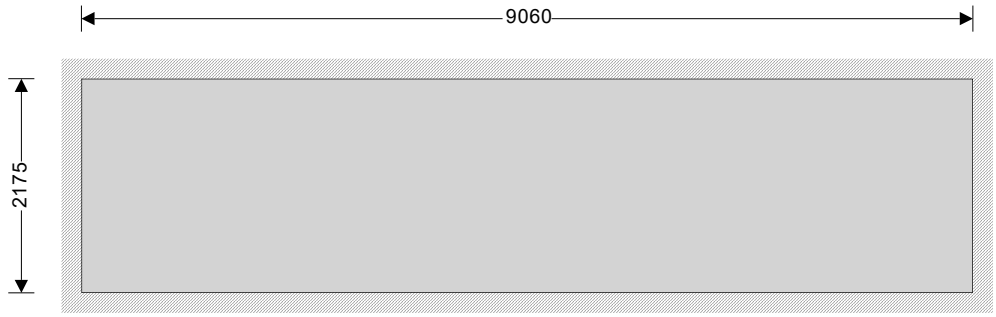
Panel height — h = **2175** mm

Panel support conditions

All edges supported, right and left continuous

Effective panel length — L_{ef} = 1.0 × L = **9060** mm

Effective panel height — h_{ef} = 1.0 × h = **2175** mm



Pier details

Thickness of pier — t_p = **290** mm

Breadth of pier — b_p = **440** mm

Spacing between piers — s_p = **1688** mm

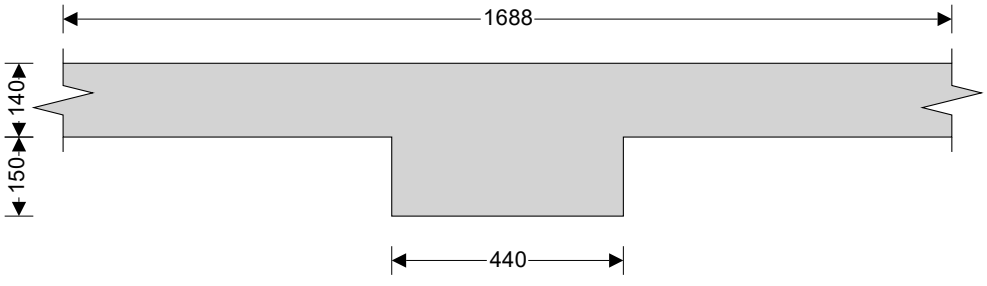
From BS5628-1 Table 5 - Stiffness coefficient for walls stiffened by piers

Stiffness coefficient — K = **1.44**

Single-leaf wall construction details

Wall thickness — t = **140** mm

Effective wall thickness — t_{ef} = t × K = **202** mm



Masonry details

Masonry type — **Aggregate concrete blocks with no voids**

Compressive strength of unit — p_{unit} = **10.4** N/mm^2

Mortar strength Class/Designation — **M6 / (ii)**

Height of masonry units — h_b = **215** mm

Density of masonry γ = **18.0** kN/m^3

Least horizontal dimension of masonry units t_{min} = **140** mm

Ratio of height to least horizontal dimension h_b / t_{min} = **1.54**

From BS5628-1 Table 2c & d - Characteristic compressive strength of masonry

Characteristic compressive strength f_k = **7.01** N/mm^2

From BS5628-1 Table 3 - Characteristic flexural strength of masonry

Plane of failure parallel to bed joints f_{kx_para} = **0.25** N/mm^2

Plane of failure perpendicular to bed joints f_{kx_perp} = **0.75** N/mm^2

Lateral loading details

Characteristic wind load on panel W_k = **0.570** kN/m^2

Vertical loading details

Dead load on top of wall G_k = **3.23** kN/m

Imposed load on top of wall Q_k = **2.53** kN/m

Partial safety factors for material strength

Category of manufacturing control **Category II**

Category of construction control **Normal**

Partial safety factor for masonry in compression γ_{mc} = **3.50**

Partial safety factor for masonry in flexure γ_{mf} = **3.00**

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

Slenderness ratio (cl 24.1)

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

Slenderness ratio SR = min(h_{ef}, L_{ef}) / t_{ef} = **10.8**

PASS - Slenderness ratio is less than maximum allowable

Vertical loading (cl 28)

Partial safety factors for design loads

Partial safety factor for design dead load γ_{fG} = **1.40**

Partial safety factor for design imposed load γ_{fQ} = **1.60**

Check vertical loads at top of wall

Design vertical load on wall $F_v = G_k \times \gamma_{fG} + Q_k \times \gamma_{fQ}$ = **8.6** kN/m

Design vertical load stress on wall $f_v = F_v / t$ = **0.061** N/mm^2

Design bending moment $M_v = G_k \times \gamma_{fG} \times e_G + Q_k \times \gamma_{fQ} \times e_Q$ = **0.0** kNm/m

Resultant eccentricity at the top of the wall $e_x = M_v / F_v$ = **0** mm

From BS5628-1 Table 7 - Capacity reduction factor

Capacity reduction factor β = **1.00**

Allowable stress capacity $f_{cap} = \beta \times f_k / \gamma_{mc}$ = **2.002** N/mm^2

PASS - Allowable stress capacity exceeds design vertical load stress on wall

Horizontal loading (cl 32)

Limiting dimensions (cl 32.3)

Area of panel $A_p = h \times L$ = **19.7** m^2

Limiting area of panel $A_{max} = 2025 \times t_{ef}^2$ = **82.6** m^2

PASS - Area of panel does not exceed limiting area of panel

Limiting panel dimension $L_{max} = 50 \times t_{ef}$ = **10100** mm

PASS - Limiting panel dimension is not exceeded

Partial safety factors for design loads

Partial safety factor for design wind load	γ_{fW} = **1.40**
Partial safety factor for design dead load	γ_{fG} = **0.90**

Design moments of resistance in panels (cl 32.4.2)

Self weight of wall	$S_{wt} = 0.375 \times h \times t \times \gamma$ = **2.06** kN/m
Design vertical compressive stress	$g_d = \gamma_{fG} \times (G_k + S_{wt}) / t$ = **0.03** N/mm^2
Enhanced flexural strength of masonry	$f_{ka_para} = f_{kx_para} + \gamma_{mf} \times g_d$ = **0.35** N/mm^2
Flange width	$b_f = \min(s_p, b_p + 12 \times t)$ = **1688** mm
Neutral axis depth	$y_2 = [(b_f \times t^2/2) + (b_p \times (t_p-t) \times (t_p+t)/2)] / (b_f \times t + b_p \times (t_p-t))$ = **102** mm
	$y_1 = t_p - y_2$ = **188** mm
Moment of inertia	$I = b_f \times t^3/12 + b_f \times t \times (y_2-t/2)^2 + b_p \times (t_p-t)^3/12 + b_p \times (t_p-t) \times (y_1-(t_p-t)/2)^2$
	I = **1594449071** mm^4
Section modulus of pier and flange	$Z = I / (\max(y_1, y_2) \times s_p)$ = **5015157** mm^3/m
Parallel design moment of resistance	$M_{para} = f_{ka_para} \times Z / \gamma_{mf}$ = **0.588** kNm/m
Section modulus of wall	$Z = t^2 / 6$ = **3266667** mm^3/m
Perpendicular design moment of resistance	$M_{perp} = f_{kx_perp} \times Z / \gamma_{mf}$ = **0.817** kNm/m
Elastic design moment of resistance	$M_d = M_{para}$ = **0.588** kNm/m

Design moment in panels (cl 32.4.2)

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

Bending moment coefficient	α = **0.125**
Design moment in wall	$M = \alpha \times W_k \times \gamma_{fW} \times h^2$ = **0.472** kNm/m

PASS - Resistance moment exceeds design moment

MASONRY WALL PANEL DESIGN

In accordance with BS5628-1:2005

Tedds calculation version 1.2.15

Masonry panel details

Garage Long wall - Unreinforced masonry wall without openings

Panel length L = **6600** mm

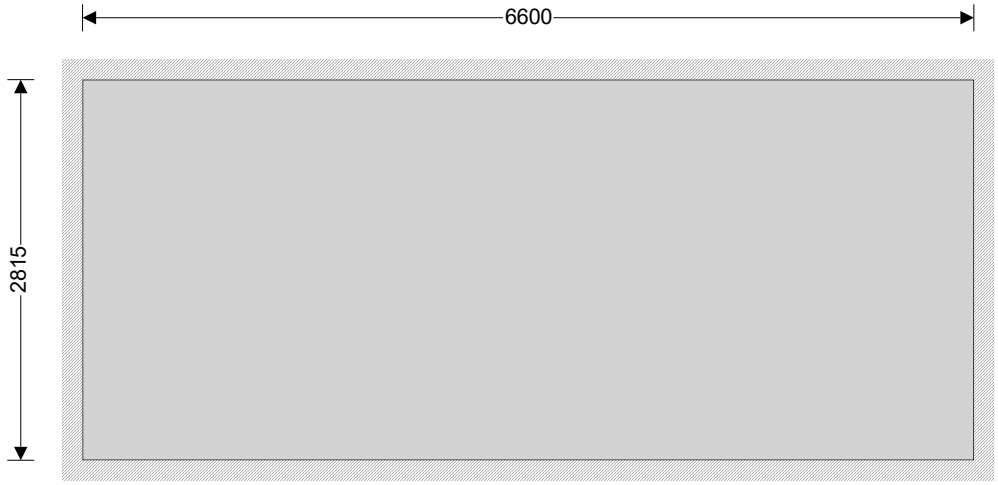
Panel height h = **2815** mm

Panel support conditions

All edges supported, right and left continuous

Effective panel length L_{ef} = 1.0 × L = **6600** mm

Effective panel height h_{ef} = 1.0 × h = **2815** mm



Pier details

Thickness of pier t_p = **220** mm

Breadth of pier b_p = **440** mm

Spacing between piers s_p = **2200** mm

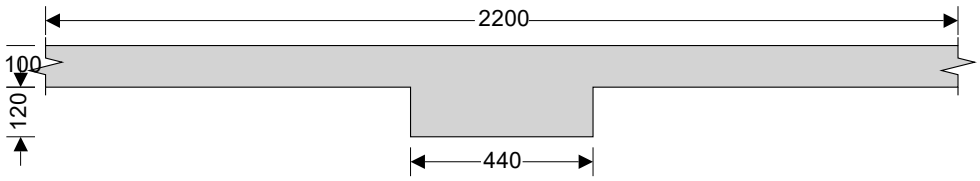
From BS5628-1 Table 5 - Stiffness coefficient for walls stiffened by piers

Stiffness coefficient K = **1.52**

Single-leaf wall construction details

Wall thickness t = **100** mm

Effective wall thickness t_{ef} = t × K = **152** mm



Masonry details

Masonry type **Aggregate concrete blocks with no voids**

Compressive strength of unit p_{unit} = **7.3** N/mm^2

Mortar strength Class/Designation	**M6 / (ii)**
Height of masonry units	h_b = **215** mm
Density of masonry	γ = **18.0** kN/m^3
Least horizontal dimension of masonry units	t_{min} = **100** mm
Ratio of height to least horizontal dimension	h_b / t_{min} = **2.15**

From BS5628-1 Table 2d - Characteristic compressive strength of masonry

Characteristic compressive strength	f_k = **6.40** N/mm^2

From BS5628-1 Table 3 - Characteristic flexural strength of masonry

Plane of failure parallel to bed joints	f_{kx_para} = **0.25** N/mm^2
Plane of failure perpendicular to bed joints	f_{kx_perp} = **0.60** N/mm^2

Lateral loading details

Characteristic wind load on panel	W_k = **0.803** kN/m^2

Partial safety factors for material strength

Category of manufacturing control	**Category II**
Category of construction control	**Normal**
Partial safety factor for masonry in compression	γ_{mc} = **3.50**
Partial safety factor for masonry in flexure	γ_{mf} = **3.00**
Partial safety factor for masonry in shear	γ_{mv} = **2.50**

Horizontal loading (cl 32)

Limiting dimensions (cl 32.3)

Area of panel	$A_p = h \times L$ = **18.6** m^2
Limiting area of panel	$A_{max} = 2025 \times t_{ef}^2$ = **46.8** m^2
	PASS - Area of panel does not exceed limiting area of panel
Limiting panel dimension	$L_{max} = 50 \times t_{ef}$ = **7600** mm
	PASS - Limiting panel dimension is not exceeded

Partial safety factors for design loads

Partial safety factor for design wind load	γ_{fW} = **1.40**
Partial safety factor for design dead load	γ_{fG} = **0.90**

Design moments of resistance in panels (cl 32.4.2)

Design vertical compressive stress	$g_d = \gamma_{fG} \times G_k / t$ = **0.00** N/mm^2
Enhanced flexural strength of masonry	$f_{ka_para} = f_{kx_para} + \gamma_{mf} \times g_d$ = **0.25** N/mm^2
Flange width	$b_f = \min(s_p, b_p + 12 \times t)$ = **1640** mm
Neutral axis depth	$y_2 = [(b_f \times t^2/2) + (b_p \times (t_p-t) \times (t_p+t)/2)] / (b_f \times t + b_p \times (t_p-t))$ = **77** mm
	$y_1 = t_p - y_2$ = **143** mm
Moment of inertia	$I = b_f \times t^3/12 + b_f \times t \times (y_2-t/2)^2 + b_p \times (t_p-t)^3/12 + b_p \times (t_p-t) \times (y_1-(t_p-t)/2)^2$
	I = **683312276** mm^4
Section modulus of pier and flange	$Z = I / (\max(y_1, y_2) \times s_p)$ = **2168813** mm^3/m
Parallel design moment of resistance	$M_{para} = f_{ka_para} \times Z / \gamma_{mf}$ = **0.181** kNm/m
Section modulus of wall	$Z = t^2 / 6$ = **1666667** mm^3/m
Perpendicular design moment of resistance	$M_{perp} = f_{kx_perp} \times Z / \gamma_{mf}$ = **0.333** kNm/m
Elastic design moment of resistance	$M_d = M_{perp}$ = **0.333** kNm/m

