

Project Ref: 23111

M Perry Associates Ltd Parade House, The Parade Liskeard, Cornwall PL14 6AH

Telephone: 01579 345777 mail@mperryassociates.com www.mperryassociates.com

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



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 QA.19-V3-27.04.2022

The Institution of StructuralEngineers







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 <u>GENERAL</u>
- 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 <u>OBSERVATIONS</u>

- 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 <u>CONCLUSIONS</u>

- 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.

SIGNED:	

DATED: 8th November 2023



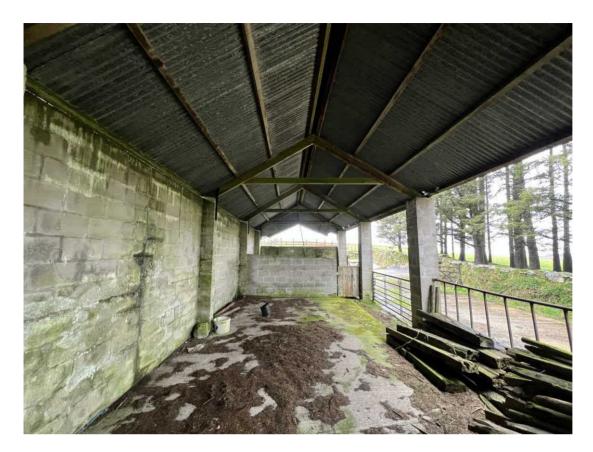


Figure 1 – General internal view.



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.





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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

The Institution of StructuralEngineers







	PROJECT TITLE HAMETETH	PROJECT: 23111	
	SECTION	SHEET: DP	
Martin Perry Associates Structural Engineering & Surveying	CALCS BY: AM	CHECKED BY: MWP	date: JUNE 2023

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

	PROJECT TITLE HAMETETH	Y FARM, BODMIN	PROJECT: 23111
	SECTION DESIGNER F	SHEET: DRA	
Martin Perry Associates Structural Engineering & Surveying	CALCS BY: AM	CHECKED BY: MWP	date: JUNE 2023

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:

- The works are to be carried out by an experienced competent contractor capable of managing general construction hazards.
 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):
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	Temp. stability /	Access	Risks to general	Asbestos	Confined spaces	Future				
	propping		public			maintenance				
	Hidden / buried	Material handling	Hot works	Working at height	COSHH	Dismantling /				
	services					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.

	PROJECT TITLE HAMETETHY F	PROJECT: 23111	
	STANDARD DATA SHEET 2 - EUROCODE		SHEET: 2
Martin Perry Associates Structural Engineering & Surveying	CALCS BY: AM	CHECKED BY: MWP	DATE: JUNE 2023

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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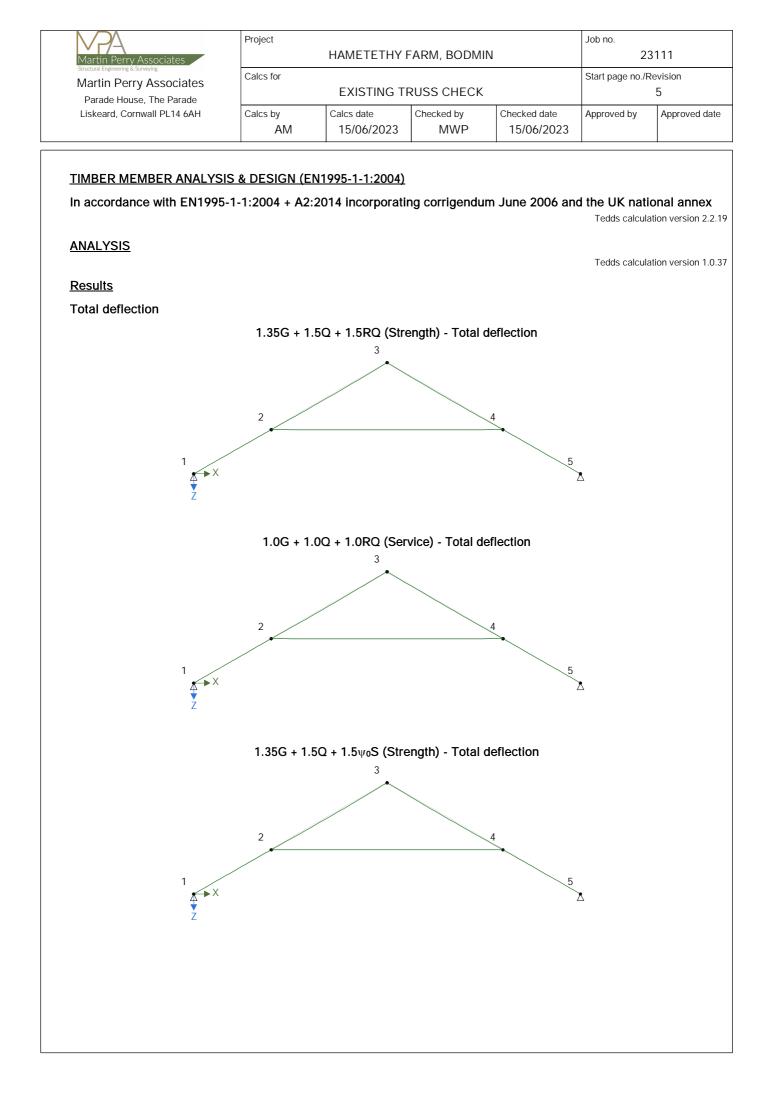
PROJECT	HAMETETHY F	FARM, BODMIN		
Designed by AM	Checked by MWP	Date Jun 2023	Project no. 23111	Sheet no. 3
		CALCULATIONS	:/	ň
GENERAL LOA	DINGS			
		now/Access prrugated sheet roofing	0.60 kN/m 0.10 kN/m	
		an agailea chicet i connig		
Masonry Pei	manent Load		2.30 kN/m	J ²

