

Project Lutterworth Fire Station - Two storey extension			Job Ref. C2978		
Section Structural Calculations			Sheet no./rev. 0		
Calc. by OG	Date Dec. 2017	Chk'd by JMS	Date Dec. 2017	App'd by	Date

STRUCTURAL CALCULATIONS

TWO STOREY EXTENSION

LUTTERWORTH FIRE STATION

GILMORTAN ROAD

LUTTERWORTH

LE17 1DZ

FOR

LEICESTERSHIRE FIRE AND RESCUE

LEICESTERSHIRE FIRE AND RESCUE SERVICE HQ

12 GEOFF MONK WAY

BIRSTALL

LEICESTER

LE4 3BU

Calculations By: **Oliver R Goodyear** MEng

Approved By: **John M Staves** CEng FStructE

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Preamble

Existing Property

The existing property is a detached monopitch roof fire station consisting of a full height appliance bay with space for 2 Fire engines on the North-West side of the building and a 2 storey office and breakout space section on the South East side of the building. The appliance bay has a mezzanine structure locate along the North-West elevation gable, accessible via a cat Ladder

The existing building appeared to be timber mono-pitch roof trusses supported by load bearing masonry. The first floor structure is unknown, but assumed to precast concrete. From trial pit investigations the existing building foundations were found to be unusually shallow with a large concrete protrusion, suggesting that the existing foundations are likely to be a raft type structure or a pile raft/ground beam configuration (see Appendix A for Geotechnical information)

Client's Proposed Development

Its proposed that a new 2 storey extension is adjoined to the North-West elevation to accommodate additional office, locker room and training room space. The proposed extension will be stepped back from the front and rear elevation and will extend out to the side by 7.2metres. The proposed roof layout will be mono-pitch style matching the original building. The proposed extension is to adjoin with the ground floor via a single doorway and the first-floor mezzanine by a doorway also.

Basic Concept Design

The proposed structure is to be similar to the existing form, in that the roof will be prefabricated mono-pitch trusses, spanning the full width of the building and supported by load bearing masonry external walls. The proposed first floor structure will be precast hollowcore planks, spanning front to back between external walls and load bearing internal walls, either side of the staircase. The proposed ground floor will be the same configuration as first floor, except precast beam and block is to be used instead of planks.

From the soils investigation carried out (see Appendix A), there are unfavourable ground conditions within the first couple of metres, such as peat and made ground, therefore the interpretive report recommends footings to either be founded at 2.6m depth or a piled solution is provided. Given the varying soil strata and health and safety implications associated with deep excavations it was deemed appropriate to support the structural walls with piled ground beams. Furthermore, the utility survey shows a significant amount of underground services in line with the existing front elevation wall and a shared sewer intersecting the site near the propped rear extension corner, therefore pile positions will be positioned in consideration of existing services and shared sewer build over application rules

Parties Involved

Client: LFRS

Architect: Unknown - LFRS

Structural Engineer: Michael Aubrey Partnership Ltd

Other Engineers: n/a

Geotechnical Engineer: AF Howland Associates

Principal Designer – Assumed to be Architect

Building Control Authority: Harborough District Council

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Loadings

Codes Used:

BS EN 1991: Eurocode 1 : Actions on Structures
 BS EN 1992: Eurocode 2 : Design of Concrete Structures
 BS EN 1993: Eurocode 3 : Design of Steel Structures
 BS EN 1995: Eurocode 5 : Design of Timber Structures
 BS EN 1996: Eurocode 6 : Design of Masonry Structures
 BS 5977-1 : Lintels - Method for assessment of load

Design Information

Load Factors ULS = 1.35DL + 1.5IL
 SLS = 1.0DL + 1.0IL

- Steelwork - Grade S275
- Masonry - Block strength min. 7.3N/mm² UNO
- Timber - Grade C24, UNO

Deflection Steel beams = span/360 for imposed load
 Steel beams = span/250 for dead + imposed load
 Timber members δ = lesser of(0.003L or 14mm)

Design Loads

Roof

		G _k	Q _k
Mono Truss	Concrete Pantile	0.65 kN/m ²	
With Ceiling	Felt & battens	0.05 kN/m ²	
18° pitch	Insulation	0.05 kN/m ²	
	Rafters	0.10 kN/m ²	
		on slope	
		<u>0.85 kN/m²</u>	
	Allow for 18° pitch DL = 0.85 / COS18° =	0.89 kN/m ²	
	19mm Chipboard	0.15 kN/m ²	
	Ceiling joists	0.10 kN/m ²	
	Services / Insulation	0.05 kN/m ²	
	MF Ceiling	0.15 kN/m ²	
	Live loading on roof		0.75 kN/m ²
	Live loading on ceiling joist		0.25 kN/m ²
		<u>1.34 kN/m²</u>	<u>1.00 kN/m²</u>

First Floor

		G _k	Q _k
150thk Hollowcore	150thk PC planks	3.00 kN/m ²	

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Span upto 7.5m	75mm screed	1.80 kN/m ²	
	Insulation	0.10 kN/m ²	
	Services / Suspended ceiling	0.25 kN/m ²	
	Live loading on floor		3.00 kN/m ²
	Internal partitions, say < 1kN/m		1.20 kN/m ²
		<u>5.15 kN/m²</u>	<u>4.20 kN/m²</u>

Ground Floor

		G_k	Q_k
Beam and Block	Finishes	0.10 kN/m ²	
	75mm screed (@22kN/m ³)	1.80 kN/m ²	
	Insulation	0.05 kN/m ²	
	225dp precast beam and block slab	2.60 kN/m ²	
	Live loading on floor		3.00 kN/m ²
	Internal partitions, say < 2kN/m		1.20 kN/m ²
		<u>4.55 kN/m²</u>	<u>4.20 kN/m²</u>

Wall Construction

External Walls

		G_k	Q_k
Brick facing cavity wall	102.5thk brickwork (@21kN/m ³)	2.15 kN/m ²	
	Insulation	0.05 kN/m ²	
	100thk blockwork (@15kN/m ³)	1.50 kN/m ²	
	Plasterboard and skim	0.15 kN/m ²	
	On Elevation	<u>3.85 kN/m²</u>	
Cavity Wall below DPC	100thk blockwork (@19kN/m ³)	1.90 kN/m ²	
	100mm lean mix (@24kN/m ³)	2.20 kN/m ²	
	100thk blockwork (@19kN/m ³)	1.90 kN/m ²	
	On Elevation	<u>6.00 kN/m²</u>	

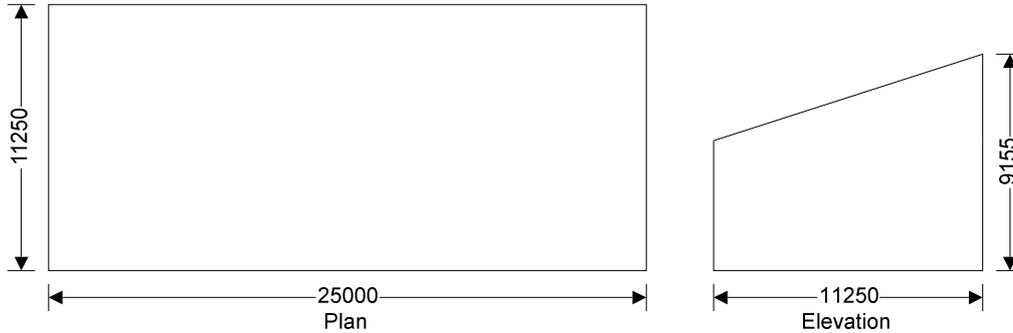
Internal Walls

140 thk Blockwork (medium density)	140thk blockwork (@15kN/m ³)	2.10 kN/m ²	
	Plasterboard and skim x 2	0.30 kN/m ²	
	On Elevation	<u>2.40 kN/m²</u>	
Stud Partitions	Studs & noggins, say	0.10 kN/m ²	
	Insulation	0.05 kN/m ²	
	Plasterboard x 2	0.30 kN/m ²	
	On Elevation	<u>0.45 kN/m²</u>	

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WIND LOADING (EN1991-1-4)

TEDDS calculation version 3.0.17



Building data

Type of roof	Monopitch			
Length of building	L = 25000 mm	Width of building	W = 11250 mm	
Pitch of roof	$\alpha_0 = 18.0$ deg			
Total height	h = 9155 mm			

Basic values

Location	Leicester			
Wind speed velocity (FigNA.1)	$v_{b,map} = 21.6$ m/s	Distance to shore	$L_{shore} = 100.00$ km	
Altitude above sea level	$A_{alt} = 125.0$ m	Altitude factor	$C_{alt} = 1.125$	
Fund wind speed velocity	$v_{b,0} = 24.3$ m/s	Direction factor	$C_{dir} = 1.00$	
Season factor	$C_{season} = 1.00$	Probability factor	$C_{prob} = 1.00$	
Basic wind speed (Exp. 4.1)	$v_b = 24.3$ m/s	Ref mean velocity pressure	$q_b = 0.362$ kN/m ²	

Orography

Orography factor not signif	$C_o = 1.0$
Terrain category	Town
Displacement height	$H_d = 0$ mm

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

Peak velocity pressure - windward wall - Wind 0 deg

Reference height	z = 5500 mm	
Exposure factor (Figure NA.7)	$C_e = 1.96$	
Exposure correction factor (Figure NA.8)	$C_{e,T} = 0.96$	
Peak velocity pressure	$q_p = 0.68$ kN/m ²	

Structural factor

Structural damping	$\delta_s = 0.100$	Size factor (Table NA.3)	$C_s = 0.81$
Dynamic factor (Figure NA.9)	$C_d = 1.00$	Structural factor	$C_s C_d = 0.917$

Peak velocity pressure - other walls and roof

Reference height	z = 9155 mm	
Exposure factor (Figure NA.7)	$C_e = 2.27$	
Exposure correction factor (Figure NA.8)	$C_{e,T} = 1.00$	
Peak velocity pressure	$q_p = 0.82$ kN/m ²	

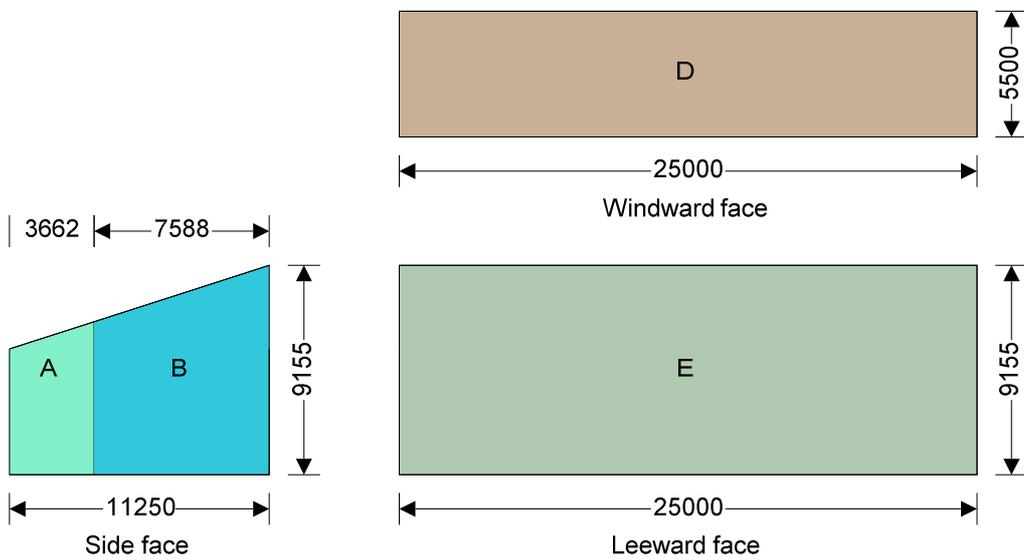
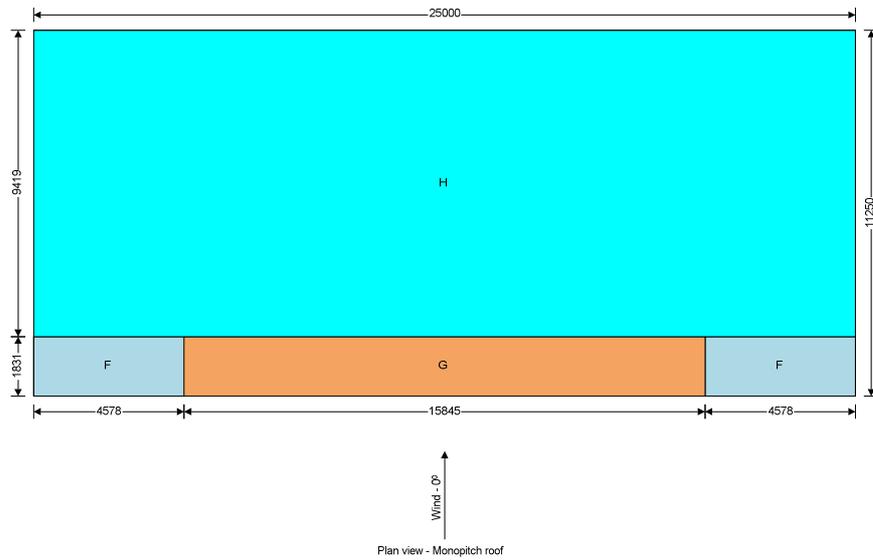
Structural factor - roof 0 deg

Structural damping	$\delta_s = 0.100$	Size factor (Table NA.3)	$C_s = 0.90$
Dynamic factor (Figure NA.9)	$C_d = 1.00$	Structural factor	$C_s C_d = 0.917$

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Peak velocity pressure for internal pressure

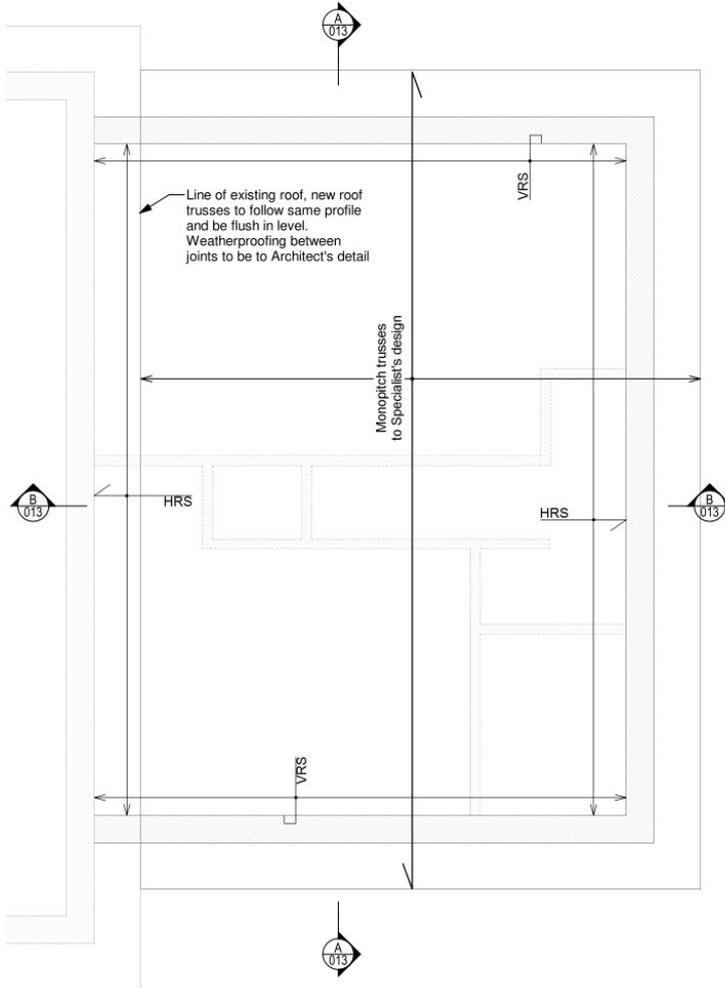
Peak velocity pressure – internal (as roof press.) $q_{p,i} = 0.82 \text{ kN/m}^2$



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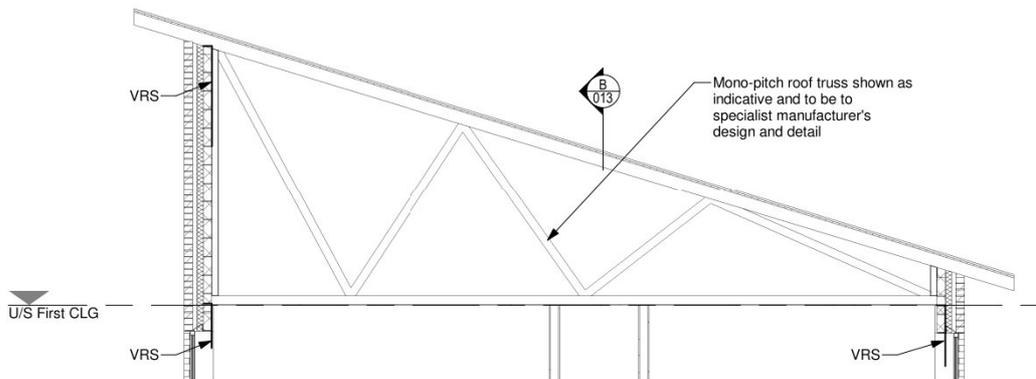
Roof Structure Design

Roof Truss Statement



Structure
Proposed Roof Level Plan
 Scale 1 : 50

Design Assumptions
The following assumptions have been made to process the design & must be confirmed/clarified on site by the contractor prior to works. If found to be otherwise, Engineer to be notified immediately.
1) Trusses to span full width of building, supported off external walls only
2) Existing roof assumed to be prefabricated roof trusses spanning full width and not supported off internal walls
Roof Trusses
Roof shown as indicative only, roof to be designed by truss roof specialist, strictly in accordance with BSEN 1991 and BS 1995. Specialist design must be co-ordinated with Engineer prior to construction



Roof truss to be designed by specialist in accordance with BS EN 1995.