Design moment in panels (cl 32.4.2)

Orthogonal strength ratio $\mu = M_{para} / M_{perp} =$ **0.54**

Using yield line analysis to calculate bending moment coefficient

Bending moment coefficient $\alpha =$ **0.017**

Design moment in wall $M = \alpha \times W_k \times \gamma_{fw} \times L^2 =$ **0.823** kNm/m

FAIL - Resistance moment is exceeded by design moment

MASONRY WALL PANEL DESIGN

In accordance with BS5628-1:2005

Tedds calculation version 1.2.15

Masonry panel details

Garage Long wall - Unreinforced masonry wall without openings

Panel length L = **6600** mm

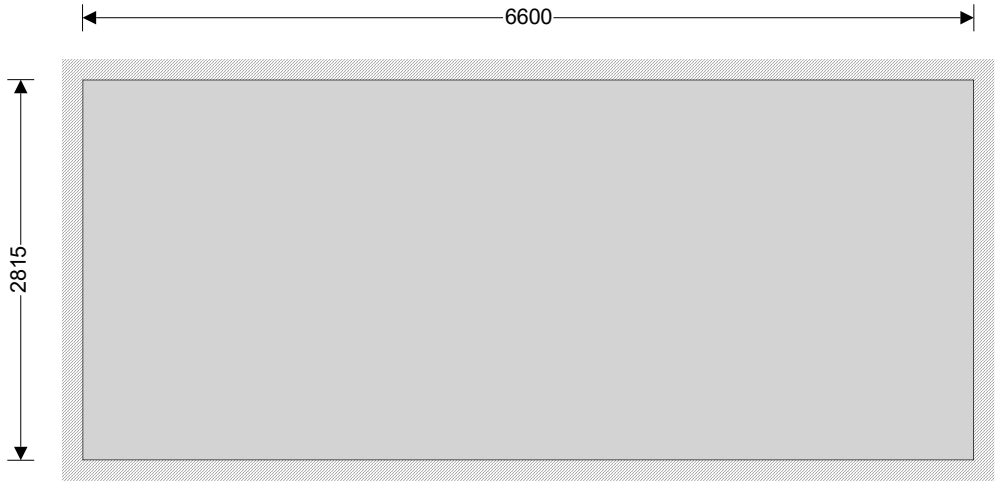
Panel height h = **2815** mm

Panel support conditions

All edges supported, right and left continuous

Effective panel length L_{ef} = 1.0 × L = **6600** mm

Effective panel height h_{ef} = 1.0 × h = **2815** mm



Pier details

Thickness of pier t_p = **290** mm

Breadth of pier b_p = **440** mm

Spacing between piers s_p = **1320** mm

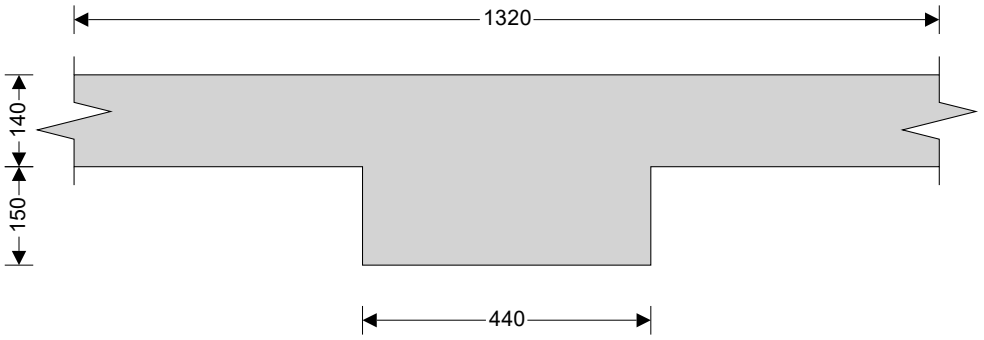
From BS5628-1 Table 5 - Stiffness coefficient for walls stiffened by piers

Stiffness coefficient K = **1.44**

Single-leaf wall construction details

Wall thickness t = **140** mm

Effective wall thickness t_{ef} = t × K = **202** mm



Masonry details

Masonry type	**Aggregate concrete blocks with no voids**
Compressive strength of unit	p_{unit} = **10.4** N/mm^2
Mortar strength Class/Designation	**M6 / (ii)**
Height of masonry units	h_b = **215** mm
Density of masonry	γ = **18.0** kN/m^3
Least horizontal dimension of masonry units	t_{min} = **140** mm
Ratio of height to least horizontal dimension	h_b / t_{min} = **1.54**

From BS5628-1 Table 2c & d - Characteristic compressive strength of masonry

Characteristic compressive strength	f_k = **7.01** N/mm^2

From BS5628-1 Table 3 - Characteristic flexural strength of masonry

Plane of failure parallel to bed joints	f_{kx_para} = **0.25** N/mm^2
Plane of failure perpendicular to bed joints	f_{kx_perp} = **0.75** N/mm^2

Lateral loading details

Characteristic wind load on panel	W_k = **0.803** kN/m^2

Partial safety factors for material strength

Category of manufacturing control	**Category I**
Category of construction control	**Normal**
Partial safety factor for masonry in compression	γ_{mc} = **3.10**
Partial safety factor for masonry in flexure	γ_{mf} = **3.00**
Partial safety factor for masonry in shear	γ_{mv} = **2.50**

Horizontal loading (cl 32)

Limiting dimensions (cl 32.3)

Area of panel	$A_p = h \times L$ = **18.6** m^2
Limiting area of panel	$A_{max} = 2025 \times t_{ef}^2$ = **82.6** m^2
	PASS - Area of panel does not exceed limiting area of panel
Limiting panel dimension	$L_{max} = 50 \times t_{ef}$ = **10100** mm
	PASS - Limiting panel dimension is not exceeded

Partial safety factors for design loads

Partial safety factor for design wind load	γ_{fW} = **1.40**
Partial safety factor for design dead load	γ_{fG} = **0.90**

Design moments of resistance in panels (cl 32.4.2)

Design vertical compressive stress	$g_d = \gamma_{fG} \times G_k / t$ = **0.00** N/mm^2
Enhanced flexural strength of masonry	$f_{ka_para} = f_{kx_para} + \gamma_{mf} \times g_d$ = **0.25** N/mm^2
Flange width	$b_f = \min(s_p, b_p + 12 \times t)$ = **1320** mm
Neutral axis depth	$y_2 = [(b_f \times t^2/2) + (b_p \times (t_p-t) \times (t_p+t)/2)] / (b_f \times t + b_p \times (t_p-t))$ = **108** mm
	$y_1 = t_p - y_2$ = **182** mm
Moment of inertia	$I = b_f \times t^3/12 + b_f \times t \times (y_2-t/2)^2 + b_p \times (t_p-t)^3/12 + b_p \times (t_p-t) \times (y_1-(t_p-t)/2)^2$
	I = **1448068947** mm^4
Section modulus of pier and flange	$Z = I / (\max(y_1, y_2) \times s_p)$ = **6032827** mm^3/m
Parallel design moment of resistance	$M_{para} = f_{ka_para} \times Z / \gamma_{mf}$ = **0.503** kNm/m
Section modulus of wall	$Z = t^2 / 6$ = **3266667** mm^3/m

Perpendicular design moment of resistance $M_{perp} = f_{kx_perp} \times Z / \gamma_{mf}$ = **0.817** kNm/m

Elastic design moment of resistance $M_d = M_{perp}$ = **0.817** kNm/m

Design moment in panels (cl 32.4.2)

Orthogonal strength ratio $\mu = M_{para} / M_{perp}$ = **0.62**

Using yield line analysis to calculate bending moment coefficient

Bending moment coefficient α = **0.016**

Design moment in wall $M = \alpha \times W_k \times \gamma_{fW} \times L^2$ = **0.764** kNm/m

PASS - Resistance moment exceeds design moment

MASONRY WALL PANEL DESIGN

In accordance with BS5628-1:2005 and BS5628-2:2005

Tedds calculation version 1.2.15

Masonry panel details

Garage Long wall - Reinforced masonry wall without openings

Panel length — L = **6600** mm

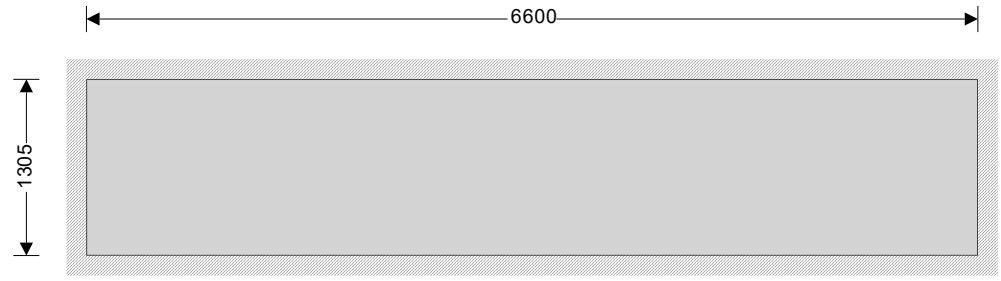
Panel height — h = **1305** mm

Panel support conditions

All edges supported

Effective panel length — L_{ef} = 1.0 × L = **6600** mm

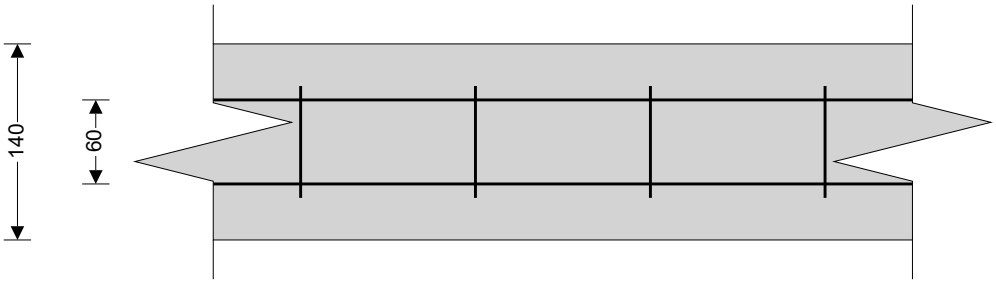
Effective panel height — h_{ef} = 1.0 × h = **1305** mm



Single-leaf wall construction details

Wall thickness — t = **140** mm

Effective wall thickness — t_{ef} = t = **140** mm



Masonry details

Masonry type — **Aggregate concrete blocks with no voids**

Compressive strength of unit — p_{unit} = **10.4** N/mm^2

Mortar strength Class/Designation — **M6 / (ii)**

Height of masonry units — h_b = **215** mm

Density of masonry — γ = **18.0** kN/m^3

Least horizontal dimension of masonry units — t_{min} = **140** mm

Ratio of height to least horizontal dimension — h_b / t_{min} = **1.54**

From BS5628-1 Table 2c & d - Characteristic compressive strength of masonry

Characteristic compressive strength — f_k = **7.01** N/mm^2

From BS5628-1 Table 3 - Characteristic flexural strength of masonry

Plane of failure parallel to bed joints — f_{kx_para} = **0.25** N/mm^2

Plane of failure perpendicular to bed joints	f_{kx_perp} = **0.75** N/mm^2

Bed joint reinforcement details

Diameter of reinforcement	ϕ = **5.00** mm
Horizontal spacing of bars	s = **60** mm
Number of courses between layers	c = **1**

Lateral loading details

Characteristic wind load on panel	W_k = **0.803** kN/m^2

Partial safety factors for material strength

Category of manufacturing control	**Category II**
Category of construction control	**Normal**
Partial safety factor for masonry in compression	γ_{mc} = **3.50**
Partial safety factor for masonry in flexure	γ_{mf} = **3.00**
Partial safety factor for masonry in shear	γ_{mv} = **2.50**

Horizontal loading (cl 32)

Limiting dimensions (cl 32.3)

Area of panel	$A_p = h \times L$ = **8.6** m^2
Limiting area of panel	$A_{max} = 2400 \times t_{ef}^2$ = **47.0** m^2
	PASS - Area of panel does not exceed limiting area of panel
Limiting panel dimension	$L_{max} = 60 \times t_{ef}$ = **8400** mm
	PASS - Limiting panel dimension is not exceeded

Partial safety factors for design loads

Partial safety factor for design wind load	γ_{fW} = **1.40**
Partial safety factor for design dead load	γ_{fG} = **0.90**

Design moments of resistance in panels (cl 32.4.2)

Self weight of wall	$S_{wt} = 0.375 \times h \times t \times \gamma$ = **1.23** kN/m
Design vertical compressive stress	$g_d = \gamma_{fG} \times (G_k + S_{wt}) / t$ = **0.01** N/mm^2
Enhanced flexural strength of masonry	$f_{ka_para} = f_{kx_para} + \gamma_{mf} \times g_d$ = **0.27** N/mm^2

