Martin Perry Associates
Structural Engineering & Surveying

M Perry Associates Ltd
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Project Ref: 23111

STRUCTURAL REPORT

ON:	COW SHED AT HAMATETHY, ST BREWARD
FOR:	MRS C SWIDERSKA
DATE:	NOVEMBER 2023
PREPARED BY:	M W PERRY B.Eng (Hons), MRICS, I.Eng, IMIStructE

1.0 INTRODUCTION

1.1 M Perry Associates Ltd have been instructed to visit the cow shed at Hamatethy, St Breward in order to inspect and report on its structural condition and suitability for conversion as part of a Class Q Permitted Development application.

1.2 Since instruction and first visual inspection M Perry Associates have worked in liaison with other project team professionals to ensure a 'joined up' approach to the task in hand; to ensure the submission of a robust case to Cornwall Council involving residential conversion of the subject barn into a single dwelling house.

We are aware of the Cornwall Council Class Q Prior Notification Guidance Note relating to the permitted development rights in respect of agricultural buildings (latest update April 2021) and the Chief Planning Officer's Advice Note relating to barn conversions and replacement dwellings in the countryside, both of which contain the following advice:

- The applicant will need to demonstrate the structural soundness of the building
- The building should be of sound construction and capable of conversion without the need for major/substantial re-building

M Perry Associates consider the subject building to be structurally sound and capable of residential conversion without any re-building. Some alterations externally are required to convert the building to human habitation, as usual, as fully shown on the submitted drawings. Internal works are shown on the submitted drawings, but these are not deemed development due to clarification within revised advice issued by the Housing Ministry on 22nd February 2018 and which is accepted by Cornwall Council (confirmed within their Class Q Prior Notification Guidance Note).

1.3 The inspection and report have been carried out in accordance with our Conditions of Appointment, a copy of which are attached.

2.0 GENERAL

2.1 The building was a single storey open-fronted cow shed of concrete block construction with a timber roof structure supporting corrugated galvanized roof sheeting.

2.2 For the purposes of this report the front elevation of the building is assumed to face north.

3.0 OBSERVATIONS

3.1 The building had internal dimensions of approximately 15.8m x 5.3m with 4 no. raised collar tie trusses forming the roof structure supporting 2 no. lines of purlins per pitch and double ridge beam at the apex.

3.2 The roof trusses spanned between solid concrete block piers at each end with steel hold-down straps present along the southern wall.

3.3 The roof structure was generally in good condition with no signs of any significant water ingress or deflection beyond normal construction tolerances and was performing well based on its current load conditions.

3.4 The external walls were constructed from solid 150mm wide concrete blockwork laid on the narrow edge with 215mm block on flat piers spaced at 3.8m centres along the southern elevation. To the north the external wall was open fronted with cattle barriers spanning between 440mm x 440mm solid block piers with the roof trusses bearing upon them.

3.5 Some vertical cracking was noted along the southern elevation, principally at the intersection between the wall panels and the piers as well as within the centre of the wall panels and to the southwest corner. These cracks were of a consistent width through the full height of the wall and appeared to be due to thermal expansion and contraction of the wall panels due to a lack of movement joints. The walls were relatively straight and true with no signs of any subsidence or settlement issues.

3.6 The floor slab internally was in good condition with no signs of any significant cracking or movement in the areas observed.

4.0 CONCLUSIONS

4.1 Overall, the building appeared to be in good condition considering its age and construction type. The masonry was of good quality and the piers relatively closely spaced in order to provide buttressing to wind forces.

4.2 The roof was performing well based on its current load conditions and providing the proposed conversion uses lightweight roofing finishes the roof structure will be suitable for reuse. The calculations appended to this report confirm this to be the case.

4.3 The building will need to be lined with a new timber-framed wall or similar in order to support internal finishes and insulation. This timber framing should be tied back to the external walls in order to ensure compliance with Building Regulations approved documents and could be designed to support any additional loads imposed on the building from its conversion. Wall panel calculations have been completed to confirm the building is stable based on the proposed layout.

4.4 The open sections of the building will be infilled with walls or glazing. If glazing is to be utilised these openings should be framed, as allowed under current planning policy guidance. The existing masonry piers have been checked by theoretical calculation and confirmed as suitable to retain.

4.5 The sub-structure was inspected and confirmed as mass concrete with no sign of significant settlement or subsidence noted.

4.6 In summary, the building was in relatively good condition and capable of conversion into a domestic dwelling without any significant structural strengthening, rebuilding or upgrading works being carried out.

[redacted]

SIGNED: ....

DATED: 8th November 2023

APPENDIX A
PHOTOGRAPHS

Figure 1 – General internal view.

Figure 2 – Trusses supported by solid block piers also providing buttressing to south wall.

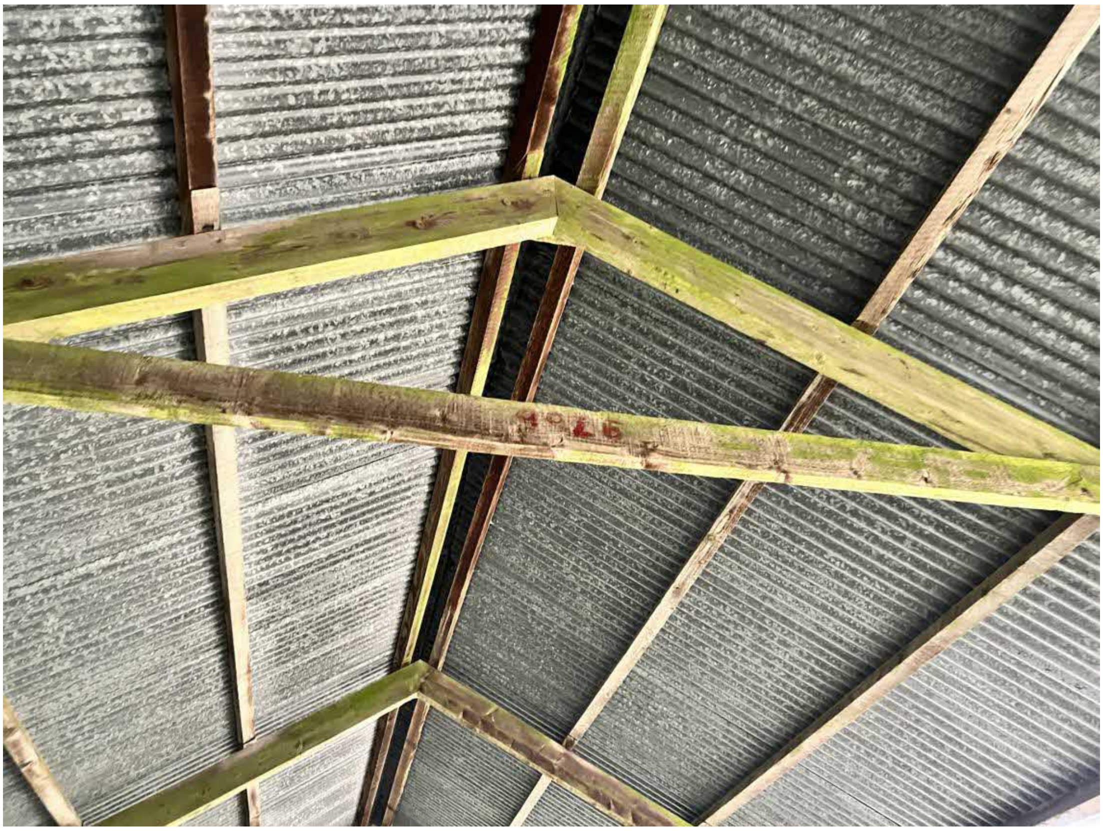

Figure 3 – Roof structure performing adequately based on current load conditions.

Figure 4 – Vertical cracking to south elevation due to thermal movement.

APPENDIX B
CALCULATIONS

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Project: 23111

STRUCTURAL CALCULATIONS

PROJECT:	HAMETETHY FARM, BODMIN
CLIENT:	MRS C SWIDERSKA
DATE:	JUNE 2023
PREPARED BY:	A MARANGE BEng(Hons)
CHECKED BY:	M W PERRY BEng(Hons), MRICS, IEng, AMIStructE

Managing Director M W Perry B.Eng (Hons) MRICS I.Eng AMIStructE
Technical Director B Pengelly M.Eng
M Perry Associates Ltd trading as Martin Perry Associates
Registered Office: Parade House, The Parade, Liskeard, Cornwall PL14 6AH
Registered Company No. 10944552

DESIGN PHILOSOPHY

The following structural calculations and drawings relate to Hametethy Farm, Bodmin.