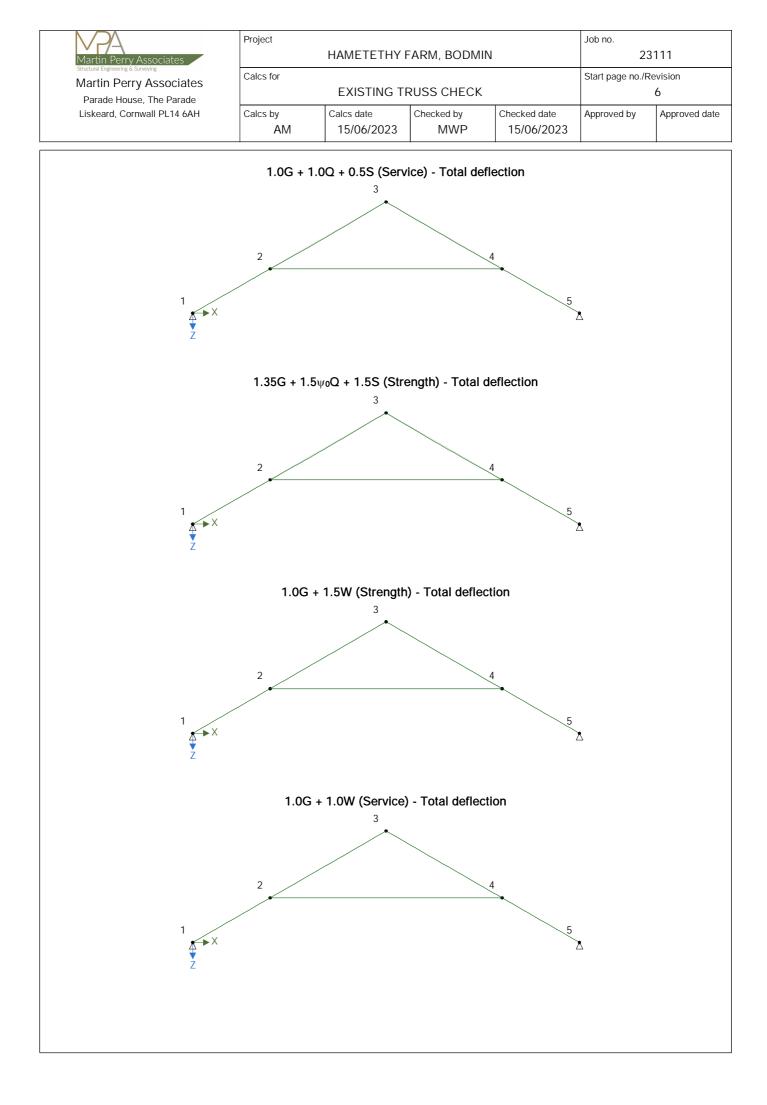


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PROJECT	HAMETETHY FAR	M BODMIN		
Designed by	Checked by	Date VUNE 2023	Project no. 23111	Sheet no. 4-
		CALCULATIONS		
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Rushin langth = 4m	D	À		
Point loads from pur	$\int_{L} \frac{\partial}{\partial t} k = 0.2 \beta \lambda$ $IL = 1.2 \beta \lambda$	$x_2 = 0.4 \text{EN}$ $x_2 = 2.4 \text{EN}$		
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	<u>/</u>	P	roject			N	Job no.	3111
Martin Structural En	Perry Associate			HAMETETHY	ARINI, BODIVII	N		
Martin Perry Associates Parade House, The Parade			Calcs for EXISTING TRUSS CHECK			Start page no./Revision 7		
	rd, Cornwall PL14		alcs by	Calcs date	Checked by	Checked date	Approved by	Approved dat
			AM	15/06/2023	MWP	15/06/2023		
		1.	35G + 1.5ψ₀C	2 + 1.5ψοRQ (S	trength) - Tota	al deflection		
			2			4		
		1 X				5	*	
							Δ	
		Z						
Node o	deflections							
Load c	ombination: ⁻	1.35G + 1.5Q	+ 1.5RQ (Sti	ength)				
Node	Defle	ection	Rotation	Co-ordinate system				
	Х	Z						
	(mm)	(mm)	(°)					
1	0	0	0.02692					
2	0.2	0.7	0.04726					
0	0	0.5	-0.07548					
3								
4	-0.2	0.7	-0.04726					
4 5	0	0.7 0	-0.04726 -0.02692					
4 5	0	0.7 0	-0.04726	/ice)				
4 5	0 ombination: 1	0.7 0	-0.04726 -0.02692	/ice) Co-ordinate system				
4 5 Load c	0 ombination: 7 Defle X	0.7 0 1.0G + 1.0Q - ection Z	-0.04726 -0.02692 + 1.0RQ (Serv Rotation	Co-ordinate				
4 5 Load c Node	0 ombination: 7 Defle X (mm)	0.7 0 1.0G + 1.0Q - ection Z (mm)	-0.04726 -0.02692 + 1.0RQ (Serv Rotation (°)	Co-ordinate				
4 5 Load c Node	0 ombination: 7 Defle X (mm) 0	0.7 0 1.0G + 1.0Q - ection Z (mm) 0	-0.04726 -0.02692 ► 1.0RQ (Serv Rotation (°) 0.01828	Co-ordinate				
4 5 Load c Node	0 ombination: 7 Defle X (mm) 0 0.1	0.7 0 1.0G + 1.0Q - ection Z (mm) 0 0.4	-0.04726 -0.02692 + 1.0RQ (Serv Rotation (°) 0.01828 0.03201	Co-ordinate				
4 5 Load c Node 1 2 3	0 ombination: 7 Defle X (mm) 0 0.1 0	0.7 0 1.0G + 1.0Q - ection Z (mm) 0 0.4 0.3	-0.04726 -0.02692 + 1.0RQ (Serv Rotation (°) 0.01828 0.03201 -0.05116	Co-ordinate				
4 5 Load c Node 1 2 3 4	0 ombination: 7 Defle X (mm) 0 0.1 0 -0.1	0.7 0 1.0G + 1.0Q - ection Z (mm) 0 0.4 0.3 0.4	-0.04726 -0.02692 ► 1.0RQ (Serv Rotation (°) 0.01828 0.03201 -0.05116 -0.03201	Co-ordinate				
4 5 Load c Node 1 2 3 4 5	0 ombination: 7 Defle X (mm) 0 0.1 0 -0.1 0 -0.1 0	0.7 0 1.0G + 1.0Q - ection Z (mm) 0 0.4 0.3 0.4 0	-0.04726 -0.02692 + 1.0RQ (Serv Rotation (°) 0.01828 0.03201 -0.05116 -0.03201 -0.01828	Co-ordinate system				
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4 5 Load c Node 1 2 3 4 5 Load c Node	0 ombination: 7 Defle X (mm) 0 0.1 0 0.1 0 0.1 0 0 0.1 0 0 0 0 Defle X (mm) 0 0 0.2	0.7 0 1.0G + 1.0Q - ection Z (mm) 0 0.4 0.3 0.4 0 1.35G + 1.5Q ection Z (mm) 0 0.7	-0.04726 -0.02692 ► 1.0RQ (Serv Rotation (°) 0.01828 0.03201 -0.05116 -0.03201 -0.01828 E + 1.5ψoS (St Rotation (°) 0.02692 0.04726	Co-ordinate system				
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Structural Engineering & Surveying Martin Perry Associates Parade House, The Parade	Calcs for EXISTING TRUSS CHECK			Start page no./Re	vision 8	
Liskeard, Cornwall PL14 6AH	Calcs by AM	Calcs date 15/06/2023	Checked by MWP	Checked date 15/06/2023	Approved by	Approved date

