

Structural Calculations	Project:	Date: JUL. '23	Designer: JL
Project No. 23-258	Subject:	Sheet No. 1	Rev:

Project Title: PROPOSED SUPERSTRUCTURE

Location: 1 WILES AVENUE, MOULTON CHAPEL,
SPALDING, LINCS. PE12 0BQ

Client: GRAHAM HYDE

Project No: 23-258

Prepared By: J. LAWRENCE

Checked By: A. WILSON CEng MStructE

Date: JUL. '23

Document Ref: SW23-258-CAL-01

Structural Calculations

Project:

Date: JUL. '23

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Project No. 23-258

Subject:

Sheet No. 2

Rev:

Loadings**Roof**

Roof Tiles	=	0.60 kN/m ²
Battens & Felt	=	0.04 kN/m ²
Trusses	=	0.20 kN/m ²
Insulation	=	0.02 kN/m ²
Plasterboard & Skim	=	0.16 kN/m ²
		<u>1.02 kN/m²</u>
Roof Imposed	=	<u>0.60 kN/m²</u>

First Floor

Floor Boards	=	0.14 kN/m ²
Joists	=	0.17 kN/m ²
Plaster Board & Skim	=	0.16 kN/m ²
Finishes	=	0.10 kN/m ²
		<u>0.57 kN/m²</u>
First Floor Imposed & Lighweight Partitions	=	<u>2.00 kN/m²</u>

Ground Floor

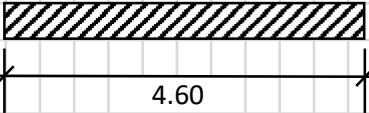
Insulation & Screed	=	1.88 kN/m ²
Beam & Block Flooring	=	2.40 kN/m ²
Finishes	=	0.10 kN/m ²
		<u>4.38 kN/m²</u>
Ground Floor Imposed & Lighweight Partitions	=	<u>2.00 kN/m²</u>

Wall

Brickwork	=	2.20 kN/m ²
Blockwork	=	1.40 kN/m ²
Render & Skim	=	0.35 kN/m ²
		<u>3.95 kN/m²</u>

Structural Calculations	Project:	Date: JUL. '23	Designer: JL
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B1 - OVER KITCHEN AREA [INNER LEAF]

Span = 4.60 m	Wall = 2.70 m x 1.75 kN/m ²
	Roof (D) = 4.40 m x 1.02 kN/m ²
	Roof (I) = 4.40 m x 0.60 kN/m ²
	First Floor (D) = 1.55 m x 0.57 kN/m ²
	First Floor (I) = 1.55 m x 2.00 kN/m ²
(D) = 23.866 kN	(D) = 23.866 kN
(I) = 13.202 kN	(I) = 13.202 kN

Loadings

Beam Self Weight					=	0.28	kN/m x	4.60	m	=	1.288	kN
Wall				(D)	=	4.73	kN/m x	4.60	m	=	21.735	kN
Roof				(D)	=	4.49	kN/m x	4.60	m	=	20.645	kN
Roof				(I)	=	2.64	kN/m x	4.60	m	=	12.144	kN
First Floor				(D)	=	0.88	kN/m x	4.60	m	=	4.064	kN
First Floor				(I)	=	3.10	kN/m x	4.60	m	=	14.260	kN
											74.136	kN

$$M_x = \frac{((47.732 \times 1.4) + (26.404 \times 1.6)) \times 4.6}{8} = 62.716 \text{ kNm}$$

(Factored)

Try 254x102x28UB [Grade S355]	Beam to be fully restrained by First floor joists.	$M_{cx} = \frac{125}{\text{PASS}}$ kNm
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$$\text{Beam Deflection [Total Load]} = \frac{5 \times 74136 \times 4600^3}{384 \times 205 \times 10^3 \times 4005 \times 10^4} = \frac{11.44}{\text{PASS}} \text{ mm}$$

$$\text{Allowable Deflection} = \frac{4600}{360} = 12.778 \text{ mm}$$

∴ 254x102x28UB is satisfactory

Bearing Check

$$\text{Factored Reaction} = \frac{((47.732 \times 1.4) + (26.404 \times 1.6))}{2} = 54.536 \text{ kN}$$

$$\text{Bearing Stress} = \frac{54.536 \times 10^3 \times 3.5}{150 \times 102} = 12.48 \text{ N/mm}^2$$

$$L_{\text{Min}} = \frac{54.536 \times 10^3 \times 3.5}{150 \times 3.5} = 364 \text{ mm}$$