MPA has been appointed to provide structural calculations for design checks on existing structural elements forming an existing barn. The design checks have been carried out in order to support the MPA Structural Report and aid the application of a Class Q Permitted Development.

The following items have been checked:

1. Timber Truss
2. Masonry Bearing Pier
3. External Masonry Wall Panel

DESIGNER RISK ASSESSMENT

The following specific risks have been identified:

Ref	Risk	Action
1	Installation of heavy beams.	Mechanical aids should be employed when installing beams. Beam can be spliced –Structural Engineer to be consulted if required.
2	Temporary support.	The contractor is to provide all necessary temporary propping to safely undertake the work and maintain structural stability, including excavation sides for retaining structures.
3	Confirmation of structural layout.	The assumptions made within the calculations and drawing should be confirmed on site, prior to commencement of works. Any differences should be discussed with Structural Engineer, in order to check calculations.
4	Working at height.	Suitable scaffold to be provided in order to carry out works safely.
5	Asbestos containing materials.	An asbestos survey should be undertaken, prior to any works proceeding, with the recommendations from the survey followed.

Notes:

1. The works are to be carried out by an experienced competent contractor capable of managing general construction hazards.
2. The following list is limited to specific risks associated with the structural design and is to be read in conjunction with other relevant designer risk assessments.
3. The following list is not necessarily exhaustive and the contractor should carry out their own assessment of potential risks.
4. Comment is made in the Risk Assessment for future demolition and use of the building as a work place only if it is considered to be out of the ordinary.
5. The Client, designers and contractor shall be mindful of their Health and Safety obligations under the CDM (2015) Regulations.

Check List (Delete if not applicable or considered a general site risk):

Temp. stability / propping	Access	Risks to general public	Asbestos	Confined spaces	Future maintenance
Hidden / buried services	Material handling	Hot works	Working at height	COSHH	Dismantling / demolition

CODES OF PRACTICE USED IN DESIGNS

BS EN 1991-1-1:2002+NA	General Actions
BS EN 1991-1-3:2003+A1:2015+NA	Snow Loads
BS EN 1991-1-4:2005 + A1:2010+NA	Wind Actions
BS EN 1992-1-1:2004 + A1:2014+NA	Design of Concrete Structures
BS EN 1993-1-1:2005 + A1:2014+NA	Design of Steel Structures
BS EN 1995-1-1-2004+ A2 + NA	Design of Timber Structures
BS EN 1996-1-1:2005+NA	Design of Masonry Structures

Refer to Data Sheet 2 for standard construction notes.

CONSTRUCTION NOTES

1) All loadings are standard loadings. Confirmation of existing materials and weights to be undertaken by others on site.
2) Calculations prepared in accordance with Architect's drawings.
3) No works should be undertaken on site until Building Regulations Approval has been granted.
4) The builder and owner are reminded that works on or adjacent to a party wall or boundary will be subject to the Party Wall etc. Act 1996 and approval from the adjoining owner must be sought 2 months prior to commencing works. Further information can be provided by this Company if required.
5) All dimensions to be taken on site and not scaled from the drawings or sketches. All details and dimensions relating to subcontractors or suppliers work must be checked and agreed between the subcontractor or supplier and the general contractor.
6) The drawings, details, and sketches to be read in conjunction with all relevant Architects' and Engineers' drawings and specifications.
7) Works to comply with current Codes of Practice, British Standards and Building Regulations.
8) Contractor to expose structure to confirm span directions, support locations and details, footings etc. and to verify that all assumptions made by the Engineer are correct prior to ordering any materials or carrying out any works on site.
9) Steelwork to have two coats of red oxide primer generally and to have two further coats of bitumen paint where supported at external walls unless indicated otherwise.
10) All internal steelwork subject to fire regulations to have two layers of 12.5mm plasterboard with joints staggered or to Architect's specification to comply with Building Regulations.
11) Where timbers are bolted together the bolt holes are to be drilled no more than 2mm diameter larger than bolt diameter. Holes to be at right angles to line of timbers. The bolts are to be tightened until local crushing is apparent around the 50mm square plate washers. If timbers are not touching, then timber packers are to be installed full depth of the joists/rafters to make up the gap.
12) Bearing of all new foundations to be 1000mm below external ground or to level of base of foundations to main building, whichever is deeper, unless indicated otherwise on drawings. Foundation details may have to be altered if ground conditions prove to be different from those assumed in the calculations, i.e. poorer ground conditions, obstructions or existing structures.
    Footings may be required to go deeper if trees are, or recently have been, in close proximity to the building.
    All foundations to be confirmed on site by the Building Control Officer.
13) All underpinning works to be checked and approved by the Local Authority Building Control Officer and to be carried out in accordance with our specification for underpinning.

The works are to be carried out in accordance with the Construction, Design and Management Regulations 2015. The Client and contractors should ensure they comply with their duties as set out in the regulations. Refer to HSE document Managing Health and Safety in Construction (CDM 2015), Approved Code of Practice for further details.

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GENERAL LOADINGS

Existing Roof

Variable Load	Snow/Access	0.60	kN/m²
Permanent Load	Corrugated sheet roofing	0.10	kN/m²

Masonry

Permanent Load		2.30	kN/m²

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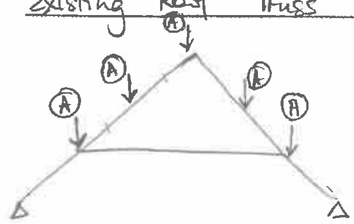
PROJECT	HAMETETHY FARM, BODMIN			
Designed by	Checked by	Date	Project no.	Sheet no.
AM	MWP	JUNE 2023	23111	4

CALCULATIONS

Existing Roof Truss Check

Roof DL = 0.1 kN/m²
IL = 0.6 kN/m²



loaded width A 1.0m
DL = 0.1 kN/m
IL = 0.6 kN/m

Purlin length = 4m

Point loads from purlin A DL = 0.2kN x2 = 0.4kN
IL = 1.2kN x2 = 2.4kN

Tedds Output
Rafter 75x200 PASS
Tie 50x175 PASS
End Reactions DL 1.2kN
IL 6.0kN

TIMBER MEMBER 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 2.2.19

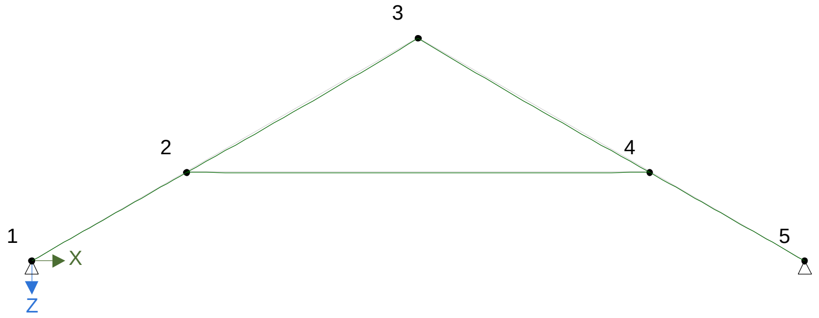
ANALYSIS

Tedds calculation version 1.0.37

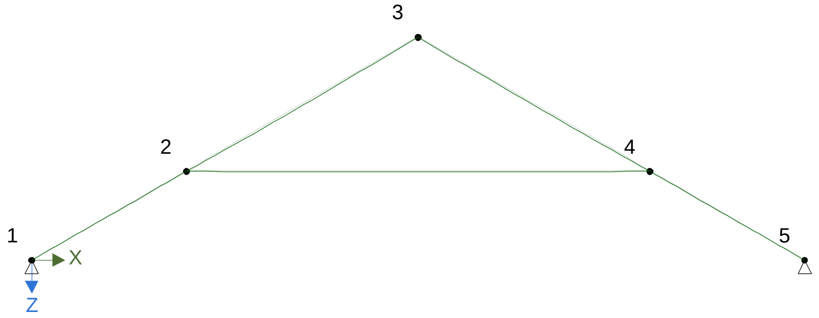
Results

Total deflection

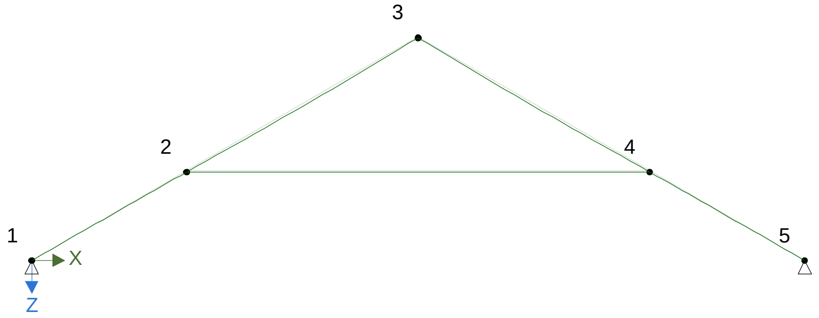
1.35G + 1.5Q + 1.5RQ (Strength) - Total deflection