Design moments in reinforced panel (cl 8.2.4)

Section modulus of wall	$Z = t^2 / 6$ = **3266667** mm^3/m
Parallel design moment of resistance	$M_{para} = f_{ka_para} \times Z / \gamma_{mf}$ = **0.298** kNm/m
Unreinforced moment of resistance	$M_u = f_{kx_perp} \times Z / \gamma_{mf}$ = **0.817** kNm/m
Characteristic tensile strength of renforcement	f_y = **500** N/mm^2
Area of tension reinforcement	$A_s = \pi \times \phi^2 / 4$ = **19.63** mm^2
Depth of tension reinforcement	$d = (t + s) / 2$ = **100.0** mm
Width of section	$b = c \times (h_b + 10\ mm)$ = **225** mm
Length of lever arm	$z = \min(1 - 0.5 \times A_s \times f_y \times \gamma_{mm} / (b \times d \times f_k \times \gamma_{ms}), 0.95) \times d$
	z = **93.8** mm
Reinforced moment of resistance	$M_r = A_s \times f_y \times z / (b \times \gamma_{ms})$ = **3.558** kNm/m
Maximum reinforced moment of resistance	$M_{r_max} = 0.4 \times f_k \times d^2 / \gamma_{mm}$ = **12.186** kNm/m
Unreinforced moment of resistance limit	$M_{u_lim} = 1.5 \times M_u$ = **1.225** kNm/m
Perpendicular design moment of resistance	$M_{perp} = \min(M_r, M_{r_max}, M_{u_lim})$ = **1.225** kNm/m
Elastic design moment of resistance	$M_d = M_{para}$ = **0.298** kNm/m

Design moment in panels (cl 32.4.2)

PW Structural Solutions Ltd
19 Coleridge Place, Bourne
Lincolnshire, PE10 9WA

Project				Job no.	
Pepper Hill Farm Cottage - New Double Garage				PW-22-01-04	
Calcs for				Start page no./Revision	
Garage South Gable End Wall				A-C/ 15	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
P D Wood	15/02/2022				

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

Bending moment coefficient α = **0.125**

Design moment in wall $M = \alpha \times W_k \times \gamma_{fw} \times h^2$ = **0.239** kNm/m

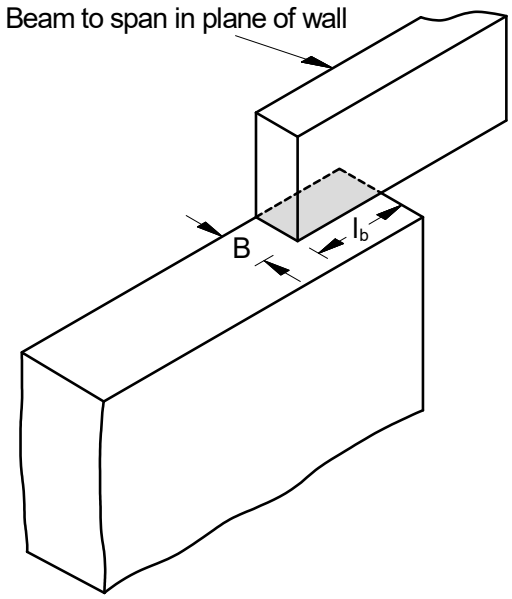
PASS - Resistance moment exceeds design moment

MASONRY BEARING DESIGN TO BS5628-1:2005

TEDDS calculation version 1.0.08

Masonry details

Masonry type	**Aggregate concrete blocks (25% or less formed voids)**
Compressive strength of unit	p_{unit} = **10.4** N/mm^2
Mortar designation	**iii**
Least horizontal dimension of masonry units	l_{unit} = **140** mm
Height of masonry units	h_{unit} = **215** mm
Category of masonry units	**Category II**
Category of construction control	**Normal**
Partial safety factor for material strength	γ_m = **3.5**
Thickness of load bearing leaf	t = **140** mm
Effective thickness of masonry wall	t_{ef} = **140** mm
Height of masonry wall	h = **2100** mm
Effective height of masonry wall	h_{ef} = **2100** mm



Bearing details

Beam spanning in plane of wall	
Width of bearing	B = **140** mm
Length of bearing	l_b = **150** mm

Compressive strength from Table 2 BS5628:Part 1 - aggregate concrete blocks (25% or less formed voids)

Mortar designation	Mortar = **"iii"**
Block compressive strength	p_{unit} = **10.4** N/mm^2
Characteristic compressive strength (Table 2c)	f_{kc} = **4.10** N/mm^2
Characteristic compressive strength (Table 2d)	f_{kd} = **8.20** N/mm^2
Height of solid block	h_{unit} = **215.0** mm
Least horizontal dimension	l_{unit} = **140.0** mm
Block ratio	ratio = h_{unit} / l_{unit} = **1.5**

Ratio between 0.6 and 4.5 - OK

Characteristic compressive strength	f_k = **6.84** N/mm^2

Loading details

Characteristic concentrated dead load	G_k = **3** kN
Characteristic concentrated imposed load	Q_k = **0** kN
Design concentrated load	F = (G_k × 1.4) + (Q_k × 1.6) = **4.5** kN
Characteristic distributed dead load	g_k = **0.0** kN/m
Characteristic distributed imposed load	q_k = **0.0** kN/m
Design distributed load	f = (g_k × 1.4) + (q_k × 1.6) = **0.0** kN/m

Masonry bearing type

Bearing type	**Type 2**
Bearing safety factor	γ_{bear} = **1.50**

Check design bearing without a spreader

Design bearing stress	f_{ca} = F / (B × l_b) + f / t = **0.215** N/mm^2
Allowable bearing stress	f_{cp} = γ_{bear} × f_k / γ_m = **2.932** N/mm^2

PASS - Allowable bearing stress exceeds design bearing stress

Check design bearing at 0.4 × h below the bearing level

Slenderness ratio	h_{ef} / t_{ef} = **15.00**
Eccentricity at top of wall	e_x = **0.0** mm

From BS5628:1 Table 7

Capacity reduction factor	β = **0.93**
Length of bearing distributed at 0.4 × h	l_d = **990** mm
Maximum bearing stress	f_{ca} = F / (l_d × t) + f / t = **0.033** N/mm^2
Allowable bearing stress	f_{cp} = β × f_k / γ_m = **1.811** N/mm^2

PASS - Allowable bearing stress at 0.4 × h below bearing level exceeds design bearing stress

VERTICAL LOADING RECTANGULAR COLUMN (BS5628-1:2005)

TEDDS calculation version 1.0.02

Compressive strength from Table 2 BS5628:Part 1 - aggregate concrete blocks (25% or less formed voids)

Mortar designation	Mortar = **"iii"**
Block compressive strength	p_{unit} = **10.4** N/mm^2
Characteristic compressive strength (Table 2c)	f_{kc} = **4.10** N/mm^2
Characteristic compressive strength (Table 2d)	f_{kd} = **8.20** N/mm^2
Height of solid block	h_{unit} = **215.0** mm
Least horizontal dimension	l_{unit} = **140.0** mm
Block ratio	ratio = h_{unit} / l_{unit} = **1.5**
	Ratio between 0.6 and 4.5 - OK
Characteristic compressive strength	f_k = **6.84** N/mm^2
Column width	b = **670** mm
Column thickness	t = **670** mm
Column height	h = **2.10** m
Column slenderness - minor axis (Clause 24.1)	λ_t = h/t = **3.13**
Column slenderness - major axis (Clause 24.1)	λ_b = h/b = **3.13**
Maximum slenderness	λ_{max} = max(λ_t, λ_b) = **3.13**
	Slenderness < 27 - OK
Partial safety factor for material (Table 4)	γ_m = **2.5**

Load eccentricity

Eccentricity of applied load about minor axis	e_{xt} = **474.0** mm
Eccentricity of applied load about major axis	e_{xb} = **0.0** mm

Capacity reduction factor: minor axis

Eccentricity due to slenderness	e_{at} = max(0 mm, t × (((h/t)^2/2400)-0.015)) = **0.0** mm
Design eccentricity	e_{tt} = 0.6×max(abs(e_{xt}), 0.05×t)+ e_{at} = **284.4** mm
	e_{mt} = max(abs(e_{xt}),e_{tt}) = **474.0** mm
Capacity reduction factors	β_{tcalc} = max(0, 1.1 × (1 - (2 × e_{mt} / t))) = **0.00**
	β_{max} = 1.0
	β_t = min (β_{tcalc}, β_{max}) = **0.00**

Capacity reduction factor: major axis

Eccentricity due to slenderness	e_{ab} = max(0 mm, b × (((h/b)^2/2400)-0.015)) = **0.0** mm
Design eccentricity	e_{tb} = 0.6× max(abs(e_{xb}), 0.05×b)+ e_{ab} = **20.1** mm
	e_{mb} = max(abs(e_{xb}),e_{tb}) = **20.1** mm
Capacity reduction factors	β_{bcalc} = max(0, 1.1 × (1 - (2 × e_{mb} / b))) = **1.03**
	β_{max} = 1.0
	β_b = min (β_{bcalc}, β_{max}) = **1.00**
Minimum capacity reduction factor	β_{min} = min(β_t, β_b) = **0.00**

Design vertical load resistance

Compressive strength correction factor	
Plan area of column	A = t × b = **0.45** m^2
For small plan area (Clause 19.1.2)	c = min(1.0, 0.7+(1.5 m^{-2})×A) = **1.00**
Design vertical load resistance	DVLR = β_{min} × t × b × c × f_k / γ_m = **0.000** kN
Applied factored vertical load on column	V = **6.450** kN

Column - Fails

PW STRUCTURAL SOLUTIONS LTD

Calculations

REF		OUTPUT

F FOUNDATIONS

F.1 FOUNDATION DESIGN DATA

F.1.1 Soil Properties

Use data for soft/firm clay from the Piling Handbook 7th edition Chapter 4

Strata 1:	Soil Type			Cohesive
	Depth		=	3.00 m
	Bulk Density		=	19.6 kN/m^3
	Saturated Density		=	19.6 kN/m^3
	Cohesion		=	50.00 kN/m^2
	Angle of Shearing Resistance		=	0.00 Deg
Water Table level			=	0.55
Allowable bearing pressure			=	50.0 kN/m^2

F.1.2 Loads On Ground

Slab:	Thickness				=	0.20 m
	Weight	=	0.20 x 25.0		=	5.00 kN/m^2
Base:	Thickness				=	0.20 m
	Weight	=	0.20 x 20.0		=	4.00 kN/m^2
Wall Loads:	Dead	=	0.14 x 18.0	x 2.40	=	6.05 kN/m
B/ 1	Roof Dead				=	3.23 kN/m
Edge Thickening		=	0.45 x 25.0	x 0.45	=	5.06 kN/m
Total Dead on Walls		=	6.05 + 3.23	+ 5.06	=	14.34 kN/m
Pressure on ground		=	14.34 / 6.740		=	2.13 kN/m^2
Total Dead pressure		=	5.00 + 4.00	+ 2.13	=	11.13 kN/m

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Calculations

REF		OUTPUT
	Wall Loads	
	Live = 2.53 kN/m	
	Pressure on ground = 2.53 / 6.740 = 0.38 kN/m^2	
	Total Ground Pressure = 11.13 + 0.38 = 11.50 kN/m	<50 OK
	F.2 RAFT FOUNDATION DESIGN	
	F.2.1 Slab Design	
BS8110 Table 3.9	Span :depth ratio = 20.0	
	Let effective depth = 0.200 - 0.040 = 0.16 m	
	Slab Maximum Span = 0.16 x 20.0 = 3.20 m	
	Internal Beams Required.	
	Provide one internal beam parallel to the long axis and two internal beams parallel	
	to the short axis. Slab span: for design = 0.00 m	
	For Design =(6.740 - 0.150 - 0.150 - 0.450)/ 1 = 5.99 m	
	With Beams =(6.740 - 0.150 - 0.150 - 0.450)/ 2 = 3.00 m	
	Design load on slab	
F/ 1	=(1.40 x 2.13)+(1.60 x 0.38) = 3.58 kN/m^2	
	Moment = 3.58 x 5.99 x 5.99 / 8 = 16.05 kNm/m	
	Reaction = 3.58 x 5.99 x 1.25 / 1 = 26.8 kN/m	
	Check the slab using TEDDS. From the output given on pages A-F/1 to A-F/4	
	200mm thick slab satisfactory	
	Use A393 mesh top and Bottom	
F/ 2	"Working" load on Beam = 11.50 x 3.00 x 1.25 = 43.06 kN/m	

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Calculations

REF		OUTPUT

F.2.2 Beam Design

Internal beams are required only to ensure that the slab does not exceed BS8110 maximum span to depth ratios. The edge beams are required to ensure load transfer of the perimeter loads into the ground through the entire foundation.

Use TEDDS raft design routine to confirm the slab reinforcement and the beam reinforcement.