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Roof External Walls

Front Elevation Wall (High Eaves)

MASONRY WALL PANEL DESIGN

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

Tedds calculation version 1.2.11

Masonry panel details

Front Wall beneath monopitch roof - Reinforced masonry wall without openings

Panel length L = 6900 mm Panel height h = 3150 mm

Panel support conditions

Outer leaf All edges supported, bottom and right continuous

Inner leaf All edges supported, bottom and right continuous

Effective height of masonry walls - Section 5.5.1.2

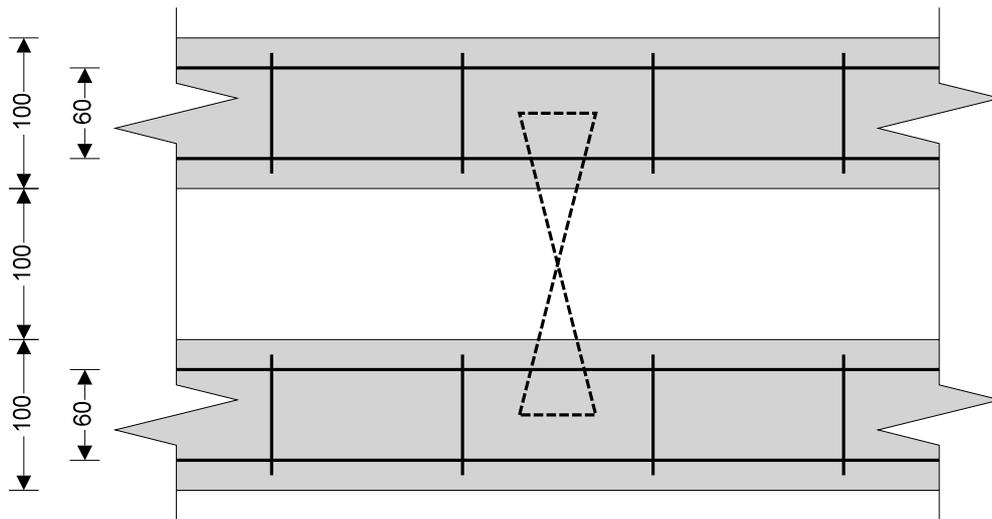
Reduction factor $\rho_2 = 1.000$ Effective height of wall $h_{ef} = 3150$ mm



Cavity wall construction details

Outer leaf thickness $t_1 = 100$ mm Cavity thickness $t_c = 100$ mm
Inner leaf thickness $t_2 = 100$ mm Effective wall thickness $t_{ef} = 126$ mm

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Masonry outer leaf details

Masonry type	Clay with water absorption less than 7% - Group 1		
Compressive strength	$f_{c1} = 20 \text{ N/mm}^2$	Norm mean comp.strength	$f_{b1} = 17 \text{ N/mm}^2$
Density of masonry	$\gamma_1 = 21 \text{ kN/m}^3$		
Mortar type	M6 - General purpose mortar		
Comp.strength of mortar	$f_{m1} = 6 \text{ N/mm}^2$	Compressive strength factor	$K = 0.50$
Char.comp.strength masonry	$f_{k1} = 6.219 \text{ N/mm}^2$		
Char.flex.str.parallel to bed	$f_{xk11} = 0.5 \text{ N/mm}^2$	Char.flex.str.perp.to bed	$f_{xk21} = 1.5 \text{ N/mm}^2$

Outer leaf bed joint reinforcement details

Diameter of reinforcement	$\phi_1 = 4.00 \text{ mm}$	Horizontal spacing of bars	$s_1 = 60 \text{ mm}$
Masonry course height	$h_{b1} = 215 \text{ mm}$	Courses between layers	$C_1 = 2$
Vertical spacing of bars	$s_{v1} = 430 \text{ mm}$	Area of reinforcement	$A_{s1} = 29.2 \text{ mm}^2/\text{m}$

Masonry inner leaf details

Masonry type	Aggregate concrete - Group 1		
Compressive strength	$f_{c2} = 3.6 \text{ N/mm}^2$	Norm mean comp.strength	$f_{b2} = 4.968 \text{ N/mm}^2$
Density of masonry	$\gamma_2 = 18 \text{ kN/m}^3$		
Mortar type	M6 - General purpose mortar		
Comp.strength of mortar	$f_{m2} = 6 \text{ N/mm}^2$	Compressive strength factor	$K = 0.75$
Char.comp.strength masonry	$f_{k2} = 3.726 \text{ N/mm}^2$		
Char.flex.str.parallel to bed	$f_{xk12} = 0.25 \text{ N/mm}^2$	Char.flex.str.perp.to bed	$f_{xk22} = 0.505 \text{ N/mm}^2$

Inner leaf bed joint reinforcement details

Diameter of reinforcement	$\phi_2 = 4.00 \text{ mm}$	Horizontal spacing of bars	$s_2 = 60 \text{ mm}$
Masonry course height	$h_{b2} = 215 \text{ mm}$	Courses between layers	$C_2 = 2$
Vertical spacing of bars	$s_{v2} = 430 \text{ mm}$	Area of reinforcement	$A_{s2} = 29.2 \text{ mm}^2/\text{m}$

Lateral loading details

Characteristic wind load on panel	$W_k = 0.900 \text{ kN/m}^2$
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Vertical loading details

Permanent load on top of inner leaf	$G_{k2} = 6 \text{ kN/m}$
Variable load on top of inner leaf	$Q_{k2} = 4.5 \text{ kN/m}$

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Partial factors for material strength

Category of manufacture	Category I	Class of execution	Class 2
Partial factor for compression	$\gamma_{Mc} = 2.70$	Partial factor for tension	$\gamma_{Mt} = 2.70$
Partial factor for shear	$\gamma_{Mv} = 2.50$	Partial factor for reinforcement	$\gamma_{Ms} = 1.15$

Slenderness ratio of masonry walls - Section 5.5.1.4

Allowable slenderness ratio	$SR_{all} = 27$	Slenderness ratio	$SR = 25.0$
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PASS - Slenderness ratio is less than maximum allowable

Unreinforced masonry walls subjected to mainly vertical loading - Section 6.1

Partial safety factors for design loads

Partial factor for dead load	$\gamma_{FG} = 1.35$	Partial factor for imposed load	$\gamma_{FQ} = 1.5$
Partial factor for wind load	$\gamma_{FW} = 0.75$		

Considering inner leaf

Reduction factor for slenderness and eccentricity - Section 6.1.2.2

Vertical load at bottom of wall	$N_{id2} = 22.505$ kN/m	Moment at bottom of wall	$M_{id2} = 0$ kNm/m
Eccentricity at bottom of wall	$e_{i2} = 7.9$ mm	Reduction factor at bottom of wall	$\Phi_{i2} = 0.843$
Vertical load at mid wall	$N_{md2} = 18.677$ kN/m	Moment at mid wall	$M_{md2} = 0$ kNm/m
Eccentricity at mid wall	$e_{mk2} = 8.1$ mm	Reduction factor at mid wall	$\Phi_{m2} = 0.436$
		Reduction factor	$\Phi_2 = 0.436$

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

Design value of vertical load	$N_{Ed2} = 22.505$ kN/m	Vertical resistance of wall	$N_{Rd2} = 60.145$ kN/m
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PASS - Design vertical resistance exceeds applied design vertical load

Reduction factor for slenderness and eccentricity - Section 6.1.2.2

Vertical load at bottom of wall	$N_{id1} = 8.93$ kN/m	Moment at bottom of wall	$M_{id1} = 0$ kNm/m
Eccentricity at bottom of wall	$e_{i1} = 21$ mm	Reduction factor at bottom of wall	$\Phi_{i1} = 0.58$
Vertical load at mid wall	$N_{md1} = 4.465$ kN/m	Moment at mid wall	$M_{md1} = 0$ kNm/m
Eccentricity at mid wall	$e_{mk1} = 35$ mm	Reduction factor at mid wall	$\Phi_{m1} = 0.023$
		Reduction factor	$\Phi_1 = 0.023$

Unreinforced masonry walls subjected to lateral loading - Section 6.3

Partial safety factors for design loads

Partial factor for dead load	$\gamma_{FG} = 1$	Partial factor for imposed load	$\gamma_{FQ} = 0$
Partial factor for wind load	$\gamma_{FW} = 1.5$		

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

Limiting h / t ratio	46.309	Height to thickness ratio	25.002 = 25.002
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PASS - Limiting height to thickness ratio is not exceeded

Design moments of resistance in panels

Self weight of wall	$S_{wt1} = 3.308$ kN/m	Design vertical comp.stress	$\sigma_{d1} = 0.008$ N/mm ²
Des.flex.str.mas.parallel to bed	$f_{xd11} = 0.185$ N/mm ²	App.flex.str.mas.parallel to bed	$f_{xd11,app} = 0.193$ N/mm ²
Elastic section modulus	$Z_1 = 1666667$ mm ³ /m	Mom.of resist parallel	$M_{Rd11} = 0.322$ kNm/m
Design strength of steel	$f_{yd} = 435$ N/mm ²	Depth of reinforcement	$d_1 = 80$ mm
Lever arm	$z_1 = 76$ mm	Mom.of resist.perpendicular	$M_{Rd21} = 0.966$ kNm/m
App.flex.str.mas.parallel to bed	$f_{xd21,app} = 0.579$ N/mm ²		
Self weight of wall	$S_{wt2} = 2.835$ kN/m	Design vertical comp.stress	$\sigma_{d2} = 0.088$ N/mm ²
Des.flex.str.mas.parallel to bed	$f_{xd12} = 0.093$ N/mm ²	App.flex.str.mas.parallel to bed	$f_{xd12,app} = 0.181$ N/mm ²
Elastic section modulus	$Z_2 = 1666667$ mm ³ /m	Mom.of resist parallel	$M_{Rd12} = 0.302$ kNm/m

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Design strength of steel $f_{yd} = 435 \text{ N/mm}^2$ Depth of reinforcement $d_2 = 80 \text{ mm}$
Lever arm $Z_2 = 75.4 \text{ mm}$ Mom.of resist.perpendicular $M_{Rd22} = 0.958 \text{ kNm/m}$
App.flex.str.mas.parallel to bed $f_{xd22,app} = 0.575 \text{ N/mm}^2$

Calculate design wind load acting on each leaf

Outer leaf design wind load $W_{k1} = 0.452 \text{ kN/m}^2$ Inner leaf design wind load $W_{k2} = 0.448 \text{ kN/m}^2$

Design moment in panels

Design moment in wall $M_{Ed1} = \gamma_{fW} \times \alpha_1 \times W_{k1} \times L^2 = 0.826 \text{ kNm/m}$

PASS - Resistance moment exceeds design moment

Design moment in wall $M_{Ed2} = \gamma_{fW} \times \alpha_2 \times W_{k2} \times L^2 = 0.850 \text{ kNm/m}$

PASS - Resistance moment exceeds design moment

Provide Brick/Block Cavity Wall construction consisting of;

- 102.5w Brickwork – min 20N/mm² strength with 7-12% assumed water absorption
- 100w Blockwork – min 3.6N/mm² strength
- Masonry Reinforcement to both inner and outer skins, using Bekaert Brickforce SBF40W60 @ 440mm vertical centres
- Wall ties as specified on drawing

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First Level GA Structure

Lintel Design

Front and Rear Elevation Windows – Clear Span = 1000mm

FIRST FLOOR LINTEL - FRONT ELEVATION

$$\begin{aligned} \text{ROOF LOAD} &= p_L = 1.34 \text{ kN/m}^2 \times 8.9 \text{ m} / 2 = 6.0 \text{ kN/m} \\ &= 1.0 \text{ kN/m}^2 \times 8.9 \text{ m} / 2 = 4.5 \text{ kN/m} \end{aligned}$$

$$\text{WALL OVER} = q = 3.5 \text{ m} \times 4 \text{ kN/m}^2 = 14.0 \text{ kN/m}$$

$$\text{TOTAL} = 24.5 \text{ kN/m} \times 1 \text{ m} \times 1.1 = \underline{\underline{27.0 \text{ kN}}}$$

(span)

$$\text{RATIO} = 2.5:1$$

REAR ELEVATION

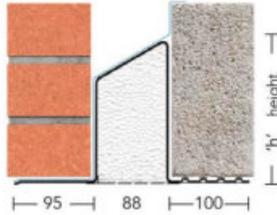
$$\begin{aligned} \text{ROOF} - p_L &= 6.0 \text{ kN/m} & \text{WALL} &= 4 \text{ kN/m}^2 \times 0.6 \text{ m} \\ &= 4.5 \text{ kN/m} & &= 2.4 \text{ kN/m} \end{aligned}$$

$$\text{TOTAL} = 12.9 \text{ kN/m} \times 1 \text{ m} \times 1.1 = \underline{\underline{14.2 \text{ kN}}}$$

$$\text{RATIO} = 8:1$$

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90-105mm Cavity



HD/K-90

For Cavity widths 90-105mm

The blockwork is built tight against inner face of the steel lintel. Place mortar bed on top of blockwork before floor units are laid to provide even distribution of load. Steel Lintels may be propped to facilitate speed of construction.

BIM	Lintel Selector	Price Lists
Installation Guide	CAD	Lintel Brochure

[Click to Copy Architect Specification](#)

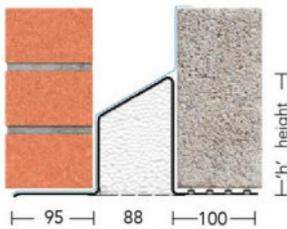
↔	↑	⊕	▲ kN 3:1	▲ kN 19:1
Length (mm)	Height (mm)	Thickness (mm)	Total UDL kN 3:1	Total UDL kN 19:1
600 1200	110	2.9	30	22
1350 1500	135	2.9	30	22

Provide Keystone Lintel – HD/K-90

Side Elevation Windows – Clear Span = 1000mm

$$\text{WALL LOAD} = 2.1\text{m} \times 4.0\text{kN/m}^2 = 8.4\text{kN/m}$$

90-105mm Cavity



S/K-90

For Cavity widths 90-105mm

The blockwork is built tight against inner face of the steel lintel. Place mortar bed on top of blockwork before floor units are laid to provide even distribution of load. Steel Lintels may be propped to facilitate speed of construction.