∴ Provide 440 Long x 215 Deep x 100 Wide Concrete Padstone

Structural Calculations

Project:

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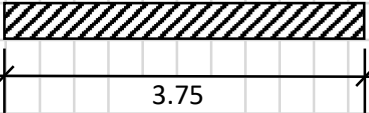
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Rev:

B2 - OVER LIVING AREA [INNER LEAF]

Span = 3.75 m	Wall = 2.70 m x 1.75 kN/m ²
	Roof (D) = 4.40 m x 1.02 kN/m ²
	Roof (I) = 4.40 m x 0.60 kN/m ²
	First Floor (D) = 0.00 m x 0.57 kN/m ²
	First Floor (I) = 0.00 m x 2.00 kN/m ²
(D) = 17.743 kN	(D) = 17.743 kN
(I) = 4.95 kN	(I) = 4.95 kN

Loadings

Beam Self Weight					=	0.25	kN/m	x		3.75	m	=	0.938	kN
Wall				(D)	=	4.73	kN/m	x		3.75	m	=	17.719	kN
Roof				(D)	=	4.49	kN/m	x		3.75	m	=	16.830	kN
Roof				(I)	=	2.64	kN/m	x		3.75	m	=	9.900	kN
First Floor				(D)	=	0.00	kN/m	x		3.75	m	=	0.000	kN
First Floor				(I)	=	0.00	kN/m	x		3.75	m	=	0.000	kN
													<u>45.386</u>	kN

$$M_x = \frac{((35.486 \times 1.4) + (9.900 \times 1.6)) \times 3.75}{8} = 30.713 \text{ kNm}$$

(Factored)

Try 254x102x28UB [Grade S355] $M_b @ 3.75 \text{ m} = \frac{41.05}{\text{PASS}} \text{ kNm}$

$$\text{Beam Deflection [Total Load]} = \frac{5 \times 45386 \times 3750^3}{384 \times 205 \times 10^3 \times 4005 \times 10^4} = \frac{3.80}{\text{PASS}} \text{ mm}$$

$$\text{Allowable Deflection} = \frac{3750}{360} = 10.417 \text{ mm}$$

$\therefore$ 254x102x28UB is satisfactory

Bearing Check

$$\text{Factored Reaction} = \frac{((35.486 \times 1.4) + (9.900 + 1.6))}{2} = 32.76 \text{ kN}$$

$$\text{Bearing Stress} = \frac{32.76 \times 10^3 \times 3.5}{100 \times 102} = 11.24 \text{ N/mm}^2$$

$$L_{\text{Min}} = \frac{32.76 \times 10^3 \times 3.5}{100 \times 3.5} = 328 \text{ mm}$$

$\therefore$ Provide 440 Long x 215 Deep x 100 Wide Concrete Padstone

Structural Calculations

Project:

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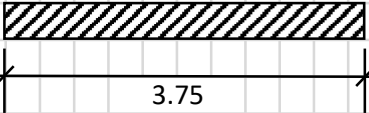
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Rev:

B3 - OVER LIVING AREA [OUTER LEAF]

Span = 3.75 m	Wall = 2.70 m x 2.20 kN/m ²
	Roof (D) = 1.00 m x 1.02 kN/m ²
	Roof (I) = 1.00 m x 0.60 kN/m ²
	First Floor (D) = 0.00 m x 0.57 kN/m ²
	First Floor (I) = 0.00 m x 2.00 kN/m ²
(D) = 13.519 kN	(D) = 13.519 kN
(I) = 1.125 kN	(I) = 1.125 kN

Loadings

Beam Self Weight				=	0.25	kN/m	x	3.75	m	=	0.938	kN
Wall			(D)	=	5.94	kN/m	x	3.75	m	=	22.275	kN
Roof			(D)	=	1.02	kN/m	x	3.75	m	=	3.825	kN
Roof			(I)	=	0.60	kN/m	x	3.75	m	=	2.250	kN
First Floor			(D)	=	0.00	kN/m	x	3.75	m	=	0.000	kN
First Floor			(I)	=	0.00	kN/m	x	3.75	m	=	0.000	kN
											<u>29.288</u>	kN

$$M_x = \frac{((27.038 \times 1.4) + (2.250 \times 1.6)) \times 3.75}{8} = 19.431 \text{ kNm}$$