1.0G + 1.0Q + 1.0RQ (Service) - Total deflection

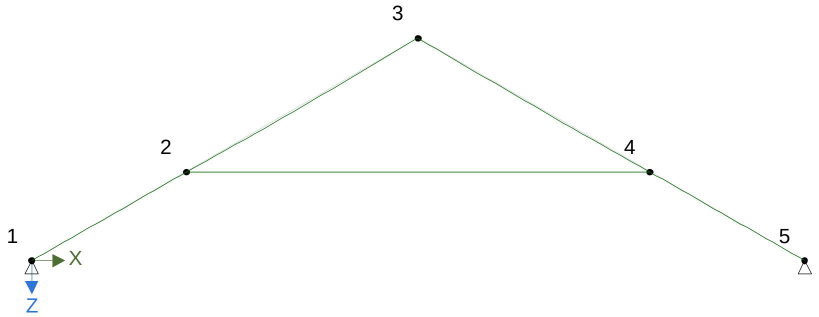


1.35G + 1.5Q + 1.5ψ_0S (Strength) - Total deflection

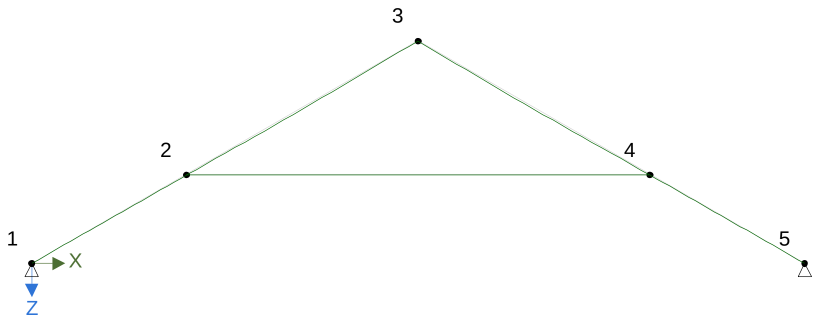


Martin Perry Associates Parade House, The Parade Liskeard, Cornwall PL14 6AH	Project HAMETETHY FARM, BODMIN				Job no. 23111	
	Calcs for EXISTING TRUSS CHECK				Start page no./Revision 6	
	Calcs by AM	Calcs date 15/06/2023	Checked by MWP	Checked date 15/06/2023	Approved by	Approved date

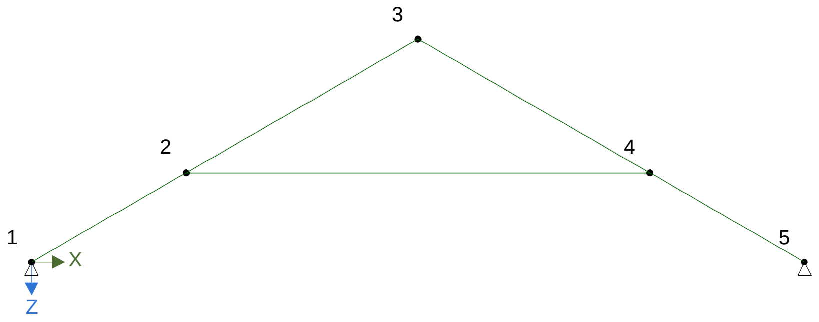
1.0G + 1.0Q + 0.5S (Service) - Total deflection



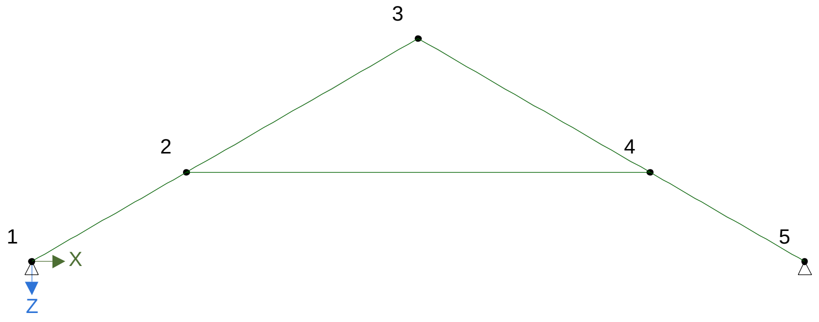
1.35G + 1.5ψ_0Q + 1.5S (Strength) - Total deflection



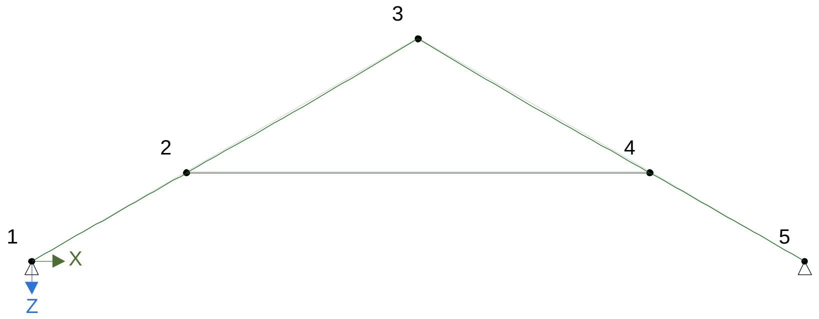
1.0G + 1.5W (Strength) - Total deflection



1.0G + 1.0W (Service) - Total deflection



1.35G + 1.5ψ_0Q + 1.5ψ_0RQ (Strength) - Total deflection



Node deflections

Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.02692	
2	0.2	0.7	0.04726	
3	0	0.5	-0.07548	
4	-0.2	0.7	-0.04726	
5	0	0	-0.02692	

Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.01828	
2	0.1	0.4	0.03201	
3	0	0.3	-0.05116	
4	-0.1	0.4	-0.03201	
5	0	0	-0.01828	

Load combination: 1.35G + 1.5Q + 1.5ψ_0S (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.02692	
2	0.2	0.7	0.04726	
3	0	0.5	-0.07548	
4	-0.2	0.7	-0.04726	
5	0	0	-0.02692	

Load combination: 1.0G + 1.0Q + 0.5S (Service)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.01828	
2	0.1	0.4	0.03201	
3	0	0.3	-0.05116	
4	-0.1	0.4	-0.03201	
5	0	0	-0.01828	

Load combination: 1.35G + 1.5ψ_0Q + 1.5S (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.0202	
2	0.1	0.5	0.03513	
3	0	0.4	-0.05622	
4	-0.1	0.5	-0.03513	
5	0	0	-0.0202	

Load combination: 1.0G + 1.5W (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.00335	
2	0	0.1	0.00505	
3	0	0.1	-0.00835	
4	0	0.1	-0.00505	
5	0	0	-0.00335	

Load combination: 1.0G + 1.0W (Service)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.00335	
2	0	0.1	0.00505	
3	0	0.1	-0.00835	
4	0	0.1	-0.00505	
5	0	0	-0.00335	

Load combination: 1.35G + 1.5ψ_0Q + 1.5ψ_0RQ (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.0202	
2	0.1	0.5	0.03513	
3	0	0.4	-0.05622	

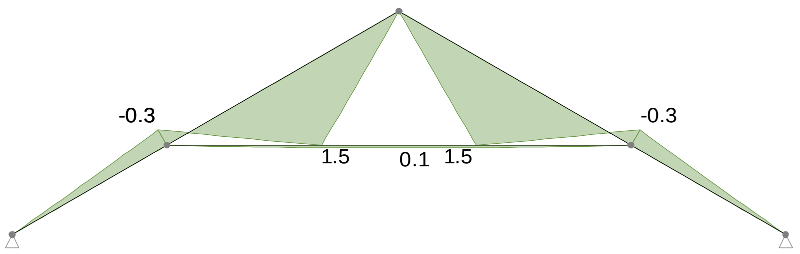
Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
4	-0.1	0.5	-0.03513	
5	0	0	-0.0202	