BIM	Lintel Selector	Price Lists
Installation Guide	CAD	Lintel Brochure

↔	↑	⊕	▲ kN 3:1	▲ kN 19:1
Length (mm)	Height (mm)	Thickness (mm)	Total UDL kN 3:1	Total UDL kN 19:1
600 1200	88	1.6	12	10
1350 1500	85	2.0	16	13

Provide Keystone Lintel – S/K-90

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Opening Through the Existing Gable to Mezzanine

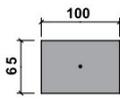
SPAN = 1000

INNER SKIN

WALL SW → SAY $2 \cdot 2 \text{ kN/m}^2 \times 2 \text{ m} = 4 \cdot 4 \text{ kN/m}$

TOTAL LOAD = $4 \cdot 4 \text{ kN/m} \times 1 \text{ m} \times 1 \cdot 1 = 4 \cdot 9 \text{ kN}$

MOMENT = $\frac{4 \cdot 9 \times 1 \cdot 1^2}{8} = 0 \cdot 75 \text{ kNm}$

100 x 65 	STANDARD LENGTHS (mm)	600	750	900	1050	1200	1350	1500	1650	1800	2100	2400	2700	3000
	Clear Span (mm)	300	450	600	750	900	1050	1200	1350	1500	1800			
	Weight (kg/m)	15.3	15.3	15.3	15.3	15.3	15.3	15.3	15.3	15.3	15.3	15.3	15.3	15.3
	UDL (kN/m)	23.9*	23.9*	13.8*	8.9	6.5	5.3	4.1	3.2	2.6	1.8			
	RM (kNm)	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8			

*UDL is restricted by shear
Minimum manufactured length is 450mm Maximum manufactured length is 3000mm

Provide 2No. 100w x 65dp PC Lintels – Supreme or Stressline

External Wall Design – Laterally Loaded Masonry

Front and Rear Elevation

MASONRY WALL PANEL DESIGN

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

Tedds calculation version 1.2.11

Masonry panel details

Front Elevation - Unreinforced masonry wall with openings

Panel length L = 7000 mm Panel height h = 2550 mm

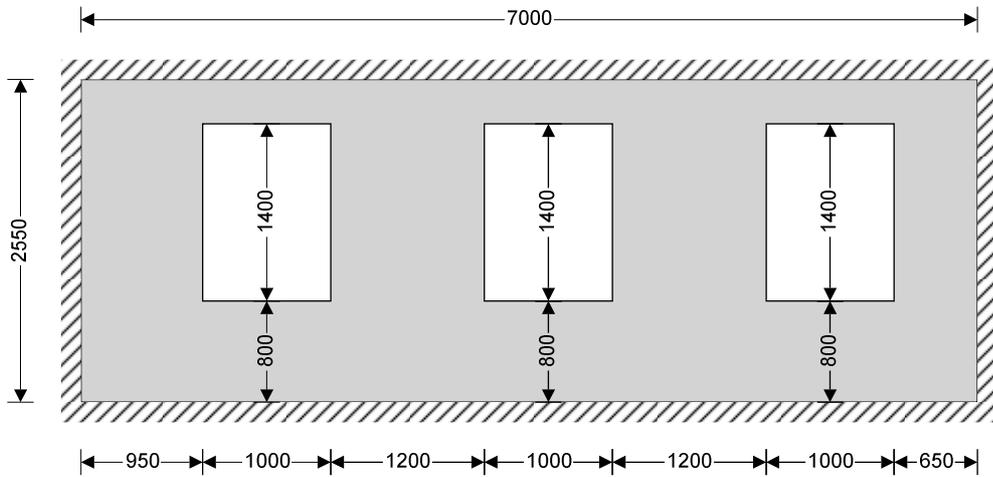
Panel support conditions

Outer leaf All edges supported, top, bottom and right continuous
Inner leaf All edges supported, top, bottom and right continuous

Effective height of masonry walls - Section 5.5.1.2

Reduction factor $p_2 = 1.000$ Effective height of wall $h_{ef} = 2550 \text{ mm}$

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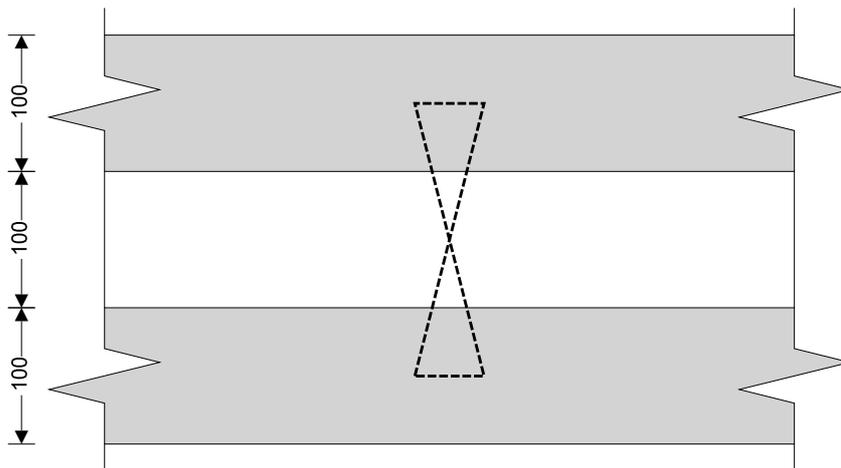


Panel opening details

Spacing length	$L_1 = 950$ mm	Opening width	$w_1 = 1000$ mm
Height to underside of opening	$h_1 = 2200$ mm	Height of opening	$o_1 = 1400$ mm
Spacing length	$L_2 = 1200$ mm	Opening width	$w_2 = 1000$ mm
Height to underside of opening	$h_2 = 2200$ mm	Height of opening	$o_2 = 1400$ mm
Spacing length	$L_3 = 1200$ mm	Opening width	$w_3 = 1000$ mm
Height to underside of opening	$h_3 = 2200$ mm	Height of opening	$o_3 = 1400$ mm

Cavity wall construction details

Outer leaf thickness	$t_1 = 100$ mm	Cavity thickness	$t_c = 100$ mm
Inner leaf thickness	$t_2 = 100$ mm	Effective wall thickness	$t_{ef} = 126$ mm



Masonry outer leaf details

Masonry type	Clay with water absorption between 7% and 12% - Group 1		
Compressive strength	$f_{c1} = 20$ N/mm ²	Norm mean comp.strength	$f_{b1} = 17$ N/mm ²
Density of masonry	$\gamma_1 = 18$ kN/m ³		
Mortar type	M4 - General purpose mortar		
Comp.strength of mortar	$f_{m1} = 4$ N/mm ²	Compressive strength factor	$K = 0.50$
Char.comp.strength masonry	$f_{k1} = 5.507$ N/mm ²		

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Char.flex.str.parallel to bed $f_{xk11} = 0.4 \text{ N/mm}^2$ Char.flex.str.perp.to bed $f_{xk21} = 1.1 \text{ N/mm}^2$

Masonry inner leaf details

Masonry type **Aggregate concrete - Group 1**

Compressive strength $f_{c2} = 3.6 \text{ N/mm}^2$ Norm mean comp.strength $f_{b2} = 4.968 \text{ N/mm}^2$

Density of masonry $\gamma_2 = 18 \text{ kN/m}^3$

Mortar type **M4 - General purpose mortar**

Comp.strength of mortar $f_{m2} = 4 \text{ N/mm}^2$ Compressive strength factor $K = 0.75$

Char.comp.strength masonry $f_{k2} = 3.491 \text{ N/mm}^2$

Char.flex.str.parallel to bed $f_{xk12} = 0.25 \text{ N/mm}^2$ Char.flex.str.perp.to bed $f_{xk22} = 0.505 \text{ N/mm}^2$

Lateral loading details

Characteristic wind load on panel $W_k = 0.750 \text{ kN/m}^2$

Vertical loading details

Permanent load on top of outer leaf $G_{k1} = 6 \text{ kN/m}$

Variable load on top of outer leaf $Q_{k1} = 4.5 \text{ kN/m}$

Partial factors for material strength

Category of manufacture **Category II** Class of execution **Class 2**

Partial factor for compression $\gamma_{Mc} = 3.00$ Partial factor for tension $\gamma_{Mt} = 2.70$

Partial factor for shear $\gamma_{Mv} = 2.50$

Slenderness ratio of masonry walls - Section 5.5.1.4

Allowable slenderness ratio $SR_{all} = 27$ Slenderness ratio $SR = 20.2$

PASS - Slenderness ratio is less than maximum allowable

Reduction factor for slenderness and eccentricity - Section 6.1.2.2

Vertical load at top of wall $N_{id2} = 0 \text{ kN/m}$ Moment at top of wall $M_{id2} = 0 \text{ kNm/m}$

Eccentricity at top of wall $e_{i2} = 5.7 \text{ mm}$ Reduction factor at top of wall $\Phi_{i2} = 0.887$

Vertical load at mid wall $N_{md2} = 3.098 \text{ kN/m}$ Moment at mid wall $M_{md2} = 0 \text{ kNm/m}$

Eccentricity at mid wall $e_{mk2} = 30.6 \text{ mm}$ Reduction factor at mid wall $\Phi_{m2} = 0.117$

Reduction factor $\Phi_2 = 0.117$

Unreinforced masonry walls subjected to lateral loading - Section 6.3

Partial safety factors for design loads

Partial factor for dead load $\gamma_{FG} = 1$ Partial factor for imposed load $\gamma_{FQ} = 0$

Partial factor for wind load $\gamma_{FW} = 1.5$

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

Limiting h / t ratio 46.11 Height to thickness ratio $20.239 = 20.239$

PASS - Limiting height to thickness ratio is not exceeded

Design moments of resistance in panels

Self weight of wall $S_{wt1} = 0 \text{ kN/m}$ Design vertical comp.stress $\sigma_{d1} = 0.06 \text{ N/mm}^2$

Des.flex.str.mas.parallel to bed $f_{xd11} = 0.148 \text{ N/mm}^2$ App.flex.str.mas.parallel to bed $f_{xd11,app} = 0.208 \text{ N/mm}^2$

Des.flex.str.mas.perp to bed $f_{xd21} = 0.407 \text{ N/mm}^2$ Elastic section modulus $Z_1 = 1666667 \text{ mm}^3/\text{m}$

Mom.of resist parallel $M_{Rd11} = 0.347 \text{ kNm/m}$ Mom.of resist.perpendicular $M_{Rd21} = 0.679 \text{ kNm/m}$

Self weight of wall $S_{wt2} = 0 \text{ kN/m}$ Design vertical comp.stress $\sigma_{d2} = 0 \text{ N/mm}^2$

Des.flex.str.mas.parallel to bed $f_{xd12} = 0.093 \text{ N/mm}^2$ App.flex.str.mas.parallel to bed $f_{xd12,app} = 0.093 \text{ N/mm}^2$

Des.flex.str.mas.perp to bed $f_{xd22} = 0.187 \text{ N/mm}^2$ Elastic section modulus $Z_2 = 1666667 \text{ mm}^3/\text{m}$

Mom.of resist parallel $M_{Rd12} = 0.154 \text{ kNm/m}$ Mom.of resist.perpendicular $M_{Rd22} = 0.312 \text{ kNm/m}$

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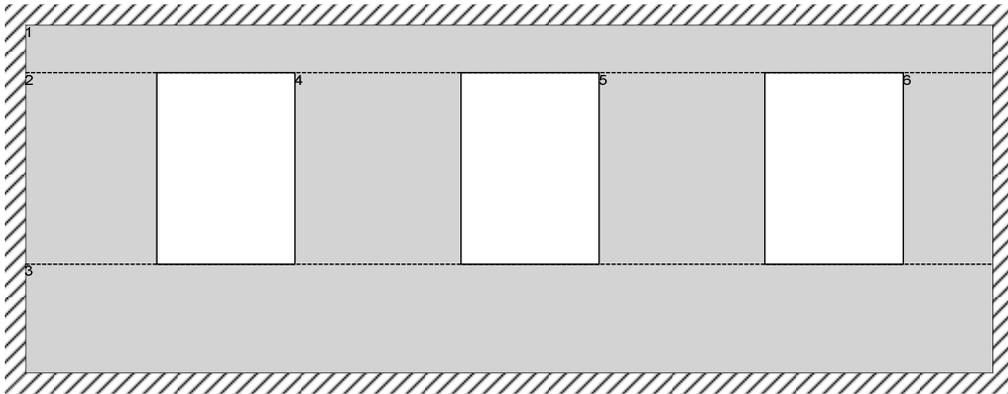
Design moment in panels

Calculate design wind load acting on each leaf

Outer leaf design wind load - parallel	$W_{k11} = M_{Rd11} \times W_k / (M_{Rd11} + M_{Rd12}) = 0.519 \text{ kN/m}^2$
Inner leaf design wind load - parallel	$W_{k12} = M_{Rd12} \times W_k / (M_{Rd11} + M_{Rd12}) = 0.231 \text{ kN/m}^2$
Outer leaf design wind load - perpendicular	$W_{k21} = M_{Rd21} \times W_k / (M_{Rd21} + M_{Rd22}) = 0.514 \text{ kN/m}^2$
Inner leaf design wind load - perpendicular	$W_{k22} = M_{Rd22} \times W_k / (M_{Rd21} + M_{Rd22}) = 0.236 \text{ kN/m}^2$

Orthogonal strength ratios

For outer leaf	$\mu_1 = f_{xd11,app} / f_{xd21} = 0.51$
For inner leaf	$\mu_2 = f_{xd12,app} / f_{xd22} = 0.49$



Considering outer leaf

Sub panel no. 1 - Right, left and top supported

Design moment of resistance $M_{Rd11} = 0.347 \text{ kNm/m}$	Design moment in sub-panel $M_{Ed11B} = 0.238 \text{ kNm/m}$
--	--

Considering inner leaf

Sub panel no. 1 - Right, left and top supported

Design moment of resistance $M_{Rd12} = 0.154 \text{ kNm/m}$	Design moment in sub-panel $M_{Ed21B} = 0.106 \text{ kNm/m}$ PASS - Resistance moment exceeds design moment
--	---

Considering outer leaf

Sub panel no. 2 - Top, bottom and left supported

Design moment of resistance $M_{Rd11} = 0.347 \text{ kNm/m}$	Design moment in sub-panel $M_{Ed12B} = 0.128 \text{ kNm/m}$ PASS - Resistance moment exceeds design moment
--	---

Considering inner leaf

Sub panel no. 2 - Top, bottom and left supported

Design moment of resistance $M_{Rd12} = 0.154 \text{ kNm/m}$	Design moment in sub-panel $M_{Ed22B} = 0.056 \text{ kNm/m}$ PASS - Resistance moment exceeds design moment
--	---

Considering outer leaf

Sub panel no. 3 - Right, left and bottom supported

Design moment of resistance $M_{Rd11} = 0.347 \text{ kNm/m}$	Design moment in sub-panel $M_{Ed13B} = 0.685 \text{ kNm/m}$ FAIL - Resistance moment is exceeded by design moment
--	--

Considering inner leaf

Sub panel no. 3 - Right, left and bottom supported

Design moment of resistance $M_{Rd12} = 0.154 \text{ kNm/m}$	Design moment in sub-panel $M_{Ed23B} = 0.305 \text{ kNm/m}$
--	--

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FAIL - Resistance moment is exceeded by design moment

Considering outer leaf

Sub panel no. 4 - Top and bottom supported

Design moment of resistance $M_{Rd11} = 0.347$ kNm/m

Design moment in sub-panel $M_{Ed14B} = 0.509$ kNm/m

FAIL - Resistance moment is exceeded by design moment

Considering inner leaf

Sub panel no. 4 - Top and bottom supported

Design moment of resistance $M_{Rd12} = 0.154$ kNm/m

Design moment in sub-panel $M_{Ed24B} = 0.226$ kNm/m

FAIL - Resistance moment is exceeded by design moment

Considering outer leaf

Sub panel no. 5 - Top and bottom supported

Design moment of resistance $M_{Rd11} = 0.347$ kNm/m

Design moment in sub-panel $M_{Ed15B} = 0.509$ kNm/m

FAIL - Resistance moment is exceeded by design moment

Considering inner leaf

Sub panel no. 5 - Top and bottom supported

Design moment of resistance $M_{Rd12} = 0.154$ kNm/m

Design moment in sub-panel $M_{Ed25B} = 0.226$ kNm/m

FAIL - Resistance moment is exceeded by design moment

Considering outer leaf

Sub panel no. 6 - Top, bottom and right supported

Design moment of resistance $M_{Rd21} = 0.679$ kNm/m

Design moment in sub-panel $M_{Ed16B} = 0.413$ kNm/m

PASS - Resistance moment exceeds design moment

Considering inner leaf

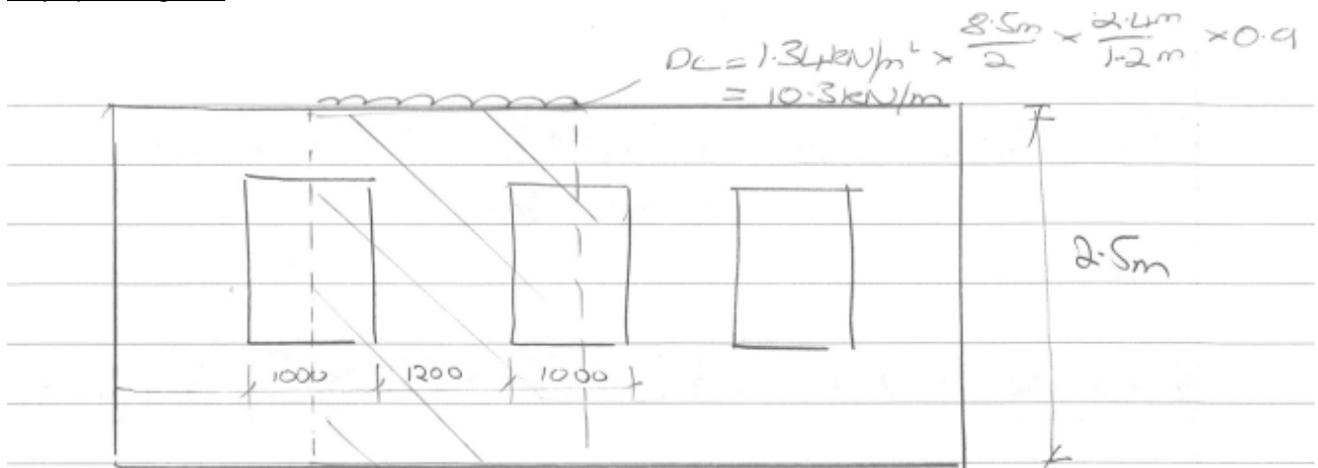
Sub panel no. 6 - Top, bottom and right supported

Design moment of resistance $M_{Rd22} = 0.312$ kNm/m

Design moment in sub-panel $M_{Ed26B} = 0.190$ kNm/m

PASS - Resistance moment exceeds design moment

The proposed configuration has inadequate 2 way flexural strength to support applied wind loads, try as one way spanning wall