Imposed Load on floor = 2.50 kN/m^2

F/ 1 Wall Dead Load = 6.05 + 3.23 = 9.28 kN/m

F/ 2 Wall Live Load = 2.53 kN/m

From the TEDDS output given on pages A-F/5 to A-F/12

Beam	Internal	Edge
Depth	= 450 mm	= 450 mm
Width	= 450 mm	= 450 mm
Top Reinforcement	= 3 x H16	= 3 x H16
Bottom Reinforcement	= 3 x H16	= 3 x H16
Links	= H10 @ 200	= H10 @ 200

RC SLAB DESIGN (BS8110:PART1:1997)

TEDDS calculation version 1.0.04

CONTINUOUS ONE WAY SPANNING SLAB DEFINITION

Overall depth of slab h = **200** mm

Sagging steel

Cover to tension reinforcement resisting sagging c_{sag} = **35** mm

Trial bar diameter D_{tryx} = **10** mm

Depth to tension steel (resisting sagging)

$d_x = h - c_{sag} - D_{tryx}/2$ = **160** mm

Hogging steel

Cover to tension reinforcement resisting hogging c_{hog} = **35** mm

Trial bar diameter $D_{tryxhog}$ = **10** mm

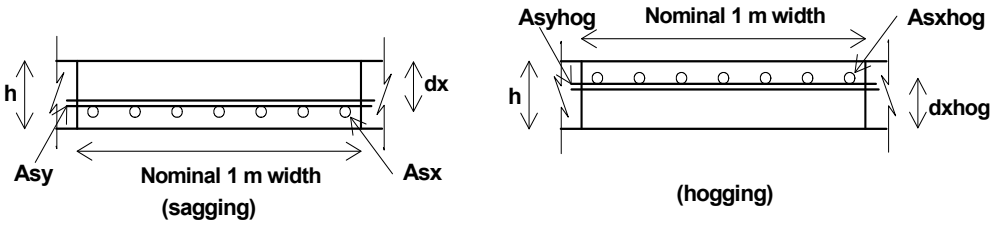
Depth to tension steel (resisting hogging)

$d_{xhog} = h - c_{hog} - D_{tryxhog}/2$ = **160** mm

Materials

Characteristic strength of reinforcement f_y = **500** N/mm^2

Characteristic strength of concrete f_{cu} = **40** N/mm^2



One-way spanning slab

(continuous)

ONE WAY SPANNING SLAB (CL 3.5.4)

MAXIMUM DESIGN MOMENTS IN SPAN

Design sagging moment (per m width of slab) m_{sx} = **10.2** kNm/m

CONCRETE SLAB DESIGN – SAGGING – OUTER LAYER OF STEEL (CL 3.5.4)

Design sagging moment (per m width of slab) m_{sx} = **10.2** kNm/m

Moment Redistribution Factor β_{bx} = **1.0**

Area of reinforcement required

$K_x = abs(m_{sx}) / (d_x^2 \times f_{cu})$ = **0.010**

$K'_x = min (0.156 , (0.402 \times (\beta_{bx} - 0.4)) - (0.18 \times (\beta_{bx} - 0.4)^2))$ = **0.156**

Outer compression steel not required to resist sagging

Slab requiring outer tension steel only - bars (sagging)

z_x = min ((0.95 × d_x),(d_x×(0.5+√(0.25-K_x/0.9)))) = **152** mm

Neutral axis depth x_x = (d_x - z_x) / 0.45 = **18** mm

Area of tension steel required

A_{sx_req} = abs(m_{sx}) / (1/γ_{ms} × f_y × z_x) = **154** mm²/m

Tension steel

Provide 10 dia bars @ 200 centres ***outer tension steel resisting sagging***

A_{sx_prov} = A_{sx} = **393** mm²/m

Area of outer tension steel provided sufficient to resist sagging

TRANSVERSE BOTTOM STEEL - INNER

Inner layer of transverse steel

Provide 10 dia bars @ 200 centres

A_{sy_prov} = A_{sy} = **393** mm²/m

MAXIMUM DESIGN MOMENTS OVER SUPPORT

Design hogging moment (per m width of slab) m_{sxhog} = **18.1** kNm/m

CONCRETE SLAB DESIGN – HOGGING – OUTER LAYER OF STEEL (CL 3.5.4)

Design hogging moment (per m width of slab) m_{sxhog} = **18.1** kNm/m

Moment Redistribution Factor β_{bx} = **1.0**

Area of reinforcement required

K_{xhog} = abs(m_{sxhog}) / (d_{xhog}^2 × f_{cu}) = **0.018**

K'$_x$ = min (0.156 , (0.402 × (β_{bx} - 0.4)) - (0.18 × (β_{bx} - 0.4)^2)) = **0.156**

Outer compression steel not required to resist hogging

Slab requiring outer tension steel only - bars (hogging)

z_{xhog} = min ((0.95 × d_{xhog}),(d_{xhog}×(0.5+√(0.25-K_{xhog}/0.9)))) = **152** mm

Neutral axis depth x_{xhog} = (d_{xhog} - z_{xhog}) / 0.45 = **18** mm

Area of tension steel required

A_{sxhog_req} = abs(m_{sxhog}) / (1/γ_{ms} × f_y × z_{xhog}) = **274** mm²/m

Tension steel

Provide 10 dia bars @ 200 centres ***outer tension steel resisting hogging***

A_{sxhog_prov} = A_{sxhog} = **393** mm²/m

Area of outer tension steel provided sufficient to resist hogging

TRANSVERSE TOP STEEL - INNER

Inner layer of transverse steel

Provide 10 dia bars @ 200 centres

A_{syhog_prov} = A_{syhog} = **393** mm²/m

Check min and max areas of steel resisting sagging

Total area of concrete A_c = h = **200000** mm²/m

Minimum % reinforcement k = **0.13** %

A_{st_min} = k × A_c = **260** mm²/m

A_{st_max} = 4 % × A_c = **8000** mm²/m

Steel defined:

Outer steel resisting sagging A_{sx_prov} = **393** mm²/m

Area of outer steel provided (sagging) OK

Inner steel resisting sagging A_{sy_prov} = **393** mm²/m

Area of inner steel provided (sagging) OK

Check min and max areas of steel resisting hogging

Total area of concrete A_c = h = **200000** mm²/m

Minimum % reinforcement k = **0.13** %

A_{st_min} = k × A_c = **260** mm²/m

A_{st_max} = 4 % × A_c = **8000** mm²/m

Steel defined:

Outer steel resisting hogging A_{sxhog_prov} = **393** mm²/m

Area of outer steel provided (hogging) OK

Inner steel resisting hogging A_{syhog_prov} = **393** mm²/m

Area of inner steel provided (hogging) OK

SHEAR RESISTANCE OF CONCRETE SLABS (CL 3.5.5)

Outer tension steel resisting sagging moments

Depth to tension steel from compression face d_x = **160** mm

Area of tension reinforcement provided (per m width of slab) A_{sx_prov} = **393** mm²/m

Design ultimate shear force (per m width of slab) V_x = **30** kN/m

Characteristic strength of concrete f_{cu} = **40** N/mm²

Applied shear stress

$v_x = V_x / d_x$ = **0.19** N/mm²

Check shear stress to clause 3.5.5.2

$v_{allowable}$ = min ((0.8 N$^{1/2}$/mm) × √(f_{cu}), 5 N/mm²) = **5.00** N/mm²

Shear stress - OK

Shear stresses to clause 3.5.5.3

Design shear stress

f_{cu_ratio} = if (f_{cu} > 40 N/mm² , 40/25 , f_{cu}/(25 N/mm²)) = **1.600**

v_{cx} = 0.79 N/mm² × min(3,100 × A_{sx_prov} / d_x)$^{1/3}$ × max(0.67,(400 mm / d_x)$^{1/4}$) / 1.25 × $f_{cu_ratio}$$^{1/3}$

v_{cx} = **0.58** N/mm^2

Applied shear stress

v_x = **0.19** N/mm^2

No shear reinforcement required

CONCRETE SLAB DEFLECTION CHECK (CL 3.5.7)

Slab span length l_x = **3.000** m

Design ultimate moment in shorter span per m width m_{sx} = **10** kNm/m

Depth to outer tension steel d_x = **160** mm

Tension steel

Area of outer tension reinforcement provided A_{sx_prov} = **393** mm^2/m

Area of tension reinforcement required A_{sx_req} = **154** mm^2/m

Moment Redistribution Factor β_{bx} = **1.00**

Modification Factors

Basic span / effective depth ratio (Table 3.9) $ratio_{span_depth}$ = **20**

The modification factor for spans in excess of 10m (ref. cl 3.4.6.4) has not been included.

$f_s = 2 \times f_y \times A_{sx_req} / (3 \times A_{sx_prov} \times \beta_{bx})$ = **130.9** N/mm^2

$factor_{tens} = \min (2 , 0.55 + (477 \text{ N/mm}^2 - f_s) / (120 \times (0.9 \text{ N/mm}^2 + m_{sx} / d_x^2)))$ = **2.000**

Calculate Maximum Span

This is a simplified approach and further attention should be given where special circumstances exist. Refer to clauses 3.4.6.4 and 3.4.6.7.

Maximum span $l_{max} = ratio_{span_depth} \times factor_{tens} \times d_x$ = **6.40** m

Check the actual beam span

Actual span/depth ratio l_x / d_x = **18.75**

Span depth limit $ratio_{span_depth} \times factor_{tens}$ = **40.00**

Span/Depth ratio check satisfied

CHECK OF NOMINAL COVER (SAGGING) – (BS8110:PT 1, TABLE 3.4)

Slab thickness h = **200** mm

Effective depth to bottom outer tension reinforcement d_x = **160.0** mm

Diameter of tension reinforcement D_x = **10** mm

Diameter of links L_{diax} = **0** mm

Cover to outer tension reinforcement

$c_{tenx} = h - d_x - D_x / 2$ = **35.0** mm

Nominal cover to links steel

c_{nomx} = c_{tenx} - L_{diax} = **35.0** mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

c_{min} = **35** mm

Cover over steel resisting sagging OK

CHECK OF NOMINAL COVER (HOGGING) – (BS8110:PT 1, TABLE 3.4)

Slab thickness h = **200** mm

Effective depth to bottom outer tension reinforcement d_{xhog} = **160.0** mm

Diameter of tension reinforcement D_{xhog} = **10** mm

Diameter of links $L_{diaxhog}$ = **0** mm

Cover to outer tension reinforcement

$c_{tenxhog}$ = h - d_{xhog} - D_{xhog} / 2 = **35.0** mm

Nominal cover to links steel

$c_{nomxhog}$ = $c_{tenxhog}$ - $L_{diaxhog}$ = **35.0** mm

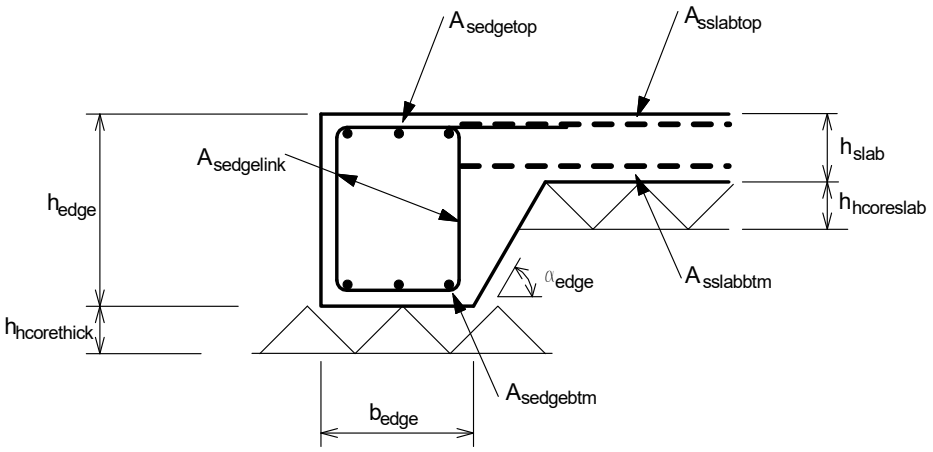
Permissable minimum nominal cover to all reinforcement (Table 3.4)

c_{min} = **35** mm

Cover OK over steel resisting hogging

RAFT FOUNDATION DESIGN (BS8110 : PART 1 : 1997)

Tedds calculation version 1.0.12



Soil and raft definition

Soil definition

Allowable bearing pressure	q_{allow} = **50.0** kN/m^2
Number of types of soil forming sub-soil	**One type only**
Soil density	**Firm to loose**
Depth of hardcore beneath slab	$h_{hcoreslab}$ = **200** mm (Dispersal allowed for bearing pressure check)
Depth of hardcore beneath thickenings	$h_{hcorethick}$ = **200** mm (Dispersal allowed for bearing pressure check)
Density of hardcore	γ_{hcore} = **20.0** kN/m^3
Basic assumed diameter of local depression	$\phi_{depbasic}$ = **2000**mm
Diameter under slab modified for hardcore	$\phi_{depslab}$ = $\phi_{depbasic}$ - $h_{hcoreslab}$ = **1800** mm
Diameter under thickenings modified for hardcore	$\phi_{depthick}$ = $\phi_{depbasic}$ - $h_{hcorethick}$ = **1800** mm