Total base reactions

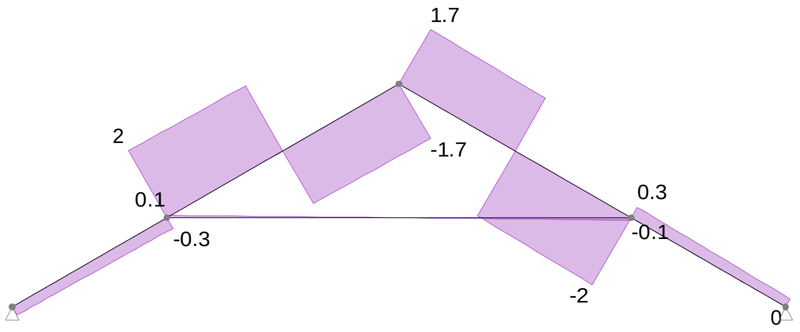
Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.5Q + 1.5RQ (Strength)	0	21.3
1.0G + 1.0Q + 1.0RQ (Service)	0	14.4
1.35G + 1.5Q + 1.5ψ_0S (Strength)	0	21.3
1.0G + 1.0Q + 0.5S (Service)	0	14.4
1.35G + 1.5ψ_0Q + 1.5S (Strength)	0	15.9
1.0G + 1.5W (Strength)	0	2.4
1.0G + 1.0W (Service)	0	2.4
1.35G + 1.5ψ_0Q + 1.5ψ_0RQ (Strength)	0	15.9

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Member results

Envelope - Strength combinations

Member	Shear force		Moment			
	Pos (m)	Max abs (kN)	Pos (m)	Max (kNm)	Pos (m)	Min (kNm)
Left Rafter	1.201	2 (max abs)	2.101	1.5 (max)	1.201	-0.3 (min)
Right Rafter	1.801	-2	0.9	1.5 (max)	1.801	-0.3 (min)
Tie	3.12	0.1	1.56	0.1	0	0

<u>Left Rafter - Span 1</u>

Partial factor for material properties and resistances

Partial factor γ_M = **1.300**

Member details

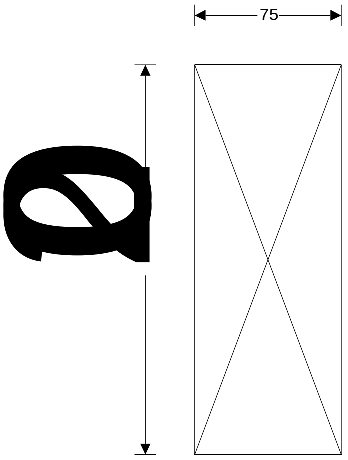
Load duration Medium-term Service class 2

Timber section details

Number of timber sections N = **1**

Breadth of sections b = **75** mm Depth of sections h = **200** mm

Timber strength class **TR26**



75x200 timber section
Cross-sectional area, A, 15000 mm^2
Section modulus, W_y, 500000 mm^3
Section modulus, W_z, 187500 mm^3
Second moment of area, I_y, 50000000 mm^4
Second moment of area, I_z, 7031250 mm^4
Radius of gyration, i_y, 57.7 mm
Radius of gyration, i_z, 21.7 mm
Timber strength class TR26
Characteristic bending strength, $f_{m.k}$, 28.3 N/mm^2
Characteristic shear strength, $f_{v.k}$, 4 N/mm^2
Characteristic compression strength parallel to grain, $f_{c.0.k}$, 22.9 N/mm^2
Characteristic compression strength perpendicular to grain, $f_{c.90.k}$, 2.6 N/mm^2
Characteristic tension strength parallel to grain, $f_{t.0.k}$, 17.6 N/mm^2
Mean modulus of elasticity, $E_{0.mean}$, 11000 N/mm^2
Fifth percentile modulus of elasticity, $E_{0.05}$, 7400 N/mm^2
Shear modulus of elasticity, G_{mean}, 690 N/mm^2
Characteristic density, ρ_k, 370 kg/m^3
Mean density, ρ_{mean}, 444 kg/m^3

Span details

Bearing length L_b = **100** mm

<u>Consider Combination 3 - 1.35G + 1.5Q + 1.5ψ_0S (Strength)</u>

Check compression parallel to the grain - cl.6.1.4

Design compressive stress $\sigma_{c,0,d}$ = **1.447** N/mm^2 Design compressive strength $f_{c,0,d}$ = **14.092** N/mm^2

Utilisation = **0.103**

PASS - Design parallel compression strength exceeds design parallel compression stress

<u>Check design at start of span</u>

Check compression perpendicular to the grain - cl.6.1.5

Des.perp.comp.stress $\sigma_{c,y,90,d}$ = **1.915** N/mm^2 Des.perp.comp.strength $f_{c,y,90,d}$ = **1.600** N/mm^2

Utilisation = **0.798**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear stress $\tau_{y,d}$ = **0.037** N/mm^2 — Design shear strength $f_{v,y,d}$ = **2.462** N/mm^2

Utilisation = **0.015**

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 $L_{e,y}$ = **2702** mm — Relative slenderness ratio $\lambda_{rel,y}$ = **0.829**

Effective length for z-axis $L_{e,z}$ = **0** mm — Relative slenderness ratio $\lambda_{rel,z}$ = **0**

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

Utilisation = **0.127**

PASS - Column stability is acceptable

<u>Consider Combination 4 - 1.0G + 1.0Q + 0.5S (Service)</u>

<u>Check design 2101 mm along span</u>

Check y-y axis deflection - Section 7.2

Final deflection without creep $\delta_{y,Final}$ = **1** mm — Allowable deflection $\delta_{y,Allowable}$ = **12** mm

Utilisation = **0.08**

PASS - Allowable deflection exceeds instantaneous deflection

<u>Right Rafter - Span 1</u>

Partial factor for material properties and resistances

Partial factor γ_M = **1.300**

Member details

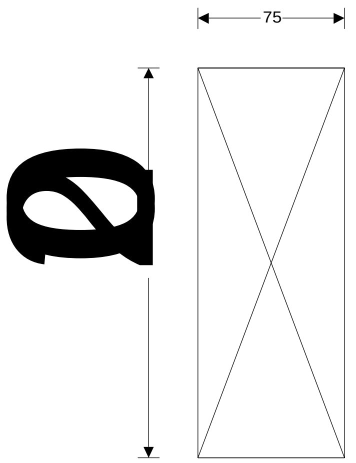
Load duration: Medium-term — Service class: 2

Timber section details

Number of timber sections N = **1**

Breadth of sections b = **75** mm — Depth of sections h = **200** mm

Timber strength class **TR26**



75x200 timber section
Cross-sectional area, A, 15000 mm^2
Section modulus, W_y, 500000 mm^3
Section modulus, W_z, 187500 mm^3
Second moment of area, I_y, 50000000 mm^4
Second moment of area, I_z, 7031250 mm^4
Radius of gyration, i_y, 57.7 mm
Radius of gyration, i_z, 21.7 mm
Timber strength class TR26
Characteristic bending strength, $f_{m.k}$, 28.3 N/mm^2
Characteristic shear strength, $f_{v.k}$, 4 N/mm^2
Characteristic compression strength parallel to grain, $f_{c.0.k}$, 22.9 N/mm^2
Characteristic compression strength perpendicular to grain, $f_{c.90.k}$, 2.6 N/mm^2
Characteristic tension strength parallel to grain, $f_{t.0.k}$, 17.6 N/mm^2
Mean modulus of elasticity, $E_{0.mean}$, 11000 N/mm^2
Fifth percentile modulus of elasticity, $E_{0.05}$, 7400 N/mm^2
Shear modulus of elasticity, G_{mean}, 690 N/mm^2
Characteristic density, r_k, 370 kg/m^3
Mean density, r_{mean}, 444 kg/m^3

Span details

Bearing length L_b = **100** mm

Consider Combination 3 - 1.35G + 1.5Q + 1.5ψ₀S (Strength)

Check compression parallel to the grain - cl.6.1.4

Design compressive stress $\sigma_{c,0,d}$ = **0.468** N/mm^2 Design compressive strength $f_{c,0,d}$ = **14.092** N/mm^2

Utilisation = **0.033**

PASS - Design parallel compression strength exceeds design parallel compression stress

Check design 900 mm along span

Check shear force - Section 6.1.7

Design shear stress $\tau_{y,d}$ = **0.296** N/mm^2 Design shear strength $f_{v,y,d}$ = **2.462** N/mm^2

Utilisation = **0.120**

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending stress $\sigma_{m,y,d}$ = **2.94** N/mm^2 Design bending strength $f_{m,y,d}$ = **17.415** N/mm^2