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$$q_p = 0.68 \text{ kN/m}^2$$

$$\text{Wind Load} = 0.68(0.8 + 0.3) = 0.75 \text{ kN/m}^2$$

ASSUME VERTICALLY LOADED PANEL

$$\text{WINDLOAD} = 0.75 \times \frac{2.4 \text{ m} \times 1.2}{1.2 \text{ m}} = 1.8 \text{ kN/m}$$

$$\begin{aligned} \text{APPLIED MOMENT} &= 1.5 \times 1.8 \text{ kN/m}^2 \times \frac{2.5^2}{8} \\ &= 2.1 \text{ kNm} \end{aligned}$$

MOMENT CAPACITY

$$f_{cd}^{\text{brick}} = \frac{0.4 \text{ N/mm}^2}{2.7} = 0.15 \text{ N/mm}^2$$

$$\begin{aligned} f_{cd}^{\text{block}} &= \frac{0.25 \text{ N/mm}^2}{2.7} + \frac{10.3 \times 1.2 \text{ m} \times 1000}{1200 \times 100} \\ &= 0.093 + 0.103 \text{ N/mm}^2 = 0.2 \text{ N/mm}^2 \end{aligned}$$

$$\begin{aligned} M_{Rd} &= (0.15 + 0.2 \text{ N/mm}^2) \times \frac{1200 \times 100^2}{6} \times 10^{-6} \\ &= 0.7 \text{ kNm} \quad \nabla \text{ APPLIED} \end{aligned}$$

WINDPOST REQ

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Windpost Design

$$\text{Total Windload} = 1.8 \text{ kN/m} \times 1.5 \times 2.5 \text{ m} = 10.1 \text{ kN}$$

Performance of WP1 and WP3 Windposts to Eurocode 3

	Size a x b x t	Design Resistance (kN) per Post (uniformly distributed) for Various Windpost Spans							
		2.5m	3.0m	3.5m	4.0m	4.5m	5.0m	5.5m	6.0m
WP1	60x60x4	4.28	2.98	2.19	-	-	-	-	-
	80x60x4	8.57	5.98	4.40	3.37	-	-	-	-
	55x60x4	3.19	2.22	-	-	-	-	-	-
	55x60x5	3.93	2.73	-	-	-	-	-	-
	65x60x4	4.68	3.26	2.39	1.83	-	-	-	-
	65x60x5	5.80	4.03	2.96	2.27	-	-	-	-
	75x60x4	6.49	4.53	3.33	2.55	-	-	-	-
	75x60x5	8.07	5.63	4.14	3.17	2.50	-	-	-
WP3	85x60x4	8.63	6.03	4.44	3.40	2.69	2.18	-	-
	85x60x5	10.75	7.51	5.54	4.24	3.35	2.72	2.24	-
	95x60x5	13.83	9.72	7.17	5.50	4.35	3.52	2.91	2.45
	105x60x5	16.24	12.22	9.04	6.94	5.49	4.45	3.68	3.09
	115x60x5	16.24	15.03	11.16	8.59	6.80	5.51	4.56	3.83
	115x60x6	16.24	17.20	12.77	9.82	7.78	6.31	5.22	4.38
	115x65x8	16.24	19.20	16.97	13.06	10.34	8.39	6.93	5.83

Note: Table based on tie spacing of 225mm. Figures in bold indicate capacity limited by tie capacity.

Provide Ancon Windpost WP3 – 85x60x5

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First Floor Structure

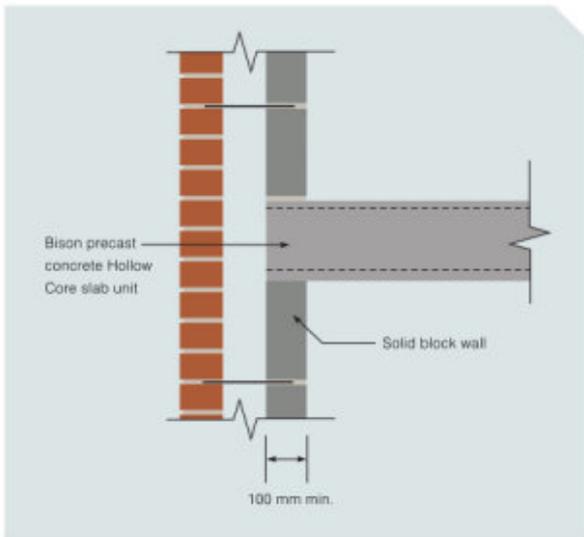
Hollowcore Planks

Disproportionate Collapse

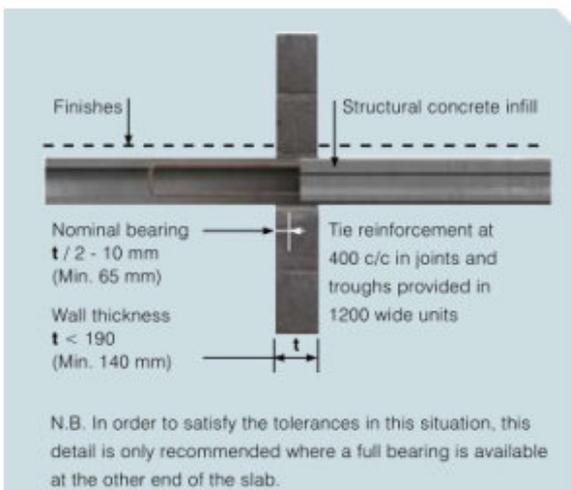
Consequence class – Class 2A effective anchorage to suspended floors

Effective anchorage provided by following details;

EXTERNAL LOAD BEARING CAVITY WALL DETAIL



For Internal walls, its assumed minimal wall thickness is desired therefore 140mm – narrow wall detail required where planks share an internal wall bearing



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Worst Case Span

Over Watch office door – Clear Span = 4550
Design Span = 4750

SDL = 2.15kN/m²
Imposed Load = 3.0 kN/m² (training room/classroom)
Partitions – Assume lightweight partitions = 1.2 kN/m²

Hollowcore Load/Span Table (Non-Composite)

Overall Structural Depth(mm)	Unit Depth (mm)	Self Weight	Self Weight + 1.5kN/m ² for Architectural Finishes (or other Dead load) + Imposed (Live) Load shown below								
			0.75	1.5	2.0	2.5	3.0	4.0	5.0	10.0	15.0
150	150	2.48	9.05	8.40	8.05	7.70	7.45	6.95	6.55	5.25	4.50
150	150	2.95	8.50	7.95	7.65	7.35	7.10	6.65	6.30	5.10	4.40
200	200	2.97	10.70	10.05	9.55	9.30	8.95	8.40	7.95	6.45	5.55
250	250	3.46	12.00	11.30	10.85	10.50	10.15	9.60	9.10	7.40	3

150 thick Hollowcore achieves the spans required

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Ground Level GA Structure

Lintel Design

Front and Rear Elevation Windows – Clear Span = 1000mm

CLEAR SPAN = 1000

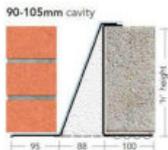
DESIGN SPAN = 1100

LOADING – ROOF DL = 6.0 kN/m
 CL = 4.5 kN/m

FIRST FLOOR DL = 5.15 kN/m² × 4.7/2 = 12.1 kN/m
 CL = 4.2 kN/m² × 4.7/2 = 9.9 kN/m

WALL SW DL = 3.85 kN/m² × 6.5m = 25.0 kN/m

TOTAL = 57.5 kN/m × 1.1m = 63.2 kN



XCFS/K-90

For Cavity widths 90-105mm

The blockwork is built tight against inner face of the steel lintel. Place mortar bed on top of blockwork before floor units are laid to provide even distribution of load. Steel Lintels may be propped to facilitate speed of construction.

BIM	Lintel Selector
Installation Guide	CAD
Price Lists	
Lintel Brochure	

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Length (mm)	Height (mm)	Thickness Inner	Thickness Outer	Total UDL kN 19:1
600 1800	235	5.0	2.9	100
1950 2400	235	5.0	2.9	90

Provide Keystone Lintel – XCFS/K-90

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BEARING CHECK

$$\text{INNER SKIN LOAD} = \frac{57.5 \text{ kN}}{2} - (2.2 \times 6.5 \text{ m}) \times \frac{1.1}{2} = 24 \text{ kN}$$

$$\text{BEARING STRESS} = \frac{24 \times 1000}{150 \times 100} = 1.6 \text{ N/mm}^2$$

$$\begin{aligned} \text{7-SIN BLOCK} \rightarrow f_d &= \frac{0.75 (7.3 \times 1.38)^{0.7} \times 4^{0.3}}{2.7} \\ &= 2.1 \text{ N/mm}^2 > \text{APPLIED} \end{aligned}$$

Lintel to have min 150 bearing

Internal Wall – Opening through to Locker Room – Clear Span = 1000mm

LOADING

$$\begin{aligned} \text{HOLLOW CORE } D_L &= 5.15 \text{ kN/m}^2 \times 9 \text{ m} / 2 = 23.2 \text{ kN/m} \\ K &= 4.2 \text{ kN/m}^2 \times 9 \text{ m} / 2 = 18.9 \text{ kN/m} \end{aligned}$$

$$\text{WALL SW} = 0.5 \text{ m} \times 2.4 \text{ kN/m} = 1.05 \text{ kN/m}$$

$$\text{TOTAL LOAD} = 43.2 \text{ kN/m} \times 1 \text{ m} \times 1.1 = 47.5 \text{ kN}$$

$$\begin{aligned} \text{MOMENT} &= 5.6 \text{ kNm} \\ &9.2 \text{ kNm (USE)} \end{aligned}$$

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HI-STRENGTH LINTELS

Designed to Eurocode 2 BS EN 1992 : 2004. Load Span Table (Units KN/m)

Section Size		110 x 100	140 x 100	150 x 140	215 x 100	215 x 140
Product Code		HSS10	HSR15	HSS15	HSR22	HSR21
Fire Resistance		R30	R30	R60	R30	R60
Lintel Length	Clear Span					
900	700	41.38	71.20	91.75	102.40	134.40
1200	900	26.48	46.45	65.68	82.00	107.50
1200	1000	21.88	38.66	54.66	74.50	97.70
1500	1200	15.67	27.98	39.56	58.40	79.15
1800	1500	10.34	18.68	26.41	44.03	52.25
2100	1800	7.34	13.35	18.88	31.22	37.05

Provide 140w x 150dp High Strength PC Lintel

BEARING

LOAD = 30kN

$$\sigma = \frac{30000}{140 \times 150} = 1.4 \text{ N/mm}^2$$

$$7.3 \text{ N-140 Block} = \frac{0.75(7.3 \times 1.3)^{0.7} \times 4^{0.3}}{2.7} = 2.03 \text{ N/mm}^2$$

Lintel to have min 150 bearing

BEARING ONTO EXISTING BRICK

$$\sigma_b = \frac{30000 \text{ N}}{100 \text{ mm} \times 140 \text{ mm}} = 2.14 \text{ N/mm}^2$$

PROVIDE PADSTONE

$$L_{req} = \frac{30000}{1.46 \text{ N/mm}^2 \times 100} = 200 \text{ mm} \rightarrow \boxed{330 \times 100 \times 215}$$

Lintel to bear onto existing gable wall. Onto 330lg x 100w x 215dp Concrete Padstone

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Internal Wall – Opening at gable end of Locker room

$$\text{HOLLOWCORE } R = 5.15 \text{ kN/m}^2 \times 5.3 \text{ m} / 2 = 13.6 \text{ kN/m}$$

$$L = 6.2 \text{ kN/m}^2 \times 5.3 \text{ m} / 2 = 16.4 \text{ kN/m}$$

$$V_{\text{WALL}} = 1.05 \text{ kN/m}$$

$$\text{TOTAL LOAD} = 26.0 \text{ kN/m} \times 1 \text{ m} \times 1.1 = 28.6 \text{ kN}$$

$$\text{Moment} = 3.9 \text{ kNm}$$

Provide 140w x 150dp High Strength PC Lintel

Internal Wall – Opening into EMAS Accommodation

Same Loading as above lintel, but 100w wall

HI-STRENGTH LINTELS

Designed to Eurocode 2 BS EN 1992 : 2004. Load Span Table (Units KN/m)

Section Size		110 x 100	140 x 100	150 x 140	215 x 100	215 x 140
Product Code		HSS10	HSR15	HSS15	HSR22	HSR21
Fire Resistance		R30	R30	R60	R30	R60
Lintel Length	Clear Span					
900	700	41.38	71.20	91.75	102.40	134.40
1200	900	26.48	46.45	65.68	82.00	107.50
1200	1000	21.88	38.66	54.66	74.50	97.70
1500	1200	15.67	27.98	39.56	58.40	79.15

Provide 100w x 140dp High Strength PC Lintel

$$\text{BEARING STRESS} = \frac{50 \times 0 \times 1000}{100 \times 150} = 1.33 \text{ N/mm}^2$$

$$7.3 \text{ N BLOCK} = 2.12 \text{ N/mm}^2$$

CHECK 100 BEARING

$$0 = \frac{20000}{100 \times 100} = 2.0 \text{ N/mm}^2 < 2.12 \text{ } \checkmark \text{ PASS}$$

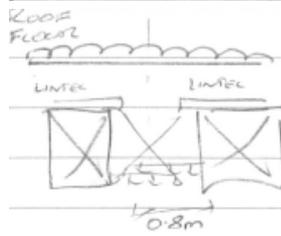
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Masonry Design – Vertically Loaded

Between Windows of Front External Wall

STIFFNESS FROM WINDPOST - $85 \times 60 \times 5 = 1137000 \text{ mm}^4$
 $- 1137000 \text{ mm}^4 \times 205000 = \frac{2.33 \times 10^{11} \text{ Nmm}}{1000 \text{ Pt}} = 6.6734367 \text{ mm}^4$
 Equiv. Pier = $\sqrt[3]{\frac{6.6734367 \times 2}{2.5}} = 200 \text{ mm} \times 100 \text{ mm}$

$T_{EFF} = \sqrt[3]{100^3 + 100^3} = 126 \text{ mm} \times \text{pt}$
 $N_{EFF} = 2550$ $P_t \rightarrow \text{from wind post} = 1.4$
 $\therefore T_{EFF} = 176.4 \text{ mm}$



LOADING

LINTEL LOAD = $\frac{31.4 \text{ kN}}{0.8 \text{ m}} \times 2$
 $= 78.5 \text{ kN/m}$

ROOF LOAD = 1 kN/m

FLOOR LOAD = $13.25 \text{ kN/m}^2 \times \frac{3.6 \text{ m}}{2} = 23.9 \text{ kN/m}$

WALL LOAD = $7.8 \text{ m} \times 1.6 \text{ kN/m} \times 1.35 = 17.4 \text{ kN/m}$

TOTAL = 134.8 kN/m

$e_i = \frac{78.5 \times \frac{100}{6}}{134.8} = 9.7 + \frac{2550}{450} = 15.4 \text{ mm}$

$\phi = 0.69$

$e_m = (9.7 \times 0.5) + \frac{2550}{450} = 10.5 \text{ mm}$

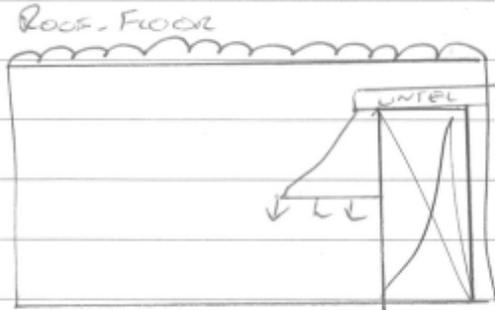
$\phi_m = 0.6$

$N_{rd} = 0.643 \times 100 \text{ mm} \times 2.1 \text{ N/mm}^2$
 $= 135 \text{ kN/m} \checkmark \rightarrow \text{DEEMED ACCEPTABLE}$

100 mm wide inner skin blockwork – 7.3N/mm² Strength deemed adequate

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Door opening Rear elevation



$$\text{Lintel Load} = \frac{29.0 \text{ kN}}{0.8 \text{ m}} = 36.3 \text{ kN/m}$$

$$\text{ROOF} = 15 \text{ kN/m}$$

$$\text{Floor} = 13.25 \times \frac{4.2}{2} = 27.8 \text{ kN/m}$$

$$W_{acc} = 1.65 \text{ kN/m}^2 \times 1.35 \times 5 \text{ m} = 11.1 \text{ kN/m}$$

$$\text{TOTAL} = 90.2 \text{ kN/m}$$

$$\lambda = \frac{2550}{\sqrt[3]{100+100}} = \frac{2550}{127.6} = 20$$

$$e_i = \frac{36.3 \times \frac{100}{2}}{90.2} + \frac{2550}{480} = 12.4 \text{ mm} \quad \phi_i = 0.75$$

$$e_m = 9.7 \text{ mm} \quad \phi_m = 0.53$$

$$N_{rd} = 0.53 \times 100 \times 2.1 \text{ N/mm}^2 = 111.3 \text{ kN/m} > \text{APPLIED} \checkmark$$

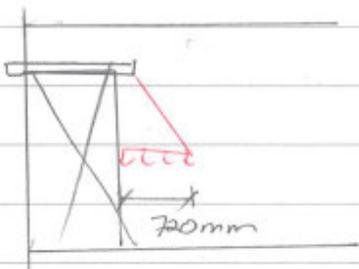
100 mm wide inner skin blockwork – 7.3N/mm² Strength deemed adequate