(Factored)

Try 254x102x28UB
[Grade S355]

$$M_b @ 3.75 \text{ m} = \frac{25.7}{\text{PASS}} \text{ kNm}$$

$$\text{Beam Deflection [Total Load]} = \frac{5 \times 29288 \times 3750^3}{384 \times 205 \times 10^3 \times 4005 \times 10^4} = \frac{2.45}{\text{PASS}} \text{ mm}$$

$$\text{Allowable Deflection} = \frac{3750}{360} = 10.417 \text{ mm}$$

∴ 254x102x28UB is satisfactory**Bearing Check**

$$\text{Factored Reaction} = \frac{((27.038 \times 1.4) + (2.250 + 1.6))}{2} = 20.726 \text{ kN}$$

$$\text{Bearing Stress} = \frac{20.726 \times 10^3 \times 3.5}{100 \times 102} = 7.11 \text{ N/mm}^2$$

$$L_{\text{Min}} = \frac{20.726 \times 10^3 \times 3.5}{100 \times 3.5} = 207 \text{ mm}$$

∴ Provide 440 Long x 215 Deep x 100 Wide Concrete Padstone

Structural Calculations

Project:

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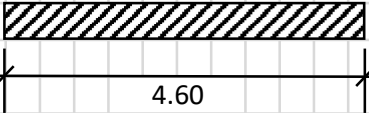
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Rev:

B4 - OVER KITCHEN AREA [INNER LEAF]

Span = 4.60 m	Wall = 2.70 m x 2.20 kN/m ²
	Roof (D) = 1.00 m x 1.02 kN/m ²
	Roof (I) = 1.00 m x 0.60 kN/m ²
	First Floor (D) = 0.00 m x 0.57 kN/m ²
	First Floor (I) = 0.00 m x 2.00 kN/m ²
(D) = 16.652 kN	(D) = 16.652 kN
(I) = 1.38 kN	(I) = 1.38 kN

Loadings

Beam Self Weight				=	0.28	kN/m	x	4.60	m	=	1.288	kN
Wall			(D)	=	5.94	kN/m	x	4.60	m	=	27.324	kN
Roof			(D)	=	1.02	kN/m	x	4.60	m	=	4.692	kN
Roof			(I)	=	0.60	kN/m	x	4.60	m	=	2.760	kN
First Floor			(D)	=	0.00	kN/m	x	4.60	m	=	0.000	kN
First Floor			(I)	=	0.00	kN/m	x	4.60	m	=	0.000	kN
											<u>36.064</u>	kN

$$M_x = \frac{((33.304 \times 1.4) + (2.760 \times 1.6)) \times 4.6}{8} = 29.349 \text{ kNm}$$

(Factored)

Try 254x102x28UB [Grade S355] Beam to be fully restrained by First floor joists.

$$M_{cx} = \frac{125}{\text{PASS}} \text{ kNm}$$

$$\text{Beam Deflection [Total Load]} = \frac{5 \times 36064 \times 4600^3}{384 \times 205 \times 10^3 \times 4005 \times 10^4} = 5.57 \text{ mm}$$

PASS

$$\text{Allowable Deflection} = \frac{4600}{360} = 12.778 \text{ mm}$$

∴ 254x102x28UB is satisfactory

Bearing Check

$$\text{Factored Reaction} = \frac{((33.304 \times 1.4) + (2.760 \times 1.6))}{2} = 25.521 \text{ kN}$$

$$\text{Bearing Stress} = \frac{25.521 \times 10^3 \times 3.5}{150 \times 102} = 5.84 \text{ N/mm}^2$$

$$L_{\text{Min}} = \frac{25.521 \times 10^3 \times 3.5}{150 \times 3.5} = 170 \text{ mm}$$

∴ Provide 440 Long x 215 Deep x 100 Wide Concrete Padstone

Structural Calculations

Project:

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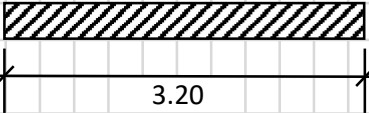
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Rev:

B5 - OVER KITCHEN / DINING ROOM

Span = 3.20 m	Wall = 2.70 m x 2.10 kN/m ²
	Roof (D) = 0.00 m x 1.02 kN/m ²
	Roof (I) = 0.00 m x 0.60 kN/m ²
	First Floor (D) = 1.90 m x 0.57 kN/m ²
	First Floor (I) = 1.90 m x 2.00 kN/m ²
(D) = 11.205 kN	(D) = 11.205 kN
(I) = 6.08 kN	(I) = 6.08 kN

Loadings

Beam Self Weight	=	0.25	kN/m x	3.20 m	=	0.800	kN
Wall (D)	=	5.67	kN/m x	3.20 m	=	18.144	kN
Roof (D)	=	0.00	kN/m x	3.20 m	=	0.000	kN
Roof (I)	=	0.00	kN/m x	3.20 m	=	0.000	kN
First Floor (D)	=	1.08	kN/m x	3.20 m	=	3.466	kN
First Floor (I)	=	3.80	kN/m x	3.20 m	=	12.160	kN
						<u>34.570</u>	<u>kN</u>

$$M_x = \frac{((22.410 \times 1.4) + (12.160 \times 1.6)) \times 3.20}{8} = 20.332 \text{ kNm}$$

(Factored)

Try 203x102x23UB
[Grade S355]

$$M_b @ 3.20 \text{ m} = \frac{36.7}{\text{PASS}} \text{ kNm}$$

$$\text{Beam Deflection [Total Load]} = \frac{5 \times 34570 \times 3200^3}{384 \times 205 \times 10^3 \times 2105 \times 10^4} = \frac{3.42}{\text{PASS}} \text{ mm}$$

$$\text{Allowable Deflection} = \frac{3200}{360} = 8.8889 \text{ mm}$$

∴ 203x102x23UB is satisfactory

Bearing Check

$$\text{Factored Reaction} = \frac{((22.410 \times 1.4) + (12.160 + 1.6))}{2} = 25.415 \text{ kN}$$

$$\text{Bearing Stress} = \frac{25.415 \times 10^3 \times 3.5}{100 \times 102} = 8.72 \text{ N/mm}^2$$

$$L_{\text{Min}} = \frac{25.415 \times 10^3 \times 3.5}{100 \times 3.5} = 254 \text{ mm}$$

∴ Provide 440 Long x 215 Deep x 100 Wide Concrete Padstone

Structural Calculations

Project:

Date: JUL. '23

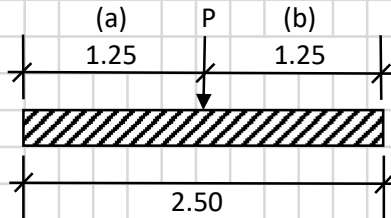
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Rev:

B6 - SPREADER BEAM

Wall	=	0.00	m	x	1.75	kN/m ²
Roof (D)	=	0.00	m	x	1.02	kN/m ²
Roof (D)	=	0.00	m	x	0.60	kN/m ²
First Floor (D)	=	0.00	m	x	0.57	kN/m ²
First Floor (D)	=	0.00	m	x	2.00	kN/m ²
Ground Floor (D)	=	0.00	m	x	4.38	kN/m ²
Ground Floor (D)	=	0.00	m	x	2.50	kN/m ²

UDL Loadings

Wall	=	0.00	kN/m	x	2.50	m	=	0.000	kN
Roof (D)	=	0.00	kN/m	x	2.50	m	=	0.000	kN
Roof (I)	=	0.00	kN/m	x	2.50	m	=	0.000	kN
First Floor (D)	=	0.00	kN/m	x	2.50	m	=	0.000	kN
First Floor (I)	=	0.00	kN/m	x	2.50	m	=	0.000	kN
Ground Floor (D)	=	0.00	kN/m	x	2.50	m	=	0.000	kN
Ground Floor (I)	=	0.00	kN/m	x	2.50	m	=	0.000	kN
								<u>0.000</u>	kN