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

Node	Defle	ection	Rotation	Co-ordinate system
	Х	Z		
	(mm)	(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: $1.35G + 1.5\psi_0Q + 1.5S$ (Strength)

Node	Defle	ection	Rotation	Co-ordinate system
	Х	Z		
	(mm)	(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	Defle	ection	Rotation	Co-ordinate system
	Х	Z		
	(mm)	(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	Defle	ection	Rotation	Co-ordinate system
	Х	Z		
	(mm)	(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: $1.35G + 1.5\psi_0Q + 1.5\psi_0RQ$ (Strength)

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

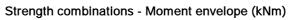
Martin Perry Associates	Project	Job no. 23111				
Structural Engineering & Surveying Martin Perry Associates Parade House, The Parade	Calcs for	Start page no./Revision 9				
Liskeard, Cornwall PL14 6AH	Calcs by AM	Calcs date 15/06/2023	Checked by MWP	Checked date 15/06/2023	Approved by	Approved date

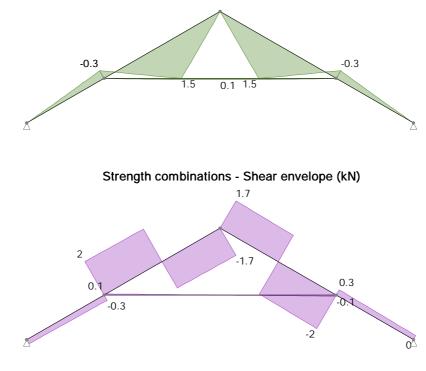
Node	Deflection		Rotation	Co-ordinate system
	X	Z		
	(mm)	(mm)	(°)	
4	-0.1	0.5	-0.03513	
5	0	0	-0.0202	

Total base reactions

Load case/combination	Fo	rce
	FX	FZ
	(kN)	(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 ₀ S (Strength)	0	21.3
1.0G + 1.0Q + 0.5S (Service)	0	14.4
1.35G + 1.5ψ ₀ Q + 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 ₀ Q + 1.5ψ ₀ RQ (Strength)	0	15.9

Forces





Martin Perry Associates	Project	Job no. 23111				
Structural Engineering & Surveying Martin Perry Associates Parade House, The Parade	Calcs for EXISTING TRUSS CHECK				Start page no./Revision 10	
Liskeard, Cornwall PL14 6AH	Calcs by AM	Calcs date 15/06/2023	Checked by MWP	Checked date 15/06/2023	Approved by	Approved date

Envelope - Strengt Member	Shear force			Moment				
	Pos Max abs		Pos	Max	Pos	Min		
	(m)	(kN)	(m)	(kNm)	(m)	(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		
Left Rafter - Span Partial factor for m Partial factor		perties and resist $\gamma_M = 1.300$	ances					
Member details Load duration		Medium-term		Service class		2		
Timber section del				JEI VICE CIA22		۷		
Number of timber se Breadth of sections Timber strength clas	ections	N = 1 b = 75 mm TR26 →		Depth of sections		h = 200 mm		
		Sec Sec Rac Rac Cha Cha Cha Cha Cha She Cha	cond moment of area dius of gyration, i _y , dius of gyration, i _z , aber strength clas ; aracteristic bending aracteristic compre- aracteristic compre- aracteristic tension an modulus of elasi n percentile modulu	ea, l_y , 5000000 mm ⁴ ea, l_z , 7031250 mm ⁴ 57.7 mm 21.7 mm 5 TR26 strength, $f_{wk'}$, 28.3 N/mm ² rength, $f_{vk'}$, 4 N/mm ² ssion strength parallel to grain ssion strength parallel to grain, $f_{10k'}$ icity, $E_{0.mean'}$ 11000 N/mm ² is of elasticity, $E_{0.05'}$, 7400 N/m ticity, $G_{mean'}$ 690 N/mm ² $r_{k'}$, 370 kg/m ³	9 grain, f _{c.90.k} , 2.6 17.6 N/mm ²			
Span details Bearing length		L _b = 100 mm						
Consider Combinat				<u>)</u>				
Check compressio		o the grain - cl.6.	1.4					
Design compressive	e stress	$\sigma_{c,0,d} =$ 1.447 N/m	m²	Design compressiv Utilisation = 0.103	e strength	$f_{c,0,d} = 14.092 \text{ N/mm}^2$		
	PAS	SS - Design parall	lel compress	sion strength excee	ds design p	parallel compression s		
Check design at st	art of spar	1						
Check design at st Check compressio			- cl.6.1.5					
Des.perp.comp.stres		σ _{c,y,90,d} = 1.915 N/	2	Des.perp.comp.str		f _{c,y,90,d} = 1.600 N/mm ²		

