

STRUCTURAL CALCULATION PACKAGE

FOR

DOUBLE-STOREY EXTENSION

AT

488 BURY OLD ROAD, PRESTWICH MANCHESTER, M25 1NL

MAY Structural Engineers

March 2020

Prepared by:

Mohamed Yusuf

Sign



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1.0 Introduction

1.1 Scope

MAY Structural Engineers have been appointed by Roman Parwany as Structural Engineer for 488 Bury Old Road, Prestwich Manchester, M25 1NL. This calculation package has been created for the proposed double-storey extension. The superstructure of the proposed extension is to be timber floor joists, timber hip roof and cavity wall of 100mm blockwork Inner leaf and 102.5mm brickwork outer leaf. The substructure is to be a mass concrete strip foundation.

MAY Structural Engineers have been appointed as a designer of all steel structural members, rafters, floor joists, and foundation. This package presents the structural calculations describing the design thereof.

1.2 Design Team

Architect: Hidayat Hatami
Structural Engineer: MAY Structural Engineers

1.3 Designers and checkers

Superstructure and Substructure designer: Mohamed Yusuf
Superstructure and Substructure checker: Mohamed Yusuf

1.4 Design Codes

The project has been designed as per the following design codes and guidance: -

Element	Design Code
Gravity loading	BS EN 1991-1-12002 Part 1-1 General actions — Densities, self-weight, imposed loads for buildings
Wind Loading	BS EN 1991-1-42005 Part 1-4 General actions — Wind actions
Snow Loading	BS EN 1991-1-32003 Part 1-3 General actions — Snow loads
Structural Steel	BS EN 1993-1-12005 Part 1-1 General rules and rules for buildings
Structural Timber	BS EN 1995-1-12004 Part 1-1 General — Common rules and rules for buildings
Masonry Structures	BS EN 1996-1-12005 Part 1-1 General rules for reinforced and unreinforced masonry structures
Concrete	BS EN 1992-1-12004 Part 1-1 General rules and rules for buildings
Building Regulations	Approved Document A - Structure (2004 Edition incorporating 2004, 2010 and 2013 amendments)

Table 1 Relevant codes used in the design

1.5 Software

The following software has been used to assist in the structural design of the project:

- TEDDS TEKLA– Structural design tools;
- Microsoft Excel – Various spreadsheet design tools.

1.6 Descriptions of the property and the site

The existing property is a double-storey semidetached house and the superstructure of the property comprises of masonry walls, and timber pitched roof. The site is located in Manchester and is approximately 53°32'16.3" N and 2°16'35.6" W

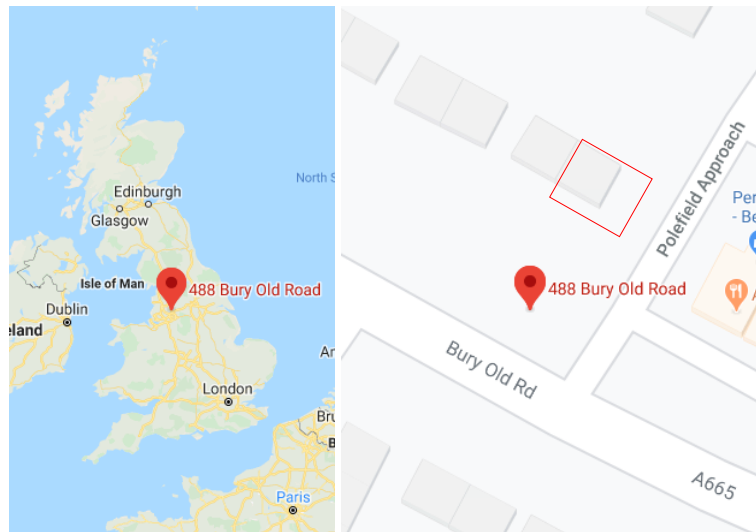


Figure 1 Site Location



Figure 2 Site plan

Site Altitude: 110m

1.7 Design Parameters

Timber

The all structural timbers (timber rafters and timber floor joists are to be a minimum of C16 timber grade and spaced 400mm centres unless noted otherwise. a minimum of 100mm bearing length should be provided to all timbers unless otherwise stated. All structural timber members have been designed as per Eurocode 5. For details and specifications refer to MAY Structural Engineers calculations and the drawings.

Steel

All steelwork has been designed to a minimum steel grade of S355 and to be protected from fire to 30-minute standard or as specified elsewhere to the satisfaction of the building control. The steelwork has been designed to BS EN 1993, refer to the calculations and the drawing for details and specifications in the appendices.

Padstones

Padstones to be grade C30 concrete minimum, at contractor's option and with the approval of the building control officer, pad stones may be constructed from 35N/mm² engineering bricks. Steel beam bearings on pad stones to be the full width of padstone, minimum 150mm.

Masonry material details

Extended gable masonry details

Outer leaf

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

Mean compressive strength of masonry unit to be 20N/mm² minimum

Mortar type to be M4 - General purpose mortar

Inner leaf

Masonry type; Aggregate concrete - Group 1

Compressive strength of masonry unit to be 7N/mm² minimum

Mortar type to be M4 - General purpose mortar

Existing Masonry Wall

The material details of the existing masonry wall were assumed as below.

Unit type assumed to be Clay - Group 1 with compressive strength of masonry unit of 20N/mm².

Mortar type is assumed to be M4 - General Purpose.

1.8 Design Variables and Assumptions

All assumptions are to be confirmed by the approved building inspector and if not correct then report back to the structural engineer for reassessment. Unless otherwise stated foundations are designed for a ground bearing capacity of 150 kN/m², which is to be verified on site. All mass concrete footings are to be taken to a minimum depth of 1.0m below external ground level unless otherwise stated or as required by the approved building inspector based on local site conditions including the presence of any medium or high water demand trees in the vicinity, whereby depths shall be as per NHBC guidelines as to building close to trees (or as stated in this document).

The existing masonry wall is intended to take an additional load. Any large cracks or wall discontinuities and/or evidence of any previous movement not apparent but exposed during construction is to be reported to engineer for further evaluation.

2.0 Loading

2.1 Gravity Loads

The table 2 below shows a summary of loadings used in the design process. The self-weight of the structural member is included in the design.

Uniformly Distributed Loads		Dead Load SLS (m ² kN/)	Imposed Load SLS (kN/m ²)
Pitched Rafter Roof	Concrete Tiles (Interlocking)	0.55	
	Battens + Felt	0.15	
	Roof Imposed Load - No Access		0.60
	Snow		0.52
Total		0.90	0.60
Ceiling Joist	Insulation	0.05	
	Ceiling Plasterboard + Skim	0.15	
	Services	0.05	
	Roof Imposed Load - No Access	0.60	0.25
Total		0.25	0.25
First Floor	Finishes	0.15	
	Tongue & groove flooring (25mm)	0.15	
	Services	0.05	
	Ceiling Plasterboard + Skim	0.15	
	Floor Imposed Load - 1,5 + 0,5 Partitions		2.00
Total		0.50	2.00

Table 2 summary of floor and roof loadings

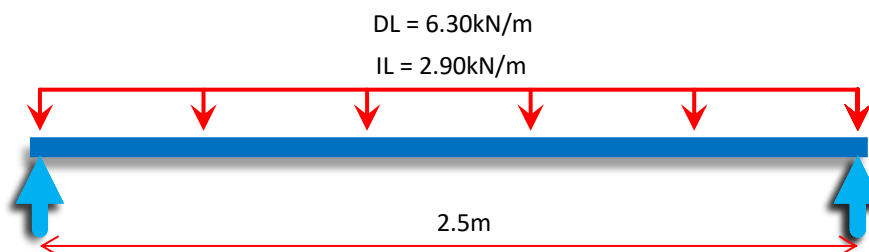
Uniformly Distributed Loads		Dead Load SLS (kN/m ²)
Cavity Wall		
Inner Leaf	Blockwork (100mm)	2.00
	Plasterboard + Skim	0.15
	Insulation	0.05
Total		2.20
Outer Leaf		
	Brickwork (102.5mm)	2.05
Total		2.05

Table 3 summary of wall self-weight

2.2 Load on structural members

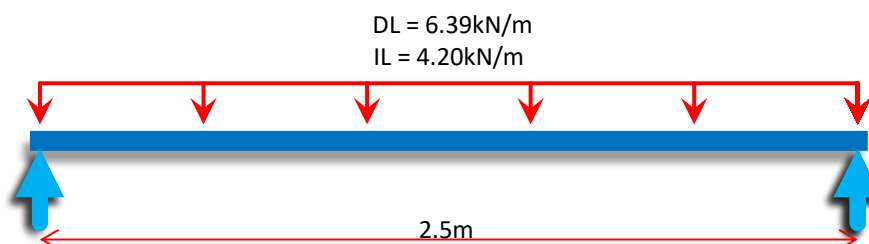
Load from	Load Type	Area Load DL(kN/m ²)	Area Load IL(kN/m ²)	With Load/ height (m)	DL (kN/m)	IL (kN/m)
Steel Beam SB1						
Inner Leaf	Full UDL	2.20		2.40	<u>5.28</u>	
First Floor	Full UDL	0.70	2.00	1.45	<u>1.02</u>	<u>2.90</u>
TOTAL					<u>6.30</u>	<u>2.90</u>

STEEL BEAM - SB1



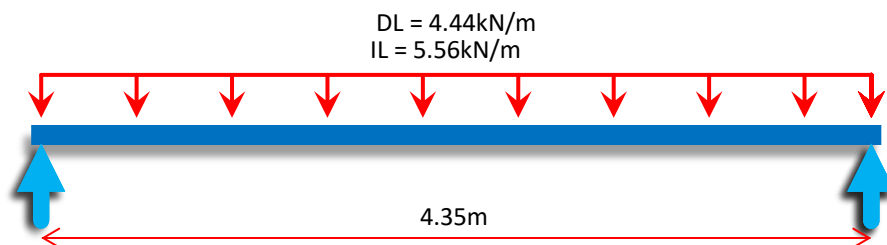
Load from	Load Type	Area Load DL(kN/m ²)	Area Load IL(kN/m ²)	With Load/ height (m)	DL (kN/m)	IL (kN/m)
Steel Beam SB2						
Outer Leaf		2.05		2.40	<u>4.92</u>	
First Floor	Full UDL	0.70	2.00	2.10	<u>1.47</u>	<u>4.20</u>
TOTAL					<u>6.39</u>	<u>4.20</u>

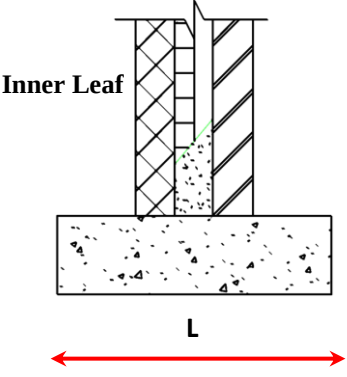
STEEL BEAM - SB2



Load from	Load Type	Area Load DL(kN/m ²)	Area Load IL(kN/m ²)	With Load/ height (m)	DL (kN/m)	IL (kN/m)
Steel Beam SB3						
Ceiling Joist	Full UDL	0.35	0.25	2.75	<u>0.96</u>	<u>0.69</u>
Rafter Pitched Roof	Full UDL	0.80	1.12	4.35	<u>3.48</u>	<u>4.87</u>
TOTAL					<u>4.44</u>	<u>5.56</u>

STEEL BEAM - SB3



Loads on foundations	Dimensions Length	Load from	Load Type	Area Load DL(kN/m ²)	Area Load IL(kN/m ²)	With Load/ height (m)	DL (kN/m)	IL (kN/m)	Point Load (kN)	
									DL (kN)	IL (kN)
	L = 700mm	Rafter Pitched Roof	Full UDL	0.80	1.12	1.30	<u>1.04</u>	<u>1.46</u>		
		Ceiling Joist	Full UDL	0.35	0.25	0.40	<u>0.14</u>	<u>0.10</u>		
		Inner Leaf	Full UDL	2.20		6.50	<u>14.30</u>			
		First Floor	Full UDL	0.70	2.00	2.10	<u>1.47</u>	<u>4.20</u>		
		Steel Beam SB3	Point Load						<u>10.10</u>	<u>11.20</u>
		point load spread at the foundation level via the masonry wall at the length of 7.0m					<u>1.44</u>	<u>1.60</u>		
		Total					<u>18.39</u>	<u>7.36</u>		
Outer Leaf		Outer Leaf	2.05	2.05		6.50	<u>13.33</u>			

3.0 Sub-Structure and Super-Structure

3.1 Sub-Structure

700mm wide and 200mm deep strip foundation is proposed for the extension perimeter walls. In the case of poor ground conditions present on-site, remove the subsoil and replaced with granular material and compacted it to achieve the required bearing pressure of a minimum 150kN/m².

3.2 Super-Structure

Roof

The hip roof structures are made of timber rafters and steel beams supporting the rafters at eave level. The member section sizes, grades and rafters spacings are shown on the drawings in appendix A.

Floors

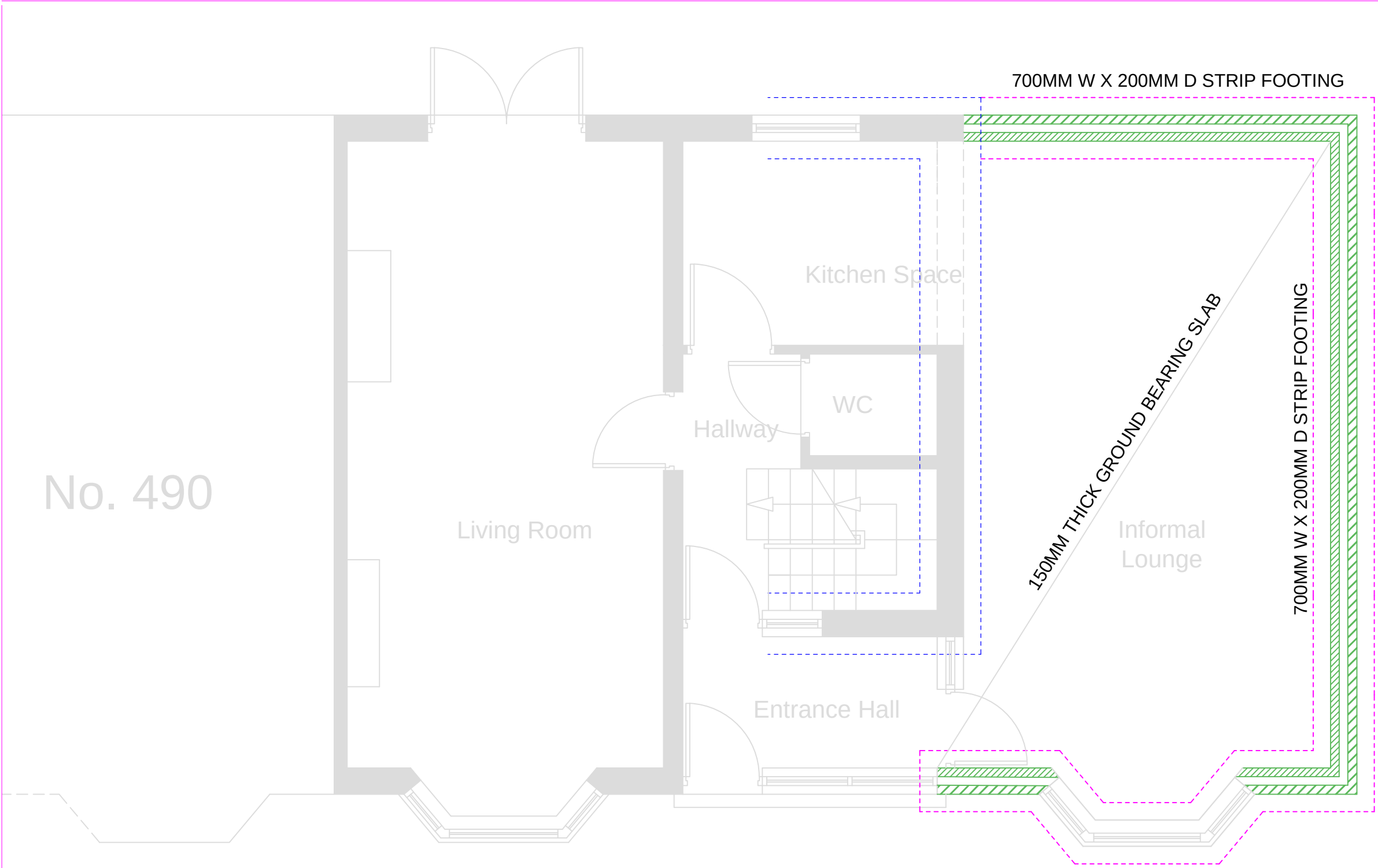
The first floor is to be timber joist spanning between masonry walls. The section properties of these members and the spacing of the floor joists are shown on the drawings in Appendix A.

Walls

The extension perimeter walls are a cavity wall of 100mm thick Inner leaf blockwork and 102.5mm thick outer leaf brickwork.

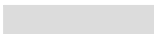
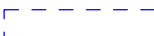
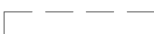
APPENDIX A

- **DRAWINGS**



GA FOUNDATION PLAN
(Scale:1:50)

KEY

-  NEW LOAD BEARING BLOCKWORK
-  NEW LOAD BEARING BRICKWORK
-  EXISTING WALL
-  EXISTING FOUNDATION
-  NEW FOUNDATION
-  DEMOLISHED

NOTES

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6. This drawing is based on the Architectural Plans. Structural Engineer to be informed of any changes to the Architectural drawings.
7. All measurements in mm unless noted otherwise.