Utilisation = **0.169**

PASS - Design bending strength exceeds design bending stress

Check combined bending and axial compression - Section 6.2.4

Utilisation = **0.171**

PASS - Combined bending and axial compression utilisation is acceptable

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

Effective length for y-axis $L_{e,y}$ = **2702** mm Relative slenderness ratio $\lambda_{rel,y}$ = **0.829**

Effective length for z-axis $L_{e,z}$ = **0** mm Relative slenderness ratio $\lambda_{rel,z}$ = **0**

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

Utilisation = **0.222**

PASS - Column stability is acceptable

Consider Combination 4 - 1.0G + 1.0Q + 0.5S (Service)

Check design 900 mm along span

Check y-y axis deflection - Section 7.2

Final deflection without creep $\delta_{y,Final}$ = **1** mm Allowable deflection $\delta_{y,Allowable}$ = **12** mm

Utilisation = **0.08**

PASS - Allowable deflection exceeds instantaneous deflection

Tie - Span 1

Partial factor for material properties and resistances

Partial factor γ_M = **1.300**

Member details

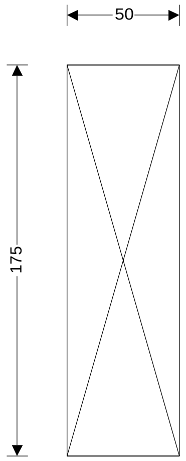
Load duration Medium-term Service class 1

Timber section details

Number of timber sections N = **1**

Breadth of sections b = **50** mm Depth of sections h = **175** mm

Timber strength class **TR26**



50x175 timber section
Cross-sectional area, A, 8750 mm^2
Section modulus, W_y, 255208.3 mm^3
Section modulus, W_z, 72917 mm^3
Second moment of area, I_y, 22330729 mm^4
Second moment of area, I_z, 1822917 mm^4
Radius of gyration, i_y, 50.5 mm
Radius of gyration, i_z, 14.4 mm
Timber strength class TR26
Characteristic bending strength, $f_{m.k}$, 28.3 N/mm^2
Characteristic shear strength, $f_{v.k}$, 4 N/mm^2
Characteristic compression strength parallel to grain, $f_{c.0.k}$, 22.9 N/mm^2
Characteristic compression strength perpendicular to grain, $f_{c.90.k}$, 2.6 N/mm^2
Characteristic tension strength parallel to grain, $f_{t.0.k}$, 17.6 N/mm^2
Mean modulus of elasticity, $E_{0.mean}$, 11000 N/mm^2
Fifth percentile modulus of elasticity, $E_{0.05}$, 7400 N/mm^2
Shear modulus of elasticity, G_{mean}, 690 N/mm^2
Characteristic density, r_k, 370 kg/m^3
Mean density, r_{mean}, 444 kg/m^3

Span details

Bearing length L_b = **100** mm

<u>Consider Combination 3 - 1.35G + 1.5Q + 1.5ψ_0S (Strength)</u>

Check compression parallel to the grain - cl.6.1.4

Design compressive stress $\sigma_{c,0,d}$ = **1.373** N/mm^2 Design compressive strength $f_{c,0,d}$ = **14.092** N/mm^2

Utilisation = **0.097**

PASS - Design parallel compression strength exceeds design parallel compression stress

<u>Check design 1560 mm along span</u>

Check bending moment - Section 6.1.6

Design bending stress $\sigma_{m,y,d}$ = **0.204** N/mm^2 Design bending strength $f_{m,y,d}$ = **17.415** N/mm^2

Utilisation = **0.012**

PASS - Design bending strength exceeds design bending stress

Check combined bending and axial compression - Section 6.2.4

Utilisation = **0.021**

PASS - Combined bending and axial compression utilisation is acceptable

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

Effective length for y-axis $L_{e,y}$ = **2808** mm Relative slenderness ratio $\lambda_{rel,y}$ = **0.984**

Effective length for z-axis $L_{e,z}$ = **0** mm Relative slenderness ratio $\lambda_{rel,z}$ = **0**

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

Utilisation = **0.151**

PASS - Column stability is acceptable

<u>Consider Combination 4 - 1.0G + 1.0Q + 0.5S (Service)</u>

<u>Check design 1560 mm along span</u>

Check y-y axis deflection - Section 7.2

Final deflection without creep $\delta_{y,Final}$ = **0.6** mm Allowable deflection $\delta_{y,Allowable}$ = **12.5** mm

Utilisation = **0.049**

PASS - Allowable deflection exceeds instantaneous deflection

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PROJECT	HAMETETHY FARM, BODMIN			
Designed by	Checked by	Date	Project no.	Sheet no.
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CALCULATIONS

MASONRY PIER

Truss Reactions (page 4)

DL = 1.2 kN
IL = 6.0 kN

Tedds Output

440x440 Masonry pier PASS.

<u>MASONRY COLUMN DESIGN</u>

In accordance with EN1996-1-1:2005 incorporating corrigenda February 2006 and July 2009 and the UK national annex

Tedds calculation version 1.0.07

<u>Design summary</u>

Description	Unit	Capacity	Maximum	Utilisation	Result
Slenderness		27.0	6.8	0.253	PASS
Lateral loading (major)	kNm	1.2	0.6	0.471	PASS
Lateral loading (minor)	kNm	1.2	0.6	0.471	PASS
Vertical loading	kN	92.8	15.0	0.161	PASS

Geometry

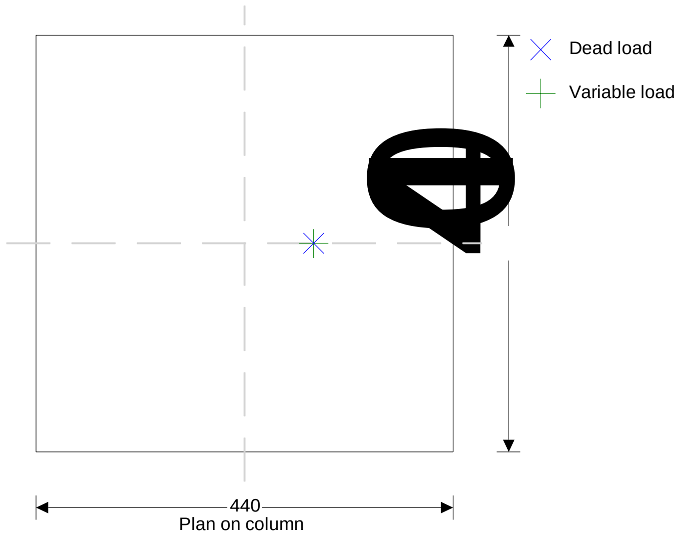
Width of column — b = **440** mm

Thickness of column — t = **440** mm

Height of column — h = **3000** mm

Reduction factor for effective height — ρ_2 = **1.0**

Effective height of column (cl 5.5.1.2) — $h_{eff} = h \times \rho_2$ = **3000** mm



Plan on column

Loading

Vertical dead load — G_k = **1.2** kN

Eccentricity of dead load in x-direction — e_{Gb} = **73** mm

Eccentricity of dead load in y-direction — e_{Gt} = **0** mm

Vertical live load — Q_k = **6.0** kN

Eccentricity of variable load in x-direction — e_{Qb} = **73** mm

Eccentricity of variable load in y-direction — e_{Qt} = **0** mm

Characteristic wind loading — W_k = **0.8** kN/m^2

Vertical wind loading — W_v = **0.0** kN

Masonry details

Masonry type — **Aggregate concrete - Group 1**

Compressive strength of masonry unit — f_c = **5.2** N/mm^2

Height of unit — h_u = **215** mm

Width of unit — w_u = **100** mm

Conditioning factor — k = **1.0**

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

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

Mean compressive strength of masonry unit — $f_b = f_c \times k \times d_{sf}$ = **7.176** N/mm^2

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

Mortar type — **M4 - General purpose mortar**

Compressive strength of masonry mortar — f_m = **4** N/mm^2

Compressive strength factor - Table NA.4 — K = **0.75**

Characteristic compressive strength of masonry - eq 3.1

$f_k = K \times f_b^{0.7} \times f_m^{0.3}$ = **4.516** N/mm^2

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

f_{xk1} = **0.15** N/mm^2

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

f_{xk2} = **0.347** N/mm^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 — γ_{Mc} = **3.00**

Slenderness ratio

Slenderness ratio minor axis (cl.5.5.2.1) — $\lambda_t = h_{eff} / t$ = **6.82**

Slenderness ratio major axis (cl.5.5.2.1) — $\lambda_b = h_{eff} / b$ = **6.82**

Maximum slenderness — $\lambda = \max(\lambda_t, \lambda_b)$ = **6.82**

PASS - Slenderness ratio is less than 27

Load combinations derived from Eq 6.10 lateral and vertical loading (utilisation)