Raft slab definition

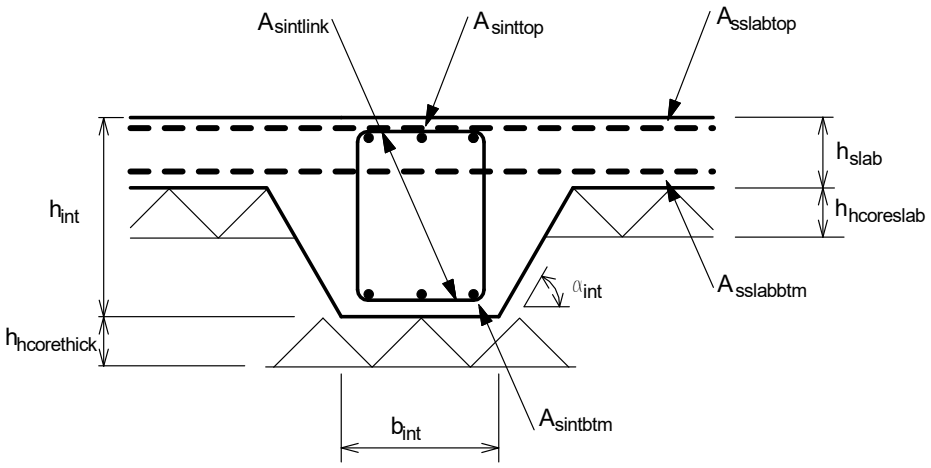
Max dimension/max dimension between joints	l_{max} = **9.215** m
Slab thickness	h_{slab} = **200** mm
Concrete strength	f_{cu} = **40** N/mm^2
Poissons ratio of concrete	ν = **0.2**
Slab mesh reinforcement strength	f_{yslab} = **500** N/mm^2
Partial safety factor for steel reinforcement	γ_s = **1.15**
From C&CA document 'Concrete ground floors' Table 5	
Minimum mesh required in top for shrinkage	**A142**
Actual mesh provided in top	**A393 ($A_{sslabtop}$ = 393 mm^2/m)**
Mesh provided in bottom	**A393 ($A_{sslabbtm}$ = 393 mm^2/m)**
Top mesh bar diameter	$\phi_{slabtop}$ = **10** mm
Bottom mesh bar diameter	$\phi_{slabbtm}$ = **10** mm
Cover to top reinforcement	c_{top} = **35** mm
Cover to bottom reinforcement	c_{btm} = **35** mm
Average effective depth of top reinforcement	$d_{tslabav}$ = h_{slab} - c_{top} - $\phi_{slabtop}$ = **155** mm
Average effective depth of bottom reinforcement	$d_{bslabav}$ = h_{slab} - c_{btm} - $\phi_{slabbtm}$ = **155** mm

Overall average effective depth	$d_{slabav} = (d_{tslabav} + d_{bslabav})/2 =$ **155** mm
Minimum effective depth of top reinforcement	$d_{tslabmin} = d_{tslabav} - \phi_{slabtop}/2 =$ **150** mm
Minimum effective depth of bottom reinforcement	$d_{bslabmin} = d_{bslabav} - \phi_{slabbtm}/2 =$ **150** mm

Edge beam definition

Overall depth	$h_{edge} =$ **600** mm
Width	$b_{edge} =$ **450** mm
Angle of chamfer to horizontal	$\alpha_{edge} =$ **45** deg
Strength of main bar reinforcement	$f_y =$ **500** N/mm^2
Strength of link reinforcement	$f_{ys} =$ **500** N/mm^2
Reinforcement provided in top	**3 H16 bars ($A_{sedgetop}$ = 603 mm^2)**
Reinforcement provided in bottom	**3 H16 bars ($A_{sedgebtm}$ = 603 mm^2)**
Link reinforcement provided	**2 H10 legs at 200 ctrs (A_{sv}/s_v = 0.785 mm)**
Bottom cover to links	$c_{beam} =$ **35** mm
Effective depth of top reinforcement	$d_{edgetop} = h_{edge} - c_{top} - \phi_{slabtop} - \phi_{edgelink} - \phi_{edgetop}/2 =$ **537** mm
Effective depth of bottom reinforcement	$d_{edgebtm} = h_{edge} - c_{beam} - \phi_{edgelink} - \phi_{edgebtm}/2 =$ **547** mm

Internal beam definition



Overall depth	$h_{int} =$ **450** mm
Width	$b_{int} =$ **400** mm
Angle of chamfer to horizontal	$\alpha_{int} =$ **45** deg
Strength of main bar reinforcement	$f_y =$ **500** N/mm^2
Strength of link reinforcement	$f_{ys} =$ **500** N/mm^2
Reinforcement provided in top	**3 H20 bars ($A_{sinttop}$ = 942 mm^2)**
Reinforcement provided in bottom	**3 H20 bars ($A_{sintbtm}$ = 942 mm^2)**
Link reinforcement provided	**2 H10 legs at 200 ctrs (A_{sv}/s_v = 0.785 mm)**
Effective depth of top reinforcement	$d_{inttop} = h_{int} - c_{top} - 2 \times \phi_{slabtop} - \phi_{inttop}/2 =$ **385** mm
Effective depth of bottom reinforcement	$d_{intbtm} = h_{int} - c_{beam} - \phi_{intlink} - \phi_{intbtm}/2 =$ **395** mm

Internal slab design checks

Basic loading

Slab self weight	$w_{slab} = 24$ kN/m$^3 \times h_{slab} =$ **4.8** kN/m^2
Hardcore	$w_{hcoreslab} = \gamma_{hcore} \times h_{hcoreslab} =$ **4.0** kN/m^2

Applied loading

Uniformly distributed dead load — w_{Dudl} = **0.0** kN/m^2

Uniformly distributed live load — w_{Ludl} = **2.5** kN/m^2

Internal slab bearing pressure check

Total uniform load at formation level — $w_{udl} = w_{slab} + w_{hcoreslab} + w_{Dudl} + w_{Ludl}$ = **11.3** kN/m^2

PASS - w_{udl} <= q_{allow} - Applied bearing pressure is less than allowable

Internal slab bending and shear check

Applied bending moments

Span of slab — $l_{slab} = \phi_{depslab} + d_{tslabav}$ = **1955** mm

Ultimate self weight udl — $w_{swult} = 1.4 \times w_{slab}$ = **6.7** kN/m^2

Self weight moment at centre — $M_{csw} = w_{swult} \times l_{slab}^2 \times (1 + \nu) / 64$ = **0.5** kNm/m

Self weight moment at edge — $M_{esw} = w_{swult} \times l_{slab}^2 / 32$ = **0.8** kNm/m

Self weight shear force at edge — $V_{sw} = w_{swult} \times l_{slab} / 4$ = **3.3** kN/m

Moments due to applied uniformly distributed loads

Ultimate applied udl — $w_{udlult} = 1.4 \times w_{Dudl} + 1.6 \times w_{Ludl}$ = **4.0** kN/m^2

Moment at centre — $M_{cudl} = w_{udlult} \times l_{slab}^2 \times (1 + \nu) / 64$ = **0.3** kNm/m

Moment at edge — $M_{eudl} = w_{udlult} \times l_{slab}^2 / 32$ = **0.5** kNm/m

Shear force at edge — $V_{udl} = w_{udlult} \times l_{slab} / 4$ = **2.0** kN/m

Resultant moments and shears

Total moment at edge — $M_{\Sigma e}$ = **1.3** kNm/m

Total moment at centre — $M_{\Sigma c}$ = **0.8** kNm/m

Total shear force — V_{Σ} = **5.2** kN/m

Reinforcement required in top

K factor — $K_{slabtop} = M_{\Sigma e}/(f_{cu} \times d_{tslabav}^2)$ = **0.001**

Lever arm — $z_{slabtop} = d_{tslabav} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabtop}/0.9)})$ = **147.3** mm

Area of steel required for bending — $A_{sslabtopbend} = M_{\Sigma e}/((1.0/\gamma_s) \times f_{yslab} \times z_{slabtop})$ = **20** mm^2/m

Minimum area of steel required — $A_{sslabmin} = 0.0013 \times h_{slab}$ = **260** mm^2/m

Area of steel required — $A_{sslabtopreq} = \max(A_{sslabtopbend}, A_{sslabmin})$ = **260** mm^2/m

PASS - $A_{sslabtopreq}$ <= $A_{sslabtop}$ - Area of reinforcement provided in top to span local depressions is adequate

Reinforcement required in bottom

K factor — $K_{slabbtm} = M_{\Sigma c}/(f_{cu} \times d_{bslabav}^2)$ = **0.001**

Lever arm — $z_{slabbtm} = d_{bslabav} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabbtm}/0.9)})$ = **147.3** mm

Area of steel required for bending — $A_{sslabbtmbend} = M_{\Sigma c}/((1.0/\gamma_s) \times f_{yslab} \times z_{slabbtm})$ = **12** mm^2/m

Area of steel required — $A_{sslabbtmreq} = \max(A_{sslabbtmbend}, A_{sslabmin})$ = **260** mm^2/m

PASS - $A_{sslabbtmreq}$ <= $A_{sslabbtm}$ - Area of reinforcement provided in bottom to span local depressions is adequate

Shear check

Applied shear stress — $v = V_{\Sigma}/d_{tslabmin}$ = **0.035** N/mm^2

Tension steel ratio — $\rho = 100 \times A_{sslabtop}/d_{tslabmin}$ = **0.262**

From BS8110-1:1997 - Table 3.8

Design concrete shear strength — v_c = **0.604** N/mm^2

PASS - v <= v_c - Shear capacity of the slab is adequate

Internal slab deflection check

Basic allowable span to depth ratio — $Ratio_{basic}$ = **26.0**

Moment factor	$M_{factor} = M_{\Sigma c}/d_{bslabav}^2 =$ **0.032** N/mm^2
Steel service stress	$f_s = 2/3 \times f_{yslab} \times A_{sslabbtmbend}/A_{sslabbtm} =$ **10.178** N/mm^2
Modification factor	$MF_{slab} = \min(2.0, 0.55 + [(477N/mm^2 - f_s)/(120 \times (0.9N/mm^2 + M_{factor}))])$
	$MF_{slab} =$ **2.000**
Modified allowable span to depth ratio	$Ratio_{allow} = Ratio_{basic} \times MF_{slab} =$ **52.000**
Actual span to depth ratio	$Ratio_{actual} = l_{slab}/ d_{bslabav} =$ **12.613**
	PASS - $Ratio_{actual}$ <= $Ratio_{allow}$ - Slab span to depth ratio is adequate

Edge beam design checks

Basic loading

Hardcore	$w_{hcorethick} = \gamma_{hcore} \times h_{hcorethick} =$ **4.0** kN/m^2
Edge beam	
Rectangular beam element	$w_{beam} = 24$ kN/m$^3 \times h_{edge} \times b_{edge} =$ **6.5** kN/m
Chamfer element	$w_{chamfer} = 24$ kN/m$^3 \times (h_{edge} - h_{slab})^2/(2 \times \tan(\alpha_{edge})) =$ **1.9** kN/m
Slab element	$w_{slabelmt} = 24$ kN/m$^3 \times h_{slab} \times (h_{edge} - h_{slab})/\tan(\alpha_{edge}) =$ **1.9** kN/m
Edge beam self weight	$w_{edge} = w_{beam} + w_{chamfer} + w_{slabelmt} =$ **10.3** kN/m

Edge load number 1

Load type	**Longitudinal line load**
Dead load	$w_{Dedge1} =$ **10.0** kN/m
Live load	$w_{Ledge1} =$ **2.6** kN/m
Ultimate load	$w_{ultedge1} = 1.4 \times w_{Dedge1} + 1.6 \times w_{Ledge1} =$ **18.2** kN/m
Longitudinal line load width	$b_{edge1} =$ **140** mm
Centroid of load from outside face of raft	$x_{edge1} =$ **70** mm

Edge beam bearing pressure check

Effective bearing width of edge beam	$b_{bearing} = b_{edge} + (h_{edge} - h_{slab})/\tan(\alpha_{edge}) =$ **850** mm
Total uniform load at formation level	$w_{udledge} = w_{Dudl}+w_{Ludl}+w_{edge}/b_{bearing}+w_{hcorethick} =$ **18.6** kN/m^2

Centroid of longitudinal and equivalent line loads from outside face of raft

Load x distance for edge load 1	$Moment_1 = w_{ultedge1} \times x_{edge1} =$ **1.3** kN
Sum of ultimate longitud'l and equivalent line loads	$\Sigma UDL =$ **18.2** kN/m
Sum of load x distances	$\Sigma Moment =$ **1.3** kN
Centroid of loads	$x_{bar} = \Sigma Moment/\Sigma UDL =$ **70** mm

Initially assume no moment transferred into slab due to load/reaction eccentricity

Sum of unfactored longitud'l and eff'tive line loads	$\Sigma UDLsls =$ **12.6** kN/m
Allowable bearing width	$b_{allow} = 2 \times x_{bar} + 2 \times h_{hcoreslab} \times \tan(30) =$ **371** mm
Bearing pressure due to line/point loads	$q_{linepoint} = \Sigma UDLsls/ b_{allow} =$ **34.0** kN/m^2
Total applied bearing pressure	$q_{edge} = q_{linepoint} + w_{udledge} =$ **52.6** kN/m^2
	q_{edge} > q_{allow} - The slab is required to resist a moment due to eccentricity

Now assume moment due to load/reaction eccentricity is resisted by slab

Bearing width required	$b_{req} = \Sigma UDLsls/(q_{allow} - w_{udledge}) =$ **402** mm
Effective bearing width at u/s of slab	$b_{reqeff} = b_{req} - 2 \times h_{hcoreslab} \times \tan(30) =$ **171** mm
Load/reaction eccentricity	$e = b_{reqeff}/2 - x_{bar} =$ **15** mm
Ultimate moment to be resisted by slab	$M_{ecc} = \Sigma UDL \times e =$ **0.3** kNm/m
From slab bending check	
Moment due to depression under slab (hogging)	$M_{\Sigma e} =$ **1.3** kNm/m
Total moment to be resisted by slab top steel	$M_{slabtop} = M_{ecc} + M_{\Sigma e} =$ **1.6** kNm/m