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C1	23.03.20	Construction issue	MY

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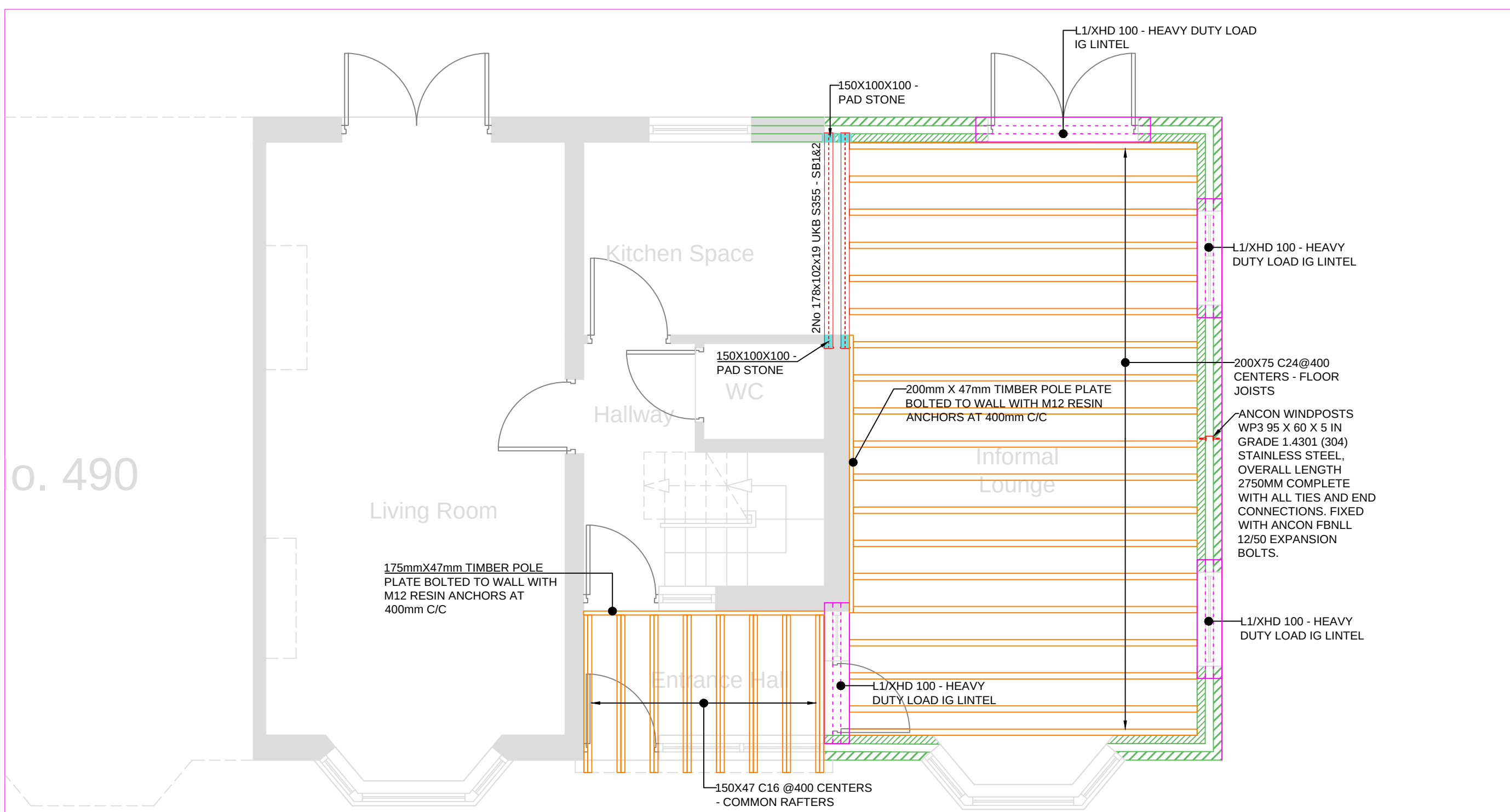
Kirkmanshulme Lane, Longsight, Manchester
M12 4TP, Tel: 07498583199.
Email: Contact@maystructuralengineers.co.uk

PROJECT:
488 Bury Old Rd,
Prestwich, Manchester,
M25 1NL

TITLE:
Foundation Plan

SCALE @ A3:	DRAWN:	DATE:
As shown	MY	22.03.20

JOB No:	DRAWING No:	REVISION:
10025	SE 100	C1



GA GROUND FLOOR PLAN SHOWING FIRST FLOOR STRUCTURES
(Scale:1:50)

KEY

- NEW LOAD BEARING BLOCKWORK
- NEW LOAD BEARING BRICKWORK
- EXISTING WALL
- EXISTING FOUNDATION
- NEW FOUNDATION
- DEMOLISHED

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M12 4TP, Tel: 07498583199.
Email: Contact@maystructuralengineers.co.uk

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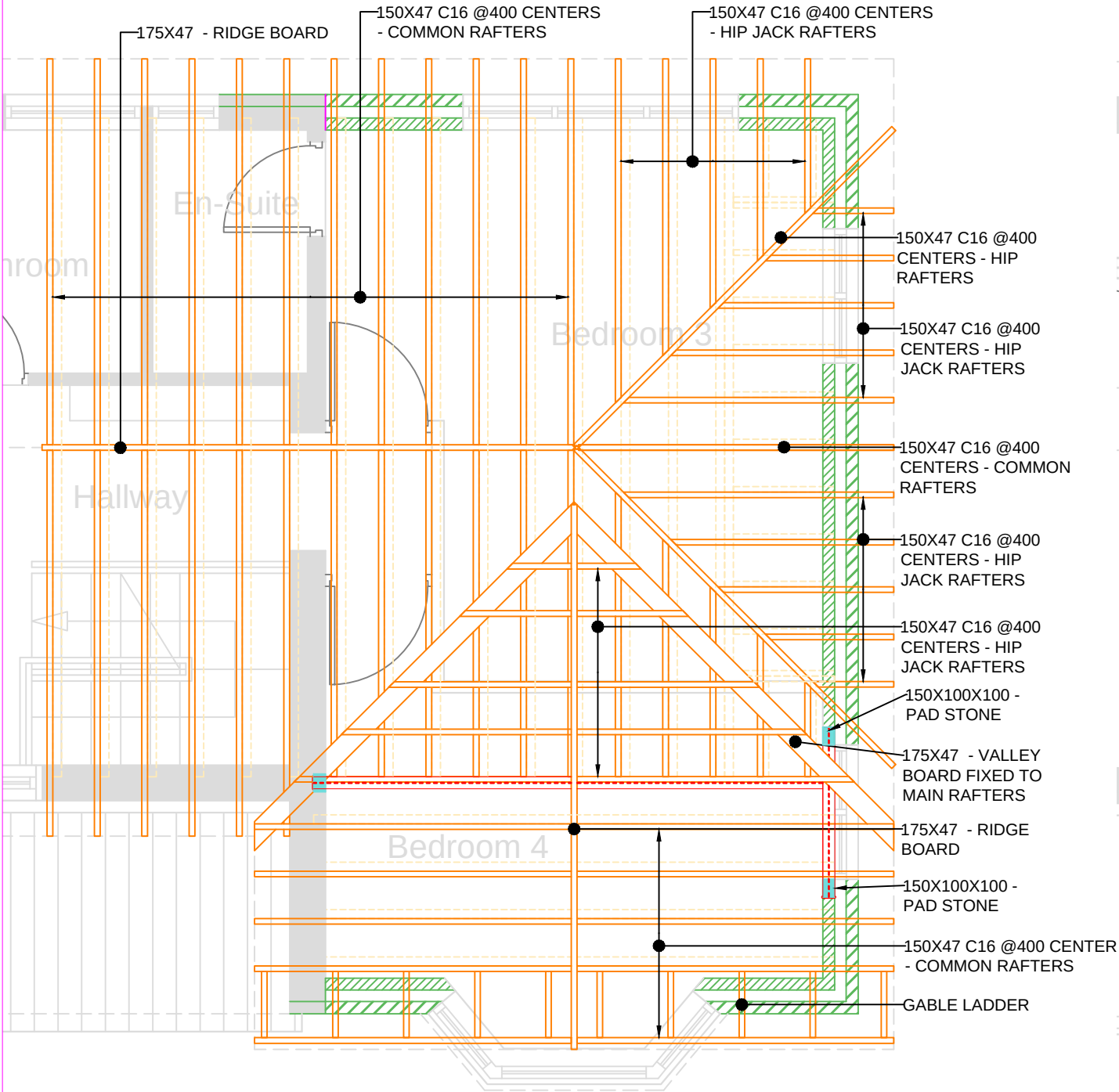
488 Bury Old Rd,
Prestwich, Manchester,
M25 1NL

TITLE:

GROUND FLOOR PLAN

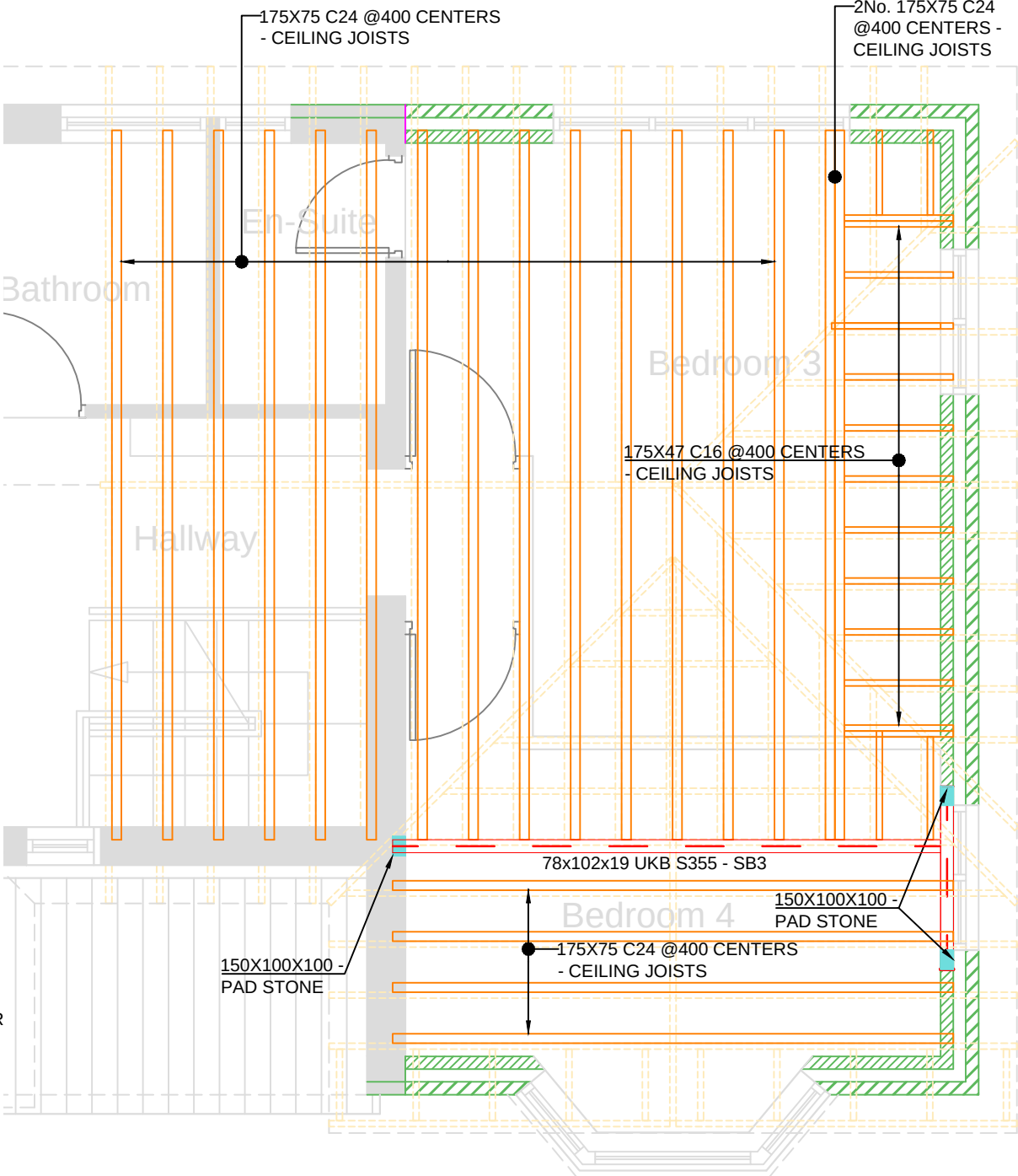
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JOB No:	DRAWING No:	REVISION:
10025	SE101	C1

Construction



GA FIRST FLOOR PLAN SHOWING ROOF STRUCTURES
(Scale:1:50)

KEY	
	NEW LOAD BEARING BLOCKWORK
	NEW LOAD BEARING BRICKWORK
	EXISTING WALL
	EXISTING FOUNDATION
	NEW FOUNDATION
	DEMOLISHED



GA FIRST FLOOR PLAN SHOWING ROOF CEILING JOISTS STRUCTURES
(Scale:1:50)

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MAY
Structural Engineers

Kirkmanshulme Lane, Longsight, Manchester
M12 4TP, Tel: 07498583199.
Email: Contact@maystructuralengineers.co.uk

PROJECT:
488 Bury Old Rd,
Prestwich, Manchester,
M25 1NL

TITLE:
FIRST FLOOR PLAN

SCALE @ A3:
As shown

DRAWN:
MY

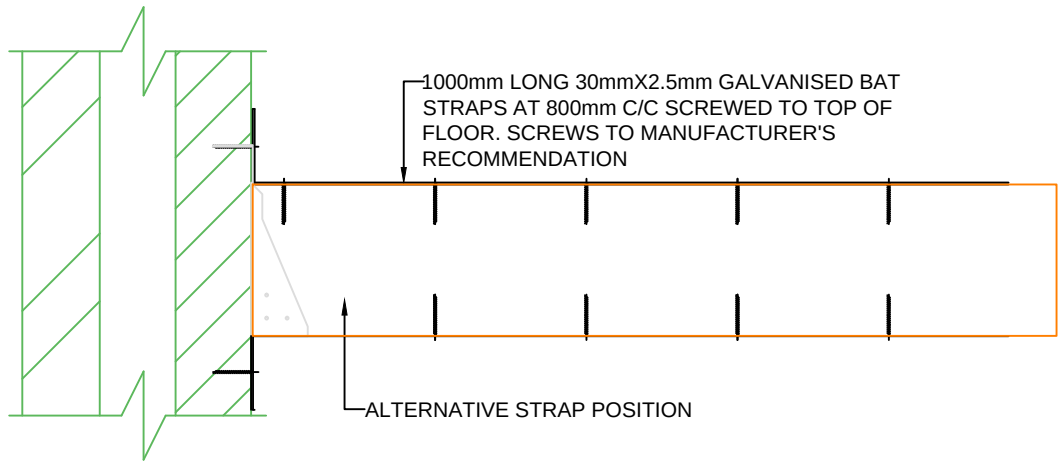
DATE:
22.03.20

JOB No:
10025

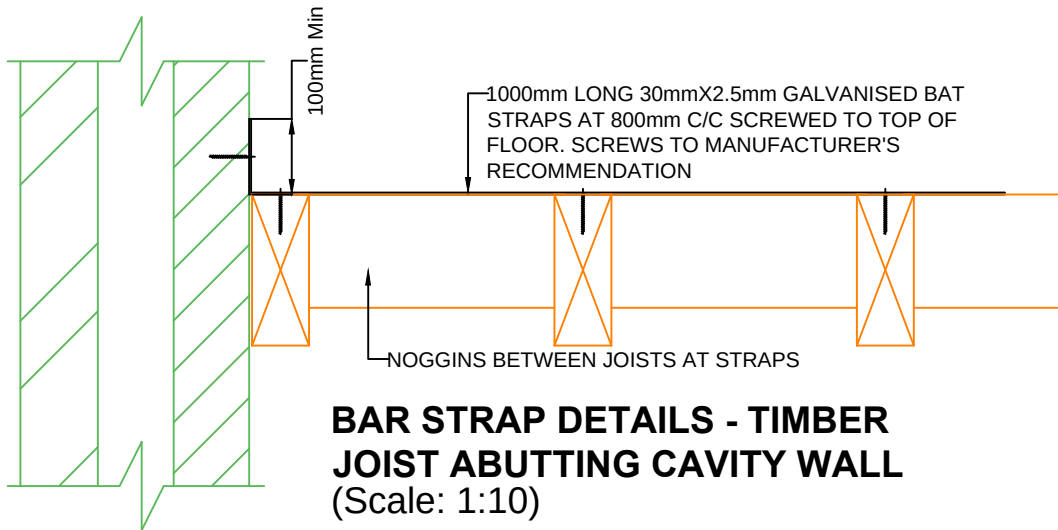
DRAWING No:
SE102

REVISION:
C1

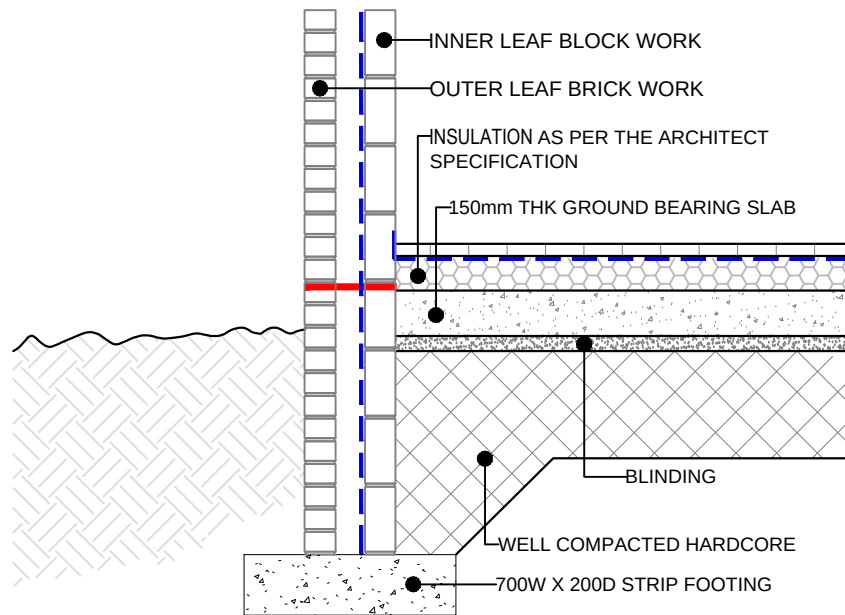
Construction



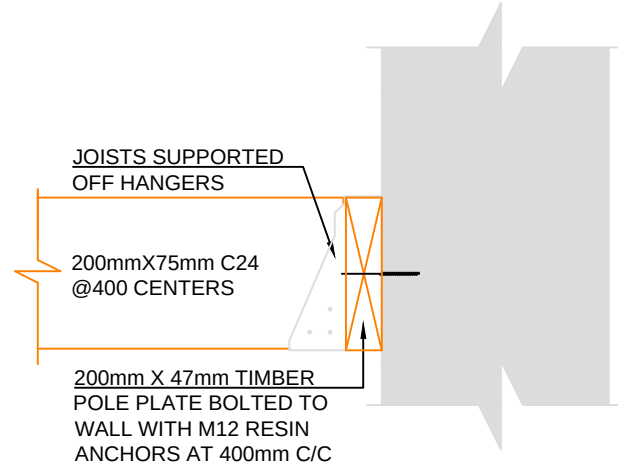
BAR STRAP DETAILS - TIMBER JOIST PERPENDICULAR TO CAVITY WALL
(Scale: 1:10)