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

Des.perp.comp.strength Utilisation = **0.798**

	Project				Job no.	
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Structural Engineering & Surveying Martin Perry Associates	Calcs for				Start page no./F	
Parade House, The Parade		EXISTING	RUSS CHECK	1		11
Liskeard, Cornwall PL14 6AH	Calcs by AM	Calcs date 15/06/2023	Checked by MWP	Checked date 15/06/2023	Approved by	Approved da
PASS - Design	•	ompression stro	ength exceeds o	design perpen	dicular compl	ression stre
Check shear force - Section						
Design shear stress	$\tau_{y,d} = 0.037 \text{ N/r}$		Design shear st Utilisation = 0.0 I <i>SS - Design sh</i> a	15	f _{v,y,d} = 2.462	
Check columns subjected to	o either compre	ssion or combir	ned compression	n and bending	- cl.6.3.2	
Effective length for y-axis	L _{e,y} = 2702 mn		Relative slender	-	$\lambda_{rel,y} = 0.829$	
Effective length for z-axis	$L_{e,z} = 0 \text{ mm}$		Relative slende		$\lambda_{rel,z} = 0$	
J. J	0,2 0			y > 0.3 columi		eck is reauli
			Utilisation = 0.1	-		
			······································		lumn stability	is accepta
Consider Combination 4 - 1.0		S (Sorvice)			2	
Check design 2101 mm alor	ng span					
Check y-y axis deflection - S	Section 7.2					
Final deflection without creep	$\delta_{y,Final} = 1 \text{ mm}$		Allowable deflection		$\delta_{\text{y,Allowable}}$ = 12 mm	
				8		
		PASS	S - Allowable def	lection exceed	ds instantane	ous deflect
Partial factor Member details	γ _M = 1.300				<u>,</u>	
Load duration	Medium-term		Service class		2	
Timber section details						
Number of timber sections	N = 1		Donth of		h 000	
Breadth of sections	b = 75 mm TP26		Depth of sectior	าร	h = 200 mm	
Timber strength class	TR26					
◄75	→					
		Characteristic shear str Characteristic compress Characteristic compress Characteristic tension s Mean modulus of elastic	, 15000 mm ² 500000 mm ³ 87500 mm ³ a, I_y , 50000000 mm ⁴ a, I_z , 7031250 mm ⁴ 17.7 mm TR26 strength, $f_{m,k}$, 28.3 N/mm ² sion strength parallel to grain, i city, $E_{0.mean'}$ 11000 N/mm ² s of elasticity, $E_{0.05'}$, 7400 icity, $G_{mean'}$, 690 N/mm ² r_k , 370 kg/m ³	rain, f _{c.0.k} , 22.9 N/mm [:] lar to grain, f _{c.90.k} , 2.6 l f _{t.0.k} , 17.6 N/mm ² r ²		
Span details Bearing length	L _b = 100 mm					

Martin Perry Associates Structural Engineering & Surveying		HAMETETHY	FARM, BODMI	N		3111
Martin Perry Associates	Calcs for	EXISTING T	RUSS CHECK		Start page no./F	Revision 12
Parade House, The Parade Liskeard, Cornwall PL14 6AH	Calcs by	Calcs date	Checked by Checked date		Approved by	Approved da
	ÂM	15/06/2023	MWP	15/06/2023		
Consider Combination 3 - 1.3	35G + 1.5Q + 1.	5ψ₀S (Strength)	<u></u>			
Check compression parallel	-					
Design compressive stress	$\sigma_{c,0,d} = 0.468 \text{ N}$	I/mm²	Design compr Utilisation = 0 .	essive strength 033	t _{c,0,d} = 14.092	2 N/mm²
PA	SS - Design pai	rallel compress	ion strength e	kceeds design p	parallel comp	ression stre
Check design 900 mm along	<u>j span</u>					
Check shear force - Section	6.1.7					
Design shear stress	$\tau_{y,d} = 0.296 \text{ N/r}$	mm²	Design shear : Utilisation = 0 .	•	$f_{v,y,d} = 2.462$	N/mm ²
		PA	ISS - Design si	hear strength e.	xceeds desigi	n shear stro
Check bending moment - Se						
Design bending stress	$\sigma_{m,y,d} = 2.94 \text{ N/}$	′mm²	Design bendin Utilisation = 0 .	• •	$f_{m,y,d} = 17.415 \text{ N/mm}^2$	
		PASS -		ng strength exce	eeds design b	ending stro
Check combined bending ar	nd axial compre	ssion - Section	6.2.4		-	-
Ũ			Utilisation = 0.	171		
	PA	SS - Combined	d bending and a	axial compress	ion utilisation	is accepta
Check columns subjected to	-		-	-		
Effective length for y-axis	L _{e,y} = 2702 mm	ſ	Relative slend		$\lambda_{rel,y} = 0.829$	
Effective length for z-axis	$L_{e,z} = 0 \text{ mm}$		Relative slend	erness ratio _{ely} > 0.3 columi	$\lambda_{rel,z} = 0$	ock is roqui
			Utilisation = 0 .			ck is requi
				PASS - Co	lumn stability	is accepta
Consider Combination 4 - 1.0	<u> 0G + 1.0Q + 0.5</u>	S (Service)				
Check design 900 mm along	1 span					
Check y-y axis deflection - S						
Final deflection without creep			Allowable defle	ection	$\delta_{y,Allowable} = 12$	2 mm
			Utilisation = 0.	08		
		PASS	S - Allowable de	eflection exceed	ds instantane	ous deflect
<u> Tie - Span 1</u>						
	oportios and ro	listanaas				
Partial factor for material pr	operties and rea	sistances				
Partial factor for material propertial factor	γ _M = 1.300	SISTALICES				
-	γ _M = 1.300	SISTUILES				
Partial factor	-	SISIGILES	Service class		1	
Partial factor Member details Load duration Timber section details	γ _M = 1.300 Medium-term	SISIGILES	Service class		1	
Partial factor Member details Load duration Timber section details Number of timber sections	γ _M = 1.300 Medium-term N = 1	SISIGILES		ons		
Partial factor Member details Load duration Timber section details	γ _M = 1.300 Medium-term	SISIGIICES	Service class Depth of section	ons	1 h = 175 mm	