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Internal Wall Supporting Lintel (Opening into Locker room)

140 Block

$$\lambda = \frac{2550}{140} = 18.2$$



$$L_{\text{intel}} = \frac{30}{0.7m} = 42.9 \text{ kN/m}$$

$$\text{Hollowcore wall} = 60 \text{ kN/m} = 1.6 \text{ kN/m}$$

$$e_i = \frac{t}{6} + \frac{2550}{450} = 29 \text{ mm} \quad \underline{\underline{\Sigma = 104.3 \text{ kN/m}}}$$

$$\phi_i = 1 - \left(\frac{29}{140} \times 2 \right) = 0.59$$

$$e_m = 11.7 \text{ mm} \quad \phi_m = 0.47$$

$$N_{rd} = 0.47 \times 140 \text{ mm} \times 2.03 = 133.5 \text{ kN/m} > \text{Applied}$$

140 mm wide blockwork – 7.3N/mm² Strength deemed adequate

Internal Wall Supporting Lintel (Opening into EMAS Accommodation)

100 Block - Restraint from adjoining wall

$$K_{eff} = \frac{1}{1 + \left(\frac{2550}{3 \times 190} \right)^2} \times 2550 = 0.8 \times 2550 = 2063 \text{ mm}$$

$$\lambda = \frac{2063}{100} = 20.6$$

$$L_{\text{intel load}} = \frac{20.0 \text{ kN}}{0.7m} = 29 \text{ kN/m}$$

$$\text{Hollowcore} = 35.0 \text{ kN/m}$$

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$w_{all} = 1.4 \text{ kN/m}$
 $\Sigma = 65.4 \text{ kN/m}$
 $e_i = 22.3 \text{ mm}$ $\phi_i = 0.55$
 $e_m = 14 \text{ mm}$ $\phi_m = 0.42$
 $N_{rd} = 0.42 \times 100 \text{ mm} \times 2.12 \text{ N/mm}^2 = 89.04 \text{ kN/m} \checkmark$

100 mm wide blockwork – 7.3N/mm² Strength deemed adequate. Relying on partition wall between Watch office and EMAS for stability

Masonry Design – Laterally Loaded

Rear External Wall

MASONRY WALL PANEL DESIGN

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

Tedds calculation version 1.2.11

Masonry panel details

Front Wall beneath monopitch roof - Unreinforced masonry wall without openings

Panel length L = **5750** mm Panel height h = **2550** mm

Panel support conditions

Outer leaf **Top, bottom and left supported, top continuous**

Inner leaf **Top, bottom and left supported, top continuous**

Effective height of masonry walls - Section 5.5.1.2

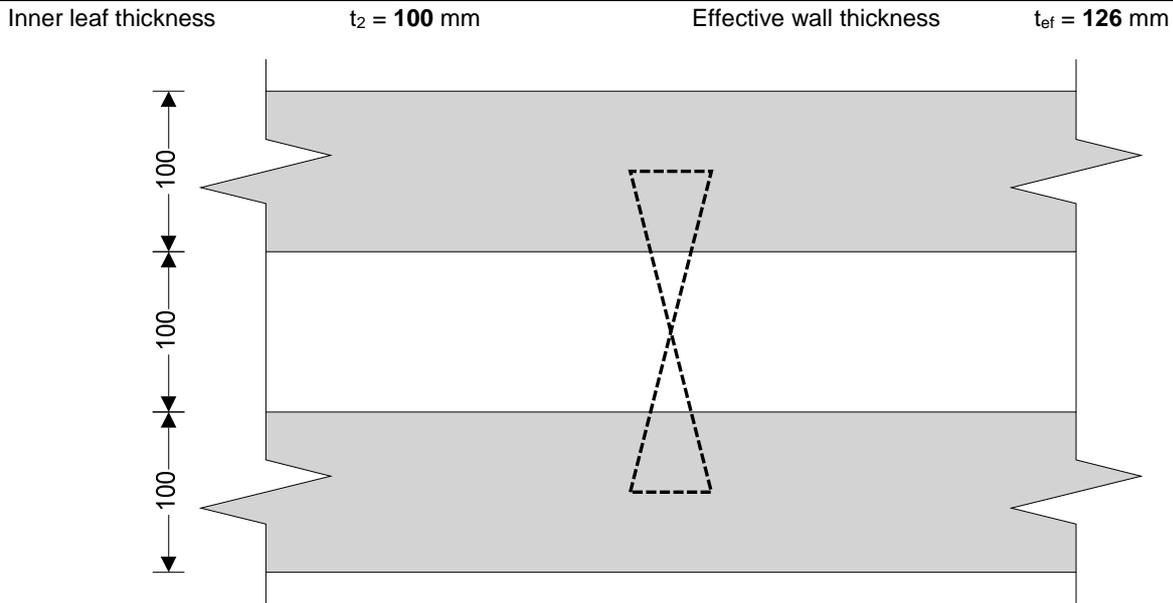
Reduction factor $\rho_2 = \mathbf{1.000}$ Effective height of wall $h_{ef} = \mathbf{2550}$ mm



Cavity wall construction details

Outer leaf thickness $t_1 = \mathbf{100}$ mm Cavity thickness $t_c = \mathbf{100}$ mm

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Masonry outer leaf details

Masonry type	Clay with water absorption less than 7% - Group 1		
Compressive strength	$f_{c1} = 20 \text{ N/mm}^2$	Norm mean comp.strength	$f_{b1} = 17 \text{ N/mm}^2$
Density of masonry	$\gamma_1 = 21 \text{ kN/m}^3$		
Mortar type	M4 - General purpose mortar		
Comp.strength of mortar	$f_{m1} = 4 \text{ N/mm}^2$	Compressive strength factor	$K = 0.50$
Char.comp.strength masonry	$f_{k1} = 5.507 \text{ N/mm}^2$		
Char.flex.str.parallel to bed	$f_{xk11} = 0.5 \text{ N/mm}^2$	Char.flex.str.perp.to bed	$f_{xk21} = 1.5 \text{ N/mm}^2$

Masonry inner leaf details

Masonry type	Aggregate concrete - Group 1		
Compressive strength	$f_{c2} = 7.3 \text{ N/mm}^2$	Norm mean comp.strength	$f_{b2} = 10.074 \text{ N/mm}^2$
Density of masonry	$\gamma_2 = 18 \text{ kN/m}^3$		
Mortar type	M4 - General purpose mortar		
Comp.strength of mortar	$f_{m2} = 4 \text{ N/mm}^2$	Compressive strength factor	$K = 0.75$
Char.comp.strength masonry	$f_{k2} = 5.727 \text{ N/mm}^2$		
Char.flex.str.parallel to bed	$f_{xk12} = 0.25 \text{ N/mm}^2$	Char.flex.str.perp.to bed	$f_{xk22} = 0.6 \text{ N/mm}^2$

Lateral loading details

Characteristic wind load on panel	$W_k = 0.750 \text{ kN/m}^2$
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Vertical loading details

Permanent load on top of outer leaf	$G_{k1} = 5.6 \text{ kN/m}$
Permanent load on top of inner leaf	$G_{k2} = 20.4 \text{ kN/m}$
Variable load on top of inner leaf	$Q_{k2} = 12.9 \text{ kN/m}$

Partial factors for material strength

Category of manufacture	Category I	Class of execution	Class 2
Partial factor for compression	$\gamma_{Mc} = 2.70$	Partial factor for tension	$\gamma_{Mt} = 2.70$
Partial factor for shear	$\gamma_{Mv} = 2.50$		

Slenderness ratio of masonry walls - Section 5.5.1.4

Allowable slenderness ratio	$SR_{all} = 27$	Slenderness ratio	$SR = 20.2$
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PASS - Slenderness ratio is less than maximum allowable

Unreinforced masonry walls subjected to mainly vertical loading - Section 6.1

Partial safety factors for design loads

Partial factor for dead load $\gamma_{fG} = 1.35$ Partial factor for imposed load $\gamma_{fQ} = 1.5$
 Partial factor for wind load $\gamma_{fW} = 0.75$

Considering outer leaf

Reduction factor for slenderness and eccentricity - Section 6.1.2.2

Vertical load at top of wall	$N_{id1} = 7.56$ kN/m	Moment at top of wall	$M_{id1} = 0$ kNm/m
Eccentricity at top of wall	$e_{i1} = 16.5$ mm	Reduction factor at top of wall	$\Phi_{i1} = 0.669$
Vertical load at mid wall	$N_{md1} = 11.175$ kN/m	Moment at mid wall	$M_{md1} = 0$ kNm/m
Eccentricity at mid wall	$e_{mk1} = 13$ mm	Reduction factor at mid wall	$\Phi_{m1} = 0.449$
		Reduction factor	$\Phi_1 = 0.449$

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

Design value of vertical load $N_{Ed1} = 11.175$ kN/m Vertical resistance of wall $N_{Rd1} = 91.595$ kN/m

PASS - Design vertical resistance exceeds applied design vertical load

Considering inner leaf

Reduction factor for slenderness and eccentricity - Section 6.1.2.2

Vertical load at top of wall	$N_{id2} = 46.89$ kN/m	Moment at top of wall	$M_{id2} = 0$ kNm/m
Eccentricity at top of wall	$e_{i2} = 6.3$ mm	Reduction factor at top of wall	$\Phi_{i2} = 0.874$
Vertical load at mid wall	$N_{md2} = 49.988$ kN/m	Moment at mid wall	$M_{md2} = 0$ kNm/m
Eccentricity at mid wall	$e_{mk2} = 6.2$ mm	Reduction factor at mid wall	$\Phi_{m2} = 0.595$
		Reduction factor	$\Phi_2 = 0.595$

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

Design value of vertical load $N_{Ed2} = 49.988$ kN/m Vertical resistance of wall $N_{Rd2} = 126.231$ kN/m

PASS - Design vertical resistance exceeds applied design vertical load

Unreinforced masonry walls subjected to lateral loading - Section 6.3

Partial safety factors for design loads

Partial factor for dead load $\gamma_{fG} = 1$ Partial factor for imposed load $\gamma_{fQ} = 0$
 Partial factor for wind load $\gamma_{fW} = 1.5$

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

Limiting h / t ratio 44.362 Height to thickness ratio 20.239 = **20.239**

PASS - Limiting height to thickness ratio is not exceeded

Design moments of resistance in panels

Self weight of wall	$S_{wt1} = 2.678$ kN/m	Design vertical comp.stress	$\sigma_{d1} = 0.083$ N/mm ²
Des.flex.str.mas.parallel to bed	$f_{xd11} = 0.185$ N/mm ²	App.flex.str.mas.parallel to bed	$f_{xd11,app} = 0.268$ N/mm ²
Des.flex.str.mas.perp to bed	$f_{xd21} = 0.556$ N/mm ²	Elastic section modulus	$Z_1 = 1666667$ mm ³ /m
Mom.of resist parallel	$M_{Rd11} = 0.447$ kNm/m	Mom.of resist.perpendicular	$M_{Rd21} = 0.926$ kNm/m
Self weight of wall	$S_{wt2} = 2.295$ kN/m	Design vertical comp.stress	$\sigma_{d2} = 0.189$ N/mm ²
Des.flex.str.mas.parallel to bed	$f_{xd12} = 0.093$ N/mm ²	App.flex.str.mas.parallel to bed	$f_{xd12,app} = 0.282$ N/mm ²
Des.flex.str.mas.perp to bed	$f_{xd22} = 0.222$ N/mm ²	Elastic section modulus	$Z_2 = 1666667$ mm ³ /m
Mom.of resist parallel	$M_{Rd12} = 0.47$ kNm/m	Mom.of resist.perpendicular	$M_{Rd22} = 0.37$ kNm/m

Calculate design wind load acting on each leaf

Outer leaf design wind load $W_{k1} = 0.365$ kN/m² Inner leaf design wind load $W_{k2} = 0.385$ kN/m²

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Design moment in panels

Design moment in wall

$$M_{Ed1} = \mu_1 \times \gamma_{fW} \times \alpha_1 \times W_{k1} \times L^2 = \mathbf{0.226 \text{ kNm/m}}$$

PASS - Resistance moment exceeds design moment

Design moment in wall

$$M_{Ed2} = \mu_2 \times \gamma_{fW} \times \alpha_2 \times W_{k2} \times L^2 = \mathbf{0.267 \text{ kNm/m}}$$

PASS - Resistance moment exceeds design moment

Provide Brick/Block Cavity Wall construction consisting of:

- 102.5w Brickwork – min 20N/mm² strength with 7-12% assumed water absorption
- 100w Blockwork – min 7.3N/mm² strength
- Masonry Reinforcement to both inner skins, using Bekaert Brickforce SBF30W60 @ 440mm vertical centres above door head to minimise expansion
- Wall ties as specified on drawing

Front External Wall

Same configuration as first floor, therefore by inspection Ancon Windpost WP3 – 85x60x5 required between windows. Wall between Watch office and EMAS can be used for stability

Gable Wall

MASONRY WALL PANEL DESIGN

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

Tedds calculation version 1.2.11

Masonry panel details

Front Elevation - Unreinforced masonry wall without openings

Panel length L = **8800 mm** Panel height h = **2550 mm**

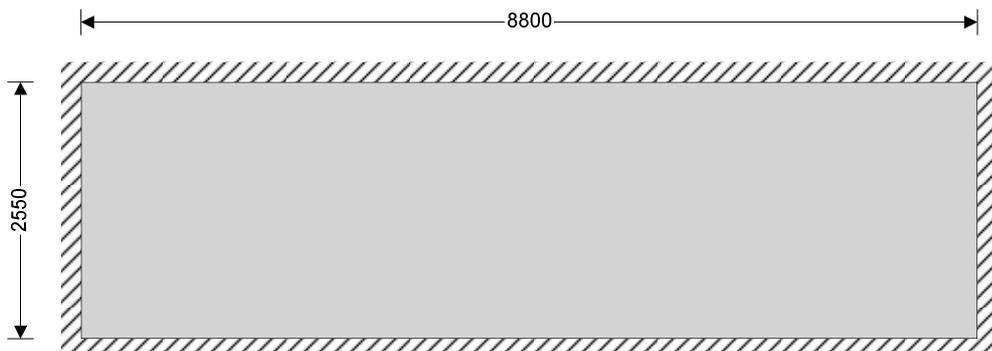
Panel support conditions

Outer leaf **All edges supported, top and right continuous**

Inner leaf **All edges supported, top and right continuous**

Effective height of masonry walls - Section 5.5.1.2

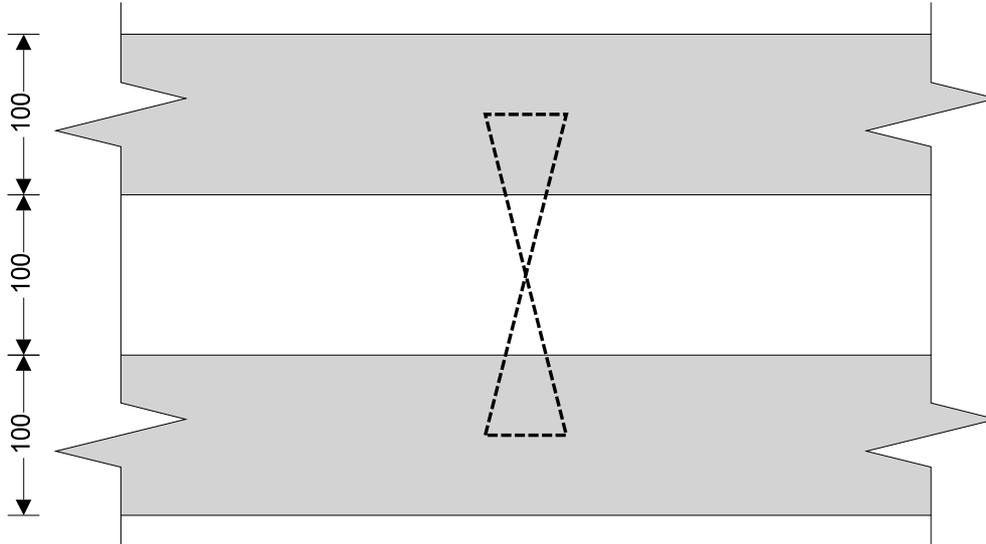
Reduction factor $\rho_2 = \mathbf{1.000}$ Effective height of wall $h_{ef} = \mathbf{2550 \text{ mm}}$



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Cavity wall construction details

Outer leaf thickness $t_1 = 100$ mm Cavity thickness $t_c = 100$ mm
 Inner leaf thickness $t_2 = 100$ mm Effective wall thickness $t_{ef} = 126$ mm



Masonry outer leaf details

Masonry type **Clay with water absorption between 7% and 12% - Group 1**
 Compressive strength $f_{c1} = 20$ N/mm² Norm mean comp.strength $f_{b1} = 17$ N/mm²
 Density of masonry $\gamma_1 = 18$ kN/m³
 Mortar type **M4 - General purpose mortar**
 Comp.strength of mortar $f_{m1} = 4$ N/mm² Compressive strength factor $K = 0.50$
 Char.comp.strength masonry $f_{k1} = 5.507$ N/mm²
 Char.flex.str.parallel to bed $f_{xk11} = 0.4$ N/mm² Char.flex.str.perp.to bed $f_{xk21} = 1.1$ N/mm²