Combination 1 — 1.35 × perm unfav + 1 × perm fav + 1.5 × variable + 1.5 × 0.5 × wind (0.117)

Combination 2 — 1.35 × perm unfav + 1 × perm fav + 1.5 × 0.7 × variable + 1.5 × wind (0.161)

The following output relates to combination 2

Reduction factor for slenderness and eccentricity about the major axis - Section 6.1.2.2

Design bending moment top or bottom of column — $M_{idb} = \mathrm{abs}(\gamma_{fGv} \times G_k \times e_{Gb} + \gamma_{fQv} \times Q_k \times e_{Qb})$ = **0.578** kNm

Design vertical load at top or bottom of column — $N_{idb} = \mathrm{abs}(\gamma_{fGv} \times G_k + \gamma_{fQv} \times Q_k)$ = **7.92** kN

Initial eccentricity - cl.5.5.1.1 — $e_{init} = h_{eff} / 450$ = **6.7** mm

Conservativley assume moment due to wind load at the top of the column is equal to that at mid height

Design moment due to horizontal load — $M_{Edb} = \gamma_{fWh} \times \alpha \times W_k \times h^2 \times t$ = **0.557** kNm

Eccentricity due to horizontal load — $e_{hb} = M_{Edb} / N_{idb}$ = **70.3** mm

Eccentricity at top or bottom of column - eq.6.5 — $e_{ib} = \max(M_{idb} / N_{idb} + e_{hb} + e_{init}, 0.05 \times b)$ = **150.0** mm

Reduction factor top or bottom of column - eq.6.4 — $\Phi_{ib} = \max(1 - 2 \times e_{ib} / b, 0)$ = **0.318**

Ratio of top and middle mnts due to eccentricity — α_{mdb} = **1.0**

Design bending moment at middle of column — $M_{mdb} = \alpha_{mdb} \times \mathrm{abs}(\gamma_{fGv} \times G_k \times e_{Gb} + \gamma_{fQv} \times Q_k \times e_{Qb})$ = **0.578** kNm

Design vertical load at middle of column — $N_{mdb} = \gamma_{fGv} \times G_k + \gamma_{fQv} \times Q_k + \gamma_{fGv} \times t \times b \times \gamma \times h / 2$ = **14.977** kN

Eccentricity due to horizontal load — $e_{hmb} = M_{Edb} / N_{mdb}$ = **37.2** mm

Eccentricity middle of column due to loads - eq.6.7 — $e_{mb} = M_{mdb} / N_{mdb} + e_{hmb} + e_{init}$ = **82.5** mm

Eccentricity at middle of column due to creep — e_{kb} = **0.0** mm

Eccentricity at middle of column - eq.6.6 — $e_{mkb} = \max(e_{mb} + e_{kb}, 0.05 \times b)$ = **82.5** mm

From eq.G.2 — $A_{1b} = 1 - 2 \times e_{mkb} / b$ = **0.625**

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

Modulus of elasticity - cl.3.7.2 — $E = K_E \times f_k$ = **4516** N/mm^2

Slenderness - eq.G.4 $\lambda_b = (h_{eff} / b) \times \sqrt{}(f_k / E) =$ **0.216**

From eq.G.3 $u_b = (\lambda_b - 0.063) / (0.73 - 1.17 \times e_{mkb} / b) =$ **0.299**

Reduction factor at middle of column - eq.G.1 $\Phi_{mb} = \max(A_{1b} \times e_e^{-(u_b \times u_b)/2}, 0) =$ **0.598**

Reduction factor for slenderness and eccentricity $\Phi_b = \min(\Phi_{ib}, \Phi_{mb}) =$ **0.318**

Reduction factor for slenderness and eccentricity about the minor axis - Section 6.1.2.2

Design bending moment top or bottom of column $M_{idt} = abs(\gamma_{fGv} \times G_k \times e_{Gt} + \gamma_{fQv} \times Q_k \times e_{Qt}) =$ **0** kNm

Design vertical load at top or bottom of column $N_{idt} = abs(\gamma_{fGv} \times G_k + \gamma_{fQv} \times Q_k) =$ **7.92** kN

Initial eccentricity - cl.5.5.1.1 $e_{init} = h_{eff} / 450 =$ **6.7** mm

Conservativley assume moment due to wind load at the top of the column is equal to that at mid height

Design moment due to horizontal load $M_{Edt} = \gamma_{fWh} \times \alpha \times W_k \times h^2 \times b =$ **0.557** kNm

Eccentricity due to horizontal load $e_{ht} = M_{Edt} / N_{idt} =$ **70.3** mm

Eccentricity at top or bottom of column - eq.6.5 $e_{it} = \max(M_{idt} / N_{idt} + e_{ht} + e_{init}, 0.05 \times t) =$ **77.0** mm

Reduction factor top or bottom of column - eq.6.4 $\Phi_{it} = \max(1 - 2 \times e_{it} / t, 0) =$ **0.65**

Ratio of top and middle mnts due to eccentricity $\alpha_{mdt} =$ **1.0**

Design bending moment at middle of column $M_{mdt} = \alpha_{mdt} \times abs(\gamma_{fGv} \times G_k \times e_{Gt} + \gamma_{fQv} \times Q_k \times e_{Qt}) =$ **0** kNm

Design vertical load at middle of column $N_{mdt} = \gamma_{fGv} \times G_k + \gamma_{fQv} \times Q_k + \gamma_{fGv} \times t \times b \times \gamma \times h / 2 =$ **14.977** kN

Eccentricity due to horizontal load $e_{hmt} = M_{Edt} / N_{mdt} =$ **37.2** mm

Eccentricity middle of column due to loads - eq.6.7 $e_{mt} = M_{mdt} / N_{mdt} + e_{hmt} + e_{init} =$ **43.8** mm

Eccentricity at middle of column due to creep $e_{kt} =$ **0.0** mm

Eccentricity at middle of column - eq.6.6 $e_{mkt} = \max(e_{mt} + e_{kt}, 0.05 \times t) =$ **43.8** mm

From eq.G.2 $A_{1t} = 1 - 2 \times e_{mkt} / t =$ **0.801**

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

Modulus of elasticity - cl.3.7.2 $E = K_E \times f_k =$ **4516** N/mm^2

Slenderness - eq.G.4 $\lambda_t = (h_{eff} / t) \times \sqrt{}(f_k / E) =$ **0.216**

From eq.G.3 $u_t = (\lambda_t - 0.063) / (0.73 - 1.17 \times e_{mkt} / t) =$ **0.249**

Reduction factor at middle of column - eq.G.1 $\Phi_{mt} = \max(A_{1t} \times e_e^{-(u_t \times u_t)/2}, 0) =$ **0.776**

Reduction factor for slenderness and eccentricity $\Phi_t = \min(\Phi_{it}, \Phi_{mt}) =$ **0.65**

Columns subjected to mainly vertical loading - Section 6.1.2

Design value of the vertical load $N_{Ed} = \max(N_{idb}, N_{mdb}, N_{idt}, N_{mdt}) =$ **14.977** kN

Design compressive strength of masonry $f_d = f_k / \gamma_{Mc} =$ **1.505** N/mm^2

Vertical resistance of column - eq.6.2 $N_{Rd} = \min(\Phi_t, \Phi_b) \times t \times b \times f_d =$ **92.765** kN

Utilisation $N_{Ed} / N_{Rd} = 0.161$

PASS - Design vertical resistance exceeds applied design vertical load

Load combinations derived from Eq 6.10 lateral and vertical loading (utilisation)

Combination 1 1.35 × perm unfav + 1 × perm fav + 1.5 × variable + 1.5 × 0.5 × wind (0.236)

Combination 2 1.35 × perm unfav + 1 × perm fav + 1.5 × 0.7 × variable + 1.5 × wind (0.471)

The following output relates to combination 2

Reduction factor for slenderness and eccentricity about the major axis - Section 6.1.2.2

Design bending moment top or bottom of column $M_{idb} = abs(\gamma_{fGv} \times G_k \times e_{Gb} + \gamma_{fQv} \times Q_k \times e_{Qb}) =$ **0.578** kNm

Design vertical load at top or bottom of column $N_{idb} = abs(\gamma_{fGv} \times G_k + \gamma_{fQv} \times Q_k) =$ **7.92** kN