Point Load

$$71.741 \times 1.4 + 20.650 \times 1.6 = 133.48 \text{ kN}$$

$$\text{Reaction @ P} = \frac{133.48 \times 1.25}{2.50} = 66.739 \text{ kN}$$

$$\text{Reaction @ R1} = (0.00 \times 1.4) + (0 \times 1.6) + 66.739 = 66.739 \text{ kN}$$

$$M = \frac{(0.000 \times 1.4) + (0.000 \times 1.6) \times 2.50}{8} + \frac{133.48 \times 1.25 \times 1.25}{2.50}$$

$$M = 83.423 \text{ kNm} \quad M_b @ 2.50 \text{ m} = 348 \text{ kNm} \quad \text{PASS}$$

Deflection Try 254x254x73UC

$$\frac{5 \times 0 \times 2500^3}{384 \times 205 \times 10^3 \times 11410 \times 10^4} + \frac{46196 \times 1250 \times 1250 \times (2500 + 1250)}{27 \times 205 \times 10^3 \times 11410 \times 10^4 \times 2500}$$

$$\sqrt{3 \times 1250 \times (2500 + 1250)} = 0.64 \text{ mm}$$

$$\text{Allowable Deflection} = \frac{2500}{360} = 6.94 \text{ mm}$$

∴ 254x254x73UC is satisfactory

Structural Calculations

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L1 - PROPOSED LINTEL OVER WINDOW

Span = 1.20 m [Clear Span]

Loadings

Wall	(D)	=	0.30 m	x	3.95 kN/m ²	=	1.185	kN/m
Roof	(D)	=	1.00 m	x	1.02 kN/m ²	=	1.020	kN/m
Roof	(I)	=	1.00 m	x	0.60 kN/m ²	=	0.600	kN/m
First Floor	(D)	=	0.00 m	x	0.57 kN/m ²	=	0.000	kN/m
First Floor	(I)	=	0.00 m	x	2.00 kN/m ²	=	0.000	kN/m
							<u>2.805</u>	kN/m

 $1.20 \text{ m} \times 2.805 \text{ kN} = 3.366 \text{ kN}$ $\therefore$ L1/S 100 IG LINTEL is satisfactory 10 kN capacity

End bearing as specified by specialist manufacturer.

Structural Calculations

Project:

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L2 - PROPOSED LINTEL OVER BI-FOLD

Span = 3.00 m [Clear Span]

Loadings

Wall	(D)	=	0.30 m	x	3.95 kN/m ²	=	1.185	kN/m
Roof	(D)	=	1.00 m	x	1.02 kN/m ²	=	1.020	kN/m
Roof	(I)	=	1.00 m	x	0.60 kN/m ²	=	0.600	kN/m
First Floor	(D)	=	0.00 m	x	0.57 kN/m ²	=	0.000	kN/m
First Floor	(I)	=	0.00 m	x	2.00 kN/m ²	=	0.000	kN/m
							<u>2.805</u>	kN/m

3.00 m x 2.805 kN = 8.415 kN

∴ L1/S 100 IG LINTEL is satisfactory 20 kN capacity

End bearing as specified by specialist manufacturer.

Structural Calculations

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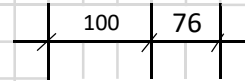
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Rev:

P1 - POST CARRYING INCOMING LOADS FROM B1, B2, B3 & B4

L = 2.50 m

(D) = 40.491 kN		(D) = 31.25 kN
(I) = 14.580 kN		(I) = 6.070 kN

FACTORED AXIAL LOAD = $(71.74 \times 1.4) + (20.65 \times 1.6) = 133.48 \text{ kN}$ Ecc x-x = $((40.49 \times 1.4) + (14.58 \times 1.6)) \times 0.176 = 14.083 \text{ kN}$ TRY 152x152x23 UC [S355] Le = 5.00 m

CHECK COMBINED AXIAL LOAD & BENDING

$$\frac{133.5}{654.0} + \frac{14.1}{50.1} = 0.49 < 1.0 \quad \text{PASS}$$
152x152x23 UC [S355] POST IS SATISFACTORY

Structural Calculations

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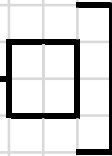
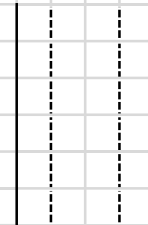
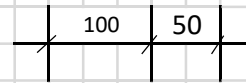
Sheet No. 12