K factor	$K_{slab} = M_{slabtop}/(f_{cu} \times d_{tslabmin}^2) =$ **0.002**
Lever arm	$z_{slab} = d_{tslabmin} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slab}/0.9)}) =$ **143** mm
Area of steel required	$A_{sslabreq} = M_{slabtop}/((1.0/\gamma_s) \times f_y \times z_{slab}) =$ **25** mm²/m

PASS - $A_{sslabreq}$ <= $A_{sslabtop}$ - Area of reinforcement provided to transfer moment into slab is adequate

The allowable bearing pressure under the edge beam will not be exceeded

Library item - B pressure with moment transfer

Edge beam bending check

Divider for moments due to udl's	$\beta_{udl} =$ **10.0**

Applied bending moments

Span of edge beam	$l_{edge} = \phi_{depthick} + d_{edgetop} =$ **2337** mm
Ultimate self weight udl	$w_{edgeult} = 1.4 \times w_{edge} =$ **14.4** kN/m
Ultimate slab udl (approx)	$w_{edgeslab} = \max(0\ kN/m, 1.4 \times w_{slab} \times ((\phi_{depthick}/2 \times 3/4) - (b_{edge} + (h_{edge} - h_{slab})/\tan(\alpha_{edge}))))$
	$w_{edgeslab} =$ **0.0** kN/m
Self weight and slab bending moment	$M_{edgesw} = (w_{edgeult} + w_{edgeslab}) \times l_{edge}^2/\beta_{udl} =$ **7.9** kNm
Self weight shear force	$V_{edgesw} = (w_{edgeult} + w_{edgeslab}) \times l_{edge}/2 =$ **16.9** kN

Moments due to applied uniformly distributed loads

Ultimate udl (approx)	$w_{edgeudl} = w_{udlult} \times \phi_{depthick}/2 \times 3/4 =$ **2.7** kN/m
Bending moment	$M_{edgeudl} = w_{edgeudl} \times l_{edge}^2/\beta_{udl} =$ **1.5** kNm
Shear force	$V_{edgeudl} = w_{edgeudl} \times l_{edge}/2 =$ **3.2** kN

Moment and shear due to load number 1

Bending moment	$M_{edge1} = w_{ultedge1} \times l_{edge}^2/\beta_{udl} =$ **9.9** kNm
Shear force	$V_{edge1} = w_{ultedge1} \times l_{edge}/2 =$ **21.2** kN

Resultant moments and shears

Total moment (hogging and sagging)	$M_{\Sigma edge} =$ **19.3** kNm
Maximum shear force	$V_{\Sigma edge} =$ **41.3** kN

Reinforcement required in top

Width of section in compression zone	$b_{edgetop} = b_{edge} =$ **450** mm
Average web width	$b_w = b_{edge} + (h_{edge}/\tan(\alpha_{edge}))/2 =$ **750** mm
K factor	$K_{edgetop} = M_{\Sigma edge}/(f_{cu} \times b_{edgetop} \times d_{edgetop}^2) =$ **0.004**
Lever arm	$z_{edgetop} = d_{edgetop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{edgetop}/0.9)}) =$ **510** mm
Area of steel required for bending	$A_{sedgetopbend} = M_{\Sigma edge}/((1.0/\gamma_s) \times f_y \times z_{edgetop}) =$ **87** mm²
Minimum area of steel required	$A_{sedgetopmin} = 0.0013 \times 1.0 \times b_w \times h_{edge} =$ **585** mm²
Area of steel required	$A_{sedgetopreq} = \max(A_{sedgetopbend}, A_{sedgetopmin}) =$ **585** mm²

PASS - $A_{sedgetopreq}$ <= $A_{sedgetop}$ - Area of reinforcement provided in top of edge beams is adequate

Reinforcement required in bottom

Width of section in compression zone	$b_{edgebtm} = b_{edge} + (h_{edge} - h_{slab})/\tan(\alpha_{edge}) + 0.1 \times l_{edge} =$ **1084** mm
K factor	$K_{edgebtm} = M_{\Sigma edge}/(f_{cu} \times b_{edgebtm} \times d_{edgebtm}^2) =$ **0.001**
Lever arm	$z_{edgebtm} = d_{edgebtm} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{edgebtm}/0.9)}) =$ **520** mm
Area of steel required for bending	$A_{sedgebtmbend} = M_{\Sigma edge}/((1.0/\gamma_s) \times f_y \times z_{edgebtm}) =$ **85** mm²
Minimum area of steel required	$A_{sedgebtmmin} = 0.0013 \times 1.0 \times b_w \times h_{edge} =$ **585** mm²
Area of steel required	$A_{sedgebtmreq} = \max(A_{sedgebtmbend}, A_{sedgebtmmin}) =$ **585** mm²

PASS - $A_{sedgebtmreq}$ <= $A_{sedgebtm}$ - Area of reinforcement provided in bottom of edge beams is adequate

Edge beam shear check

Applied shear stress	$v_{edge} = V_{\Sigma edge}/(b_w \times d_{edgetop}) =$ **0.102** N/mm²

Tension steel ratio $\rho_{edge} = 100 \times A_{sedgetop}/(b_w \times d_{edgetop}) = \mathbf{0.150}$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength $v_{cedge} = \mathbf{0.393}$ N/mm^2

v_{edge} <= v_{cedge} + 0.4N/mm^2 - Therefore minimum links required

Link area to spacing ratio required $A_{sv_upon_svreqedge} = 0.4\text{N/mm}^2 \times b_w/((1.0/\gamma_s) \times f_{ys}) = \mathbf{0.690}$ mm

Link area to spacing ratio provided $A_{sv_upon_svprovedge} = N_{edgelink} \times \pi \times \phi_{edgelink}^2/(4 \times s_{vedge}) = \mathbf{0.785}$ mm

PASS - $A_{sv_upon_svreqedge}$ <= $A_{sv_upon_svprovedge}$ - Shear reinforcement provided in edge beams is adequate

Corner design checks

Basic loading

Corner bearing pressure check

Total uniform load at formation level $w_{udlcorner} = w_{Dudl} + w_{Ludl} + w_{edge}/b_{bearing} + w_{hcorethick} = \mathbf{18.6}$ kN/m^2

PASS - $w_{udlcorner}$ <= q_{allow} - Applied bearing pressure is less than allowable

Corner beam bending check

Cantilever span of edge beam $l_{corner} = \phi_{depthick}/\sqrt{(2)} + d_{edgetop}/2 = \mathbf{1541}$ mm

Moment and shear due to self weight

Ultimate self weight udl $w_{edgeult} = 1.4 \times w_{edge} = \mathbf{14.4}$ kN/m

Average ultimate slab udl (approx) $w_{cornerslab} = \max(0\text{ kN/m}, 1.4 \times w_{slab} \times (\phi_{depthick}/(\sqrt{(2)} \times 2) - (b_{edge} + (h_{edge} - h_{slab})/\tan(\alpha_{edge}))))$

$w_{cornerslab} = \mathbf{0.0}$ kN/m

Self weight and slab bending moment $M_{cornersw} = (w_{edgeult} + w_{cornerslab}) \times l_{corner}^2/2 = \mathbf{17.2}$ kNm

Self weight and slab shear force $V_{cornersw} = (w_{edgeult} + w_{cornerslab}) \times l_{corner} = \mathbf{22.3}$ kN

Moment and shear due to udls

Maximum ultimate udl $w_{cornerudl} = ((1.4 \times w_{Dudl}) + (1.6 \times w_{Ludl})) \times \phi_{depthick}/\sqrt{(2)} = \mathbf{5.1}$ kN/m

Bending moment $M_{cornerudl} = w_{cornerudl} \times l_{corner}^2/6 = \mathbf{2.0}$ kNm

Shear force $V_{cornerudl} = w_{cornerudl} \times l_{corner}/2 = \mathbf{3.9}$ kN

Resultant moments and shears

Total design moment $M_{\Sigma corner} = M_{cornersw} + M_{cornerudl} = \mathbf{19.2}$ kNm

Total design shear force $V_{\Sigma corner} = V_{cornersw} + V_{cornerudl} = \mathbf{26.2}$ kN

Reinforcement required in top of edge beam

K factor $K_{corner} = M_{\Sigma corner}/(f_{cu} \times b_{edgetop} \times d_{edgetop}^2) = \mathbf{0.004}$

Lever arm $z_{corner} = d_{edgetop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{corner}/0.9)}) = \mathbf{510}$ mm

Area of steel required for bending $A_{scornerbend} = M_{\Sigma corner}/((1.0/\gamma_s) \times f_y \times z_{corner}) = \mathbf{86}$ mm^2

Minimum area of steel required $A_{scornermin} = A_{sedgetopmin} = \mathbf{585}$ mm^2

Area of steel required $A_{scorner} = \max(A_{scornerbend}, A_{scornermin}) = \mathbf{585}$ mm^2

PASS - $A_{scorner}$ <= $A_{sedgetop}$ - Area of reinforcement provided in top of edge beams at corners is adequate

Corner beam shear check

Average web width $b_w = b_{edge} + (h_{edge}/\tan(\alpha_{edge}))/2 = \mathbf{750}$ mm

Applied shear stress $v_{corner} = V_{\Sigma corner}/(b_w \times d_{edgetop}) = \mathbf{0.065}$ N/mm^2

Tension steel ratio $\rho_{corner} = 100 \times A_{sedgetop}/(b_w \times d_{edgetop}) = \mathbf{0.150}$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength $v_{ccorner} = \mathbf{0.365}$ N/mm^2

v_{corner} <= $v_{ccorner}$ + 0.4N/mm^2 - Therefore minimum links required

Link area to spacing ratio required $A_{sv_upon_svreqcorner} = 0.4\text{N/mm}^2 \times b_w/((1.0/\gamma_s) \times f_{ys}) = \mathbf{0.690}$ mm

Link area to spacing ratio provided $A_{sv_upon_svprovedge} = N_{edgelink} \times \pi \times \phi_{edgelink}^2/(4 \times s_{vedge}) = \mathbf{0.785}$ mm

PASS - $A_{sv}_upon_s_{vreqcorner}$ <= $A_{sv}_upon_s_{vprovedge}$ - Shear reinforcement provided in edge beams at corners is adequate

Corner beam deflection check

Basic allowable span to depth ratio — $Ratio_{basiccorner}$ = **7.0**

Moment factor — $M_{factorcorner} = M_{\Sigma corner}/(b_{edgetop} \times d_{edgetop}^2)$ = **0.148** N/mm²

Steel service stress — $f_{scorner} = 2/3 \times f_y \times A_{scornerbend}/A_{sedgetop}$ = **47.779** N/mm²

Modification factor — $MF_{corner}=\min(2.0,0.55+[(477N/mm^2-f_{scorner})/(120\times(0.9N/mm^2+M_{factorcorner}))])$

MF_{corner} = **2.000**

Modified allowable span to depth ratio — $Ratio_{allowcorner} = Ratio_{basiccorner} \times MF_{corner}$ = **14.000**

Actual span to depth ratio — $Ratio_{actualcorner} = l_{corner}/ d_{edgetop}$ = **2.870**

PASS - $Ratio_{actualcorner}$ <= $Ratio_{allowcorner}$ - Edge beam span to depth ratio is adequate

Internal beam design checks

Basic loading

Hardcore — $w_{hcorethick} = \gamma_{hcore} \times h_{hcorethick}$ = **4.0** kN/m²

Internal beam self weight — $w_{int}=24\ kN/m^3\times[(h_{int}\times b_{int})+(h_{int}-h_{slab})^2/\tan(\alpha_{int})+2\times h_{slab}\times(h_{int}-h_{slab})/\tan(\alpha_{int})]$

w_{int} = **8.2** kN/m

Internal beam bearing pressure check

Total uniform load at formation level — $w_{udlint} = w_{Dudl}+w_{Ludl}+w_{hcorethick}+24kN/m^3\times h_{int}$ = **17.3** kN/m²

PASS - w_{udlint} <= q_{allow} - Applied bearing pressure is less than allowable

Internal beam bending check

Divider for moments due to udl's — β_{udl} = **10.0**

Applied bending moments

Span of internal beam — $l_{int} = \phi_{depthick} + d_{inttop}$ = **2185** mm

Ultimate self weight udl — $w_{intult} = 1.4 \times w_{int}$ = **11.5** kN/m

Ultimate slab udl (approx) — $w_{intslab} = \max(0\ kN/m,1.4\times w_{slab}\times((\phi_{depthick}\times 3/4)-(b_{int}+2\times(h_{int}-h_{slab})/\tan(\alpha_{int}))))$

$w_{intslab}$ = **3.0** kN/m

Self weight and slab bending moment — $M_{intsw} = (w_{intult} + w_{intslab}) \times l_{int}^2/\beta_{udl}$ = **6.9** kNm

Self weight shear force — $V_{intsw} = (w_{intult} + w_{intslab}) \times l_{int}/2$ = **15.9** kN

Moments due to applied uniformly distributed loads

Ultimate udl (approx) — $w_{intudl} = w_{udlult} \times \phi_{depthick} \times 3/4$ = **5.4** kN/m

Bending moment — $M_{intudl} = w_{intudl} \times l_{int}^2/\beta_{udl}$ = **2.6** kNm