Masonry inner leaf details

Masonry type **Aggregate concrete - Group 1**
 Compressive strength $f_{c2} = 7.3$ N/mm² Norm mean comp.strength $f_{b2} = 10.074$ N/mm²
 Density of masonry $\gamma_2 = 18$ kN/m³
 Mortar type **M4 - General purpose mortar**
 Comp.strength of mortar $f_{m2} = 4$ N/mm² Compressive strength factor $K = 0.75$
 Char.comp.strength masonry $f_{k2} = 5.727$ N/mm²
 Char.flex.str.parallel to bed $f_{xk12} = 0.25$ N/mm² Char.flex.str.perp.to bed $f_{xk22} = 0.6$ N/mm²

Lateral loading details

Characteristic wind load on panel $W_k = 0.750$ kN/m²

Vertical loading details

Permanent load on top of outer leaf $G_{k1} = 9.3$ kN/m
 Permanent load on top of inner leaf $G_{k2} = 7$ kN/m

Partial factors for material strength

Category of manufacture **Category II** Class of execution **Class 2**
 Partial factor for compression $\gamma_{Mc} = 3.00$ Partial factor for tension $\gamma_{Mt} = 2.70$
 Partial factor for shear $\gamma_{Mv} = 2.50$

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Slenderness ratio of masonry walls - Section 5.5.1.4

Allowable slenderness ratio $SR_{all} = 27$ Slenderness ratio $SR = 20.2$

PASS - Slenderness ratio is less than maximum allowable

Unreinforced masonry walls subjected to mainly vertical loading - Section 6.1

Partial safety factors for design loads

Partial factor for dead load $\gamma_{fG} = 1.35$ Partial factor for imposed load $\gamma_{fQ} = 1.5$

Partial factor for wind load $\gamma_{fW} = 0.75$

Considering outer leaf

Reduction factor for slenderness and eccentricity - Section 6.1.2.2

Vertical load at top of wall $N_{id1} = 12.555$ kN/m Moment at top of wall $M_{id1} = 0$ kNm/m

Eccentricity at top of wall $e_{i1} = 20.9$ mm Reduction factor at top of wall $\Phi_{i1} = 0.583$

Vertical load at mid wall $N_{md1} = 15.653$ kN/m Moment at mid wall $M_{md1} = 0$ kNm/m

Eccentricity at mid wall $e_{mk1} = 17.9$ mm Reduction factor at mid wall $\Phi_{m1} = 0.348$

Reduction factor $\Phi_1 = 0.348$

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

Design value of vertical load $N_{Ed1} = 15.653$ kN/m Vertical resistance of wall $N_{Rd1} = 63.938$ kN/m

PASS - Design vertical resistance exceeds applied design vertical load

Considering inner leaf

Reduction factor for slenderness and eccentricity - Section 6.1.2.2

Vertical load at top of wall $N_{id2} = 9.45$ kN/m Moment at top of wall $M_{id2} = 0$ kNm/m

Eccentricity at top of wall $e_{i2} = 33.9$ mm Reduction factor at top of wall $\Phi_{i2} = 0.323$

Vertical load at mid wall $N_{md2} = 12.548$ kN/m Moment at mid wall $M_{md2} = 0$ kNm/m

Eccentricity at mid wall $e_{mk2} = 26.9$ mm Reduction factor at mid wall $\Phi_{m2} = 0.176$

Reduction factor $\Phi_2 = 0.176$

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

Design value of vertical load $N_{Ed2} = 12.548$ kN/m Vertical resistance of wall $N_{Rd2} = 33.579$ kN/m

PASS - Design vertical resistance exceeds applied design vertical load

Unreinforced masonry walls subjected to lateral loading - Section 6.3

Partial safety factors for design loads

Partial factor for dead load $\gamma_{fG} = 1$ Partial factor for imposed load $\gamma_{fQ} = 0$

Partial factor for wind load $\gamma_{fW} = 1.5$

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

Limiting h / t ratio 42.539 Height to thickness ratio 20.239 = **20.239**

PASS - Limiting height to thickness ratio is not exceeded

Design moments of resistance in panels

Self weight of wall $S_{wt1} = 2.295$ kN/m

Design vertical comp.stress $\sigma_{d1} = 0.096$ N/mm²

Des.flex.str.mas.parallel to bed $f_{xd11} = 0.148$ N/mm²

App.flex.str.mas.parallel to bed $f_{xd11,app} = 0.244$ N/mm²

Des.flex.str.mas.perp to bed $f_{xd21} = 0.407$ N/mm²

Elastic section modulus $Z_1 = 1666667$ mm³/m

Mom.of resist parallel $M_{Rd11} = 0.407$ kNm/m

Mom.of resist.perpendicular $M_{Rd21} = 0.679$ kNm/m

Self weight of wall $S_{wt2} = 2.295$ kN/m

Design vertical comp.stress $\sigma_{d2} = 0.05$ N/mm²

Des.flex.str.mas.parallel to bed $f_{xd12} = 0.093$ N/mm²

App.flex.str.mas.parallel to bed $f_{xd12,app} = 0.143$ N/mm²

Des.flex.str.mas.perp to bed $f_{xd22} = 0.222$ N/mm²

Elastic section modulus $Z_2 = 1666667$ mm³/m

Mom.of resist parallel $M_{Rd12} = 0.238$ kNm/m

Mom.of resist.perpendicular $M_{Rd22} = 0.37$ kNm/m

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Calculate design wind load acting on each leaf

Outer leaf design wind load $W_{k1} = 0.473 \text{ kN/m}^2$ Inner leaf design wind load $W_{k2} = 0.277 \text{ kN/m}^2$

Design moment in panels

Design moment in wall $M_{Ed1} = \gamma_{fW} \times \alpha_1 \times W_{k1} \times h^2 = 0.577 \text{ kNm/m}$
FAIL - Resistance moment is exceeded by design moment

Design moment in wall $M_{Ed2} = \gamma_{fW} \times \alpha_2 \times W_{k2} \times h^2 = 0.338 \text{ kNm/m}$
FAIL - Resistance moment is exceeded by design moment

Wall has inadequate lateral stiffness, therefore provide Wind post at mid-length.

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Ground Level Floor Structure

Worst Case Span

Over Beneath Locker Room – Clear Span = 4100
Design Span = 4300

SDL = 1.8 kN/m²
Imposed Load = 3.0 kN/m²
Partitions – Assume lightweight partitions = 1.2 kN/m²



Finishes				kN/m ²					
Chipboard	No		0.2						
50mm Screed	No		1.2						
75mm Screed	Yes		1.8						
Use your own Finish Load	No								
12mm plaster or plasterboard ceiling	No		0.2						
				Total Finishes					
				1.8		kN/m ²			
Partitions				kN/m ²					
Studwork	No		0.5						
100mm lt. wt. block, plastered, 2.4m high	No		1.0						
100mm med. wt. block, plastered, 2.4m high	No		2.0						
Use your own Partition Load	Yes		1.2						
				Total Partitions					
				1.2		kN/m ²			
Live Loads				kN/m ²					
Domestic	No		1.5						
Domestic Garage or Office	No		2.5						
Communal Area	Yes		3.0						
Use your own Live Load	No								
				Total Live Load					
				3.0		kN/m ²			
		Clear Span required		4.300		m			
Unit Type	Acceptable	Max Clear Span	Predicted Camber	Natural Frequency Hz	Self Weight (kN/m ²)	Centres	Structural Depth	Floor Type	
T155 Single -maximum	NO	3.05	N/A	N/A	1.78	510	155	Standard Beam and Block Floors using 1450kg/m ³ infill blocks	
T155 Single -alternate	NO	3.44	N/A	N/A	1.88	398	155		
T155 Single -narrow	NO	4.04	N/A	N/A	2.06	285	155		
T155 Double -maximum	NO	3.86	N/A	N/A	2.11	620	155		
T155 Double -alternate	NO	4.24	N/A	N/A	2.26	508	155		
T155 Double -narrow	Yes	4.75	-13.6	6.74	2.50	395	155		

225thk Beam and block adequate for span

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Foundation Structure

Foundation Statement

Extract from soils report

6. SUMMARY

1. A ground investigation was undertaken at Gilmorton Road, Lutterworth, where it is proposed to extend the existing Lutterworth Fire and Rescue Station.
2. The investigation proved a differing sequence of soils within the two exploratory holes. Made ground was found to overlie a mixed sequence of glacial soils including peat, clay and sand over stiff clay of the Blue Lias Formation.
3. Groundwater seepages were noted within the made ground and inflows were encountered at 2.6 m bgl within WS01, coincident with the base of the peat.
4. It is recommended that the structural loads are transferred through the made ground and peat and founded in stiff clay at least 2.6 m deep. An allowable bearing capacity of about 150 kNm⁻² would be appropriate in the natural cohesive soils at this depth.
5. As an alternative to a shallow foundation and to mitigate the uncertainty of the upper soils, consideration could be given to the use of a piled foundation, although it would be necessary to confirm the ground conditions to a depth greater than the finished depth of the piles.

Due to site conditions; a piled ground beam solution is deemed most appropriate

Concrete Specification

Designed concrete specification for Piled Ground Beam (75 mm nominal cover)

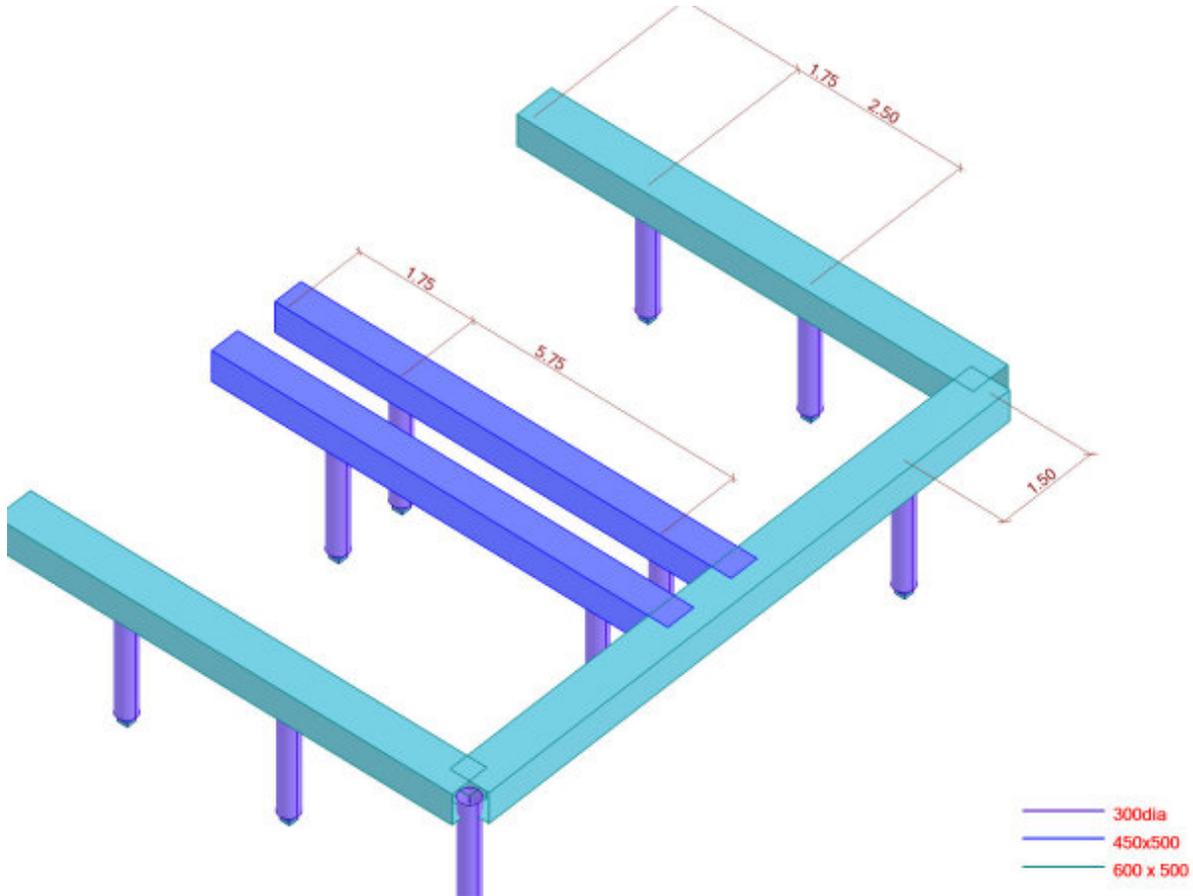
The concrete shall be produced in accordance with BS8500-2.

Compressive strength class	C28/35
Maximum water/cement ratio	0.45
Minimum cement/combination content	360 kg/m³
Allowable cement/combinations types	IIIB+SR (DC-4)
Maximum aggregate size	20 mm
Chloride content class	Cl 0,40
Consistence class	S3

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Proposed Foundation Analysis

The following Piled ground beam layout was carried out in Robot Structural Analysis 2018;



This layout was analysed for the following load cases and load combinations;

Loads - Cases

Case	Case name	Nature
1	Self-Weight	Structural
2	Masonry	Structural
3	Roof DL	Structural
4	First Floor DL	Structural
5	Ground Floor DL	Structural
6	Roof Imposed	Category A
7	First Floor IL	Category A
8	Ground Floor IL	Category A