Rev:

P2 - POST CARRYING INCOMING LOADS FROM B1 & B4

$$L = 2.50 \text{ m}$$

$$\begin{aligned} (D) &= 40.49 \text{ kN} \\ (I) &= 14.580 \text{ kN} \end{aligned}$$

WIND AREA
6.0 m

$$\text{FACTORED AXIAL LOAD} = (40.49 \times 1.4) + (14.58 \times 1.6) = 80.02 \text{ kN}$$

$$\text{Ecc x-x} = ((40.49 \times 1.4) + (14.58 \times 1.6)) \times 0.150 = 12.002 \text{ kN}$$

$$w = 0.65 \text{ kN/m}^2 \times 6.0 \text{ m} = 3.90 \text{ kN/m} \times 1.4 = 5.46 \text{ kN/m}$$

$$W = 5.46 \text{ kN/m} \times 2.50 \text{ m} = 13.65 \text{ kN}$$

$$M_w = 2.50 \text{ m} \times 13.65 \text{ kN} \div 8 = 4.266 \text{ kN}$$

$$M_{\text{total}} = 12.002 \text{ kN} + 4.266 \text{ kN} = 16.268 \text{ kN}$$

$$\text{TRY } 100 \times 100 \times 8 \text{ SHS [S355]} \quad L_e = 5.00 \text{ m}$$

CHECK COMBINED AXIAL LOAD & BENDING

$$\frac{80.0}{293.0} + \frac{16.3}{34.0} = 0.75 < 1.0 \quad \text{PASS}$$

CHECK DEFLECTION

$$\begin{aligned} \text{BEAM DEFLECTION} &= \frac{5 \times 9.75 \times 2500^3}{384 \times 205 \times 10^3 \times 400 \times 10^4} = 0.002 \text{ mm} \\ \text{[TOTAL LOAD]} & \quad \text{PASS} \end{aligned}$$

$$\text{Allowable Deflection} = \frac{2500}{360} = 6.9444 \text{ mm}$$

100x100x8 SHS [S355] POST IS SATISFACTORY

Structural Calculations

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Sheet No. 13

Rev:

T1 - ROOF JOIST

Span = 2.00 m Rafters @ 0.6 m Crs.

Load/m = (1.02 kN/m² + 0.60 kN/m²) x 0.6 m = 0.97 kN/mMoment = $\frac{0.97 \times 2^2}{8} = 0.49 \text{ kNm}$ Try 50 x 150 (C16) Timber $z = \frac{50 \times 150^2}{6} = 187500 \text{ mm}^3$ Stress = $\frac{0.49 \times 10^6}{187500} = 2.59 \text{ N/mm}^2$ K7 = $\left(\frac{300}{150}\right)^{0.11} = 1.08$ K8 = 1.1 K3 = 1.25 $\therefore \text{Allowable Stress} = 1.08 \times 1.1 \times 1.25 \times 5.3 = 7.86 \text{ N/mm}^2 \quad \therefore \text{PASS}$ Check Deflection $I = \frac{50 \times 150^3}{12} = 14.06 \times 10^6 \text{ mm}^4$ EMean = 5800 $\therefore \frac{5 \times 0.972 \times 2000}{384 \times 5800 \times 14.06 \times 10^6} = 2.48 \text{ mm}$ Allowable Deflection = 0.003 x 2000 = 6 $\therefore \text{PASS}$ $\therefore 50 \times 150 \text{ (C16) Timber Rafters @ } 0.6 \text{ m Crs. Are Satisfactory}$