Martin Perry Associates	Project	HAMETETHY I	FARM, BODMI	N	Job no. 23111		
Structural Engineering & Surveying Martin Perry Associates	Calcs for				Start page no./Revision		
Parade House, The Parade					13		
Liskeard, Cornwall PL14 6AH	Calcs by AM	Calcs date 15/06/2023	Checked by MWP	Checked date 15/06/2023	Approved by	Approved c	
	CI SG SG SG RR RR CI CI CI CI CI SI SI SI CI	Dx175 timber section ross-sectional area, A, 875 ection modulus, W _y , 25520 ection modulus, W _z , 7291 econd moment of area, I _y adius of gyration, i _y , 50.5 r imber strength class TR2 haracteristic bending strer haracteristic compression haracteristic tension strem ean modulus of elasticity, fth percentile modulus of elasticity, haracteristic density, r _k 3 ean density, r _{mean} , 444 kg	$\begin{array}{c} \text{D8.3 mm}^3 \\ \text{7 mm}^3 \\ \text{22330729 mm}^4 \\ \text{1822917 mm}^4 \\ \text{nm} \\ \text{nm} \\ \text{nm} \\ \text{26} \\ \text{h}, f_{wk}, 28.3 \text{ N/mm}^2 \\ \text{strength parallel to grain, f} \\ \text{strength parallel to grain, f} \\ \text{gtm parallel to grain, f} \\ \text{E}_{0.mean'} 11000 \text{ N/mm}^2 \\ \text{elasticity, E}_{0.05'}, 7400 \text{ I} \\ \text{G}_{mean'} 690 \text{ N/mm}^2 \\ 70 \text{ kg/m}^3 \\ \end{array}$	rain, f _{c.0.k} , 22.9 N/mm ² ar to grain, f _{c.90.k} , 2.6 N/n _{1.0.k} , 17.6 N/mm ²	nm²		
Span details Bearing length	L _b = 100 mm						
Consider Combination 3 - 1.3	35G + 1.5Q + 1.	.5ψ₀S (Strength)					
Consider Combination 3 - 1.3 Check compression parallel Design compressive stress		1.6.1.4	Design compr Utilisation = 0	essive strength . 097	f _{c,0,d} = 14.092	2 N/mm²	
Check compression parallel Design compressive stress	to the grain - c $\sigma_{c,0,d} = 1.373$ N	1.6.1.4	Utilisation = 0	.097			
Check compression parallel Design compressive stress PA Check design 1560 mm alor	to the grain - c σ _{c,0,d} = 1.373 № <i>SS - Design pa</i> ng span	I.6.1.4 N/mm ²	Utilisation = 0	.097			
Check compression parallel Design compressive stress <i>PA</i> Check design 1560 mm alor Check bending moment - Se	to the grain - c σ _{c,0,d} = 1.373 № <i>SS - Design pa</i> ng span ection 6.1.6	I.6.1.4 W/mm ² Inrallel compressi	Utilisation = 0 Ion strength e.	.097 xceeds design µ	parallel compl	ression sti	
Check compression parallel Design compressive stress PA Check design 1560 mm alor	to the grain - c σ _{c,0,d} = 1.373 № <i>SS - Design pa</i> ng span	I. 6.1.4 W/mm ² Drallel compression	Utilisation = 0 fon strength e. Design bendir Utilisation = 0	.097 xceeds design µ ng strength .012	f _{m,y,d} = 17.41	ression str 5 N/mm²	
Check compression parallel Design compressive stress <i>PA</i> <u>Check design 1560 mm alor</u> Check bending moment - Se Design bending stress	to the grain - c σ _{c,0,d} = 1.373 M <i>SS - Design pa</i> ng span ection 6.1.6 σ _{m,y,d} = 0.204 M	I.6.1.4 W/mm ² orallel compression N/mm ² PASS - 1	Utilisation = 0 fon strength e. Design bendir Utilisation = 0 Design bendir	.097 xceeds design µ	f _{m,y,d} = 17.41	ression str 5 N/mm²	
Check compression parallel Design compressive stress <i>PA</i> Check design 1560 mm alor Check bending moment - Se	to the grain - c σ _{c,0,d} = 1.373 M <i>SS - Design pa</i> ng span ection 6.1.6 σ _{m,y,d} = 0.204 M	I.6.1.4 W/mm ² orallel compression N/mm ² PASS - 1	Utilisation = 0 fon strength e. Design bendir Utilisation = 0 Design bendir	.097 xceeds design µ ng strength .012 ng strength exce	f _{m,y,d} = 17.41	ression str 5 N/mm²	
Check compression parallel Design compressive stress <i>PA</i> <u>Check design 1560 mm alor</u> Check bending moment - Se Design bending stress	to the grain - c σ _{c,0,d} = 1.373 Γ <i>SS - Design pa</i> ng span ection 6.1.6 σ _{m,y,d} = 0.204 Γ nd axial compre	I.6.1.4 W/mm ² orallel compression N/mm ² PASS - 1	Utilisation = 0 on strength e. Design bendir Utilisation = 0 Design bendir 6.2.4 Utilisation = 0	.097 xceeds design µ ng strength .012 ng strength exce .021	parallel compl f _{m.y.d} = 17.41 eeds design b	ression str 5 N/mm² pending str	
Check compression parallel Design compressive stress <i>PA</i> <u>Check design 1560 mm alor</u> Check bending moment - Se Design bending stress Check combined bending ar Check columns subjected to	to the grain - c σ _{c,0,d} = 1.373 M <i>SS - Design pa</i> ng span ection 6.1.6 σ _{m,y,d} = 0.204 M nd axial compre- <i>P</i> A p either compre-	I.6.1.4 W/mm ² <i>prallel compressi</i> N/mm ² <i>PASS - 1</i> ession - Section of <i>ASS - Combined</i> ession or combin	Utilisation = 0 on strength e. Design bendir Utilisation = 0 Design bendir 6.2.4 Utilisation = 0 bending and	.097 xceeds design µ ng strength .012 ng strength exce .021 axial compress	parallel compl f _{m.y.d} = 17.41 eeds design b ion utilisation	ression str 5 N/mm² pending str	
Check compression parallel Design compressive stress PA Check design 1560 mm alor Check bending moment - Se Design bending stress Check combined bending ar Check columns subjected to Effective length for y-axis	to the grain - c σ _{c,0,d} = 1.373 M <i>SS - Design pa</i> ag <u>span</u> ection 6.1.6 σ _{m,y,d} = 0.204 M nd axial compre <i>Pa</i> o either compre L _{e,y} = 2808 mr	I.6.1.4 W/mm ² <i>prallel compressi</i> N/mm ² <i>PASS - 1</i> ession - Section of <i>ASS - Combined</i> ession or combin	Utilisation = 0 on strength e. Design bendir Utilisation = 0 Design bendir 6.2.4 Utilisation = 0 bending and ed compressi Relative slenc	.097 xceeds design p ng strength .012 ng strength exce .021 axial compress ion and bending lerness ratio	fm.y,d = 17.41 fm.y,d = 17.41 fm.y,d = 17.41 ceeds design b fon utilisation g - cl.6.3.2 $\lambda_{rel,y} = 0.984$	ression str 5 N/mm² pending str	
Check compression parallel Design compressive stress <i>PA</i> <u>Check design 1560 mm alor</u> Check bending moment - Se Design bending stress Check combined bending ar Check columns subjected to	to the grain - c σ _{c,0,d} = 1.373 M <i>SS - Design pa</i> ng span ection 6.1.6 σ _{m,y,d} = 0.204 M nd axial compre- <i>P</i> A p either compre-	I.6.1.4 W/mm ² <i>prallel compressi</i> N/mm ² <i>PASS - 1</i> ession - Section of <i>ASS - Combined</i> ession or combin	Utilisation = 0 on strength e. Design bendir Utilisation = 0 Design bendir 6.2.4 Utilisation = 0 bending and ed compressi Relative slenc Relative slenc λ	.097 xceeds design p ng strength .012 ng strength exce .021 axial compress lon and bending lerness ratio lerness ratio rel,y > 0.3 column	fm.y,d = 17.41 fm.y,d = 17.41 fm.y,	ression str 5 N/mm² Dending str	