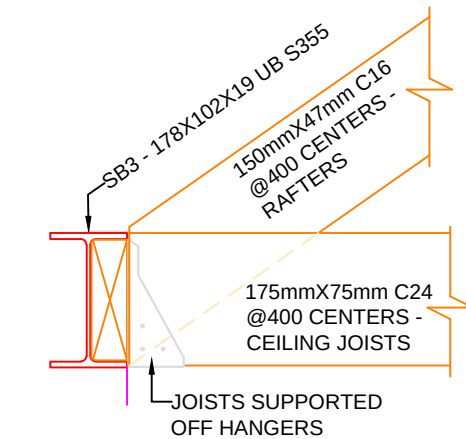
BAR STRAP DETAILS - TIMBER JOIST ABUTTING CAVITY WALL
(Scale: 1:10)



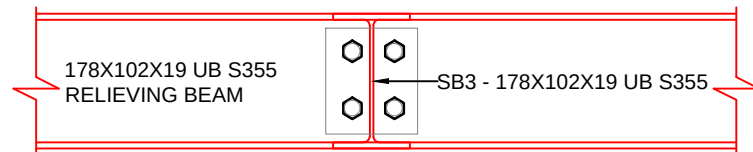
TYPICAL FOUNDATION DETAILS
Scale:1:25



POLE PLATE DETAILS
(Scale: 1:10)

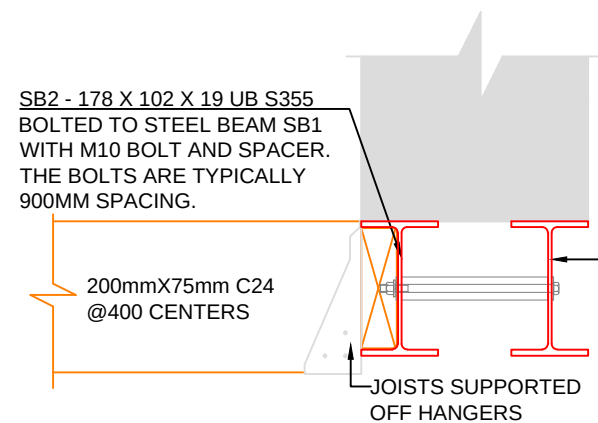


STEEL BEAM SUPPORTING CEILING JOISTS AND RAFTERS
(Scale: 1:10)



STEEL BEAM TO BEAM CONNECTIONS
(Scale: 1:10)

140X120X10 S355 END PLATE
4NO M16, BOLT GRADE 8.8
6MM CONTINUOUS FILLET WELD



TWIN BEAMS SHOWING JOISTS PERPENDICULAR TO WALL
(Scale: 1:10)

NOTE:

STEELWORK

- ALL STEELWORK ARE TO BE GRADE S355
- ALL STEELWORK ARE TO BE PROTECTED FROM FIRE TO 30-MINUTE STANDARD OR AS SPECIFIED ELSEWHERE TO THE SATISFACTION OF THE BUILDING CONTROL.
- EXTERNAL STEELWORK TO BE GALVANISED TO BS EN ISO 1461, TO RECEIVE A MIN MEAN COATING THICKNESS OF 140 MICROMETERS.
- ALL SITE DRILLED HOLES TO BE MADE GOOD TO EQUIVALENT OF ORIGINAL FINISHES SPECIFICATION.
- ALL BOLTED CONNECTIONS ARE TO INCLUDE A MINIMUM OF TWO M16 GRADE 8.8 BOLTS PER MEMBER UNLESS SPECIFICALLY INDICATED OTHERWISE ON DETAILS.
- ALL BOLTS ARE TO BE GRADE 8.8 GALVANISED TO BS 7371-6.
- ALL CONNECTIONS TO BE WELDED, UNLESS NOTED OTHERWISE.
- ALL WELDS TO BE MINIMUM 6mm CONTINUOUS FILLET WELDS UNLESS SPECIFICALLY NOTED OTHERWISE.
- WELDS TO BE OF GOOD VISUAL QUALITY.
- SITE WELDING IS NOT PERMITTED.

PAD STONES

- PAD STONES TO BE GRADE C30 CONCRETE MINIMUM, AT CONTRACTOR'S OPTION AND WITH THE APPROVAL OF THE BUILDING CONTROL OFFICER, PAD STONES MAY BE CONSTRUCTED FROM 35N/MM² ENGINEERING BRICKS.
- STEEL BEAM BEARINGS ON PAD STONES TO BE THE FULL WIDTH OF PAD STONE, MINIMUM 150MM UNLESS NOTED OTHERWISE.

JOIST HANGERS

- ONLY PROPRIETARY JOIST HANGERS MUST BE USED COMPLYING WITH BS EN845-1 AND THESE MUST BE INSTALLED IN STRICT ACCORDANCE WITH THE MANUFACTURER'S RECOMMENDATIONS.

STRAPS

- FIX STRAPS TO NOGGINS OR RAFTERS WITH 3.4 X 35MM SQUARE TWIST NAILS, EVENLY SPACED ALONG THE LENGTH OF THE STRAP. THE 90DEG BEND OF THE STRAP IS TO BE HELD TIGHT AGAINST THE CAVITY FACE OF THE INNER LEAF OF BLOCK WORK. 30 X 5MM GALVANISED MILD STEEL STRAP EXTENDING ACROSS MIN 3 RAFTERS AND HELD TIGHT AGAINST MASONRY WALL. STRAPS AT 2M CENTERS TO ALL EXPOSED GABLE WALLS.

MASONRY

- BRICKS WITH WATER ABSORPTION FROM 7% TO 12%
- CLAY BRICKS UNIT STRENGTH 20N/MM²
- CONCRETE BLOCKS STRENGTH 7N/MM²
- UNITS CATEGORY II
- MORTAR DESIGNATION M4/(III)

CONCRETE

- ALL FOUNDATIONS TO BE A MINIMUM OF 1000MM BELOW GROUND LEVEL, EXACT DEPTH TO BE AGREED ON SITE WITH BUILDING CONTROL OFFICER TO SUIT SITE CONDITIONS.
- FOOTINGS NEAR BOUNDARIES SHOULD NOT BE LOCATED HIGHER OR LOWER THAN EXISTING FOUNDATIONS UNLESS APPROVED.
- FOUNDATION CONCRETE STRENGTH IS TO BE C30/37

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Rev	Date	Description	By
C1	23.03.20	Construction issue	MY

MAY
Structural Engineers

Kirkmanshulme Lane, Longsight, Manchester
M12 4TP, Tel: 07498583199.
Email: Contact@maystructuralengineers.co.uk

PROJECT:

488 Bury Old Rd,
Prestwich, Manchester,
M25 1NL

TITLE:

DETAILS AND GENERAL
NOTE

SCALE @ A3:	DRAWN:	DATE:
As shown	MY	22.03.20

JOB No:	DRAWING No:	REVISION:
10025	SE 200	C1

Construction

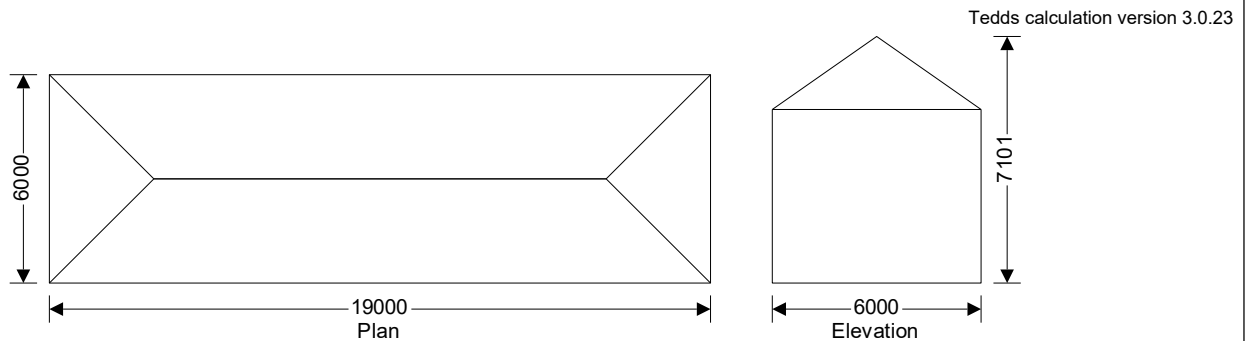
APPENDIX B

- **WIND AND SNOW LOADING**

Project 488 BURY OLD RD, MANCHESTER, M25 1NL				Job no. 10025	
Calcs for WIND				Start page no./Revision 1	
Calcs by M	Calcs date 17/03/2020	Checked by	Checked date	Approved by	Approved date

WIND LOADING

In accordance with EN1991-1-4:2005+A1:2010 and the UK national annex



Building data

Type of roof	Hipped
Length of building	L = 19000 mm
Width of building	W = 6000 mm
Height to eaves	H = 5000 mm
Pitch of main slope	$\alpha_0 = 35.0$ deg
Pitch of gable slope	$\alpha_{90} = 35.0$ deg
Total height	h = 7101 mm

Basic values

Location	Manchester
Wind speed velocity (FigureNA.1)	$V_{b,map} = 22.5$ m/s
Distance to shore	$L_{shore} = 49.00$ km
Altitude above sea level	$A_{alt} = 110.0$ m
Altitude factor	$C_{alt} = A_{alt}/1m \times 0.001 + 1 = 1.110$
Fundamental basic wind velocity	$V_{b,0} = V_{b,map} \times C_{alt} = 25.0$ m/s
Direction factor	$C_{dir} = 0.73$
Season factor	$C_{season} = 1.00$
Shape parameter K	K = 0.2
Exponent n	n = 0.5
Air density	$\rho = 1.226$ kg/m ³
Probability factor	$C_{prob} = [(1 - K \times \ln(-\ln(1-p)))/(-K \times \ln(-\ln(0.98)))]^n = 1.00$
Basic wind velocity (Exp. 4.1)	$V_b = C_{dir} \times C_{season} \times V_{b,0} \times C_{prob} = 18.2$ m/s
Reference mean velocity pressure	$q_b = 0.5 \times \rho \times V_b^2 = 0.204$ kN/m ²

Orography

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

The velocity pressure for the windward face of the building with a 90 degree wind is to be considered as 1 part as the height h is less than b (cl.7.2.2)

Peak velocity pressure - windward wall - Wind 0 deg

Reference height (at which q is sought)	z = 5000 mm
---	-------------

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Displacement height (sheltering effects excluded) $h_{dis} = 0$ mm
 Exposure factor (Figure NA.7) $C_e = 1.94$
 Exposure correction factor (Figure NA.8) $C_{e,T} = 0.82$
 Peak velocity pressure $q_p = C_e \times C_{e,T} \times q_b = 0.33$ kN/m²

Structural factor

Structural damping $\delta_s = 0.100$
 Height of element $h_{part} = 5000$ mm
 Size factor (Table NA.3) $C_s = 0.828$
 Dynamic factor (Figure NA.9) $C_d = 1.009$
 Structural factor $C_s C_d = C_s \times C_d = 0.835$

Peak velocity pressure - windward wall - Wind 90 deg

Reference height (at which q is sought) $z = 5000$ mm
 Displacement height (sheltering effects excluded) $h_{dis} = 0$ mm
 Exposure factor (Figure NA.7) $C_e = 1.94$
 Exposure correction factor (Figure NA.8) $C_{e,T} = 0.82$
 Peak velocity pressure $q_p = C_e \times C_{e,T} \times q_b = 0.33$ kN/m²

Structural factor

Structural damping $\delta_s = 0.100$
 Height of element $h_{part} = 5000$ mm
 Size factor (Table NA.3) $C_s = 0.876$
 Dynamic factor (Figure NA.9) $C_d = 1.038$
 Structural factor $C_s C_d = C_s \times C_d = 0.909$

Peak velocity pressure - roof

Reference height (at which q is sought) $z = 7101$ mm
 Displacement height (sheltering effects excluded) $h_{dis} = 0$ mm
 Exposure factor (Figure NA.7) $C_e = 2.16$
 Exposure correction factor (Figure NA.8) $C_{e,T} = 0.87$
 Peak velocity pressure $q_p = C_e \times C_{e,T} \times q_b = 0.38$ kN/m²

Structural factor - roof 0 deg

Structural damping $\delta_s = 0.100$
 Height of element $h_{part} = 7101$ mm
 Size factor (Table NA.3) $C_s = 0.830$
 Dynamic factor (Figure NA.9) $C_d = 1.009$
 Structural factor $C_s C_d = C_s \times C_d = 0.837$

Structural factor - roof 90 deg

Structural damping $\delta_s = 0.100$
 Height of element $h_{part} = 7101$ mm
 Size factor (Table NA.3) $C_s = 0.874$
 Dynamic factor (Figure NA.9) $C_d = 1.038$
 Structural factor $C_s C_d = C_s \times C_d = 0.907$

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) $q_{p,i} = 0.38$ kN/m²

Pressures and forces

Net pressure $p = C_s C_d \times q_p \times C_{pe} - q_{p,i} \times C_{pi}$
 Net force $F_w = p_w \times A_{ref}$

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

Zone	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (kN/m ²)	Net pressure p (kN/m ²)	Area A_{ref} (m ²)	Net force F_w (kN)
F (-ve)	-0.33	0.38	-0.18	9.85	-1.80
G (-ve)	-0.33	0.38	-0.18	20.63	-3.76
H (-ve)	-0.13	0.38	-0.12	28.12	-3.34
I (-ve)	-0.60	0.38	-0.27	38.77	-10.37
J (-ve)	-1.10	0.38	-0.43	7.94	-3.39
K (-ve)	-0.67	0.38	-0.29	11.88	-3.43
L (-ve)	-1.03	0.38	-0.41	9.17	-3.72
M (-ve)	-0.60	0.38	-0.27	12.80	-3.42

Total vertical net force $F_{w,v} = -27.21$ kN

Total horizontal net force $F_{w,h} = 4.75$ kN

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

Zone	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (kN/m ²)	Net pressure p (kN/m ²)	Area A_{ref} (m ²)	Net force F_w (kN)
A	-1.20	0.33	-0.40	14.20	-5.72
B	-0.80	0.33	-0.29	15.80	-4.64
D	0.80	0.33	0.14	95.00	13.43
E	-0.51	0.33	-0.21	95.00	-20.39

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,WE} = -20.4$ kN

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

Lack of correlation (cl.7.2.2(3) – Note) $f_{corr} = 0.86$ as h/W is 1.183

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h}) = 33.1$ kN

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

Zone	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (kN/m ²)	Net pressure p (kN/m ²)	Area A_{ref} (m ²)	Net force F_w (kN)
F (+ve)	0.80	0.38	0.37	9.85	3.64
G (+ve)	0.53	0.38	0.28	20.63	5.86
H (+ve)	0.50	0.38	0.27	28.12	7.69
I (+ve)	-0.60	0.38	-0.08	38.77	-2.99
J (+ve)	-1.10	0.38	-0.24	7.94	-1.88
K (+ve)	-0.67	0.38	-0.10	11.88	-1.17
L (+ve)	-1.03	0.38	-0.22	9.17	-1.97
M (+ve)	-0.60	0.38	-0.08	12.80	-0.99

Total vertical net force $F_{w,v} = 6.72$ kN

Total horizontal net force $F_{w,h} = 13.32$ kN

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

Zone	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (kN/m ²)	Net pressure p (kN/m ²)	Area A_{ref} (m ²)	Net force F_w (kN)
A	-1.20	0.33	-0.21	14.20	-3.01
B	-0.80	0.33	-0.10	15.80	-1.63
D	0.80	0.33	0.33	95.00	31.52
E	-0.51	0.33	-0.02	95.00	-2.30

Overall loading

Equiv leeward net force for overall section

$$F_l = F_{w,WE} = -2.3 \text{ kN}$$

Net windward force for overall section

$$F_w = F_{w,WD} = 31.5 \text{ kN}$$

Lack of correlation (cl.7.2.2(3) – Note)

$$f_{corr} = 0.86 \text{ as } h/W \text{ is } 1.183$$

Overall loading overall section

$$F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h}) = 40.4 \text{ kN}$$

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

Zone	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (kN/m ²)	Net pressure p (kN/m ²)	Area A_{ref} (m ²)	Net force F_w (kN)
F (-ve)	-0.33	0.38	-0.19	1.76	-0.34
G (-ve)	-0.33	0.38	-0.19	2.20	-0.42
H (-ve)	-0.13	0.38	-0.12	7.03	-0.86
I (-ve)	-0.60	0.38	-0.28	7.03	-1.99
J (-ve)	-1.10	0.38	-0.46	3.96	-1.80
L (-ve)	-1.03	0.38	-0.43	4.39	-1.90
M (-ve)	-0.60	0.38	-0.28	7.03	-1.99
N (-ve)	-0.47	0.38	-0.24	105.77	-25.10

Total vertical net force

$$F_{w,v} = -28.18 \text{ kN}$$

Total horizontal net force

$$F_{w,h} = 1.24 \text{ kN}$$

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

Zone	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (kN/m ²)	Net pressure p (kN/m ²)	Area A_{ref} (m ²)	Net force F_w (kN)
A	-1.20	0.33	-0.43	6.00	-2.59
B	-0.80	0.33	-0.31	24.00	-7.51
C	-0.50	0.33	-0.22	65.00	-14.57
D	0.72	0.33	0.14	30.00	4.08
E	-0.33	0.33	-0.17	30.00	-5.24

Overall loading

Equiv leeward net force for overall section

$$F_l = F_{w,WE} = -5.2 \text{ kN}$$

Net windward force for overall section

$$F_w = F_{w,WD} = 4.1 \text{ kN}$$

Lack of correlation (cl.7.2.2(3) – Note)

$$f_{corr} = 0.85 \text{ as } h/L \text{ is } 0.374$$

Overall loading overall section

$$F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h}) = 9.0 \text{ kN}$$

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

Zone	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (kN/m ²)	Net pressure p (kN/m ²)	Area A_{ref} (m ²)	Net force F_w (kN)
F (+ve)	0.80	0.38	0.39	1.76	0.69
G (+ve)	0.53	0.38	0.30	2.20	0.66
H (+ve)	0.50	0.38	0.29	7.03	2.02
I (+ve)	-0.60	0.38	-0.09	7.03	-0.65
J (+ve)	-1.10	0.38	-0.27	3.96	-1.05
L (+ve)	-1.03	0.38	-0.24	4.39	-1.07
M (+ve)	-0.60	0.38	-0.09	7.03	-0.65
N (+ve)	-0.47	0.38	-0.05	105.77	-4.96