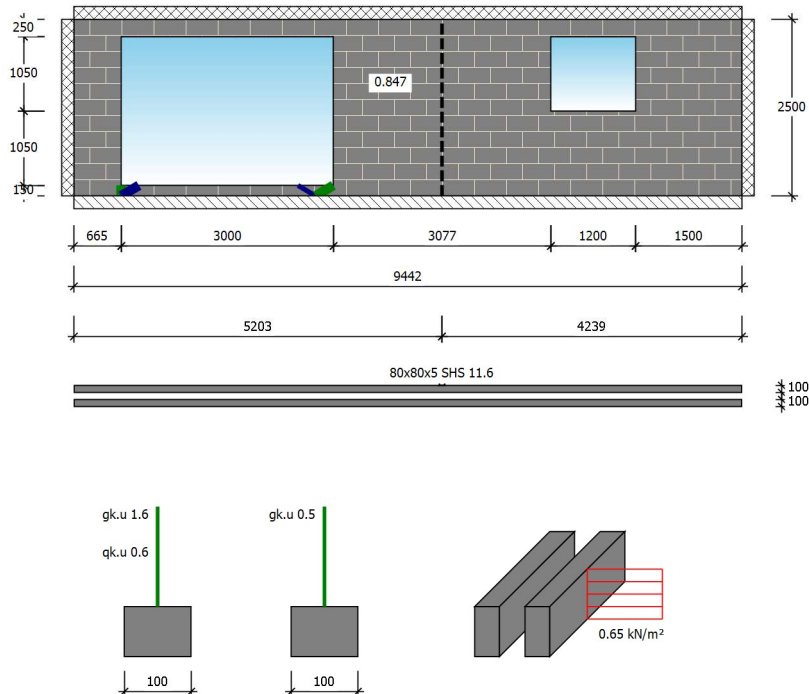
Shields Wilson

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Sheet : /
Made by : JL
Date : 18 July 2023 / Ver. 2022.06.07
Checked : APW
Approved :

Two Way Spanning, Vertically and Laterally Loaded, Cavity Wall Design to BS EN 1996-1-1:2005+A1:2012 MASONRY CHECK 1

**Summary of Design Data**

EuroCode National Annex
Wall Dimensions
Wind Post L reduction
Support Conditions
Lateral Loads
Opening load span direction

Using UK values A1 2012

$h=2.500$ m, $h_{ef}=2.031$ m (Eqn. 5.8), $L=9.442$ m, $L_{ef}=9.442$ m
Wind post assumed to act as stiffener, $L=5.203$ m, $L_{ef}=5.203$ m
Bottom Simple, Top Cont., Left Cont., Right Cont.

 $W_x=0.65$ kN/m²opening at $X=0.665$, $Y=0.15$ - Two way spanningopening at $X=6.742$, $Y=1.2$ - Two way spanning $t_1=100$, $t_2=100$, $t_{ef}=126$

Cavity Wall (mm)

Limiting Dimensions

 $\lambda=16.1 < \lambda_{lim}=27$, $L/t_{ef}=41.3$, $H/t_{ef}=19.8$, Hence $H/t_{ef} < 49.7$

0.597

OK

Outer-Leaf DesignPartial Safety Factor (γ_{mc}/γ_{mf})

Unit Material

Construction Class 2, Unit Manufacture I

2.7/2.7 Table NA.1

Concrete Blocks, Group 1, $\gamma=20$ kN/m³Normalised mean compressive strength $f_b=3.6$ N/mm²

Mortar Material

M2, $f_m=2$ N/mm²

Unit Ratio

Unit height=215, Least horizontal dimensions=100

2.15

Compressive Strength (f_k) $k=0.75$, $\alpha=0.7$, $\beta=0.3$ 2.26 N/mm² Table NA.4

Loads from above

Dead Load=0.5 kN/m

Section Properties

Area=1000 cm²/m, $Z_p=1667$ cm³/mFlexural Strength f_{xk2} (Perpendicular)0.4 N/mm² Table NA.6Flexural Strength f_{xk1} (Parallel) $f_{xk1}=0.2$, $g_d=0.03$ N/mm² $f_{xk1}=f_{xk1}+\min(g_d, 0.15 \cdot f_k/\gamma_{mc})/\gamma_{mf}$ 0.281 N/mm² Table NA.6

Critical axial compressive case

 $1.35(\gamma_{tk} \cdot h + g_k)$

Max local stress @

 $X=0$ m, $Y=1.125$ m $< f_k \cdot 0.9$ (small area red.) γ_{mc} 0.1 N/mm²

OK

Critical axial buckling case

 $1.35(\gamma_{tk} \cdot h + g_k)$

Max axial buckling force @

 $X=0.333$ m, $Y=1.25$ m averaged over width of 0.665 m

10.47 kN/m

Moments from Lateral Load

 $M_{wx,top}=0.000$ kN.m, $M_{wx,mid}=0.000$ kN.mCapacity reduction factor top, Φ $e_x=0.0$ mm, $h_{ef}=750$ mm, $t_{ef}=126.0$ mm, $t=100.0$ mm