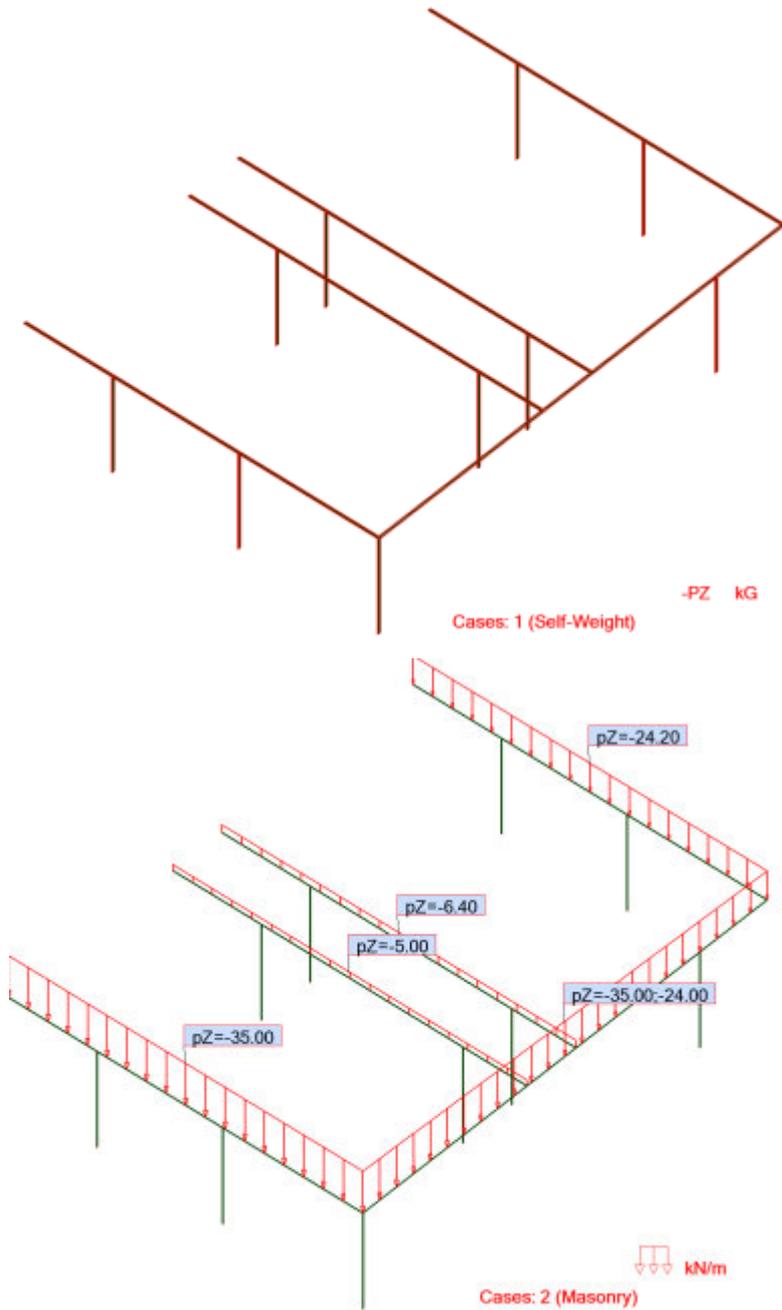
Combinations

- Cases: 10to12

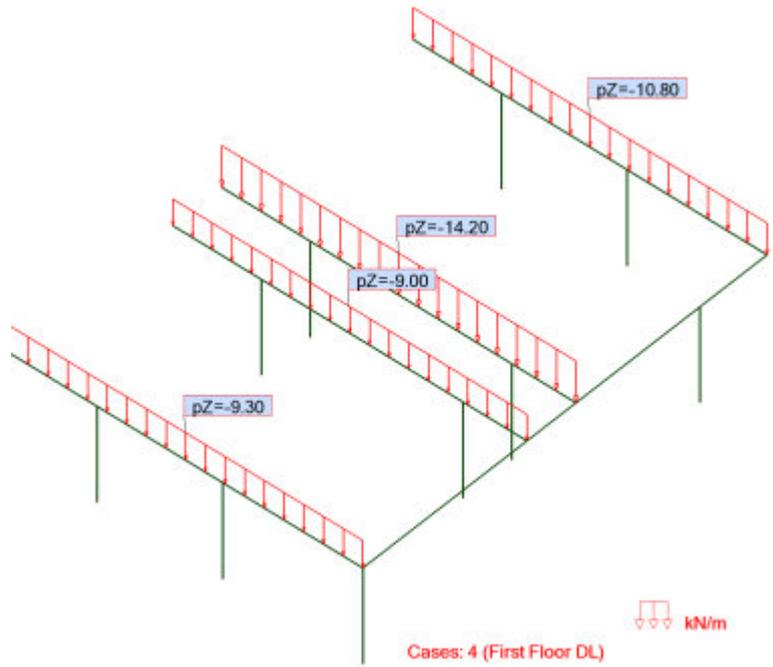
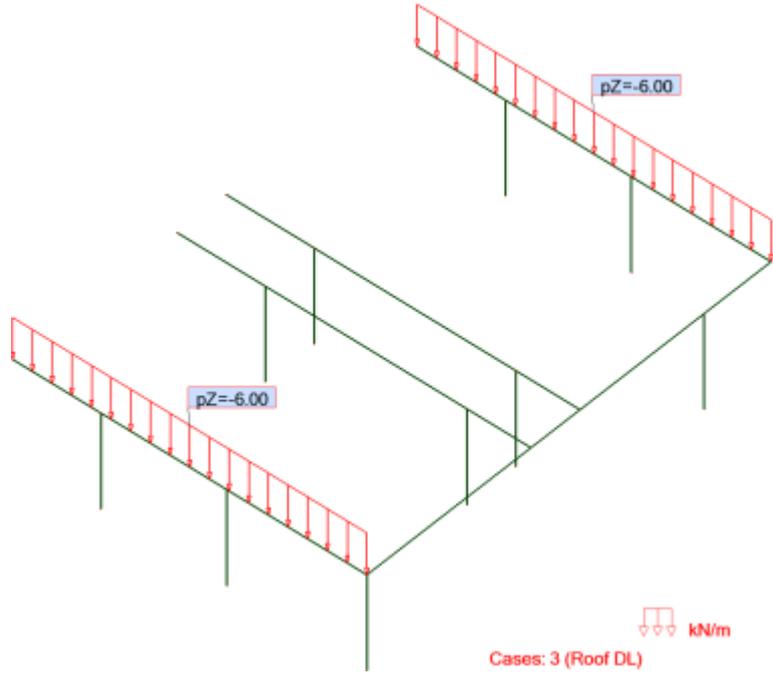
Combinations	Name	Combination type	Definition
10 (C)	ULS DL + IL	ULS	$(1+2+3+4+5)*1.35+(6+7+8)*1.50$
11 (C)	SLS - DL + IL	SLS:QPR	$(1+2+3+4+5+6+7+8)*1.00$
12 (C)	SLS - IL	SLS	$(6+7+8)*1.00$

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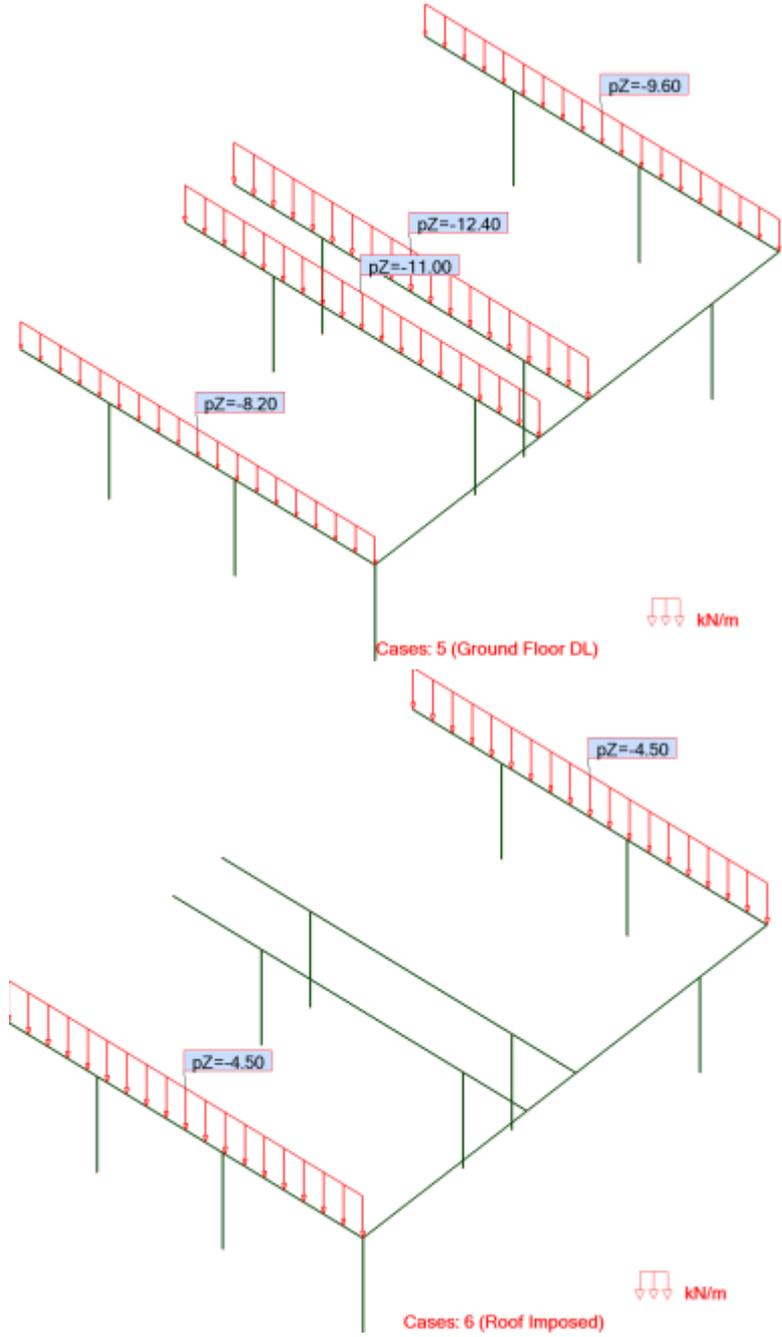
Loadings



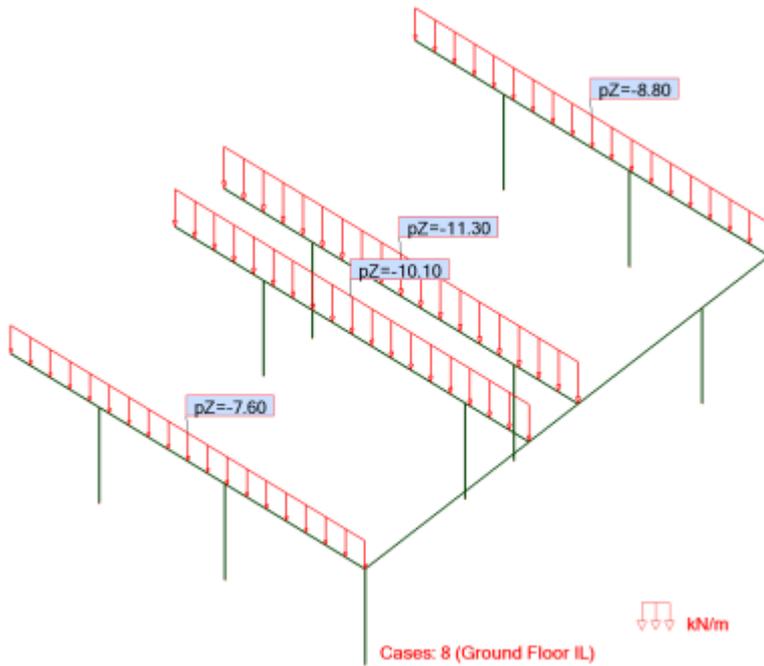
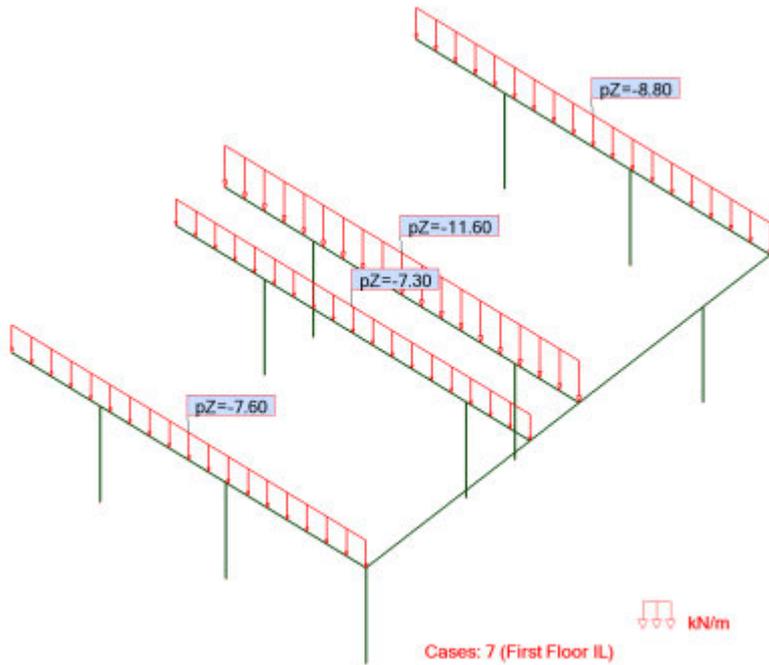
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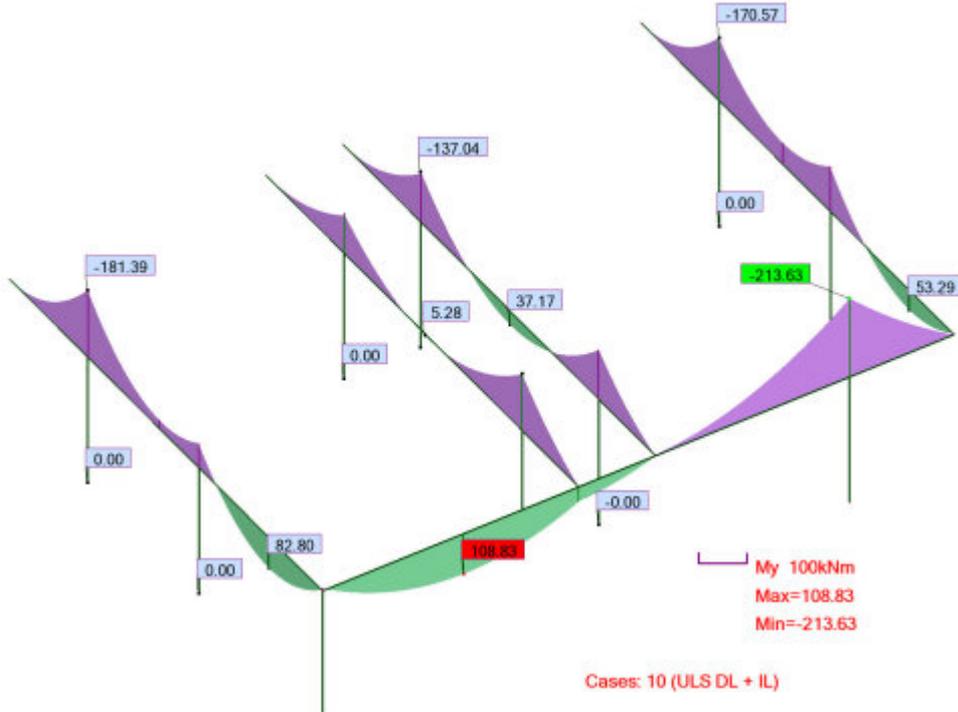
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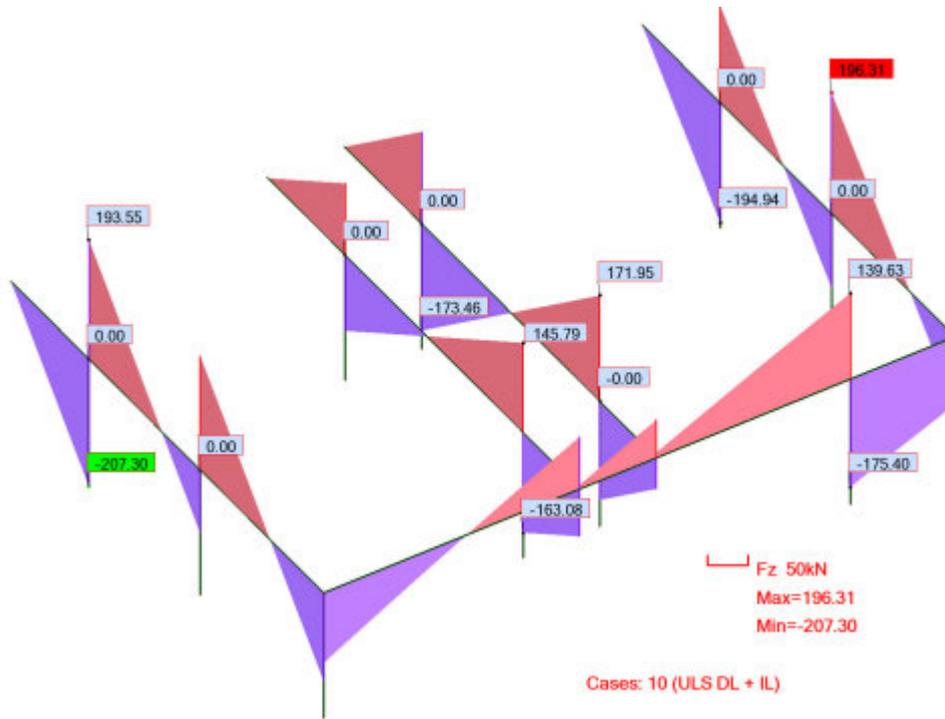
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Load Effects

Bending Moment

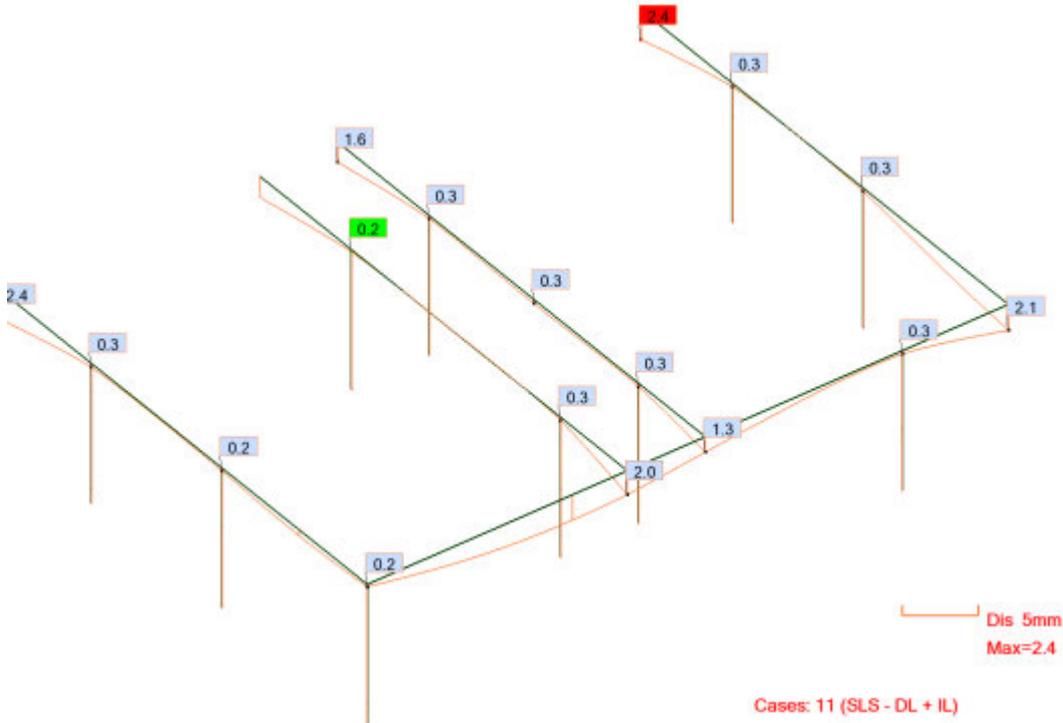


Shear Force

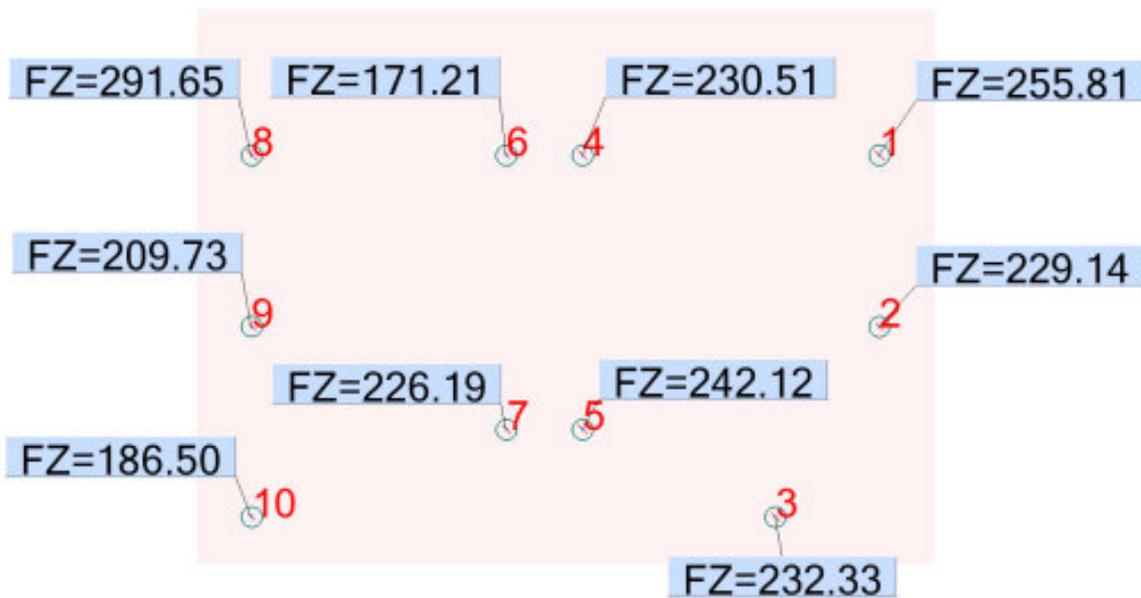


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Deflection



Reactions



Pile Design – By specialist for the above loads, it is anticipated that a 300 dia – 450 dia pile is required