Initial eccentricity - cl.5.5.1.1 $e_{init} = h_{eff} / 450 =$ **6.7** mm

Conservativley assume moment due to wind load at the top of the column is equal to that at mid height

Design moment due to horizontal load $M_{Edb} = \gamma_{fWh} \times \alpha \times W_k \times h^2 \times t =$ **0.557** kNm

Eccentricity due to horizontal load $e_{hb} = M_{Edb} / N_{idb} =$ **70.3** mm

Eccentricity at top or bottom of column - eq.6.5 — $e_{ib} = \max(M_{idb} / N_{idb} + e_{hb} + e_{init}, 0.05 \times b) = \mathbf{150.0}$ mm

Reduction factor top or bottom of column - eq.6.4 — $\Phi_{ib} = \max(1 - 2 \times e_{ib} / b, 0) = \mathbf{0.318}$

Ratio of top and middle mnts due to eccentricity — $\alpha_{mdb} = \mathbf{1.0}$

Design bending moment at middle of column — $M_{mdb} = \alpha_{mdb} \times abs(\gamma_{fGv} \times G_k \times e_{Gb} + \gamma_{fQv} \times Q_k \times e_{Qb}) = \mathbf{0.578}$ kNm

Design vertical load at middle of column — $N_{mdb} = \gamma_{fGv} \times G_k + \gamma_{fQv} \times Q_k + \gamma_{fGv} \times t \times b \times \gamma \times h / 2 = \mathbf{14.977}$ kN

Eccentricity due to horizontal load — $e_{hmb} = M_{Edb} / N_{mdb} = \mathbf{37.2}$ mm

Eccentricity middle of column due to loads - eq.6.7 — $e_{mb} = M_{mdb} / N_{mdb} + e_{hmb} + e_{init} = \mathbf{82.5}$ mm

Eccentricity at middle of column due to creep — $e_{kb} = \mathbf{0.0}$ mm

Eccentricity at middle of column - eq.6.6 — $e_{mkb} = \max(e_{mb} + e_{kb}, 0.05 \times b) = \mathbf{82.5}$ mm

From eq.G.2 — $A_{1b} = 1 - 2 \times e_{mkb} / b = \mathbf{0.625}$

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

Modulus of elasticity - cl.3.7.2 — $E = K_E \times f_k = \mathbf{4516}$ N/mm^2

Slenderness - eq.G.4 — $\lambda_b = (h_{eff} / b) \times \sqrt{(f_k / E)} = \mathbf{0.216}$

From eq.G.3 — $u_b = (\lambda_b - 0.063) / (0.73 - 1.17 \times e_{mkb} / b) = \mathbf{0.299}$

Reduction factor at middle of column - eq.G.1 — $\Phi_{mb} = \max(A_{1b} \times e_e^{-(u_b \times u_b)/2}, 0) = \mathbf{0.598}$

Reduction factor for slenderness and eccentricity — $\Phi_b = \min(\Phi_{ib}, \Phi_{mb}) = \mathbf{0.318}$

Reduction factor for slenderness and eccentricity about the minor axis - Section 6.1.2.2

Design bending moment top or bottom of column — $M_{idt} = abs(\gamma_{fGv} \times G_k \times e_{Gt} + \gamma_{fQv} \times Q_k \times e_{Qt}) = \mathbf{0}$ kNm

Design vertical load at top or bottom of column — $N_{idt} = abs(\gamma_{fGv} \times G_k + \gamma_{fQv} \times Q_k) = \mathbf{7.92}$ kN

Initial eccentricity - cl.5.5.1.1 — $e_{init} = h_{eff} / 450 = \mathbf{6.7}$ mm

Conservativley assume moment due to wind load at the top of the column is equal to that at mid height

Design moment due to horizontal load — $M_{Edt} = \gamma_{fWh} \times \alpha \times W_k \times h^2 \times b = \mathbf{0.557}$ kNm

Eccentricity due to horizontal load — $e_{ht} = M_{Edt} / N_{idt} = \mathbf{70.3}$ mm

Eccentricity at top or bottom of column - eq.6.5 — $e_{it} = \max(M_{idt} / N_{idt} + e_{ht} + e_{init}, 0.05 \times t) = \mathbf{77.0}$ mm

Reduction factor top or bottom of column - eq.6.4 — $\Phi_{it} = \max(1 - 2 \times e_{it} / t, 0) = \mathbf{0.65}$

Ratio of top and middle mnts due to eccentricity — $\alpha_{mdt} = \mathbf{1.0}$

Design bending moment at middle of column — $M_{mdt} = \alpha_{mdt} \times abs(\gamma_{fGv} \times G_k \times e_{Gt} + \gamma_{fQv} \times Q_k \times e_{Qt}) = \mathbf{0}$ kNm

Design vertical load at middle of column — $N_{mdt} = \gamma_{fGv} \times G_k + \gamma_{fQv} \times Q_k + \gamma_{fGv} \times t \times b \times \gamma \times h / 2 = \mathbf{14.977}$ kN

Eccentricity due to horizontal load — $e_{hmt} = M_{Edt} / N_{mdt} = \mathbf{37.2}$ mm

Eccentricity middle of column due to loads - eq.6.7 — $e_{mt} = M_{mdt} / N_{mdt} + e_{hmt} + e_{init} = \mathbf{43.8}$ mm

Eccentricity at middle of column due to creep — $e_{kt} = \mathbf{0.0}$ mm

Eccentricity at middle of column - eq.6.6 — $e_{mkt} = \max(e_{mt} + e_{kt}, 0.05 \times t) = \mathbf{43.8}$ mm

From eq.G.2 — $A_{1t} = 1 - 2 \times e_{mkt} / t = \mathbf{0.801}$

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

Modulus of elasticity - cl.3.7.2 — $E = K_E \times f_k = \mathbf{4516}$ N/mm^2

Slenderness - eq.G.4 — $\lambda_t = (h_{eff} / t) \times \sqrt{(f_k / E)} = \mathbf{0.216}$

From eq.G.3 — $u_t = (\lambda_t - 0.063) / (0.73 - 1.17 \times e_{mkt} / t) = \mathbf{0.249}$

Reduction factor at middle of column - eq.G.1 — $\Phi_{mt} = \max(A_{1t} \times e_e^{-(u_t \times u_t)/2}, 0) = \mathbf{0.776}$

Reduction factor for slenderness and eccentricity — $\Phi_t = \min(\Phi_{it}, \Phi_{mt}) = \mathbf{0.65}$

Columns subjected to mainly vertical loading - Section 6.1.2

Design value of the vertical load — $N_{Ed} = \max(N_{idb}, N_{mdb}, N_{idt}, N_{mdt}) = \mathbf{14.977}$ kN

Design compressive strength of masonry — $f_d = f_k / \gamma_{Mc} = \mathbf{1.505}$ N/mm^2

Vertical resistance of column - eq.6.2 — $N_{Rd} = \min(\Phi_t, \Phi_b) \times t \times b \times f_d = \mathbf{92.765}$ kN

Utilisation — $N_{Ed} / N_{Rd} = 0.161$

PASS - Design vertical resistance exceeds applied design vertical load

Flexural strength of masonry

Self weight at middle of column $S_{wt} = 0.5 \times h \times t \times b \times \gamma =$ **5.227** kN

Design compressive strength of masonry $f_d = f_k / \gamma_{Mc} =$ **1.505** N/mm^2

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

Design flex masonry strength parallel to bed joints $f_{xd1} = f_{xk1} / \gamma_{Mc} =$ **0.05** N/mm^2

Apparent design flex strength parallel to bed joints $f_{xd1,app} = f_{xd1} + \sigma_d =$ **0.083** N/mm^2

Column subject to lateral loading about the major axis - Section 6.3

Elastic section modulus of column $Z_b = t \times b^2 / 6 =$ **14197333** mm^3

Moment of resist parallel to bed joints - eq.6.15 $M_{Rd1b} = f_{xd1,app} \times Z_b =$ **1.181** kNm

Bending moment coefficient $\alpha =$ **0.125**

Design moment in column $M_{Edb} = \gamma_{fWh} \times \alpha \times W_k \times h^2 \times t =$ **0.557** kNm

$M_{Edb} / M_{Rd1b} =$ **0.471**

PASS - Moment resistance greater than design moment in column

Column subject to lateral loading about the minor axis - Section 6.3

Elastic section modulus of column $Z_t = b \times t^2 / 6 =$ **14197333** mm^3

Moment of resist parallel to bed joints - eq.6.15 $M_{Rd1t} = f_{xd1,app} \times Z_t =$ **1.181** kNm