Check compression parallel Design compressive stress PA Check design 1560 mm alor Check bending moment - Se Design bending stress Check combined bending ar Check columns subjected to Effective length for y-axis	to the grain - c σ _{c,0,d} = 1.373 M <i>SS - Design pa</i> ag <u>span</u> ection 6.1.6 σ _{m,y,d} = 0.204 M nd axial compre <i>Pa</i> o either compre L _{e,y} = 2808 mr	I.6.1.4 W/mm ² <i>prallel compressi</i> N/mm ² <i>PASS - 1</i> ession - Section of <i>ASS - Combined</i> ession or combin	Utilisation = 0 on strength e. Design bendir Utilisation = 0 Design bendir 6.2.4 Utilisation = 0 bending and ed compressi Relative slenc Relative slenc	.097 xceeds design p ng strength .012 ng strength exce .021 axial compress ion and bending derness ratio derness ratio rel,y > 0.3 column .151	fm.y,d = 17.41 fm.y,d = 17.41 fm.y,	ression str 5 N/mm ² Dending str 1 is accepta Pack is requi	
Check compression parallel Design compressive stress <i>PA</i> Check design 1560 mm alor Check bending moment - Se Design bending stress Check combined bending ar Check columns subjected to Effective length for y-axis Effective length for z-axis	to the grain - c $\sigma_{c,0,d} = 1.373$ M <i>SS - Design pai</i> and span ection 6.1.6 $\sigma_{m,y,d} = 0.204$ M and axial compresion <i>Pai</i> b either compresion Le,y = 2808 mr Le,z = 0 mm	I.6.1.4 W/mm ² <i>prallel compressi</i> N/mm ² <i>PASS - I</i> ession - Section of <i>ASS - Combined</i> ession or combined m	Utilisation = 0 on strength e. Design bendir Utilisation = 0 Design bendir 6.2.4 Utilisation = 0 bending and ed compressi Relative slenc Relative slenc λ	.097 xceeds design p ng strength .012 ng strength exce .021 axial compress ion and bending derness ratio derness ratio rel,y > 0.3 column .151	fm,y,d = 17.41 fm,y,d = 17.41 fm,y,d = 17.41 fm,y,	ression str 5 N/mm ² Dending str 1 is accepta Pick is requi	
Check compression parallel Design compressive stress <i>PA</i> Check design 1560 mm alor Check bending moment - Se Design bending stress Check combined bending ar Check columns subjected to Effective length for y-axis Effective length for z-axis	to the grain - c $\sigma_{c,0,d} = 1.373$ M SS - Design para and span ection 6.1.6 $\sigma_{m,y,d} = 0.204$ M and axial comprese $Para perither comprese L_{e,y} = 2808 mrL_{e,z} = 0 mmDG + 1.0Q + 0.5$	I.6.1.4 W/mm ² <i>prallel compressi</i> N/mm ² <i>PASS - I</i> ession - Section of <i>ASS - Combined</i> ession or combined m	Utilisation = 0 on strength e. Design bendir Utilisation = 0 Design bendir 6.2.4 Utilisation = 0 bending and ed compressi Relative slenc Relative slenc λ	.097 xceeds design p ng strength .012 ng strength exce .021 axial compress ion and bending derness ratio derness ratio rel,y > 0.3 column .151	fm,y,d = 17.41 fm,y,d = 17.41 fm,y,d = 17.41 fm,y,	ression str 5 N/mm² pending str is accepta eck is requi	
Check compression parallel Design compressive stress <i>PA</i> Check design 1560 mm alor Check bending moment - Se Design bending stress Check combined bending ar Check columns subjected to Effective length for y-axis Effective length for z-axis Consider Combination 4 - 1.0 Check design 1560 mm alor	to the grain - c $\sigma_{c,0,d} = 1.373$ M SS - Design para and span ection 6.1.6 $\sigma_{m,y,d} = 0.204$ M and axial comprese Pa be either comprese $L_{e,y} = 2808$ mr $L_{e,z} = 0$ mm DG + 1.0Q + 0.5 and span	I.6.1.4 W/mm ² <i>prallel compressi</i> N/mm ² <i>PASS - I</i> ession - Section of <i>ASS - Combined</i> ession or combined m	Utilisation = 0 on strength e. Design bendir Utilisation = 0 Design bendir 6.2.4 Utilisation = 0 bending and ed compressi Relative slenc Relative slenc λ	.097 xceeds design p ng strength .012 ng strength exce .021 axial compress ion and bending derness ratio derness ratio rel,y > 0.3 column .151	fm,y,d = 17.41 fm,y,d = 17.41 fm,y,d = 17.41 fm,y,	ression str 5 N/mm ² Dending str 1 is accepta Pack is requi	
Check compression parallel Design compressive stress <i>PA</i> Check design 1560 mm alor Check bending moment - Se Design bending stress Check combined bending ar Check columns subjected to Effective length for y-axis Effective length for z-axis Consider Combination 4 - 1.0 Check design 1560 mm alor Check y-y axis deflection - S	to the grain - c $\sigma_{c,0,d} = 1.373$ M SS - Design para and span ection 6.1.6 $\sigma_{m,y,d} = 0.204$ M and axial comprese $Para be either comprese L_{e,y} = 2808 mrL_{e,z} = 0 mmDG + 1.0Q + 0.5and spanSection 7.2$	I.6.1.4 W/mm ² <i>prallel compressi</i> N/mm ² <i>PASS - I</i> ession - Section of <i>ASS - Combined</i> ession or combined m <i>SS (Service)</i>	Utilisation = 0 on strength e. Design bendir Utilisation = 0 Design bendir 6.2.4 Utilisation = 0 bending and ed compressi Relative slenc Relative slenc λ_i Utilisation = 0	.097 xceeds design μ ng strength .012 ng strength exce .021 axial compress ion and bending lerness ratio lerness ratio $r_{el,y} > 0.3$ column .151 PASS - Co	fm.y.d = 17.41 fm.y.d = 17.41 fm.y.	ression str 5 N/mm ² pending str o is accepta eck is requi	
Check compression parallel Design compressive stress <i>PA</i> Check design 1560 mm alor Check bending moment - Se Design bending stress Check combined bending ar Check columns subjected to Effective length for y-axis Effective length for z-axis Consider Combination 4 - 1.0 Check design 1560 mm alor	to the grain - c $\sigma_{c,0,d} = 1.373$ M SS - Design para and span ection 6.1.6 $\sigma_{m,y,d} = 0.204$ M and axial comprese $Para be either comprese L_{e,y} = 2808 mrL_{e,z} = 0 mmDG + 1.0Q + 0.5and spanSection 7.2$	I.6.1.4 W/mm ² <i>prallel compressi</i> N/mm ² <i>PASS - I</i> ession - Section of <i>ASS - Combined</i> ession or combined m <i>SS (Service)</i>	Utilisation = 0 on strength e. Design bendir Utilisation = 0 Design bendir 6.2.4 Utilisation = 0 bending and ed compressi Relative slenc Relative slenc λ	.097 xceeds design p ng strength .012 ng strength exce .021 axial compress tion and bending lerness ratio lerness ratio rel,y > 0.3 column .151 PASS - Co	fm,y,d = 17.41 fm,y,d = 17.41 fm,y,d = 17.41 fm,y,	ression str 5 N/mm ² pending str 5 is accepta eck is requi	