Shear force — $V_{intudl} = w_{intudl} \times l_{int}/2$ = **5.9** kN

Resultant moments and shears

Total moment (hogging and sagging) — $M_{\Sigma int}$ = **9.5** kNm

Maximum shear force — $V_{\Sigma int}$ = **21.8** kN

Reinforcement required in top

Width of section in compression zone — $b_{inttop} = b_{int}$ = **400** mm

Average web width — $b_{wint} = b_{int} + h_{int}/\tan(\alpha_{int})$ = **850** mm

K factor — $K_{inttop} = M_{\Sigma int}/(f_{cu} \times b_{inttop} \times d_{inttop}^2)$ = **0.004**

Lever arm — $z_{inttop} = d_{inttop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{inttop}/0.9)})$ = **366** mm

Area of steel required for bending — $A_{sinttopbend} = M_{\Sigma int}/((1.0/\gamma_s) \times f_y \times z_{inttop})$ = **60** mm²

Minimum area of steel — $A_{sinttopmin} = 0.0013 \times b_{wint} \times h_{int}$ = **497** mm²

Area of steel required — $A_{sinttopreq} = \max(A_{sinttopbend}, A_{sinttopmin})$ = **497** mm²

PASS - $A_{sinttopreq}$ <= $A_{sinttop}$ - Area of reinforcement provided in top of internal beams is adequate

Reinforcement required in bottom

Width of section in compression zone $b_{intbtm} = b_{int} + 2 \times (h_{int} - h_{slab})/\tan(\alpha_{int}) + 0.2 \times l_{int}$ = **1337** mm

K factor $K_{intbtm} = M_{\Sigma int}/(f_{cu} \times b_{intbtm} \times d_{intbtm}^2)$ = **0.001**

Lever arm $z_{intbtm} = d_{intbtm} \times \min(0.95, 0.5 + \sqrt{}(0.25 - K_{intbtm}/0.9))$ = **375** mm

Area of steel required for bending $A_{sintbtmbend} = M_{\Sigma int}/((1.0/\gamma_s) \times f_y \times z_{intbtm})$ = **58** mm^2

Minimum area of steel required $A_{sintbtmmin} = 0.0013 \times 1.0 \times b_{wint} \times h_{int}$ = **497** mm^2

Area of steel required $A_{sintbtmreq} = \max(A_{sintbtmbend}, A_{sintbtmmin})$ = **497** mm^2

PASS - $A_{sintbtmreq}$ <= $A_{sintbtm}$ - Area of reinforcement provided in bottom of internal beams is adequate

Internal beam shear check

Applied shear stress $v_{int} = V_{\Sigma int}/(b_{wint} \times d_{inttop})$ = **0.067** N/mm^2

Tension steel ratio $\rho_{int} = 100 \times A_{sinttop}/(b_{wint} \times d_{inttop})$ = **0.288**

From BS8110-1:1997 - Table 3.8

Design concrete shear strength v_{cint} = **0.488** N/mm^2

v_{int} <= v_{cint} + 0.4N/mm^2 - Therefore minimum links required

Link area to spacing ratio required $A_{sv_upon_svreqint} = 0.4\text{N/mm}^2 \times b_{wint}/((1.0/\gamma_s) \times f_{ys})$ = **0.782** mm

Link area to spacing ratio provided $A_{sv_upon_svprovint} = N_{intlink} \times \pi \times \phi_{intlink}^2/(4 \times s_{vint})$ = **0.785** mm

PASS - $A_{sv_upon_svreqint}$ <= $A_{sv_upon_svprovint}$ - Shear reinforcement provided in internal beams is adequate

CONCRETE SPECIFICATION

In accordance with BS8500-1:2015 + A2:2019 incorporating corrigenda No.1 and No.2

Tedds calculation version 1.1.01

Element definition

Element description	Structural Concrete
Intended working life	At least 50 years
Type of concrete	Reinforced, normal weight
Maximum aggregate size	20 mm
Allowance for deviation between minimum and nominal covers	
	Δc_{dev} = **10** mm
Allowance for deviation (XA class)	Δc_{dev_s} = **5** mm

Exposure classes

Corrosion induced by carbonation (XC classes) - Table A.1

Class description	Dry or permanently wet
Class designation	XC1

Chemical attack (XA classes) - Table A.10

ACEC class	AC-1
Section width	t = **600** mm
Hydraulic gradient (note B)	Less than or equal to 5
Design chemical class	DC-1

Concrete requirements and specification

Consistence class	S3

Minimum allowable nominal covers

For exposure class XC1 - Table A.4	25 mm
For design chemical class DC-1 - Table A.10	30 mm
Specified nominal cover	35 mm

PASS - The specified nominal cover is adequate

Minimum strength class with 35 mm cover

For exposure class XC1 - Table A.4	C20/25
For design chemical class DC-1 - Table A.12	C25/30
Specified strength class	C32/40

PASS - The specified strength class is adequate

Maximum water/cement ratio with 35 mm cover, 20 mm aggregate and C32/40 concrete

For exposure class XC1 - Table A.4	0.70
For design chemical class DC-1 - Table A.12	0.65
Specified maximum water/cement ratio	0.60

PASS - The specified maximum water cement ratio is acceptable

Minimum cement content with 35 mm cover and C32/40 concrete

For exposure class XC1 - Table A.4	240 kg/m^3
For design chemical class DC-1 - Table A.12	260 kg/m^3
For water/cement ratio of 0.60 - Table A.7	280 kg/m^3
Specified minimum cement content	280 kg/m^3

PASS - The specified minimum cement content is adequate

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Project				Job no.	
Pepper Hill Farm Cottage - New Double Garage				PW-22-01-04	
Calcs for				Start page no./Revision	
Concrete Specification				SPEC/ 2	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
P D Wood	17/02/2022				

Recommended cements/combinations with 35 mm cover and C32/40 concrete

For exposure class XC1 - Table A.4	All in Table A.6
For design chemical class DC-1 - Table A.12	All in Table A.6
Recommended cement/combinations types	CEM I, CEM I-SR0, CEM I-SR3, IIA, IIB-M, IIB-S, IIB-P, IIB-Q, IIB-V, IIB-P + SR, IIB-Q + SR, IIB-V + SR, IIIA, IIIA + SR, IIIB, IIIB + SR,IVB-P, IVB-Q, IVB-V

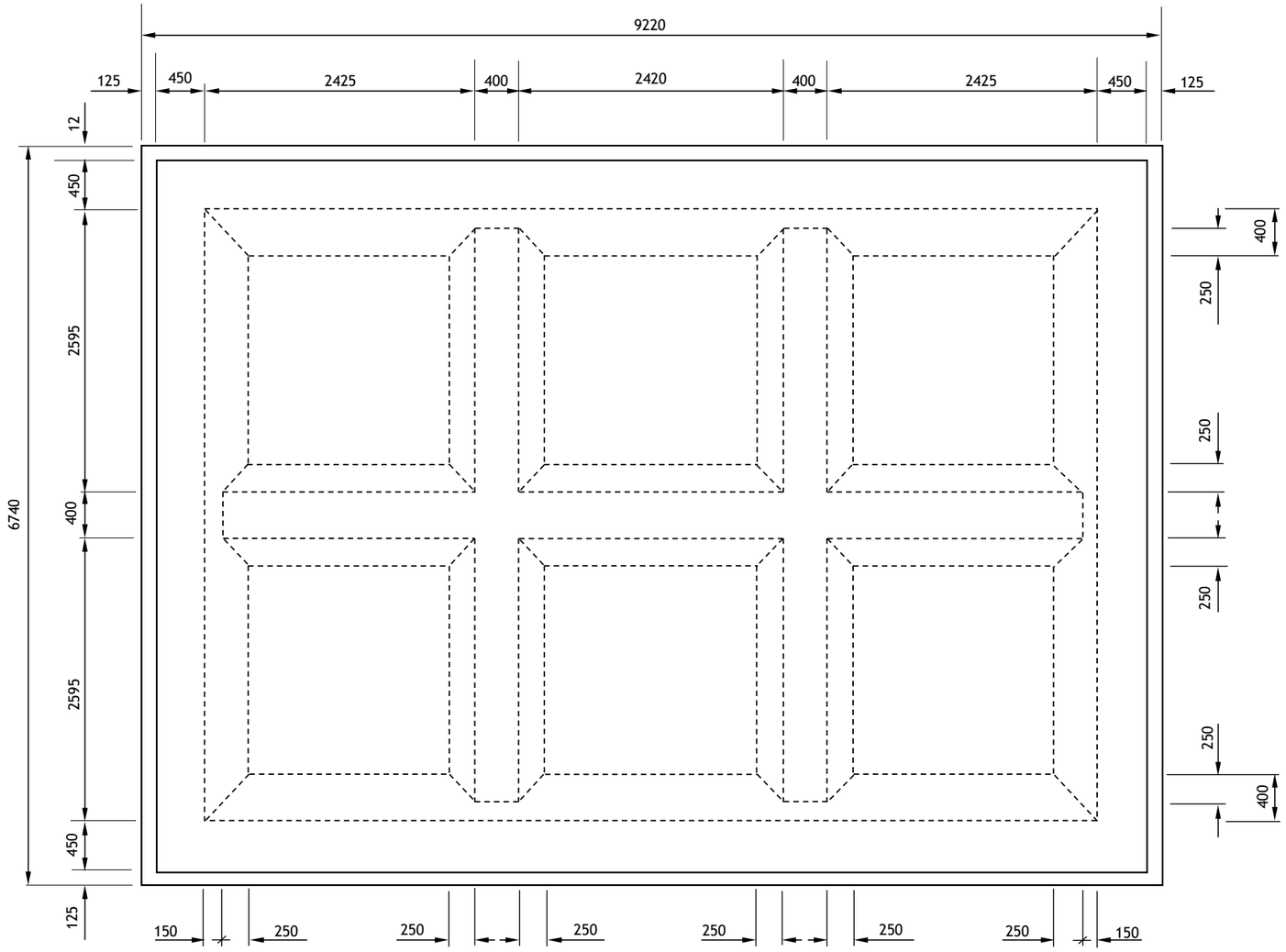
PW Structural Solutions Ltd
19 Coleridge Place, Bourne
Lincolnshire, PE10 9WA

Project				Job no.	
Pepper Hill Farm Cottage - New Double Garage				PW-22-01-04	
Calcs for				Start page no./Revision	
Concrete Specification				SPEC/ 3	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
P D Wood	17/02/2022				

Designed concrete specification for Structural Concrete with 35 mm cover

The concrete shall be produced in accordance with BS8500-2.

Compressive strength class	C32/40
Maximum water/cement ratio	0.60
Minimum cement/combination content	280 kg/m^3
Recommended cement/combinations types	CEM I, CEM I-SR0, CEM I-SR3, IIA, IIB-M, IIB-S, IIB-P, IIB-Q, IIB-V, IIB-P + SR, IIB-Q + SR, IIB-V + SR, IIIA, IIIA + SR, IIIB, IIIB + SR,IVB-P, IVB-Q, IVB-V (DC-1)
Maximum aggregate size	20 mm
Chloride content class	Cl 0, 40
Consistence class	S3



FOUNDATION PLAN (1:50)

NOTES:

1 All dimensions to be confirmed on site.

2 Structural concrete to be grade C32/40. Minimum cement content 280kg/m^3; maximum water:cement ratio 0.6 . Cement types to be agreed with engineer.

3 Blinding concrete to be GEN 1 to BS8500.

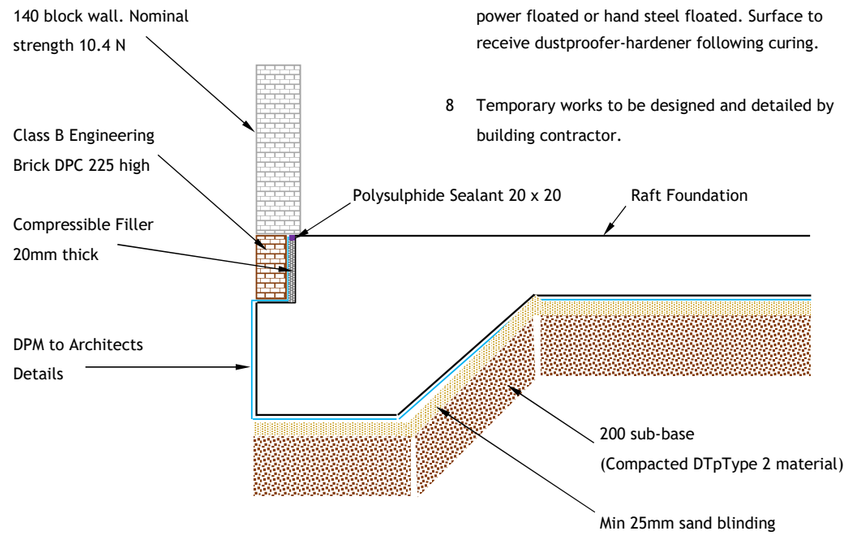
4 All reinforcement to be grade B500B. Splices and laps to be in acordance with BS8110.

5 Cover to reinforcement to be 35mm.

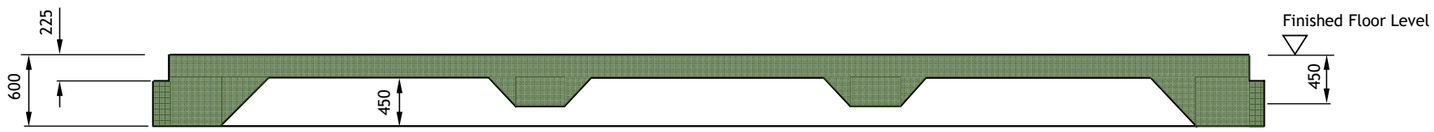
6 Finishes to external sides of the raft foundation to be fair faced.