Bending moment coefficient $\alpha =$ **0.125**

Design moment in column $M_{Edt} = \gamma_{fWh} \times \alpha \times W_k \times h^2 \times b =$ **0.557** kNm

$M_{Edt} / M_{Rd1t} =$ **0.471**

PASS - Moment resistance greater than design moment in column

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PROJECT	HAMETETHY FARM, BODMIN			
Designed by	Checked by	Date	Project no.	Sheet no.
AM	MWP	JUNE 2023	23111	20

CALCULATIONS

WALL CHECK

Roof DL = 0.1 kN/m²
IL = 0.6 kN/m²

Supported width = 0.5m

loading DL = 0.05 kN/m
IL = 0.3 kN/m

Tekla Output

150mm 7N Blockwork wall PASS

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 1.2.20

Summary table

	Allowable	Actual	Utilisation	
Slenderness ratio	27;	20.0;	0.741;	PASS
Vertical loading on wall	82.735 kN/m;	11.453 kN/m;	0.138;	PASS
Height to thickness ratio	80.000;	20.000;	0.250;	PASS
Design moment to wall	0.718 kNm/m;	0.686 kNm/m;	0.956;	PASS

Masonry panel details

Unreinforced masonry wall without openings

Panel length L = **4000** mm

Panel height h = **3000** mm

Panel support conditions

Right, left and bottom supported continuously

Effective height of masonry walls - Section 5.5.1.2

Reduction factor ρ_2 = **1.000**

Effective height of wall - eq 5.2 $h_{ef} = \rho_2 \times h$ = **3000** mm

Martin Perry Associates Parade House, The Parade Liskeard, Cornwall PL14 6AH	Project HAMETETHY FARM, BODMIN				Job no. 23111	
	Calcs for EXISTING MASONRY WALL PANEL CHECK				Start page no./Revision 22	
	Calcs by AM	Calcs date 15/06/2023	Checked by MWP	Checked date 15/06/2023	Approved by	Approved date

Masonry details

Masonry type **Aggregate concrete - Group 1**

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

Height of unit h_u = **215** mm

Width of unit w_u = **150** mm

Conditioning factor k = **1.0**

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

Shape factor - Table A.1 d_{sf} = **1.28**

Norm. mean compressive strength of masonry $f_b = f_c \times k \times d_{sf}$ = **9.344** N/mm^2

Density of masonry γ = **18** kN/m^3

Mortar type **M4 - General purpose mortar**

Compressive strength of masonry mortar f_m = **4** N/mm^2

Compressive strength factor - Table NA.4 K = **0.75**

Characteristic compressive strength of masonry - eq 3.1

$f_k = K \times f_b^{0.7} \times f_m^{0.3}$ = **5.433** N/mm^2

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

f_{xk1} = **0.217** N/mm^2

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

f_{xk2} = **0.517** N/mm^2

Lateral loading details

Characteristic wind load on panel W_k = **0.750** kN/m^2

Vertical loading details

Permanent load on top of wall G_k = **0.05** kN/m at an eccentricity of 25 mm

Variable load on top of wall Q_k = **0.3** kN/m at an eccentricity of 25 mm

Partial factors for material strength

Category of manufacturing control **Category II**

Class of execution control **Class 2**

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

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

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

Slenderness ratio of masonry walls - Section 5.5.1.4

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

Slenderness ratio $SR = h_{ef} / t_{ef}$ = **20.0**

PASS - Slenderness ratio is less than maximum allowable

Unreinforced masonry walls subjected to mainly vertical loading - Section 6.1

Partial safety factors for design loads

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

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

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

Check vertical loads

Reduction factor for slenderness and eccentricity - Section 6.1.2.2

Vertical load at bottom of wall $N_{id} = \gamma_{fG} \times (G_k + \gamma \times t \times h) + \gamma_{fQ} \times Q_k$ = **11.453** kN/m

Moment at bottom of wall due to vertical load $M_{id} = \gamma_{fG} \times G_k \times e_G + \gamma_{fQ} \times Q_k \times e_Q$ = **0.013** kNm/m

Initial eccentricity - cl.5.5.1.1 $e_{init} = h_{ef} / 450$ = **6.7** mm

Moment at bottom of wall due to horizontal load M_{Eid} = **0.129** kNm/m

Eccentricity at bottom of wall due to horizontal load $e_h = M_{Eid} / N_{id}$ = **11.3** mm

Eccentricity at bottom of wall - eq.6.5 $e_i = \max(M_{id} / N_{id} + e_h + e_{init}, 0.05 \times t)$ = **19.1** mm

Reduction factor at bottom of wall - eq.6.4 $\Phi_i = \max(1 - 2 \times e_i / t, 0)$ = **0.745**

Vertical load at middle of wall $N_{md} = \gamma_{fG} \times (G_k + \gamma \times t \times h / 2) + \gamma_{fQ} \times Q_k$ = **5.985** kN/m

Moment at middle of wall due to vertical load $M_{md} = \gamma_{fG} \times G_k \times e_G + \gamma_{fQ} \times Q_k \times e_Q$ = **0.013** kNm/m

Moment at middle of wall due to horizontal load M_{Emd} = **0.129** kNm/m

Eccentricity at middle of wall due to horizontal load $e_{hm} = M_{Emd} / N_{md}$ = **21.6** mm

Eccentricity at middle of wall due to loads - eq.6.7 $e_m = M_{md} / N_{md} + e_{hm} + e_{init}$ = **30.4** mm

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

Eccentricity at middle of wall - eq.6.6 $e_{mk} = \max(e_m + e_k, 0.05 \times t)$ = **30.4** mm

From eq.G.2 $A_1 = 1 - 2 \times e_{mk} / t$ = **0.594**

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

Modulus of elasticity - cl.3.7.2 $E = K_E \times f_k$ = **5433** N/mm^2

Slenderness - eq.G.4 $\lambda = (h_{ef} / t_{ef}) \times \sqrt{(f_k / E)}$ = **0.632**

From eq.G.3 $u = (\lambda - 0.063) / (0.73 - 1.17 \times e_{mk} / t)$ = **1.156**

Reduction factor at middle of wall - eq.G.1 $\Phi_m = \max(A_1 \times e_e^{-(u \times u)/2}, 0)$ = **0.305**

Reduction factor for slenderness and eccentricity $\Phi = \min(\Phi_i, \Phi_m)$ = **0.305**

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

Design value of the vertical load $N_{Ed} = \max(N_{id}, N_{md})$ = **11.453** kN/m

Design compressive strength of masonry $f_d = f_k / \gamma_{Mc}$ = **1.811** N/mm^2

Vertical resistance of wall - eq.6.2 $N_{Rd} = \Phi \times t \times f_d$ = **82.735** kN/m

PASS - Design vertical resistance exceeds applied design vertical load

Unreinforced masonry walls subjected to lateral loading - Section 6.3

Partial safety factors for design loads

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

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

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

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

Length to thickness ratio L / t = **26.667**

Limiting height to thickness ratio - Figure F.3 80

Height to thickness ratio h / t = **20**

PASS - Limiting height to thickness ratio is not exceeded

Design moments of resistance in panels

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

Design compressive strength of masonry $f_d = f_k / \gamma_{Mc}$ = **1.811** N/mm^2

Design vertical compressive stress $\sigma_d = \min(\gamma_{fG} \times (G_k + S_{wt}) / t, 0.15 \times \Phi \times f_d)$ = **0** N/mm^2

Design flexural strength of masonry parallel to bed joints

$f_{xd1} = f_{xk1} / \gamma_{Mt}$ = **0.08** N/mm^2

Apparent design flexural strength of masonry parallel to bed joints

$f_{xd1,app} = f_{xd1} + \sigma_d$ = **0.081** N/mm^2

Design flexural strength of masonry perpendicular to bed joints

$f_{xd2} = f_{xk2} / \gamma_{Mt}$ = **0.191** N/mm^2

Elastic section modulus of wall $Z = t^2 / 6$ = **3750000** mm^3/m

Moment of resistance parallel to bed joints - eq.6.15

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

Moment of resistance perpendicular to bed joints - eq.6.15

$M_{Rd2} = f_{xd2} \times Z$ = **0.718** kNm/m

Design moment in panels

Orthogonal strength ratio $\mu = f_{xd1,app} / f_{xd2}$ = **0.42**

Using yield line analysis to calculate bending moment coefficient

Bending moment coefficient α = **0.038**

Design moment in wall $M_{Ed} = \gamma_{fW} \times \alpha \times W_k \times L^2$ = **0.686** kNm/m

PASS - Resistance moment exceeds design moment