Martin Per Structural Engineering	ry Associate	2S	Para Liskear Teler mail@r	I Perry Associates Ltd de House, The Parade d, Cornwall PL14 6AH ohone: 01579 345777 nperryassociates.com nperryassociates.com
PROJECT	HAMETETHY F	RM, BODMIN		
Designed by	Checked by	Date	Project no.	Sheet no.
Am	MWP	JUNE 2023	23111	14
		CALCULATIONS		
	MASONR	1 PIER		
Truss Roactions	(page 4)			
$\begin{cases} t = 1, \\ t = 6 \end{cases}$	io EN			
Tedds Output				
440	X440 Masonry prer	PASS		

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

MASONRY COLUMN DESIGN

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

Design summary

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

Thickness of column

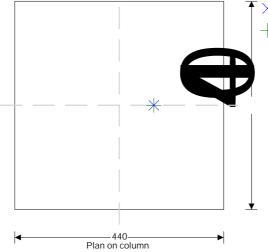
Height of column

Reduction factor for effective height

Effective height of column (cl 5.5.1.2)

b = **440** mm t = **440** mm

- ρ₂ = **1.0**
- h_{eff} = $h \times \rho_2$ = **3000** mm



h = **3000** mm

Dead load <

Variable load

Loading

Vertical dead load
Eccentricity of dead load in x-direction
Eccentricity of dead load in y-direction
Vertical live load
Eccentricity of variable load in x-direction
Eccentricity of variable load in y-direction
Characteristic wind loading
Vertical wind loading

Masonry details

Masonry type Compressive strength of masonry unit Height of unit Width of unit Conditioning factor

$W_v = 0.0 \ kN$ Aggregate concrete - Group 1

 $f_c = 5.2 \text{ N/mm}^2$ h_u = **215** mm $w_u = 100 \text{ mm}$ k = **1.0**

G_k = **1.2** kN e_{Gb} = **73** mm $e_{Gt} = 0 mm$ $Q_k = 6.0 \text{ kN}$ e_{Qb} = **73** mm $e_{Qt} = \mathbf{0} \text{ mm}$ $W_k = 0.8 \text{ kN/m}^2$

Martin Perry Associates	Project	HAMETETHY	- ARM, BODMI	IN	Job no.	3111
Structural Engineering & Surveying	Calcs for					
Martin Perry Associates Parade House, The Parade	MAS	SONRY COLUM	IN BEARING C	CHECK		16
Liskeard, Cornwall PL14 6AH	Calcs by AM	Calcs date 15/06/2023	Checked by MWP	Checked date 15/06/2023	Approved by	Approved date
- Conditioning to the air dry c	ondition in accord	ance with cl.7.3	2			
Shape factor - Table A.1		$d_{sf} = 1.38$				
Mean compressive strength o	f masonry unit	$f_b = f_c \times k \times$	d _{sf} = 7.176 N/r	mm²		
Density of masonry		γ = 18 kN/n	1 ³			
Mortar type		M4 - Gene	ral purpose m	ortar		
Compressive strength of mas	•	f _m = 4 N/mr	n²			
Compressive strength factor -		K = 0.75				
Characteristic compressive st	rength of masonry	•				
			′ × fm ^{0.3} = 4.516			
Characteristic flexural strengt	n of masonry navi			the bed joints - 1	adie NA.6	
Characteristic flexural strengt	h of masonry bay	f _{xk1} = 0.15 [ng a plane of fai		ular to the hod io	nts - Tablo N	۵.6
Characteristic nexural strengt	n or masonry navi	f _{xk2} = 0.347		uiai to the bed Joi	IIIS - I ADIE IVI	٦.0
		Txk2 - 0.347	11/11111			
Partial factors for material s	-	Catagory	1			
Category of manufacturing co Class of execution control	Introl	Category I Class 2	I			
Partial factor for masonry in c	omprossivo flovur					
		γινις – 3.00				
Slenderness ratio			(
Slenderness ratio minor axis		$\lambda_t = h_{eff} / t =$				
Slenderness ratio major axis	(cl.5.5.2.1)	$\lambda_{\rm b} = h_{\rm eff} / b$				
Maximum slenderness		$\lambda = \max(\lambda_t,$	λb) = 6.82			
				PASS - Slende	erness ratio i	s less than 27
Load combinations derived	-		-			
	$1.35 \times \text{perm unf}$					
Combination 2	1.35 × perm unf	fav + 1 × perm fa	av + 1.5 × 0.7 :	× variable + 1.5 ×	wind (0.161)	
The following output relates	s to combination	2				
Reduction factor for slender	ness and eccent	ricity about the	major axis -	Section 6.1.2.2		
Design bending moment top of	or bottom of colum	n 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(γ	$\gamma_{\rm fGv} \times G_k + \gamma_{\rm fQv}$	× Q _k) = 7.92 kN		
Initial eccentricity - cl.5.5.1.1		$e_{init} = h_{eff} / 2$	150 = 6.7 mm			
Conservativley assume mome	ent due to wind loa	ad at the top of t	he column is e	equal to that at mic	d height	
Design moment due to horizo	ntal load	$M_{Edb} = \gamma_{fWh}$	$\times \alpha \times W_k \times h^2$	× t = 0.557 kNm		
Eccentricity due to horizontal	load	$e_{hb} = M_{Edb}$ /	N _{idb} = 70.3 mr	n		
Eccentricity at top or bottom of	of column - eq.6.5	e _{ib} = max(N	Nidb / Nidb + ehb	+ e_{init} , 0.05 × b) =	150.0 mm	
Reduction factor top or botton	n of column - eq.6	.4 $\Phi_{ib} = max(2)$	1 - 2 × e _{ib} / b, 0) = 0.318		
Ratio of top and middle mnts						
Design bending moment at m				$\hat{\mathbf{b}}_k \times \mathbf{e}_{Gb} + \gamma_{fQv} \times \mathbf{Q}_k$		
Design vertical load at middle		$N_{mdb} = \gamma_{fGv}$	$\times G_k + \gamma_{fQv} \times Q_i$	$_{\rm k}$ + $\gamma_{\rm fGv}$ × t × b × γ	× h / 2 = 14.9	77 kN
Eccentricity due to horizontal			/ N _{mdb} = 37.2 I			
				e _{init} = 82.5 mm		
Eccentricity middle of column		e _{kb} = 0.0 m	m			
Eccentricity at middle of colur						
Eccentricity at middle of colur Eccentricity at middle of colur		e _{mkb} = max		5 × b) = 82.5 mm		
Eccentricity at middle of colur Eccentricity at middle of colur From eq.G.2	nn - eq.6.6	$e_{mkb} = max$ $A_{1b} = 1 - 2$	(e _{mb} + e _{kb} , 0.05 × e _{mkb} / b = 0.6			
Eccentricity at middle of colur Eccentricity at middle of colur	nn - eq.6.6 elasticity factor	e _{mkb} = max A _{1b} = 1 - 2 : K _E = 1000		25		