7 Finished surface to be flat and level. Finish to be power floated or hand steel floated. Surface to receive dustproofer-hardener following curing.

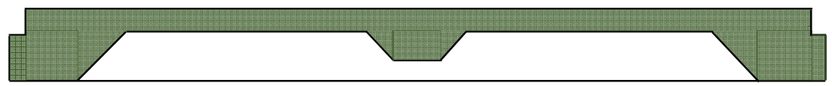
8 Temporary works to be designed and detailed by building contractor.



TYPICAL CONSTRUCTION DETAIL

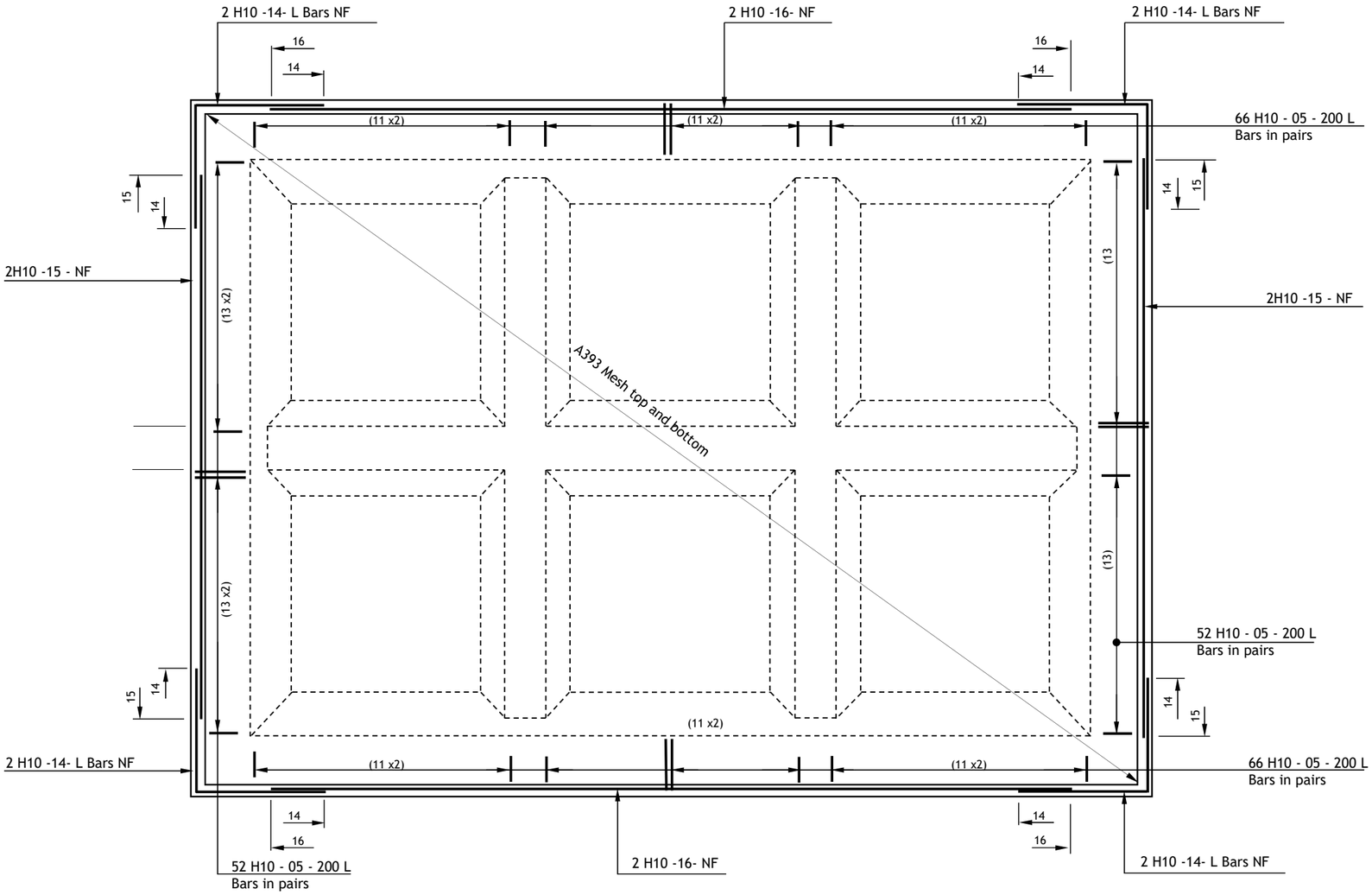


FOUNDATION LONGITUDINAL SECTION (1:50)



FOUNDATION TRANSVERSE SECTION (1:50)

PROJECT TITLE: New Garage - Pepper Hill Farm Cottage	PROJECT NO. PW-22-01-04	DRAWN BY: P D Wood	CHECKED BY:	REVISION P 2		PW STRUCTURAL SOLUTIONS LTD
DRAWING TITLE: Foundation General Arrangement	DRAWING NO. SK/ 1	DATE 17 February 2022	DATE	DATE 2217/02/2022		



SLAB AND EDGE REINFORCEMENT PLAN (1:50)

NOTES:

1 All dimensions to be confirmed on site.

2 Structural concrete to be grade C32/40. Minimum cement content 280kg/m^3; maximum water:cement ratio 0.6 . Cement types to be agreed with engineer.

3 Blinding concrete to be GEN 1 to BS8500.

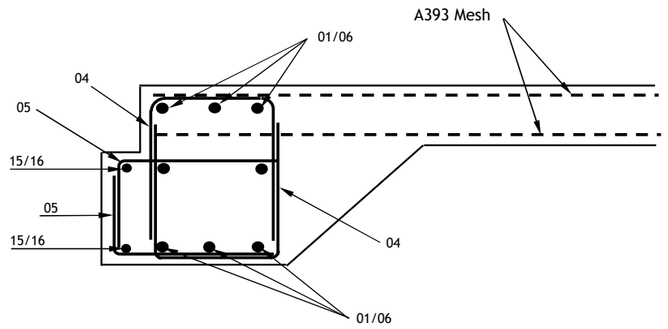
4 All reinforcement to be grade B500B. Splices and laps to be in acordance with BS8110.

5 Cover to reinforcement to be 35mm.

6 Finishes to external sides of the raft foundation to be fair faced.

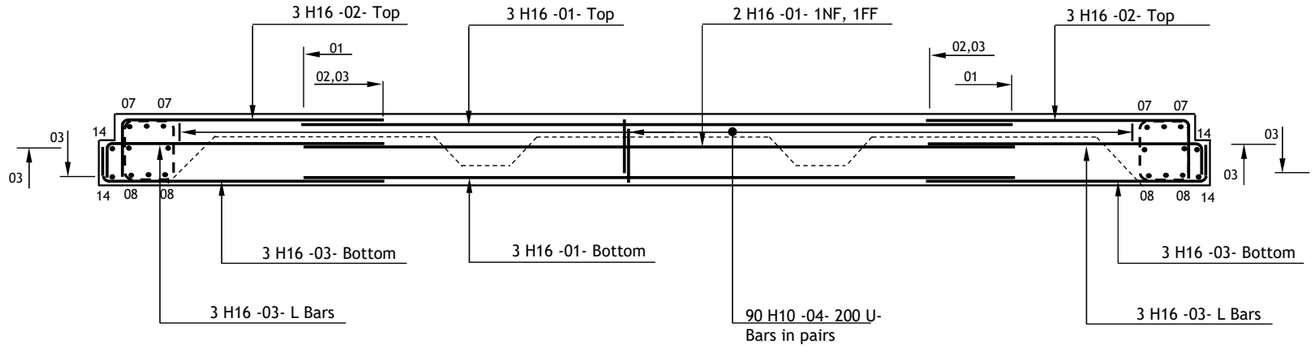
7 Finished surface to be flat and level. Finish to be power floated or hand steel floated. Surface to receive dustproofer-hardener following curing.

8 Temporary works to be designed and detailed by building contractor.

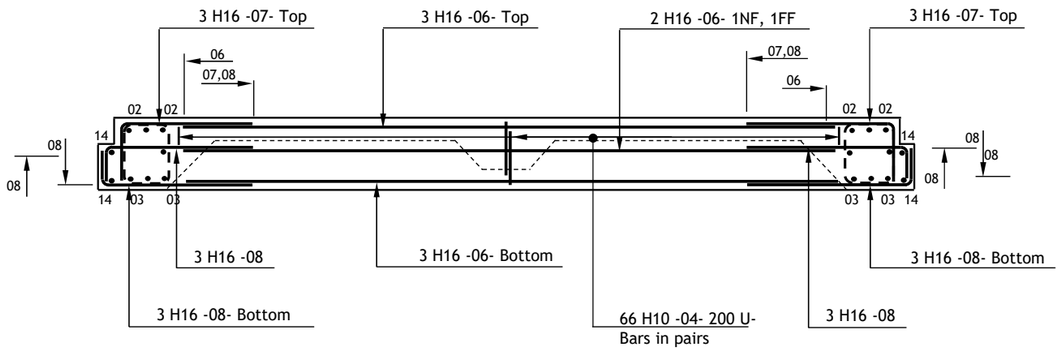


TYPICAL EDGE REINFORCING DETAIL

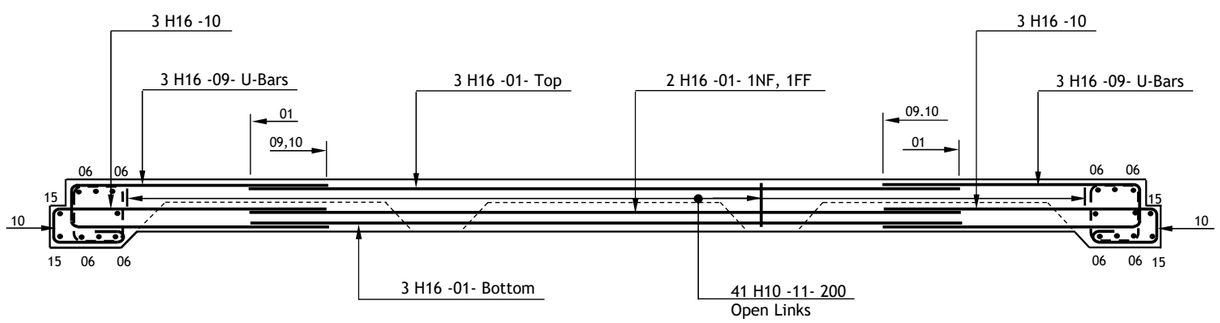
PROJECT TITLE: **New Garage - Pepper Hill Farm Cottage**	PROJECT NO. PW-22-01-04	DRAWN BY: P D Wood	CHECKED BY:	REVISION P 1		PW STRUCTURAL SOLUTIONS LTD
DRAWING TITLE: **Foundation Reinforcement Drawing Sheet 1**	DRAWING NO. SK/ 2	DATE 22 February 2022	DATE	DATE 22 February 2022		



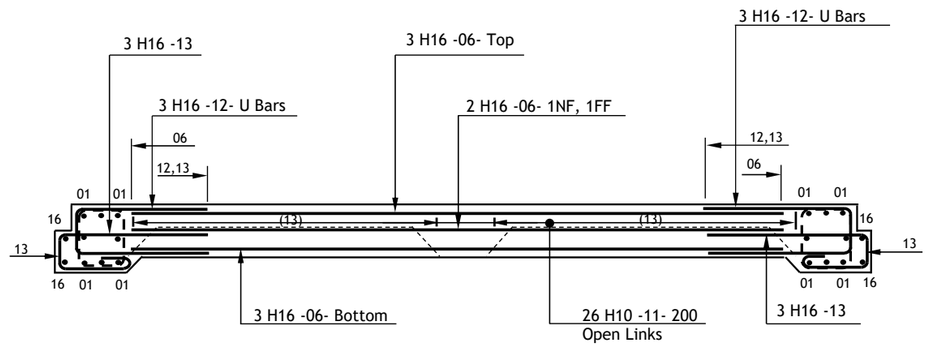
LONG EDGE BEAMS - REINFORCEMENT ELEVATION (2 No. thus)



SHORT EDGE BEAMS - REINFORCEMENT ELEVATION (2 No. thus)



LONG INTERNAL BEAM - REINFORCEMENT ELEVATION (1 No. thus)



SHORT INTERNAL BEAMS - REINFORCEMENT ELEVATION (2 No. thus)

NOTES:

1 All dimensions to be confirmed on site.

2 Structural concrete to be grade C32/40. Minimum cement content 280kg/m^3; maximum water:cement ratio 0.6 . Cement types to be agreed with engineer.

3 Blinding concrete to be GEN 1 to BS8500.

4 All reinforcement to be grade B500B. Splices and laps to be in acordance with BS8110.

5 Cover to reinforcement to be 35mm.

building contractor.

6 Finishes to external sides of the raft foundation to be fair faced.

7 Finished surface to be flat and level. Finish to be power floated or hand steel floated. Surface to receive dustproofer-hardener following curing.

8 Temporary works to be designed and detailed by

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DRAWING TITLE: **Foundation Reinforcement Drawing Sheet 2**	DRAWING NO. SK/ 3	DATE 23 February 2022	DATE	DATE		