Total vertical net force $F_{w,v} = -4.12$ kN

Total horizontal net force $F_{w,h} = 2.91$ kN

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

Zone	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (kN/m ²)	Net pressure p (kN/m ²)	Area A_{ref} (m ²)	Net force F_w (kN)
A	-1.20	0.33	-0.24	6.00	-1.45
B	-0.80	0.33	-0.12	24.00	-2.94
C	-0.50	0.33	-0.03	65.00	-2.20
D	0.72	0.33	0.33	30.00	9.79
E	-0.33	0.33	0.02	30.00	0.47

Overall loading

Equiv leeward net force for overall section

$$F_l = F_{w,WE} = 0.5 \text{ kN}$$

Net windward force for overall section

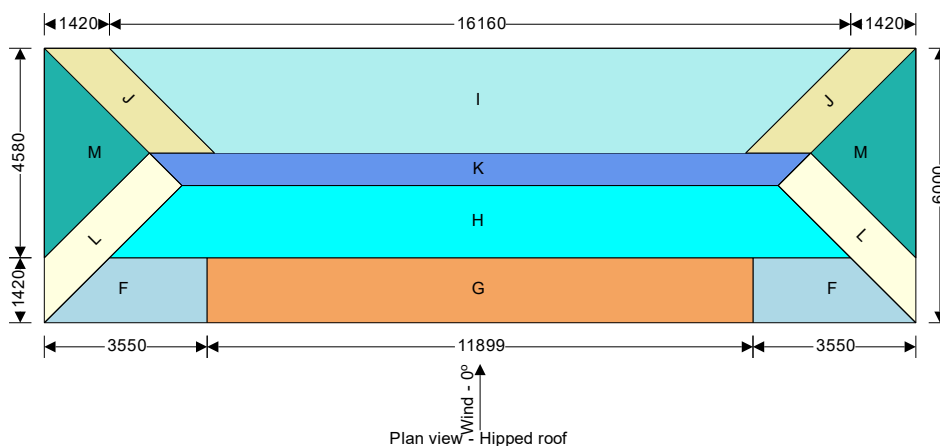
$$F_w = F_{w,wD} = 9.8 \text{ kN}$$

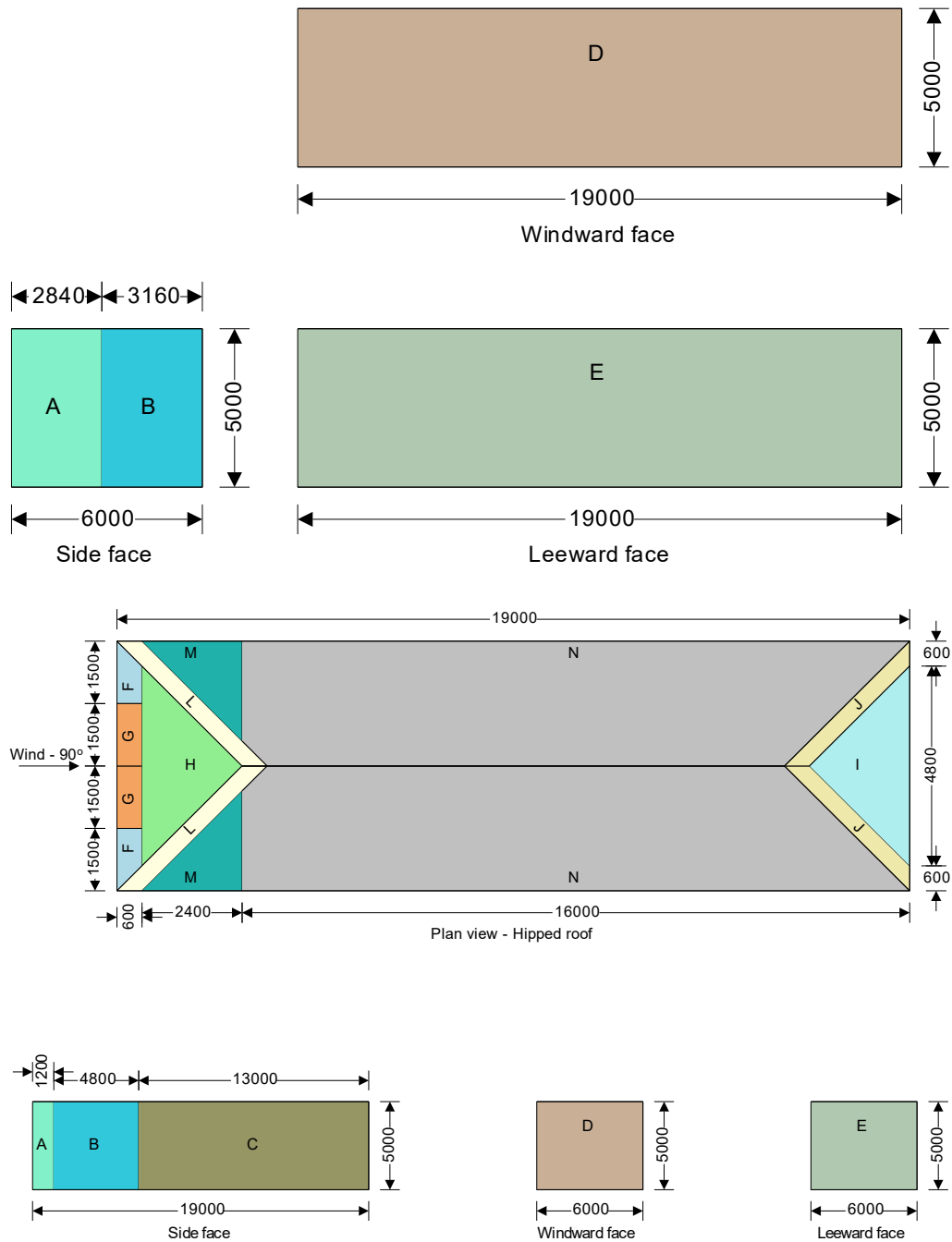
Lack of correlation (cl.7.2.2(3) – Note)

$$f_{corr} = 0.85 \text{ as } h/L \text{ is } 0.374$$

Overall loading overall section

$$F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h}) = 10.4 \text{ kN}$$





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SNOW LOADING

In accordance with EN1991-1-3:2003+A1:2015 incorporating corrigenda dated December 2004 and March 2009 and the UK national annex NA+A1:2015 to BS EN 1991-1-3:2003+A1:2015 incorporating Corrigendum No.1

Tedds calculation version 1.0.10

Characteristic ground snow load

Location	Manchester
Site altitude above sea level (user modified value)	A = 110 m
Zone number	Z = 3.0
Density of snow	$\gamma = 2.00 \text{ kN/m}^3$
Characteristic ground snow load	$s_k = ((0.15 + (0.1 \times Z + 0.05)) + ((A - 100\text{m}) / 525\text{m})) \times 1\text{kN/m}^2 = 0.52 \text{ kN/m}^2$
Exposure coefficient (Normal)	$C_e = 1.0$
Thermal coefficient	$C_t = 1.0$
Snow fence	Not present

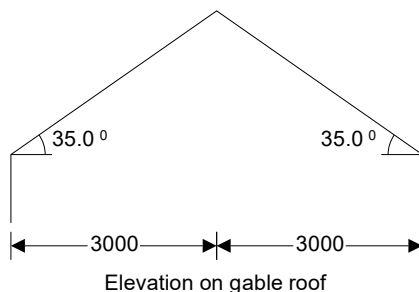
Building details

Roof type	Duopitch
Width of roof (left on elevation)	$b_1 = 3.00 \text{ m}$
Width of roof (right on elevation)	$b_2 = 3.00 \text{ m}$
Slope of roof (left on elevation)	$\alpha_1 = 35.00 \text{ deg}$
Slope of roof (right on elevation)	$\alpha_2 = 35.00 \text{ deg}$

Shape coefficients

Shape coefficient roof (Table 5.2)	$\mu_{2_a1_T52} = 0.67$
Shape coefficient roof (Table 5.2)	$\mu_{2_a2_T52} = 0.67$
Shape coefficient roof (Table UK NA.2)	$\mu_{1_a1_NA2} = 1.00$
Shape coefficient roof (Table UK NA.2)	$\mu_{1_a2_NA2} = 1.00$

Case (i)	$\mu_{2_a1_T52}$	$\mu_{2_a2_T52}$	Shape coef	Coef	Loading (kN/m ²)
			$\mu_{2_a1_T52}$	0.667	0.35
Case (ii)		$\mu_{1_a2_NA2}$	$\mu_{2_a2_T52}$	0.667	0.35
			$\mu_{1_a1_NA2}$	1.000	0.52
Case (iii)	$\mu_{1_a1_NA2}$		$\mu_{1_a2_NA2}$	1.000	0.52



Loadcase 1 Table 5.2

Loading to roof 1 (LHS)	$s_{1_1} = \mu_{2_a1_T52} \times C_e \times C_t \times s_k = 0.35 \text{ kN/m}^2$
Loading to roof 2 (RHS)	$s_{2_1} = \mu_{2_a2_T52} \times C_e \times C_t \times s_k = 0.35 \text{ kN/m}^2$



MAY Structural Engineers
151 Kirkmanchulme Lane
Manchester
M12 4TP

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Loadcase 2 UK NA.2

Loading to roof 1 (LHS)

$$s_{1_2} = 0 \times C_e \times C_t \times s_k = \mathbf{0.00 \text{ kN/m}^2}$$

Loading to roof 2 (RHS)

$$s_{2_2} = \mu_{1_2_NA2} \times C_e \times C_t \times s_k = \mathbf{0.52 \text{ kN/m}^2}$$

Loadcase 3 UK NA.2

Loading to roof 1 (LHS)

$$s_{1_3} = \mu_{1_1_NA2} \times C_e \times C_t \times s_k = \mathbf{0.52 \text{ kN/m}^2}$$

Loading to roof 2 (RHS)

$$s_{2_3} = 0 \times C_e \times C_t \times s_k = \mathbf{0.00 \text{ kN/m}^2}$$

APPENDIX C

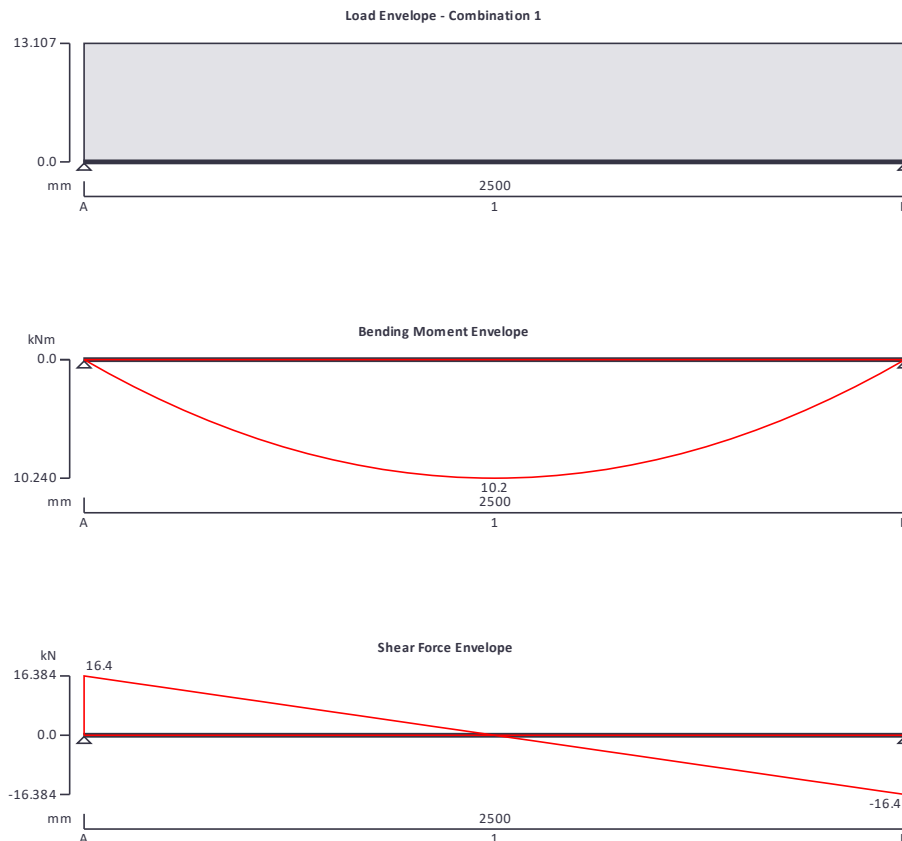
- **STEEL MEMBERS**

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STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Masonry inner leaf - Permanent full UDL 5.28 kN/m

first floor - Permanent full UDL 1.02 kN/m

first floor - Variable full UDL 2.9 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

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Variable $\times 1.50$

Analysis results

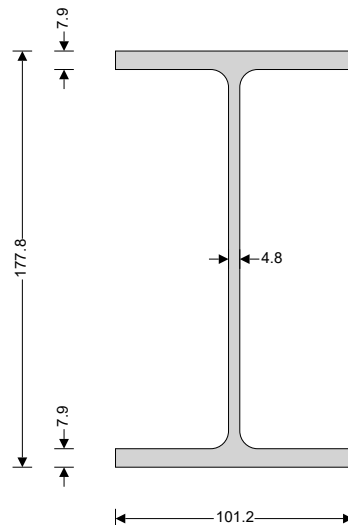
Maximum moment	$M_{\max} = 10.2 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 16.4 \text{ kN}$	$V_{\min} = -16.4 \text{ kN}$
Deflection	$\delta_{\max} = 0.5 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 16.4 \text{ kN}$	$R_{A_{\min}} = 16.4 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{\text{Permanent}}} = 8.1 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_{\text{Variable}}} = 3.6 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 16.4 \text{ kN}$	$R_{B_{\min}} = 16.4 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{\text{Permanent}}} = 8.1 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_{\text{Variable}}} = 3.6 \text{ kN}$	

Section details

Section type **UKB 178x102x19 (Tata Steel Advance)**
Steel grade **S355**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	$t = \max(t_f, t_w) = 7.9 \text{ mm}$
Nominal yield strength	$f_y = 355 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 470 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT,A} = 1.000$
	$K_{LT,B} = 1.000$

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Classification of cross sections - Section 5.5

$$\varepsilon = \sqrt{235 \text{ N/mm}^2 / f_y} = 0.81$$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section

$$c = d = 146.8 \text{ mm}$$

$$c / t_w = 37.6 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = 40.6 \text{ mm}$$

$$c / t_f = 6.3 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 162 \text{ mm}$$

Shear area factor

$$\eta = 1.000$$

$$h_w / t_w < 72 \times \varepsilon / \eta$$

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 16.4 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 985 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 201.9 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment

$$M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 10.2 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 60.8 \text{ kNm}$$

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 6.9 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 0.518 \text{ mm}$$

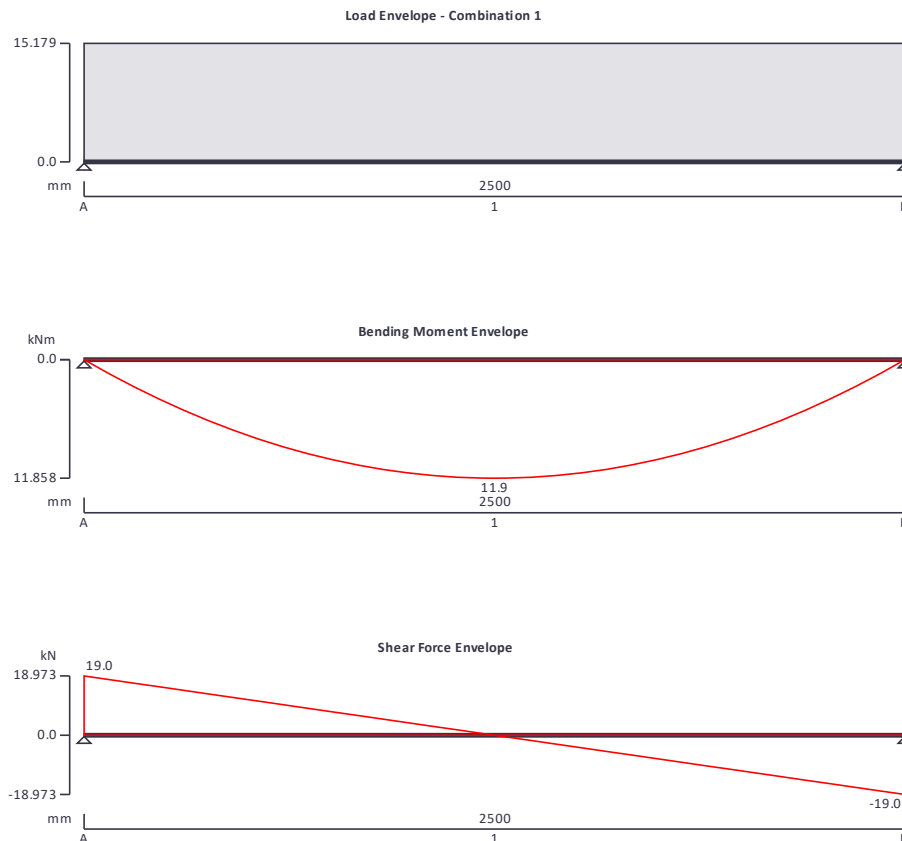
PASS - Maximum deflection does not exceed deflection limit

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STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Masonry inner leaf - Permanent full UDL 4.92 kN/m

first floor - Permanent full UDL 1.47 kN/m

first floor - Variable full UDL 4.2 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

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Variable $\times 1.50$

Analysis results

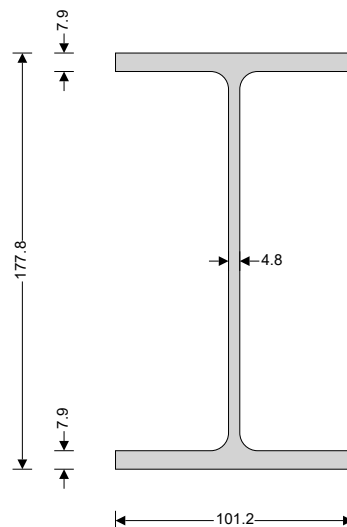
Maximum moment	$M_{\max} = 11.9 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 19 \text{ kN}$	$V_{\min} = -19 \text{ kN}$
Deflection	$\delta_{\max} = 0.8 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 19 \text{ kN}$	$R_{A_{\min}} = 19 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{\text{Permanent}}} = 8.2 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_{\text{Variable}}} = 5.3 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 19 \text{ kN}$	$R_{B_{\min}} = 19 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{\text{Permanent}}} = 8.2 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_{\text{Variable}}} = 5.3 \text{ kN}$	

Section details

Section type	UKB 178x102x19 (Tata Steel Advance)
Steel grade	S355

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	$t = \max(t_f, t_w) = 7.9 \text{ mm}$
Nominal yield strength	$f_y = 355 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 470 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT,A} = 1.000$
	$K_{LT,B} = 1.000$