0.900

Capacity reduction factor mid, Φ_m $e_{hm}=0.000$ mm, $h_{ef}=0.750$

0.884

 $F_r=\Phi \cdot f_k \cdot t_k/\gamma_{mc}$ $0.884 \times 2.04 \times 100/2.7$

66.7 kN/m

 F_d/F_r $10.5/66.7$

0.157

OK

 $M_{ro}=f_{xk2} \cdot Z_p/\gamma_{mf}$ $0.4 \times 1667/2.7$

0.247 kN.m/m

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$M_{ro} = f_{xk1} \cdot Z_b / \gamma_{mf}$ 0.281x1667/2.7 0.173 kN.m/m

Inner-Leaf Design

Partial Safety Factor (γ_{mc}/γ_{mf}) Construction Class 2, Unit Manufacture I 2.7/2.7 Table NA.1
Unit Material Concrete Blocks, Group 1, $\gamma = 20 \text{ kN/m}^3$
Normalised mean compressive strength $f_b = 3.6 \text{ N/mm}^2$
Mortar Material M2, $f_m = 2 \text{ N/mm}^2$
Unit Ratio Unit height=215, Least horizontal dimensions=100 2.15
Compressive Strength (f_k) $k = 0.75, \alpha = 0.7, \beta = 0.3$ 2.26 N/mm² Table NA.4
Loads from above Dead Load=1.6 kN/m, Live Load=0.6 kN/m
Section Properties Area=1000 cm²/m, $Z_p = 1667 \text{ cm}^3/\text{m}$
Flexural Strength f_{xk2} (Perpendicular) 0.4 N/mm² Table NA.6
Flexural Strength f_{xk1} (Parallel) $f_{xk1} = 0.2, g_d = 0.041 \text{ N/mm}^2$
 $f_{xk1} = f_{xk1} + \min(g_d, 0.15 \cdot f_k / \gamma_{mc}) / \gamma_{mf}$ 0.311 N/mm² Table NA.6
Critical axial compressive case
Max local stress @ $X=0 \text{ m}, Y=1.125 \text{ m} < f_k \cdot 0.9$ (small area red.) γ_{mc} 0.17 N/mm² OK
Critical axial buckling case
Max axial buckling force @ $1.35(\gamma_{tk} \cdot h + g_{ku}) + 1.05 q_{ku}$
Moments from Lateral Load $X=0.333 \text{ m}, Y=1.25 \text{ m}$ averaged over width of 0.665 m 17.36 kN/m
 $M_{wx, \text{top}} = 0.000 \text{ kN.m}, M_{wx, \text{mid}} = 0.000 \text{ kN.m}$
Capacity reduction factor top, Φ $e_x = 0.0 \text{ mm}, h_{ef} = 750 \text{ mm}, t_{ef} = 126.0 \text{ mm}, t = 100.0 \text{ mm}$ 0.900
Capacity reduction factor mid, Φ_m $e_{hm} = 0.000 \text{ mm}, h_{ef} = 0.750$ 0.884
 $F_r = \Phi \cdot f_k \cdot t_k / \gamma_{mc}$ 0.884x2.04x100/2.7 66.7 kN/m
 F_d / F_r 17.4/66.7 0.260 OK
 $M_{ri} = f_{xk2} \cdot Z_p / \gamma_{mf}$ 0.4x1667/2.7 0.247 kN.m/m
 $M_{ri} = f_{xk1} \cdot Z_b / \gamma_{mf}$ 0.311x1667/2.7 0.192 kN.m/m

Design for Lateral Loads

Design Lateral Load W_d 1.5 W_x 0.975 kN/m²
Wind Post Data Base pinned, Top pinned, Major axis bending
Leaf Continuity at Wind Posts Inner leaf continuous, outer leaf continuous
Wind posts at 5.203m 80x80x5 SHS 11.6 (S 355), $M_{el} = 12.123 \text{ kN.m}$
Yield Line Analysis Load Factor λ_p 1.181
 $U_t = 1 / \lambda_p$ 1 / 1.181 0.847 OK
Wind Post Design Full restrained moment capacity implicitly checked in yield line analysis