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Ground Beam Design

Ground Beam beneath Gable Wall

M (hog) = 215kNm

M (sag) = 110kNm

Shear = 225kN

RC MEMBER DESIGN

In accordance with EN1992-1-1:2004 incorporating Corrigenda January 2008 and the UK national annex

Tedds calculation version 3.0.06

Concrete details (Table 3.1 - Strength and deformation characteristics for concrete)

Concrete strength class	C28/35	Char. comp. cylinder strength	$f_{ck} = 28 \text{ N/mm}^2$
Design comp conc. strength	$f_{cwd} = 18.7 \text{ N/mm}^2$	Maximum aggregate size	$h_{agg} = 20 \text{ mm}$

Reinforcement details

Char. yield strength of reinf.	$f_{yk} = 500 \text{ N/mm}^2$	Partial factor for reinf. steel	$\gamma_s = 1.15$
Design yield strength of reinf.	$f_{yd} = 435 \text{ N/mm}^2$		

Nominal cover to reinforcement

Nominal cover to top reinf	$c_{nom_t} = 75 \text{ mm}$	Nominal cover to bottom reinf	$c_{nom_b} = 75 \text{ mm}$
Nominal cover to side reinf	$c_{nom_s} = 75 \text{ mm}$		

Fire resistance

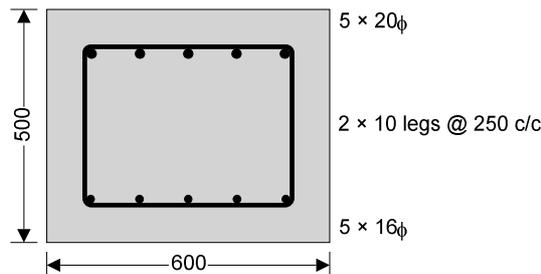
Standard fire resistance period	R = 60 min	No. sides exposed to fire	3
Minimum width of beam	$b_{min} = 120 \text{ mm}$		

Section 1 - Ground Beam

Rectangular section details

Section width	b = 600 mm	Section depth	h = 500 mm
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PASS - Minimum dimensions for fire resistance met



Positive moment - section 6.1

Design bending moment	M = 110.0 kNm	Effective depth tension reinf.	d = 407 mm
Area of tension reinf. req'd	$A_{s,req} = 654 \text{ mm}^2$	Area of tension reinf. prov	$A_{s,prov} = 1005 \text{ mm}^2$
Min area of reinf. (exp.9.1N)	$A_{s,min} = 351 \text{ mm}^2$	Max area reinf. (cl.9.2.1.1(3))	$A_{s,max} = 12000 \text{ mm}^2$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Maximum crack width	$w_k = 0.3 \text{ mm}$	Min area reinf req'd (exp.7.1)	$A_{sc,min} = 445 \text{ mm}^2$
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PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment	$M_{QP} = 95.0 \text{ kNm}$		
Actual tension bar spacing	$S_{bar} = 103.5 \text{ mm}$	Max bar spacing (Table 7.3N)	$S_{bar,max} = 194.5 \text{ mm}$

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

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Negative moment - section 6.1

Design bending moment	$M = 215.0$ kNm	Effective depth tension reinf.	$d = 405$ mm
Area of tension reinf. req'd	$A_{s,req} = 1319$ mm ²	Area of tension reinf. prov	$A_{s,prov} = 1571$ mm ²
Min area of reinf. (exp.9.1N)	$A_{s,min} = 350$ mm ²	Max area reinf. (cl.9.2.1.1(3))	$A_{s,max} = 12000$ mm ²

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Maximum crack width	$w_k = 0.3$ mm	Min area reinf req'd (exp.7.1)	$A_{s,min} = 442$ mm ²
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PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment	$M_{QP} = 185.0$ kNm	Max bar spacing (Table 7.3N)	$S_{bar,max} = 107.3$ mm
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PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing (Section 8.2)

Top bar spacing	$S_{top} = 82.5$ mm	Min allow. top bar spacing	$S_{top,min} = 25.0$ mm
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PASS - Actual bar spacing exceeds minimum allowable

Bottom bar spacing	$S_{bot} = 87.5$ mm	Min allow. bottom bar spacing	$S_{bot,min} = 25.0$ mm
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PASS - Actual bar spacing exceeds minimum allowable

Section in shear (section 6.2)

Angle of comp. shear strut	$\theta_{max} = 45$ deg	Strength reduction factor	$v_1 = 0.533$
Compression chord coefficient	$\alpha_{cw} = 1.00$	Minimum area of shear reinf.	$A_{sv,min} = 508$ mm ² /m
Shear force at support	$V_{Ed,max} = 250$ kN	Max design shear resistance	$V_{Rd,max} = 1119$ kN

PASS - Design shear force at support is less than maximum design shear resistance

Design shear force	$V_{Ed} = 250$ kN	Area shear reinf. req'd	$A_{sv,req} = 613$ mm ² /m
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PASS - Area of shear reinforcement provided exceeds minimum required

Max. long. spacing - exp.9.6N	$s_{vl,max} = 304$ mm		
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PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Provide 600w x 500dp Beam:

- 5H20's Top
- 5H16's Bottom
- H10 Links @ 200 centres – 2 Legs

Ground Beam beneath Front/Rear Wall

M (hog) = 190kNm
M (sag) = 90kNm
Shear = 210kN

RC MEMBER DESIGN

In accordance with EN1992-1-1:2004 incorporating Corrigenda January 2008 and the UK national annex

Tedds calculation version 3.0.06

Concrete details (Table 3.1 - Strength and deformation characteristics for concrete)

Concrete strength class	C28/35	Char. comp. cylinder strength	$f_{ck} = 28$ N/mm ²
Design comp conc. strength	$f_{c,d} = 18.7$ N/mm ²	Maximum aggregate size	$h_{agg} = 20$ mm

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Reinforcement details

Char. yield strength of reinf. $f_{yk} = 500 \text{ N/mm}^2$ Partial factor for reinf. steel $\gamma_s = 1.15$
Design yield strength of reinf. $f_{yd} = 435 \text{ N/mm}^2$

Nominal cover to reinforcement

Nominal cover to top reinf $c_{nom,t} = 75 \text{ mm}$ Nominal cover to bottom reinf $c_{nom,b} = 75 \text{ mm}$
Nominal cover to side reinf $c_{nom,s} = 75 \text{ mm}$

Fire resistance

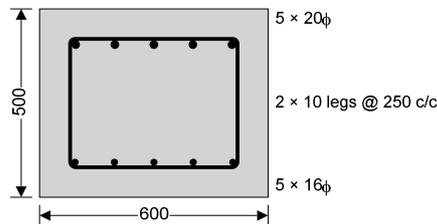
Standard fire resistance period $R = 60 \text{ min}$ No. sides exposed to fire 3
Minimum width of beam $b_{min} = 120 \text{ mm}$

Section 1 - Ground Beam

Rectangular section details

Section width $b = 600 \text{ mm}$ Section depth $h = 500 \text{ mm}$

PASS - Minimum dimensions for fire resistance met



Positive moment - section 6.1

Design bending moment $M = 90.0 \text{ kNm}$ Effective depth tension reinf. $d = 407 \text{ mm}$
Area of tension reinf. req'd $A_{s,req} = 535 \text{ mm}^2$ Area of tension reinf. prov $A_{s,prov} = 1005 \text{ mm}^2$
Min area of reinf. (exp.9.1N) $A_{s,min} = 351 \text{ mm}^2$ Max area reinf. (cl.9.2.1.1(3)) $A_{s,max} = 12000 \text{ mm}^2$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Maximum crack width $w_k = 0.3 \text{ mm}$ Min area reinf req'd (exp.7.1) $A_{sc,min} = 445 \text{ mm}^2$

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment $M_{QP} = 78.0 \text{ kNm}$
Actual tension bar spacing $S_{bar} = 103.5 \text{ mm}$ Max bar spacing (Table 7.3N) $S_{bar,max} = 249.2 \text{ mm}$

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Negative moment - section 6.1

Design bending moment $M = 190.0 \text{ kNm}$ Effective depth tension reinf. $d = 405 \text{ mm}$
Area of tension reinf. req'd $A_{s,req} = 1154 \text{ mm}^2$ Area of tension reinf. prov $A_{s,prov} = 1571 \text{ mm}^2$
Min area of reinf. (exp.9.1N) $A_{s,min} = 350 \text{ mm}^2$ Max area reinf. (cl.9.2.1.1(3)) $A_{s,max} = 12000 \text{ mm}^2$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Maximum crack width $w_k = 0.3 \text{ mm}$ Min area reinf req'd (exp.7.1) $A_{sc,min} = 442 \text{ mm}^2$

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment $M_{QP} = 165.0 \text{ kNm}$
Actual tension bar spacing $S_{bar} = 102.5 \text{ mm}$ Max bar spacing (Table 7.3N) $S_{bar,max} = 153.2 \text{ mm}$

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing (Section 8.2)

Top bar spacing $S_{top} = 82.5 \text{ mm}$ Min allow. top bar spacing $S_{top,min} = 25.0 \text{ mm}$

PASS - Actual bar spacing exceeds minimum allowable

Bottom bar spacing $S_{bot} = 87.5 \text{ mm}$ Min allow. bottom bar spacing $S_{bot,min} = 25.0 \text{ mm}$

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PASS - Actual bar spacing exceeds minimum allowable

Section in shear (section 6.2)

Angle of comp. shear strut	$\theta_{max} = 45 \text{ deg}$	Strength reduction factor	$V_1 = 0.533$
Compression chord coefficient	$\alpha_{cw} = 1.00$	Minimum area of shear reinf.	$A_{sv,min} = 508 \text{ mm}^2/\text{m}$
Shear force at support	$V_{Ed,max} = 210 \text{ kN}$	Max design shear resistance	$V_{Rd,max} = 1130 \text{ kN}$
PASS - Design shear force at support is less than maximum design shear resistance			
Design shear force	$V_{Ed} = 210 \text{ kN}$	Area shear reinf. req'd	$A_{sv,req} = 510 \text{ mm}^2/\text{m}$
Area of shear reinf prov.	$A_{sv,prov} = 628 \text{ mm}^2/\text{m}$	PASS - Area of shear reinforcement provided exceeds minimum required	

Max. long. spacing - exp.9.6N $s_{vl,max} = 304 \text{ mm}$

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Provide 600w x 500dp Beam:

- 5H20's Top
- 5H16's Bottom
- H10 Links @ 200 centres – 2 Legs

Internal Beams beneath Internal Walls

M (hog) = 155kNm

M (sag) = 40

Shear = 180kN

RC MEMBER DESIGN

In accordance with EN1992-1-1:2004 incorporating Corrigenda January 2008 and the UK national annex

Concrete details (Table 3.1 - Strength and deformation characteristics for concrete)

Concrete strength class	C28/35	Char. comp. cylinder strength	$f_{ck} = 28 \text{ N/mm}^2$
Design comp conc. strength	$f_{c wd} = 18.7 \text{ N/mm}^2$	Maximum aggregate size	$h_{agg} = 20 \text{ mm}$

Reinforcement details

Char. yield strength of rinf.	$f_{yk} = 500 \text{ N/mm}^2$	Partial factor for reinf. steel	$\gamma_s = 1.15$
Design yield strength of reinf.	$f_{yd} = 435 \text{ N/mm}^2$		

Nominal cover to reinforcement

Nominal cover to top reinf	$c_{nom,t} = 75 \text{ mm}$	Nominal cover to bottom reinf	$c_{nom,b} = 75 \text{ mm}$
Nominal cover to side reinf	$c_{nom,s} = 75 \text{ mm}$		

Fire resistance

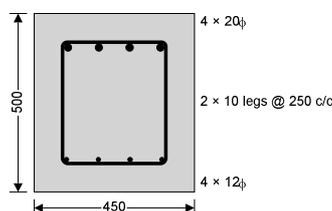
Standard fire resistance period	$R = 60 \text{ min}$	No. sides exposed to fire	3
Minimum width of beam	$b_{min} = 120 \text{ mm}$		

Section 1 - Ground Beam

Rectangular section details

Section width	$b = 450 \text{ mm}$	Section depth	$h = 500 \text{ mm}$
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PASS - Minimum dimensions for fire resistance met



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Positive moment - section 6.1

Design bending moment	$M = 40.0$ kNm	Effective depth tension reinf.	$d = 409$ mm
Area of tension reinf. req'd	$A_{s,req} = 237$ mm ²	Area of tension reinf. prov	$A_{s,prov} = 452$ mm ²
Min area of reinf. (exp.9.1N)	$A_{s,min} = 265$ mm ²	Max area reinf. (cl.9.2.1.1(3))	$A_{s,max} = 9000$ mm ²

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Maximum crack width	$w_k = 0.3$ mm	Min area reinf req'd (exp.7.1)	$A_{sc,min} = 337$ mm ²
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PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment	$M_{QP} = 35.0$ kNm	Max bar spacing (Table 7.3N)	$s_{bar,max} = 251.1$ mm
Actual tension bar spacing	$s_{bar} = 89.3$ mm		

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Negative moment - section 6.1

Design bending moment	$M = 155.0$ kNm	Effective depth tension reinf.	$d = 405$ mm
Area of tension reinf. req'd	$A_{s,req} = 948$ mm ²	Area of tension reinf. prov	$A_{s,prov} = 1257$ mm ²
Min area of reinf. (exp.9.1N)	$A_{s,min} = 262$ mm ²	Max area reinf. (cl.9.2.1.1(3))	$A_{s,max} = 9000$ mm ²

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Maximum crack width	$w_k = 0.3$ mm	Min area reinf req'd (exp.7.1)	$A_{sc,min} = 331$ mm ²
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PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment	$M_{QP} = 135.0$ kNm	Max bar spacing (Table 7.3N)	$s_{bar,max} = 143$ mm
Actual tension bar spacing	$s_{bar} = 86.7$ mm		

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing (Section 8.2)

Top bar spacing	$s_{top} = 66.7$ mm	Min allow. top bar spacing	$s_{top,min} = 25.0$ mm
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PASS - Actual bar spacing exceeds minimum allowable

Bottom bar spacing	$s_{bot} = 77.3$ mm	Min allow. bottom bar spacing	$s_{bot,min} = 25.0$ mm
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PASS - Actual bar spacing exceeds minimum allowable

Section in shear (section 6.2)

Angle of comp. shear strut	$\theta_{max} = 45$ deg	Strength reduction factor	$v_1 = 0.533$
Compression chord coefficient	$\alpha_{cw} = 1.00$	Minimum area of shear reinf.	$A_{sv,min} = 381$ mm ² /m
Shear force at support	$V_{Ed,max} = 180$ kN	Max design shear resistance	$V_{Rd,max} = 842$ kN

PASS - Design shear force at support is less than maximum design shear resistance

Design shear force	$V_{Ed} = 180$ kN	Area shear reinf. req'd	$A_{sv,req} = 440$ mm ² /m
Area of shear reinf prov.	$A_{sv,prov} = 628$ mm ² /m		

PASS - Area of shear reinforcement provided exceeds minimum required

Max. long. spacing - exp.9.6N	$s_{vl,max} = 304$ mm
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PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Provide 450w x 500dp Beam:

- 4H20's Top
- 4H16's Bottom
- H10 Links @ 200 centres – 2 Legs