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	Calcs for STEEL BEAM - SB2				Start page no./Revision 3	
	Calcs by M	Calcs date 18/03/2020	Checked by	Checked date	Approved by	Approved date

Classification of cross sections - Section 5.5

$$\varepsilon = \sqrt{235 \text{ N/mm}^2 / f_y} = 0.81$$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section

$$c = d = 146.8 \text{ mm}$$

$$c / t_w = 37.6 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = 40.6 \text{ mm}$$

$$c / t_f = 6.3 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 162 \text{ mm}$$

Shear area factor

$$\eta = 1.000$$

$$h_w / t_w < 72 \times \varepsilon / \eta$$

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 19 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 985 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 201.9 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment

$$M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 11.9 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 60.8 \text{ kNm}$$

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 6.9 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 0.75 \text{ mm}$$

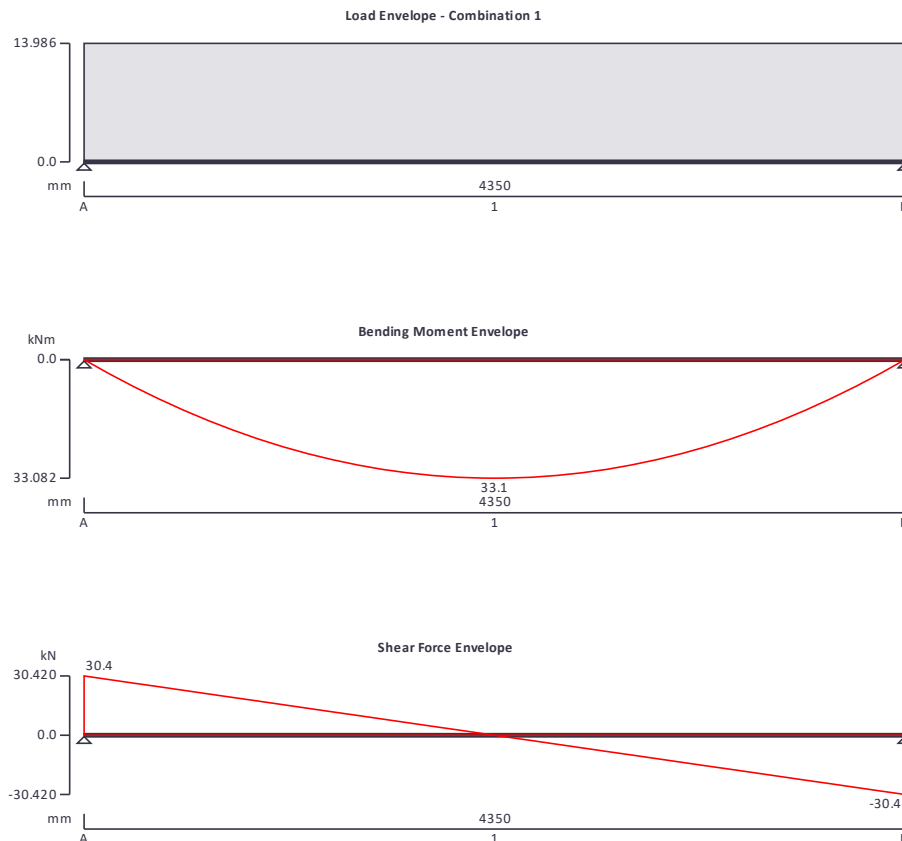
PASS - Maximum deflection does not exceed deflection limit

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Calcs for STEEL BEAM - SB3				Start page no./Revision 1	
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STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

CIELING JOISTS - Permanent full UDL 0.96 kN/m

CIELING JOISTS - Variable full UDL 0.96 kN/m

PITCHED RAFTER - Permanent full UDL 3.48 kN/m

PITCHED RAFTER - Variable full UDL 4.2 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

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Support B

Permanent $\times 1.35$

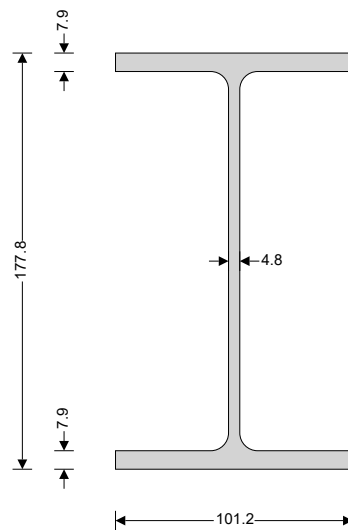
Variable $\times 1.50$

Analysis results

Maximum moment	$M_{\max} = 33.1 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 30.4 \text{ kN}$	$V_{\min} = -30.4 \text{ kN}$
Deflection	$\delta_{\max} = 8.4 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 30.4 \text{ kN}$	$R_{A_{\min}} = 30.4 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{\text{Permanent}}} = 10.1 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_{\text{Variable}}} = 11.2 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 30.4 \text{ kN}$	$R_{B_{\min}} = 30.4 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{\text{Permanent}}} = 10.1 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_{\text{Variable}}} = 11.2 \text{ kN}$	

Section details

Section type	UKB 178x102x19 (Tata Steel Advance)
Steel grade	S355
EN 10025-2:2004 - Hot rolled products of structural steels	
Nominal thickness of element	$t = \max(t_f, t_w) = 7.9 \text{ mm}$
Nominal yield strength	$f_y = 355 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 470 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LTA} = 1.000$
	$K_{LTB} = 1.000$

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Classification of cross sections - Section 5.5

$$\varepsilon = \sqrt{235 \text{ N/mm}^2 / f_y} = 0.81$$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section

$$c = d = 146.8 \text{ mm}$$

$$c / t_w = 37.6 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = 40.6 \text{ mm}$$

$$c / t_f = 6.3 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 162 \text{ mm}$$

Shear area factor

$$\eta = 1.000$$

$$h_w / t_w < 72 \times \varepsilon / \eta$$

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 30.4 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 985 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 201.9 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment

$$M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 33.1 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 60.8 \text{ kNm}$$

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 12.1 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 8.448 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

APPENDIX D

- **TIMBER MEMBERS**

Project 488 BURY OLD RD, MANCHESTER, M25 1NL				Job no. 10025	
Calcs for COMMON RAFTERS				Start page no./Revision 1	
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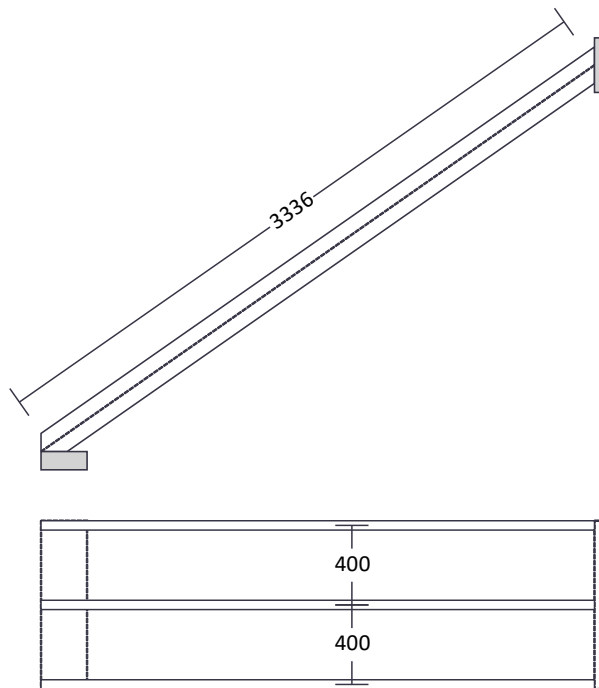
TIMBER RAFTER ANALYSIS & DESIGN (EN1995-1-1:2004)

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 2.2.00

Rafter details

Description	47 x 150 C16 timber rafters
Rafter spacing	$S_{\text{Rafter}} = 400 \text{ mm}$
Rafter inclination	$\theta_{\text{Rafter}} = 35 \text{ deg}$



Forces input on Rafter

Permanent load on slope	$F_{G_Rafter} = 0.70 \text{ kN/m}^2$
Imposed load on plan	$F_{Q_Rafter} = 0.60 \text{ kN/m}^2$
Snow load on plan	$F_{S_Rafter} = 0.52 \text{ kN/m}^2$

Rafter loading details

Distributed loads

Permanent load on slope	$p_G = F_{G_Rafter} \times S_{\text{Rafter}} = 0.28 \text{ kN/m}$
Imposed load on slope	$p_Q = F_{Q_Rafter} \times S_{\text{Rafter}} \times \cos(\theta_{\text{Rafter}}) = 0.20 \text{ kN/m}$
Snow load on slope	$p_S = F_{S_Rafter} \times S_{\text{Rafter}} \times \cos(\theta_{\text{Rafter}}) = 0.17 \text{ kN/m}$

ANALYSIS

Tedds calculation version 1.0.28

Loading

Self weight included (Permanent x 1)

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Calcs for COMMON RAFTERS				Start page no./Revision 2	
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Load combination factors

Load combination	Permanent	Imposed	Snow	Wind
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00

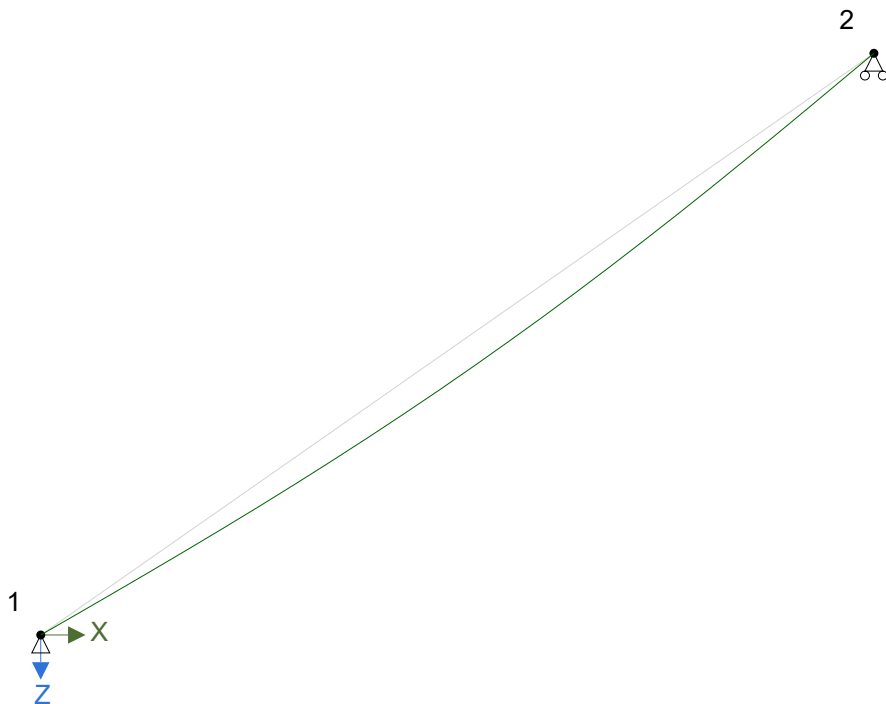
Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.28 kN/m at 0 m to 3.336 m
Member	Imposed	UDL	GlobalZ	0.2 kN/m at 0 m to 3.336 m
Member	Snow	UDL	GlobalZ	0.17 kN/m at 0 m to 3.336 m

Results

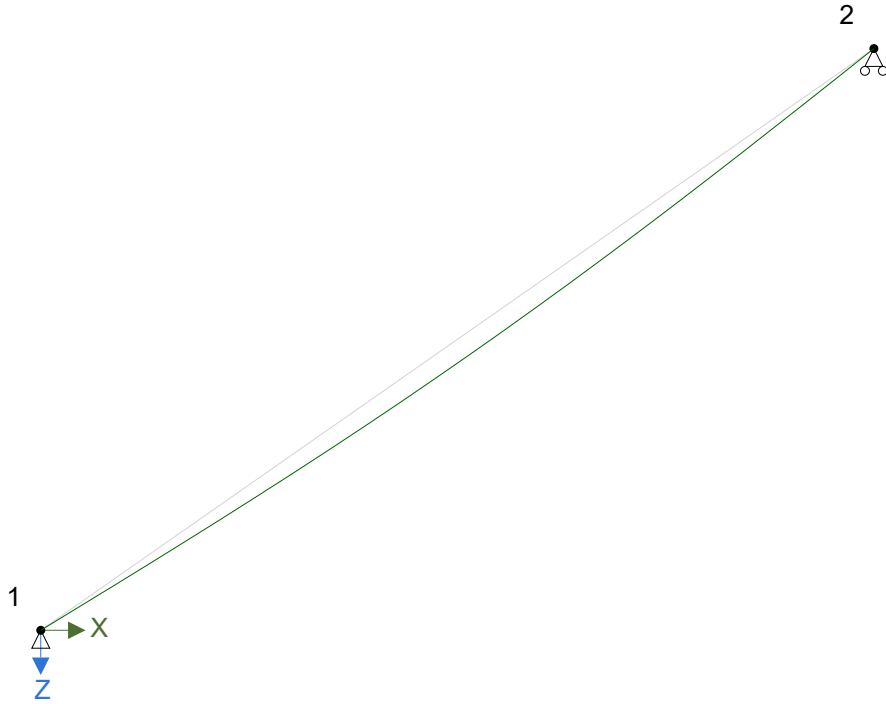
Total deflection

1.35G + 1.50Q (Strength) - Total deflection



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1.00G + 1.00Q (Service) - Total deflection



Node deflections

Load combination: 1.35G + 1.50Q (Strength)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.48184	
2	0	0	-0.48184	

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.34192	
2	0	0	-0.34192	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	2.3
1.00G + 1.00Q (Service)	0	1.7

Element end forces

Load combination: 1.35G + 1.50Q (Strength)

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Calcs for COMMON RAFTERS				Start page no./Revision 4	
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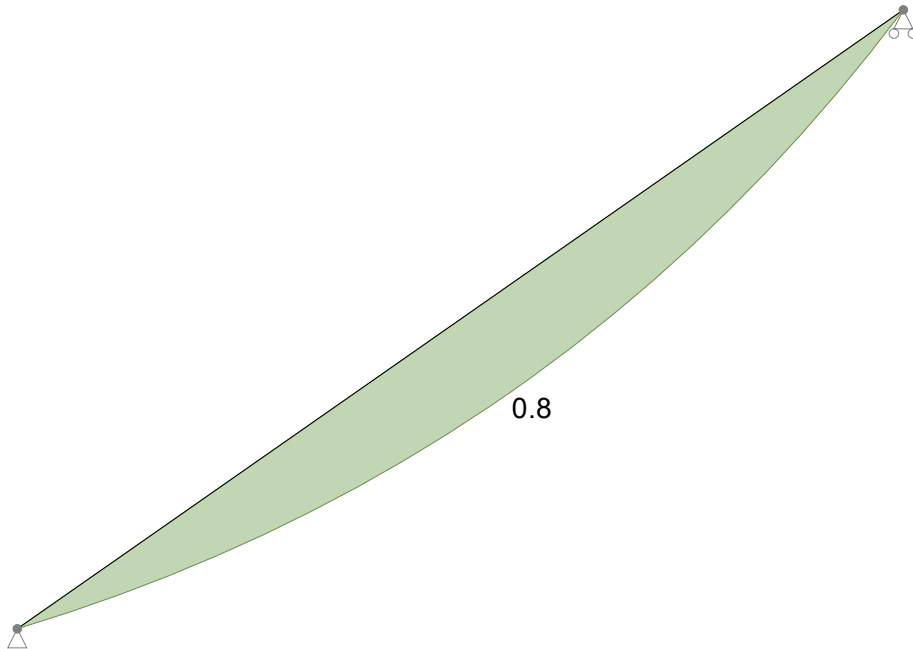
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.336	1	-0.7	-1	0
		2	-0.7	-1	0

Load combination: 1.00G + 1.00Q (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.336	1	-0.5	-0.7	0
		2	-0.5	-0.7	0

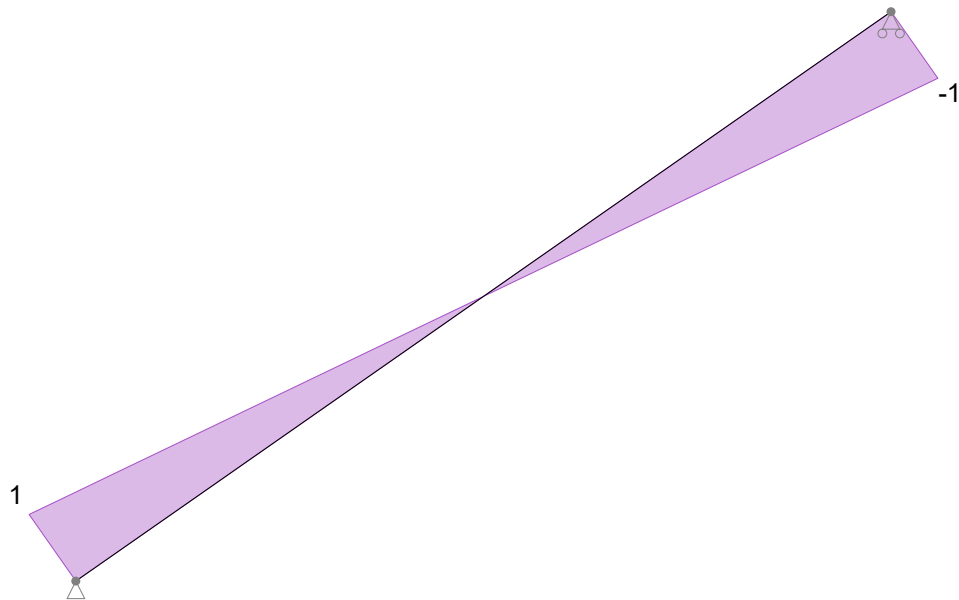
Forces

Strength combinations - Moment envelope (kNm)



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Calcs for COMMON RAFTERS				Start page no./Revision 5	
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Strength combinations - Shear envelope (kN)



Member results

Envelope - Strength combinations

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Member	0	1 (max abs)		0 (min)	
	1.668	0		0.8 (max)	
	3.336	-1		0 (min)	

Member - Span 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 $\gamma_M = 1.300$

Member details

Load duration - cl.2.3.1.2 Medium-term

Service class - cl.2.3.1.3 2

Timber section details

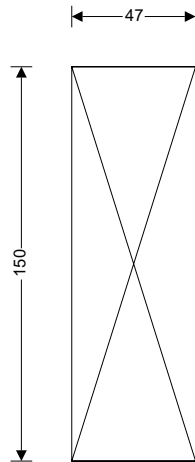
Number of timber sections in member N = 1

Breadth of sections b = 47 mm

Depth of sections h = 150 mm

Timber strength class - EN 338:2016 Table 1 C16

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	Calcs for	COMMON RAFTERS			Start page no./Revision 6	
	Calcs by M	Calcs date 18/03/2020	Checked by	Checked date	Approved by	Approved date



47x150 timber section

Cross-sectional area, A , 7050 mm²
Section modulus, W_y , 176250 mm³
Section modulus, W_z , 55225 mm³
Second moment of area, I_y , 13218750 mm⁴
Second moment of area, I_z , 1297787 mm⁴
Radius of gyration, i_y , 43.3 mm
Radius of gyration, i_z , 13.6 mm
Timber strength class C16
Characteristic bending strength, $f_{m,k}$, 16 N/mm²
Characteristic shear strength, $f_{v,k}$, 3.2 N/mm²
Characteristic compression strength parallel to grain, $f_{t,0,k}$, 17 N/mm²
Characteristic compression strength perpendicular to grain, $f_{t,90,k}$, 2.2 N/mm²
Characteristic tension strength parallel to grain, $f_{d,k}$, 8.5 N/mm²
Mean modulus of elasticity, $E_{0,mean}$, 8000 N/mm²
Fifth percentile modulus of elasticity, $E_{0.05}$, 5400 N/mm²
Shear modulus of elasticity, G_{mean} , 500 N/mm²
Characteristic density, ρ_k , 310 kg/m³
Mean density, ρ_{mean} , 370 kg/m³

Span details

Bearing length $L_b = 100$ mm

Consider Combination 1 - 1.35G + 1.50Q (Strength)

Modification factors

Duration of load and moisture content - Table 3.1 $k_{mod} = 0.8$
Deformation factor - Table 3.2 $k_{def} = 0.8$
Depth factor for tension - exp.3.1 $k_{h,t} = 1$
Bending stress re-distribution factor - cl.6.1.6(2) $k_m = 0.7$
Crack factor for shear resistance - cl.6.1.7(2) $k_{cr} = 0.67$
System strength factor - cl.6.6 $k_{sys} = 1.1$

Check compression parallel to the grain - cl.6.1.4

Design axial compression $P_d = 0.671$ kN
Design compressive stress $\sigma_{c,0,d} = P_d / A = 0.095$ N/mm²
Design compressive strength $f_{c,0,d} = k_{mod} \times k_{sys} \times f_{c,0,k} / \gamma_M = 11.508$ N/mm²
 $\sigma_{c,0,d} / f_{c,0,d} = 0.008$

PASS - Design parallel compression strength exceeds design parallel compression stress

Check design at start of span

Check shear force - Section 6.1.7

Design shear force $F_{y,d} = 0.959$ kN
Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) = 0.305$ N/mm²
Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.166$ N/mm²
 $\tau_{y,d} / f_{v,y,d} = 0.141$

PASS - Design shear strength exceeds design shear stress

Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis bending $L_{e,y} = 0.9 \times 3336$ mm = 3002 mm
Slenderness ratio $\lambda_y = L_{e,y} / i_y = 69.337$
Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = 1.238$
Effective length for z-axis bending $L_{e,z} = 0$ mm
Slenderness ratio $\lambda_z = L_{e,z} / i_z = 0$

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Relative slenderness ratio - exp. 6.22

$$\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = 0$$

$\lambda_{rel,y} > 0.3$ column stability check is required

Straightness factor

$$\beta_c = 0.2$$

Instability factors - exp.6.25, 6.26, 6.27 & 6.28

$$k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) = 1.361$$

$$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) = 0.470$$

$$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) = 0.520$$

$$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) = 1.064$$

Column stability checks - exp.6.23 & 6.24

$$\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) = 0.016$$

$$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = 0.008$$

PASS - Column stability is acceptable

Check design 1668 mm along span

Check bending moment - Section 6.1.6

Design bending moment

$$M_{y,d} = 0.8 \text{ kNm}$$

Design bending stress

$$\sigma_{m,y,d} = M_{y,d} / W_y = 4.538 \text{ N/mm}^2$$

Design bending strength

$$f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m,k} / \gamma_M = 10.831 \text{ N/mm}^2$$

$$\sigma_{m,y,d} / f_{m,y,d} = 0.419$$

PASS - Design bending strength exceeds design bending stress

Check beams subjected to either bending or combined bending and compression - cl.6.3.3

Lateral buckling factor - exp.6.34

$$K_{crit} = 1.000$$

Beam stability check - exp.6.33

$$\sigma_{m,y,d} / (K_{crit} \times f_{m,y,d}) = 0.419$$

PASS - Beam stability is acceptable

Consider Combination 2 - 1.00G + 1.00Q (Service)

Check design 1668 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection

$$\delta_y = 6.4 \text{ mm}$$

Quasi-permanent variable load factor

$$\psi_2 = 0.3$$

Final deflection with creep

$$\delta_{y,Final} = \delta_y \times (1 + k_{def}) = 11.5 \text{ mm}$$

Allowable deflection

$$\delta_{y,Allowable} = L_{m1_s1} / 250 = 13.3 \text{ mm}$$

$$\delta_{y,Final} / \delta_{y,Allowable} = 0.865$$

PASS - Allowable deflection exceeds final deflection

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Calcs for CEILING JOISTS				Start page no./Revision 1	
Calcs by M	Calcs date 21/03/2020	Checked by	Checked date	Approved by	Approved date

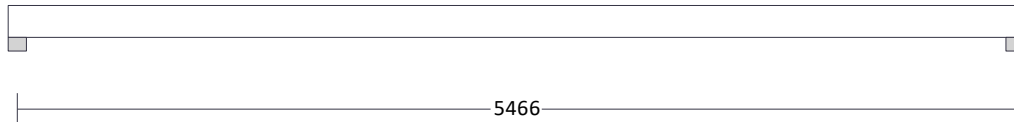
TIMBER JOIST ANALYSIS & DESIGN (EN1995-1-1:2004)

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 2.2.00

Joist details

Description 75 x 175 C24 timber joists
Joist spacing $s_{Joist} = 400$ mm



Forces input on Joist

Vertical permanent load on joist $F_{G_Joist} = 0.35$ kN/m²
Vertical imposed load on joist $F_{Q_Joist} = 0.35$ kN/m²

Joist loading details

Distributed loads

Vertical permanent load on joist $p_G = F_{G_Joist} \times s_{Joist} = 0.14$ kN/m
Vertical imposed load on joist $p_Q = F_{Q_Joist} \times s_{Joist} = 0.14$ kN/m

ANALYSIS

Tedds calculation version 1.0.28

Loading

Self weight included (Permanent x 1)

Load combination factors

Load combination	Permanent	Imposed	Snow	Wind
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.14 kN/m at 0 m to 5.466 m
Member	Imposed	UDL	GlobalZ	0.14 kN/m at 0 m to 5.466 m

Results

Total deflection

1.35G + 1.50Q (Strength) - Total deflection



1.00G + 1.00Q (Service) - Total deflection



Node deflections

Load combination: 1.35G + 1.50Q (Strength)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.49941	
2	0	0	-0.49941	

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.35347	
2	0	0	-0.35347	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	2.6
1.00G + 1.00Q (Service)	0	1.8

Element end forces

Load combination: 1.35G + 1.50Q (Strength)

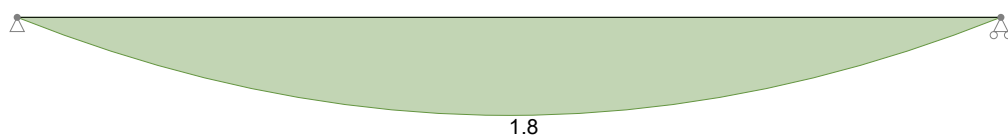
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	5.466	1	0	-1.3	0
		2	0	-1.3	0

Load combination: 1.00G + 1.00Q (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	5.466	1	0	-0.9	0
		2	0	-0.9	0

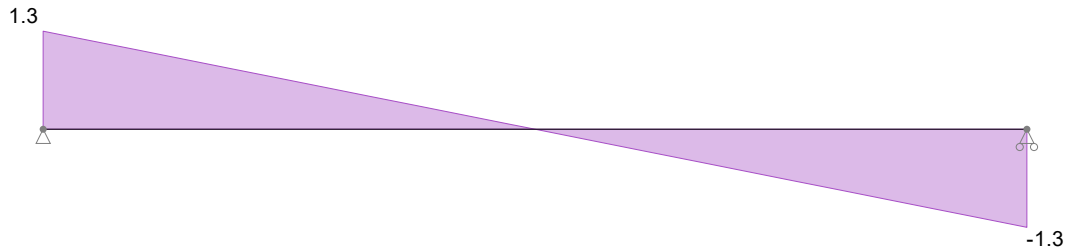
Forces

Strength combinations - Moment envelope (kNm)



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Strength combinations - Shear envelope (kN)



Member results

Envelope - Strength combinations

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Member	0	1.3 (max abs)		0 (min)	
	2.733	0		1.8 (max)	
	5.466	-1.3		0 (min)	

Member - Span 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 $\gamma_M = 1.300$

Member details

Load duration - cl.2.3.1.2 Long-term

Service class - cl.2.3.1.3 2

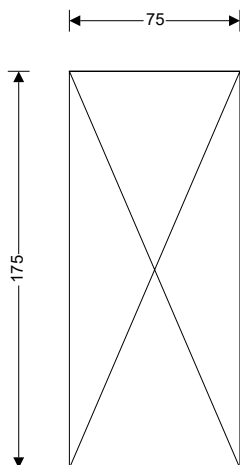
Timber section details

Number of timber sections in member N = 1

Breadth of sections b = 75 mm

Depth of sections h = 175 mm

Timber strength class - EN 338:2016 Table 1 C24



75x175 timber section

Cross-sectional area, A, 13125 mm²

Section modulus, W_y, 382812.5 mm³

Section modulus, W_z, 164063 mm³

Second moment of area, I_y, 33496094 mm⁴

Second moment of area, I_z, 6152344 mm⁴

Radius of gyration, i_y, 50.5 mm

Radius of gyration, i_z, 21.7 mm

Timber strength class C24

Characteristic bending strength, f_{m,k}, 24 N/mm²

Characteristic shear strength, f_{vk}, 4 N/mm²

Characteristic compression strength parallel to grain, f_{0,k}, 21 N/mm²

Characteristic compression strength perpendicular to grain, f_{90,k}, 2.5 N/mm²

Characteristic tension strength parallel to grain, f_{t0,k}, 14.5 N/mm²

Mean modulus of elasticity, E_{0,mean}, 11000 N/mm²

Fifth percentile modulus of elasticity, E_{0,05}, 7400 N/mm²

Shear modulus of elasticity, G_{mean}, 690 N/mm²

Characteristic density, ρ_k, 350 kg/m³

Mean density, ρ_{mean}, 420 kg/m³

Span details

Bearing length L_b = 100 mm

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Consider Combination 1 - 1.35G + 1.50Q (Strength)

Modification factors

Duration of load and moisture content - Table 3.1	$k_{mod} = 0.7$
Deformation factor - Table 3.2	$k_{def} = 0.8$
Bending stress re-distribution factor - cl.6.1.6(2)	$k_m = 0.7$
Crack factor for shear resistance - cl.6.1.7(2)	$k_{cr} = 0.67$
System strength factor - cl.6.6	$k_{sys} = 1.1$

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis	$F_{c,y,90,d} = 1.29 \text{ kN}$
Effective contact length	$L_{b,ef} = L_b = 100 \text{ mm}$
Design perpendicular compressive stress - exp.6.4	$\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef}) = 0.172 \text{ N/mm}^2$
Design perpendicular compressive strength	$f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c,90,k} / \gamma_M = 1.481 \text{ N/mm}^2$
	$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d}) = 0.116$

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force	$F_{y,d} = 1.29 \text{ kN}$
Design shear stress - exp.6.60	$\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) = 0.220 \text{ N/mm}^2$
Design shear strength	$f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.369 \text{ N/mm}^2$
	$\tau_{y,d} / f_{v,y,d} = 0.093$

PASS - Design shear strength exceeds design shear stress

Check design 2733 mm along span

Check bending moment - Section 6.1.6

Design bending moment	$M_{y,d} = 1.763 \text{ kNm}$
Design bending stress	$\sigma_{m,y,d} = M_{y,d} / W_y = 4.605 \text{ N/mm}^2$
Design bending strength	$f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m,k} / \gamma_M = 14.215 \text{ N/mm}^2$
	$\sigma_{m,y,d} / f_{m,y,d} = 0.324$

PASS - Design bending strength exceeds design bending stress

Check beams subjected to either bending or combined bending and compression - cl.6.3.3

Lateral buckling factor - exp.6.34	$k_{crit} = 1.000$
Beam stability check - exp.6.33	$\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d}) = 0.324$

PASS - Beam stability is acceptable

Consider Combination 2 - 1.00G + 1.00Q (Service)

Check design 2733 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection	$\delta_y = 10.7 \text{ mm}$
Quasi-permanent variable load factor	$\psi_2 = 0.3$
Final deflection with creep	$\delta_{y,Final} = \delta_y \times (1 + k_{def}) = 19.3 \text{ mm}$
Allowable deflection	$\delta_{y,Allowable} = L_{m1,s1} / 250 = 21.9 \text{ mm}$
	$\delta_{y,Final} / \delta_{y,Allowable} = 0.881$

PASS - Allowable deflection exceeds final deflection

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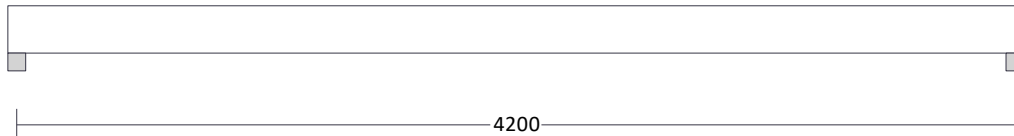
TIMBER JOIST ANALYSIS & DESIGN (EN1995-1-1:2004)

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 2.2.00

Joist details

Description 75 x 200 C24 timber joists
Joist spacing $S_{Joist} = 400$ mm



Forces input on Joist

Vertical permanent load on joist $F_{G_Joist} = 0.50$ kN/m²
Vertical imposed load on joist $F_{Q_Joist} = 2.00$ kN/m²

Joist loading details

Distributed loads

Vertical permanent load on joist $p_G = F_{G_Joist} \times S_{Joist} = 0.20$ kN/m
Vertical imposed load on joist $p_Q = F_{Q_Joist} \times S_{Joist} = 0.80$ kN/m

ANALYSIS

Tedds calculation version 1.0.28

Geometry

Span	Length (m)	Section	Start Support	End Support
1	4.2	75x200	Pinned	Roller Pin X
75x200: Area 150 cm ² , Inertia Major 5000 cm ⁴ , Inertia Minor 703 cm ⁴ , Shear area parallel to Minor 125 cm ² , Shear area parallel to Major = 125 cm ²				
C24 (EC5): Density 420 kg/m ³ , Youngs 11 kN/mm ² , Shear 0.69 kN/mm ² , Thermal 0 °C ⁻¹				

Loading

Self weight included (Permanent x 1)

Load combination factors

Load combination	Permanent	Imposed	Snow	Wind
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.35G + 1.50Q + $\psi_s 1.50S$ (Strength)	1.35	1.50	0.75	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00
1.00G + 1.00Q + $\psi_s 1.00S$ (Service)	1.00	1.00	0.50	0.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.2 kN/m at 0 m to 4.2 m
Member	Imposed	UDL	GlobalZ	0.8 kN/m at 0 m to 4.2 m

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Results

Total deflection

Member results

Load combination: 1.35G + 1.50Q (Strength)

Member	Position (m)	Deflection (mm)	Axial deflection (mm)
Member	0	0	0
	2.1	11.8 (max)	0
	4.2	0	0

Member results

Load combination: 1.35G + 1.50Q + ψ_s 1.50S (Strength)

Member	Position (m)	Deflection (mm)	Axial deflection (mm)
Member	0	0	0
	2.1	11.8 (max)	0
	4.2	0	0

Member results

Load combination: 1.00G + 1.00Q (Service)

Member	Position (m)	Deflection (mm)	Axial deflection (mm)
Member	0	0	0
	2.1	8.1 (max)	0
	4.2	0	0

Member results

Load combination: 1.00G + 1.00Q + ψ_s 1.00S (Service)

Member	Position (m)	Deflection (mm)	Axial deflection (mm)
Member	0	0	0
	2.1	8.1 (max)	0
	4.2	0	0

Reactions

Load case: Permanent

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	0.5	0
2	0	0.5	0

Load case: Imposed

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	1.7	0
2	0	1.7	0

Load case: Snow

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	0	0
2	0	0	0

Load case: Wind

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	0	0
2	0	0	0

Load combination: 1.35G + 1.50Q (Strength)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	3.3	0
2	0	3.3	0

Load combination: 1.35G + 1.50Q + ψ_s 1.50S (Strength)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	3.3	0
2	0	3.3	0

Load combination: 1.00G + 1.00Q (Service)

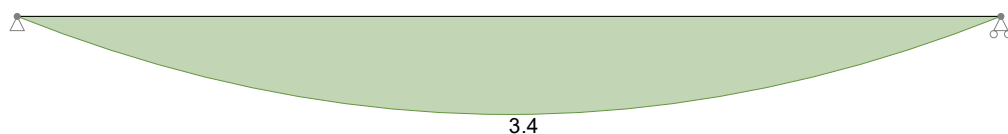
Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	2.2	0
2	0	2.2	0

Load combination: 1.00G + 1.00Q + ψ_s 1.00S (Service)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	2.2	0
2	0	2.2	0

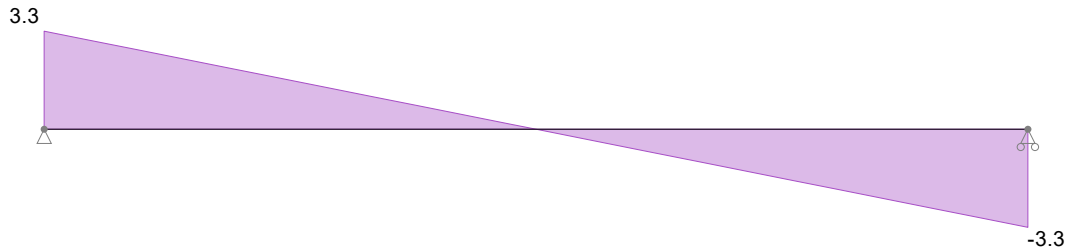
Forces

Strength combinations - Moment envelope (kNm)



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Strength combinations - Shear envelope (kN)



Member - Span 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 $\gamma_M = 1.300$

Member details

Load duration - cl.2.3.1.2 Long-term

Service class - cl.2.3.1.3 1

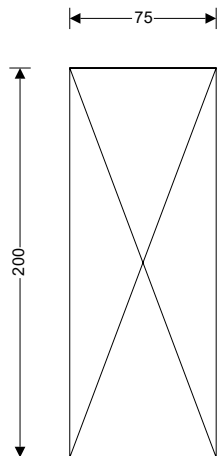
Timber section details

Number of timber sections in member $N = 1$

Breadth of sections $b = 75 \text{ mm}$

Depth of sections $h = 200 \text{ mm}$

Timber strength class - EN 338:2016 Table 1 **C24**



75x200 timber section

Cross-sectional area, A , 15000 mm²

Section modulus, W_y , 500000 mm³

Section modulus, W_z , 187500 mm³

Second moment of area, I_y , 50000000 mm⁴

Second moment of area, I_z , 7031250 mm⁴

Radius of gyration, i_y , 57.7 mm

Radius of gyration, i_z , 21.7 mm

Timber strength class C24

Characteristic bending strength, $f_{m,k}$, 24 N/mm²

Characteristic shear strength, $f_{v,k}$, 4 N/mm²

Characteristic compression strength parallel to grain, $f_{0,k}$, 21 N/mm²

Characteristic compression strength perpendicular to grain, $f_{90,k}$, 2.5 N/mm²

Characteristic tension strength parallel to grain, $f_{t,k}$, 14.5 N/mm²

Mean modulus of elasticity, $E_{0,mean}$, 11000 N/mm²

Fifth percentile modulus of elasticity, $E_{0,05}$, 7400 N/mm²

Shear modulus of elasticity, G_{mean} , 690 N/mm²

Characteristic density, ρ_k , 350 kg/m³

Mean density, ρ_{mean} , 420 kg/m³

Span details

Bearing length $L_b = 75 \text{ mm}$

Consider Combination 2 - 1.35G + 1.50Q + ψ_s 1.50S (Strength)

Modification factors

Duration of load and moisture content - Table 3.1 $k_{mod} = 0.7$

Deformation factor - Table 3.2 $k_{def} = 0.6$

Bending stress re-distribution factor - cl.6.1.6(2) $k_m = 0.7$

Crack factor for shear resistance - cl.6.1.7(2) $k_{cr} = 0.67$

System strength factor - cl.6.6 $k_{sys} = 1.1$

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Check design 2100 mm along span

Check bending moment - Section 6.1.6

Design bending moment

$$M_{y,d} = 3.425 \text{ kNm}$$

Design bending stress

$$\sigma_{m,y,d} = M_{y,d} / W_y = 6.851 \text{ N/mm}^2$$

Design bending strength

$$f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m,k} / \gamma_M = 14.215 \text{ N/mm}^2$$

$$\sigma_{m,y,d} / f_{m,y,d} = 0.482$$

PASS - Design bending strength exceeds design bending stress

Consider Combination 3 - 1.00G + 1.00Q (Service)

Check design 2100 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection

$$\delta_y = 8.1 \text{ mm}$$

Quasi-permanent variable load factor

$$\psi_2 = 0.3$$

Final deflection with creep

$$\delta_{y,Final} = \delta_y \times (1 + k_{def}) = 12.9 \text{ mm}$$

Allowable deflection

$$\delta_{y,Allowable} = \text{Min}(L_{m1_s1} / 250, 14 \text{ mm}) = 14 \text{ mm}$$

$$\delta_{y,Final} / \delta_{y,Allowable} = 0.925$$

PASS - Allowable deflection exceeds final deflection

APPENDIX E

- **MASONRY WALLS**

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Calcs for MASONRY WALL PANEL				Start page no./Revision 1	
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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 1.2.16

Masonry panel details

Unreinforced masonry wall with openings

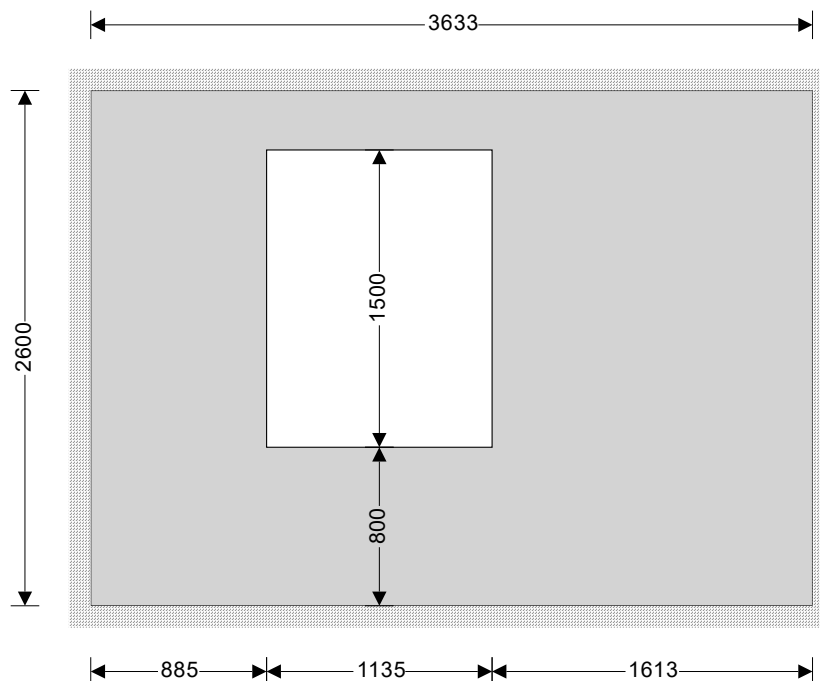
Panel length $L = 3633$ mm
Panel height $h = 2600$ mm

Panel support conditions

Outer leaf **All edges supported**
Inner leaf **All edges supported**

Effective height of masonry walls - Section 5.5.1.2

Reduction factor $\rho_2 = 1.000$
Effective height of wall - eq 5.2 $h_{ef} = \rho_2 \times h = 2600$ mm



Panel opening details

Spacing length $L_1 = 885$ mm
Opening width $w_1 = 1135$ mm
Height to underside of lintel $h_1 = 2300$ mm
Height of opening $o_1 = 1500$ mm

Cavity wall construction details

Outer leaf thickness $t_1 = 100$ mm
Cavity thickness $t_c = 50$ mm
Inner leaf thickness $t_2 = 100$ mm

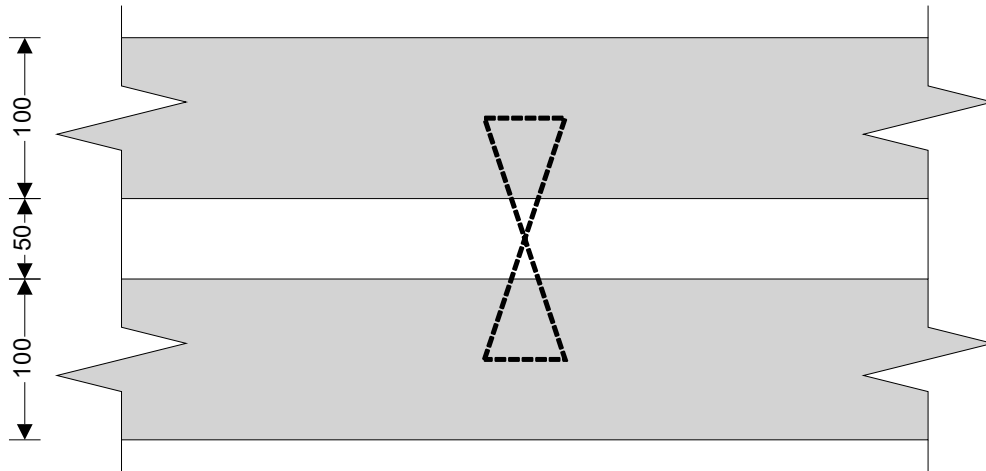
Effective thickness of masonry walls - Section 5.5.1.3

Relative E factor $K_{tef} = 1.000$

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Effective thickness - eq 5.11

$$t_{ef} = (k_{lef} \times t_1^3 + t_2^3)^{1/3} = 126 \text{ mm}$$



Masonry outer leaf details

Masonry type	Clay with water absorption between 7% and 12% - Group 1
Compressive strength of masonry	$f_{c1} = 20 \text{ N/mm}^2$
Height of unit	$h_{u1} = 215 \text{ mm}$
Width of unit	$w_{u1} = 100 \text{ mm}$
Conditioning factor	$k_1 = 1.0$
- Conditioning to the air dry condition in accordance with cl.7.3.2	
Shape factor - Table A.1	$d_{sf1} = 1.38$
Norm. mean compressive strength of masonry	$f_{b1} = f_{c1} \times k_1 \times d_{sf1} = 27.6 \text{ N/mm}^2$
Density of masonry	$\gamma_1 = 18 \text{ kN/m}^3$
Mortar type	M6 - General purpose mortar
Compressive strength of masonry mortar	$f_{m1} = 6 \text{ N/mm}^2$
Compressive strength factor - Table NA.4	$K = 0.50$
Characteristic compressive strength of masonry - eq 3.1	$f_{k1} = K \times f_{b1}^{0.7} \times f_{m1}^{0.3} = 8.731 \text{ N/mm}^2$
Characteristic flexural strength of masonry having a plane of failure parallel to the bed joints - Table NA.6	$f_{xk11} = 0.4 \text{ N/mm}^2$
Characteristic flexural strength of masonry having a plane of failure perpendicular to the bed joints - Table NA.6	$f_{xk21} = 1.1 \text{ N/mm}^2$

Masonry inner leaf details

Masonry type	Aggregate concrete - Group 1
Compressive strength of masonry	$f_{c2} = 7.3 \text{ N/mm}^2$
Height of unit	$h_{u2} = 215 \text{ mm}$
Width of unit	$w_{u2} = 100 \text{ mm}$
Conditioning factor	$k_2 = 1.0$
- Conditioning to the air dry condition in accordance with cl.7.3.2	
Shape factor - Table A.1	$d_{sf2} = 1.38$
Norm. mean compressive strength of masonry	$f_{b2} = f_{c2} \times k_2 \times d_{sf2} = 10.074 \text{ N/mm}^2$
Density of masonry	$\gamma_2 = 18 \text{ kN/m}^3$
Mortar type	M6 - General purpose mortar
Compressive strength of masonry mortar	$f_{m2} = 6 \text{ N/mm}^2$

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Compressive strength factor - Table NA.4

$$K = 0.75$$

Characteristic compressive strength of masonry - eq 3.1

$$f_{k2} = K \times f_{b2}^{0.7} \times f_{m2}^{0.3} = 6.468 \text{ N/mm}^2$$

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

$$f_{xk12} = 0.25 \text{ N/mm}^2$$

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

$$f_{xk22} = 0.6 \text{ N/mm}^2$$

Lateral loading details

Characteristic wind load on panel

$$W_k = 0.620 \text{ kN/m}^2$$

Partial factors for material strength

Category of manufacturing control

Category II

Class of execution control

Class 2

Partial factor for masonry in compressive flexure

$$\gamma_{Mc} = 3.00$$

Partial factor for masonry in tensile flexure

$$\gamma_{Mt} = 2.70$$

Partial factor for masonry in shear

$$\gamma_{Mv} = 2.50$$

Partial safety factors for design loads

Partial safety factor for permanent load

$$\gamma_{fG} = 1.35$$

Partial safety factor for variable imposed load

$$\gamma_{fQ} = 1.5$$

Partial safety factor for variable 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

$$N_{id1} = \gamma_{fG} \times G_{k1} + \gamma_{fQ} \times Q_{k1} = 0 \text{ kN/m}$$

Moment at top of wall due to vertical load

$$M_{id1} = \gamma_{fG} \times G_{k1} \times e_{G1} + \gamma_{fQ} \times Q_{k1} \times e_{Q1} = 0 \text{ kNm/m}$$

Initial eccentricity - cl.5.5.1.1

$$e_{init} = h_{ef} / 450 = 5.8 \text{ mm}$$

Moment at top of wall due to horizontal load

$$M_{Eid1} = 0 \text{ kNm/m}$$

Eccentricity at top of wall due to horizontal load

$$e_{h1} = 0 \text{ mm}$$

Eccentricity at top of wall - eq.6.5

$$e_{i1} = \max(e_{h1} + e_{init}, 0.05 \times t_1) = 5.8 \text{ mm}$$

Reduction factor at top of wall - eq.6.4

$$\Phi_{i1} = \max(1 - 2 \times e_{i1} / t_1, 0) = 0.884$$

Vertical load at middle of wall

$$N_{md1} = \gamma_{fG} \times (G_{k1} + \gamma_1 \times t_1 \times h / 2) + \gamma_{fQ} \times Q_{k1} = 3.159 \text{ kN/m}$$

Moment at middle of wall due to vertical load

$$M_{md1} = \gamma_{fG} \times G_{k1} \times e_{G1} + \gamma_{fQ} \times Q_{k1} \times e_{Q1} = 0 \text{ kNm/m}$$

Moment at middle of wall due to horizontal load

$$M_{Emd1} = 0.059 \text{ kNm/m}$$

Eccentricity at middle of wall due to horizontal load

$$e_{hm1} = M_{Emd1} / N_{md1} = 18.7 \text{ mm}$$

Eccentricity at middle of wall due to loads - eq.6.7

$$e_{m1} = M_{md1} / N_{md1} + e_{hm1} + e_{init} = 24.5 \text{ mm}$$

Eccentricity at middle of wall due to creep

$$e_{k1} = 0 \text{ mm}$$

Eccentricity at middle of wall - eq.6.6

$$e_{mk1} = \max(e_{m1} + e_{k1}, 0.05 \times t_1) = 24.5 \text{ mm}$$

From eq.G.2

$$A_{11} = 1 - 2 \times e_{mk1} / t_1 = 0.51$$

Short term secant modulus of elasticity factor

$$K_E = 1000$$

Modulus of elasticity - cl.3.7.2

$$E_1 = K_E \times f_{k1} = 8731 \text{ N/mm}^2$$

Slenderness - eq.G.4

$$\lambda_1 = (h_{ef} / t_{ef}) \times \sqrt{(f_{k1} / E_1)} = 0.653$$

From eq.G.3

$$u_1 = (\lambda_1 - 0.063) / (0.73 - 1.17 \times e_{mk1} / t_1) = 1.33$$

Reduction factor at middle of wall - eq.G.1

$$\Phi_{m1} = \max(A_{11} \times e^{-u_1 \times u_1^2}, 0) = 0.211$$

Reduction factor for slenderness and eccentricity

$$\Phi_1 = \min(\Phi_{i1}, \Phi_{m1}) = 0.211$$

Considering inner leaf

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Reduction factor for slenderness and eccentricity - Section 6.1.2.2

Vertical load at top of wall	$N_{id2} = \gamma_{fG} \times G_{k2} + \gamma_{fQ} \times Q_{k2} = 0 \text{ kN/m}$
Moment at top of wall due to vertical load	$M_{id2} = \gamma_{fG} \times G_{k2} \times e_{G2} + \gamma_{fQ} \times Q_{k2} \times e_{Q2} = 0 \text{ kNm/m}$
Initial eccentricity - cl.5.5.1.1	$e_{init} = h_{ef} / 450 = 5.8 \text{ mm}$
Moment at top of wall due to horizontal load	$M_{Eid2} = 0 \text{ kNm/m}$
Eccentricity at top of wall due to horizontal load	$e_{h2} = 0 \text{ mm}$
Eccentricity at top of wall - eq.6.5	$e_{i2} = \max(e_{h2} + e_{init}, 0.05 \times t_2) = 5.8 \text{ mm}$
Reduction factor at top of wall - eq.6.4	$\Phi_{i2} = \max(1 - 2 \times e_{i2} / t_2, 0) = 0.884$
Vertical load at middle of wall	$N_{md2} = \gamma_{fG} \times (G_{k2} + \gamma_2 \times t_2 \times h / 2) + \gamma_{fQ} \times Q_{k2} = 3.159 \text{ kN/m}$
Moment at middle of wall due to vertical load	$M_{md2} = \gamma_{fG} \times G_{k2} \times e_{G2} + \gamma_{fQ} \times Q_{k2} \times e_{Q2} = 0 \text{ kNm/m}$
Moment at middle of wall due to horizontal load	$M_{Emd2} = 0.042 \text{ kNm/m}$
Eccentricity at middle of wall due to horizontal load	$e_{hm2} = M_{Emd2} / N_{md2} = 13.2 \text{ mm}$
Eccentricity at middle of wall due to loads - eq.6.7	$e_{m2} = M_{md2} / N_{md2} + e_{hm2} + e_{init} = 19 \text{ mm}$
Eccentricity at middle of wall due to creep	$e_{k2} = 0 \text{ mm}$
Eccentricity at middle of wall - eq.6.6	$e_{mk2} = \max(e_{m2} + e_{k2}, 0.05 \times t_2) = 19 \text{ mm}$
From eq.G.2	$A_{12} = 1 - 2 \times e_{mk2} / t_2 = 0.621$
Short term secant modulus of elasticity factor	$K_E = 1000$
Modulus of elasticity - cl.3.7.2	$E_2 = K_E \times f_{k2} = 6468 \text{ N/mm}^2$
Slenderness - eq.G.4	$\lambda_2 = (h_{ef} / t_{ef}) \times \sqrt{(f_{k2} / E_2)} = 0.653$
From eq.G.3	$u_2 = (\lambda_2 - 0.063) / (0.73 - 1.17 \times e_{mk2} / t_2) = 1.16$
Reduction factor at middle of wall - eq.G.1	$\Phi_{m2} = \max(A_{12} \times e^{-u_2 \times u_2 / 2}, 0) = 0.317$
Reduction factor for slenderness and eccentricity	$\Phi_2 = \min(\Phi_{i2}, \Phi_{m2}) = 0.317$

Unreinforced masonry walls subjected to lateral loading - Section 6.3

Partial safety factors for design loads

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

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

Length to thickness ratio	$L / t_{ef} = 28.835$
Limiting height to thickness ratio - Figure F.1	80
Height to thickness ratio	$h / t_{ef} = 20.636$

PASS - Limiting height to thickness ratio is not exceeded

Design moments of resistance in panels

Considering outer leaf

Self weight at top of wall	$S_{wt1} = 0 \text{ kN/m}$
Design compressive strength of masonry	$f_{d1} = f_{k1} / \gamma_{Mc} = 2.910 \text{ N/mm}^2$
Design vertical compressive stress	$\sigma_{d1} = \min(\gamma_{fG} \times (G_{k1} + S_{wt1}) / t_1, 0.15 \times \Phi_1 \times f_{d1}) = 0 \text{ N/mm}^2$
Design flexural strength of masonry parallel to bed joints	$f_{xd11} = f_{xk11} / \gamma_{Mt} = 0.148 \text{ N/mm}^2$
Apparent design flexural strength of masonry parallel to bed joints	$f_{xd11,app} = f_{xd11} + \sigma_{d1} = 0.148 \text{ N/mm}^2$
Design flexural strength of masonry perpendicular to bed joints	$f_{xd21} = f_{xk21} / \gamma_{Mt} = 0.407 \text{ N/mm}^2$
Elastic section modulus of wall	$Z_1 = t_1^2 / 6 = 1666667 \text{ mm}^3/\text{m}$
Moment of resistance parallel to bed joints - eq.6.15	

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$$M_{Rd11} = f_{xd11,app} \times Z_1 = \mathbf{0.247 \text{ kNm/m}}$$

Moment of resistance perpendicular to bed joints - eq.6.15

$$M_{Rd21} = f_{xd21} \times Z_1 = \mathbf{0.679 \text{ kNm/m}}$$

Considering inner leaf

Self weight at top of wall

$$S_{w12} = \mathbf{0 \text{ kN/m}}$$

Design compressive strength of masonry

$$f_{d2} = f_{k2} / \gamma_{Mc} = \mathbf{2.156 \text{ N/mm}^2}$$

Design vertical compressive stress

$$\sigma_{d2} = \min(\gamma_{FG} \times (G_{k2} + S_{w12}) / t_2, 0.15 \times \Phi_2 \times f_{d2}) = \mathbf{0 \text{ N/mm}^2}$$

Design flexural strength of masonry parallel to bed joints

$$f_{xd12} = f_{xk12} / \gamma_{Mt} = \mathbf{0.093 \text{ N/mm}^2}$$

Apparent design flexural strength of masonry parallel to bed joints

$$f_{xd12,app} = f_{xd12} + \sigma_{d2} = \mathbf{0.093 \text{ N/mm}^2}$$

Design flexural strength of masonry perpendicular to bed joints

$$f_{xd22} = f_{xk22} / \gamma_{Mt} = \mathbf{0.222 \text{ N/mm}^2}$$

Elastic section modulus of wall

$$Z_2 = t_2^2 / 6 = \mathbf{1666667 \text{ mm}^3/\text{m}}$$

Moment of resistance parallel to bed joints - eq.6.15

$$M_{Rd12} = f_{xd12,app} \times Z_2 = \mathbf{0.154 \text{ kNm/m}}$$

Moment of resistance perpendicular to bed joints - eq.6.15

$$M_{Rd22} = f_{xd22} \times Z_2 = \mathbf{0.37 \text{ kNm/m}}$$

Design moment in panels

Calculate design wind load acting on each leaf

Outer leaf design wind load - parallel

$$W_{k11} = M_{Rd11} \times W_k / (M_{Rd11} + M_{Rd12}) = \mathbf{0.382 \text{ kN/m}^2}$$

Inner leaf design wind load - parallel

$$W_{k12} = M_{Rd12} \times W_k / (M_{Rd11} + M_{Rd12}) = \mathbf{0.238 \text{ kN/m}^2}$$

Outer leaf design wind load - perpendicular

$$W_{k21} = M_{Rd21} \times W_k / (M_{Rd21} + M_{Rd22}) = \mathbf{0.401 \text{ kN/m}^2}$$

Inner leaf design wind load - perpendicular

$$W_{k22} = M_{Rd22} \times W_k / (M_{Rd21} + M_{Rd22}) = \mathbf{0.219 \text{ kN/m}^2}$$

Orthogonal strength ratios

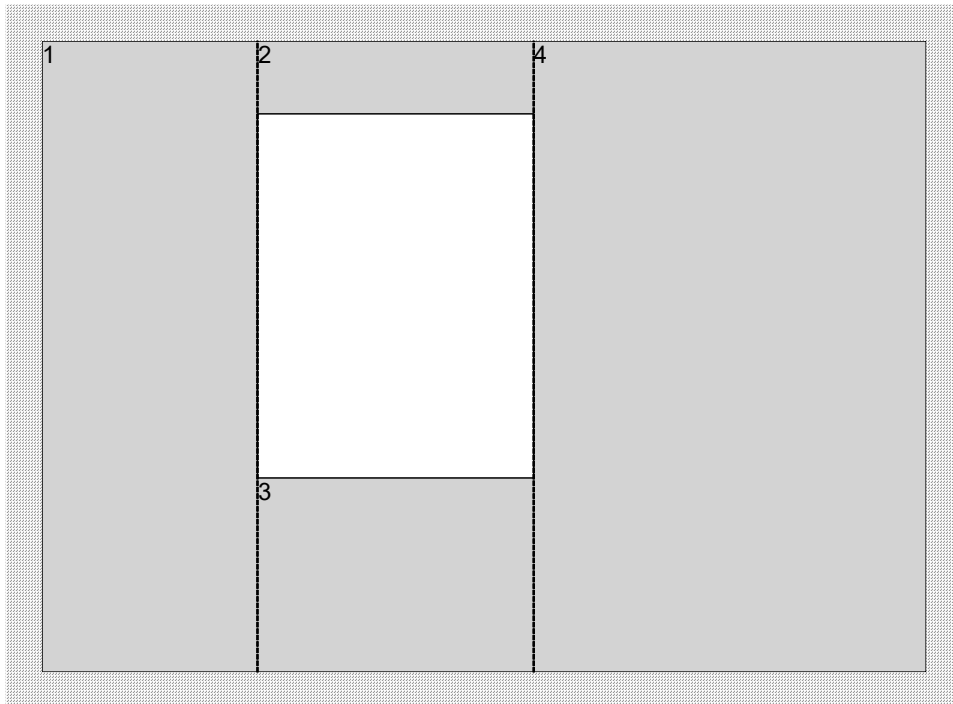
For outer leaf

$$\mu_1 = f_{xd11,app} / f_{xd21} = \mathbf{0.36}$$

For inner leaf

$$\mu_2 = f_{xd12,app} / f_{xd22} = \mathbf{0.42}$$

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

Ratio panel height to length $h_{s1A} / L_{s1A} = 2.94$

Considering outer leaf

Perpendicular design moment of resistance $M_{Rd21} = 0.679 \text{ kNm/m}$

Using yield line analysis to calculate bending moment coefficient

Bending moment coefficient $\alpha_{s11A} = 1.078$

Design moment in sub-panel $M_{Ed11A} = \gamma_{fW} \times \alpha_{s11A} \times W_{k21} \times L_{s1A}^2 = 0.508 \text{ kNm/m}$

PASS - Resistance moment exceeds design moment

Considering inner leaf

Perpendicular design moment of resistance $M_{Rd22} = 0.370 \text{ kNm/m}$

Using yield line analysis to calculate bending moment coefficient

Bending moment coefficient $\alpha_{s21A} = 1.079$

Design moment in sub-panel $M_{Ed21A} = \gamma_{fW} \times \alpha_{s21A} \times W_{k22} \times L_{s1A}^2 = 0.277 \text{ kNm/m}$

PASS - Resistance moment exceeds design moment

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

Ratio panel height to length $h_{s2A} / L_{s2A} = 0.26$

Considering outer leaf

Parallel design moment of resistance $M_{Rd11} = 0.247 \text{ kNm/m}$

Using yield line analysis to calculate bending moment coefficient

Bending moment coefficient $\alpha_{s12A} = 0.356$

Design moment in sub-panel $M_{Ed12A} = \mu_1 \times \gamma_{fW} \times \alpha_{s12A} \times W_{k11} \times L_{s2A}^2 = 0.095 \text{ kNm/m}$

PASS - Resistance moment exceeds design moment

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Considering inner leaf

Parallel design moment of resistance $M_{Rd12} = 0.154 \text{ kNm/m}$

Using yield line analysis to calculate bending moment coefficient

Bending moment coefficient $\alpha_{s22A} = 0.356$

Design moment in sub-panel $M_{Ed22A} = \mu_2 \times \gamma_{fW} \times \alpha_{s22A} \times W_{k12} \times L_{s2A}^2 = 0.068 \text{ kNm/m}$

PASS - Resistance moment exceeds design moment

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

Ratio panel height to length $h_{s3A} / L_{s3A} = 0.70$

Considering outer leaf

Perpendicular design moment of resistance $M_{Rd21} = 0.679 \text{ kNm/m}$

Using yield line analysis to calculate bending moment coefficient

Bending moment coefficient $\alpha_{s13A} = 0.170$

Design moment in sub-panel $M_{Ed13A} = \gamma_{fW} \times \alpha_{s13A} \times W_{k21} \times L_{s3A}^2 = 0.132 \text{ kNm/m}$

PASS - Resistance moment exceeds design moment

Considering inner leaf

Perpendicular design moment of resistance $M_{Rd22} = 0.370 \text{ kNm/m}$

Using yield line analysis to calculate bending moment coefficient

Bending moment coefficient $\alpha_{s23A} = 0.166$

Design moment in sub-panel $M_{Ed23A} = \gamma_{fW} \times \alpha_{s23A} \times W_{k22} \times L_{s3A}^2 = 0.070 \text{ kNm/m}$

PASS - Resistance moment exceeds design moment

Sub panel no. 4 - Top, bottom and right supported

Ratio panel height to length $h_{s4A} / L_{s4A} = 1.61$

Considering outer leaf

Parallel design moment of resistance $M_{Rd11} = 0.247 \text{ kNm/m}$

Using yield line analysis to calculate bending moment coefficient

Bending moment coefficient $\alpha_{s14A} = 0.423$

Design moment in sub-panel $M_{Ed14A} = \mu_1 \times \gamma_{fW} \times \alpha_{s14A} \times W_{k11} \times L_{s4A}^2 = 0.229 \text{ kNm/m}$

PASS - Resistance moment exceeds design moment

Considering inner leaf

Parallel design moment of resistance $M_{Rd12} = 0.154 \text{ kNm/m}$

Using yield line analysis to calculate bending moment coefficient

Bending moment coefficient $\alpha_{s24A} = 0.387$

Design moment in sub-panel $M_{Ed24A} = \mu_2 \times \gamma_{fW} \times \alpha_{s24A} \times W_{k12} \times L_{s4A}^2 = 0.150 \text{ kNm/m}$

PASS - Resistance moment exceeds design moment

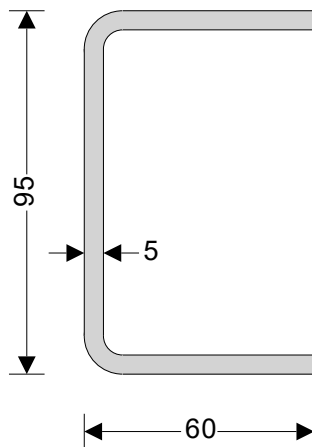
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WINDPOST DESIGN (BS5950:PART 1:2000)

TEDDS calculation version 1.0.02

Section properties - 95 × 60 × 5 channel section windpost

Section depth	d = 95 mm
Section breadth	b = 60 mm
Section thickness	t = 5 mm
Radius of curves	r = 5 mm
Area of section	Area = 9.771 cm ²
Section modulus	Z = 28.929 cm ³
Moment of inertia	I = 137.412 cm ⁴



Material properties

Design strength	$p_y = 180.00 \text{ N/mm}^2$
Youngs modulus	$E = 200000 \text{ N/mm}^2$

Section classification

Constant	$\varepsilon = [(275 / p_y) \times (E / 205000)]^{0.5} = 1.221$
Web with neutral axis at mid-depth	$d / t = 19.000 \leq 52 \times \varepsilon$ (1) Plastic
Outstand element of compression flange	$b / t = 12.000 \leq 11.0 \times \varepsilon$ (3) Semi-compact
Section classification	(3) Semi-compact

Windpost design details

Height of windpost	Height = 2600 mm
Length of loaded panel	Length = 4722 mm
Windpost support type	Simply supported

Loading details

Partial safety factor for wind loading	$\gamma_f = 1.40$
Unfactored wind loading	$w = 0.620 \text{ kN/m}^2$
Reaction at supports	$R = w \times \gamma_f \times \text{Height} \times \text{Length} / 2 = 5.328 \text{ kN}$

Check bending stress

Bending moment	$M = w \times \gamma_f \times \text{Length} \times \text{Height}^2 / 8 = 3.463 \text{ kNm}$
Applied bending stress	$p_{app} = M / Z = 119.721 \text{ N/mm}^2$

PASS - Applied bending stress is less than maximum bending stress

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Check deflection

Maximum deflection

$$\delta_{\max} = \text{Height} / 360 = \mathbf{7.222 \text{ mm}}$$

Actual deflection

$$\delta_{\text{act}} = 5 \times w \times \text{Length} \times \text{Height}^4 / (384 \times E \times I) = \mathbf{6.339 \text{ mm}}$$

PASS - Actual deflection is less than maximum deflection

Check wall ties

Brick tie capacity in tension and compression

$$f_t = \mathbf{4.500 \text{ kN}}$$

Partial safety factor for wall ties

$$\gamma_m = \mathbf{3.00}$$

Total load on windpost

$$F = w \times \gamma_f \times \text{Height} \times \text{Length} = \mathbf{10.657 \text{ kN}}$$

Maximum spacing between ties

$$s = \text{Height} \times f_t / (F \times \gamma_m) = \mathbf{366 \text{ mm}}$$

Wall ties provided at a maximum of 225 mm centres

APPENDIX F

- **STRIP FOUNDATIONS**

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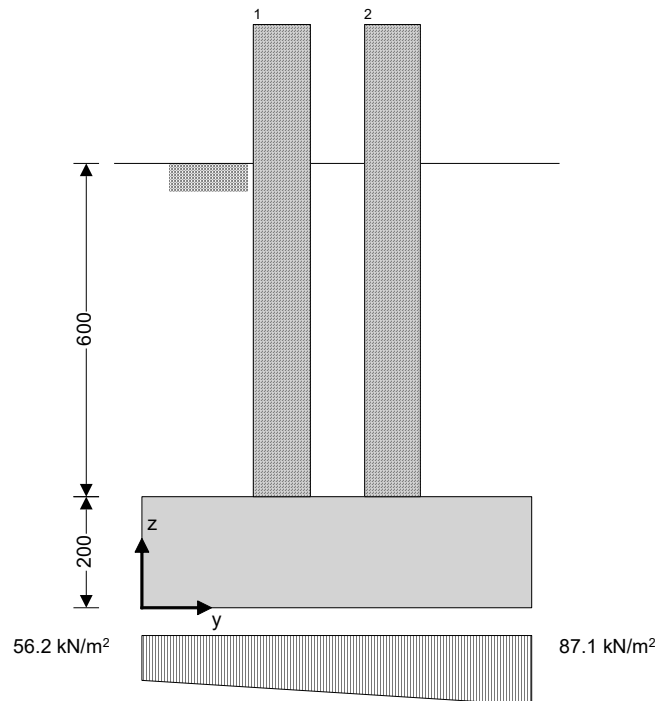
FOUNDATION ANALYSIS (EN1997-1:2004)

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

TEDDS calculation version 3.2.21

Strip foundation details - considering a one metre strip

Length of foundation	$L_x = 1000$ mm
Width of foundation	$L_y = 700$ mm
Foundation area	$A = L_x \times L_y = 0.700$ m ²
Depth of foundation	$h = 200$ mm
Depth of soil over foundation	$h_{\text{soil}} = 600$ mm
Level of water	$h_{\text{water}} = 0$ mm
Density of water	$\gamma_{\text{water}} = 9.8$ kN/m ³
Density of concrete	$\gamma_{\text{conc}} = 25.0$ kN/m ³



Wall no.1 details

Width of wall	$l_{y1} = 103$ mm
position in y-axis	$y_1 = 251$ mm

Wall no.2 details

Width of wall	$l_{y2} = 100$ mm
position in y-axis	$y_2 = 450$ mm

Soil properties

Density of soil	$\gamma_{\text{soil}} = 18.0$ kN/m ³
Characteristic cohesion	$c'_k = 0$ kN/m ²
Characteristic effective shear resistance angle	$\phi'_k = 30$ deg
Characteristic friction angle	$\delta_k = 20$ deg

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Foundation loads

Self weight

$$F_{\text{swt}} = h \times \gamma_{\text{conc}} = \mathbf{5.0 \text{ kN/m}^2}$$

Soil weight

$$F_{\text{soil}} = h_{\text{soil}} \times \gamma_{\text{soil}} = \mathbf{10.8 \text{ kN/m}^2}$$

Wall no.1 loads per linear metre

Permanent load in z

$$F_{\text{Gz1}} = \mathbf{13.3 \text{ kN}}$$

Wall no.2 loads per linear metre

Permanent load in z

$$F_{\text{Gz2}} = \mathbf{18.4 \text{ kN}}$$

Variable load in z

$$F_{\text{Qz2}} = \mathbf{7.4 \text{ kN}}$$

Bearing resistance (Section 6.5.2)

Forces on foundation per linear metre

Force in z-axis

$$F_{\text{dz}} = A \times (F_{\text{swt}} + F_{\text{soil}}) + F_{\text{Gz1}} + F_{\text{Gz2}} + F_{\text{Qz2}} = \mathbf{50.2 \text{ kN}}$$

Moments on foundation per linear metre

Moment in y-axis

$$M_{\text{dy}} = A \times (F_{\text{swt}} + F_{\text{soil}}) \times L_y / 2 + F_{\text{Gz1}} \times y_1 + F_{\text{Gz2}} \times y_2 + F_{\text{Qz2}} \times y_2 = \mathbf{18.8 \text{ kNm}}$$

Eccentricity of base reaction

Eccentricity of base reaction in y-axis

$$e_y = M_{\text{dy}} / F_{\text{dz}} - L_y / 2 = \mathbf{25 \text{ mm}}$$

Pad base pressures

$$q_1 = F_{\text{dz}} \times (1 - 6 \times e_y / L_y) / (L_y \times 1000 \text{ mm}) = \mathbf{56.2 \text{ kN/m}^2}$$

$$q_2 = F_{\text{dz}} \times (1 + 6 \times e_y / L_y) / (L_y \times 1000 \text{ mm}) = \mathbf{87.1 \text{ kN/m}^2}$$

Minimum base pressure

$$q_{\text{min}} = \min(q_1, q_2) = \mathbf{56.2 \text{ kN/m}^2}$$

Maximum base pressure

$$q_{\text{max}} = \max(q_1, q_2) = \mathbf{87.1 \text{ kN/m}^2}$$

Presumed bearing capacity

Presumed bearing capacity

$$P_{\text{bearing}} = \mathbf{150.0 \text{ kN/m}^2}$$

PASS - Presumed bearing capacity exceeds design base pressure