Martin Perry Associates	Project	HAMETETHY	FARM, BODM	IN	Job no.	3111
Structural Engineering & Surveying Martin Perry Associates Parade House, The Parade	Calcs for M	ASONRY COLUM	IN BEARING C	СНЕСК	Start page no./F	Revision 17
Liskeard, Cornwall PL14 6AH	Calcs by AM	Calcs date 15/06/2023	Checked by MWP	Checked date 15/06/2023	Approved by	Approved date
Slenderness - eq.G.4		$\lambda_{b} = (h_{eff} / k)$	o) × √(f _k / E) = (0.216		
From eq.G.3		$u_b = (\lambda_b - 0)$.063) / (0.73 -	1.17 × e _{mkb} / b) = 0	0.299	
Reduction factor at middle of	column - eq.G.1	$\Phi_{\sf mb}$ = max	$(A_{1b} \times e_{e^{-(u_b \times u_b)}})$	^{/2} , 0) = 0.598		
Reduction factor for slenderne	ess and eccentri	city $\Phi_{b} = min(\Phi_{b})$	$\Phi_{ib}, \Phi_{mb}) = 0.31$	8		
Reduction factor for slender	ness and ecce	ntricity about the	e minor axis -	Section 6.1.2.2		
Design bending moment top of	or bottom of colu	וmn M _{idt} = abs(ז	$\gamma_{\rm fGv} imes G_k imes e_{\rm Gt} +$	$\gamma_{fQv} \times Q_k \times e_{Qt}$ =	0 kNm	
Design vertical load at top or l	bottom of colum	n N _{idt} = abs(γ	$\gamma_{\rm fGv} \times G_k + \gamma_{\rm fQv} >$	< Q _k) = 7.92 kN		
Initial eccentricity - cl.5.5.1.1		$e_{init} = h_{eff} / A_{eff}$	450 = 6.7 mm			
Conservativley assume mome	ent due to wind I	load at the top of t	he column is e	equal to that at mi	d height	
Design moment due to horizo	ntal load	$M_{Edt} = \gamma_{fWh}$	$\times \alpha \times W_k \times h^2$	× b = 0.557 kNm		
Eccentricity due to horizontal		$e_{ht} = M_{Edt} /$	N _{idt} = 70.3 mm			
Eccentricity at top or bottom c				e_{init} , 0.05 × t) = 77	7.0 mm	
Reduction factor top or botton			$1 - 2 \times e_{it} / t, 0$			
Ratio of top and middle mnts	•					
Design bending moment at m		-	x abs(very x G	$_{\rm k} \times e_{\rm Gt} + \gamma_{\rm fQv} \times Q_{\rm k}$:	$(x \in O_t) = 0 k N m$	ı
Design vertical load at middle				$x + \gamma_{fGv} \times t \times b \times \gamma$		
Eccentricity due to horizontal			$/ N_{mdt} = 37.2 \text{ m}$		~ 117 2 - 14.71	
Eccentricity middle of column						
Eccentricity at middle of colur		$e_{kt} = 0.0 \text{ m}$		Jinit – 43.0 mini		
Eccentricity at middle of colur				× t) = 43.8 mm		
From eq.G.2	nn - cq.0.0		$\times e_{mkt} / t = 0.80$			
Short term secant modulus of	elasticity factor		$\sim \text{CHRUT} = 0.00$			
Modulus of elasticity - cl.3.7.2			= 4516 N/mm ²	2		
•			= 4310 k/mm			
Slenderness - eq.G.4					240	
From eq.G.3				$.17 \times e_{mkt} / t) = 0.2$	249	
Reduction factor at middle of			$(A_{1t} \times e_e^{-(u_t \times u_t)/2})$	0) = 0.776		
Reduction factor for slenderne		,	it, Φ_{mt}) = 0.65			
Columns subjected to main	-	•				
Design value of the vertical lo				Nmdt) = 14.977 kt	N	
Design compressive strength	•	-	= 1.505 N/mm ²			
Vertical resistance of column	- eq.6.2		-	≺ f _d = 92.765 kN		
Utilisation		$N_{Ed} / N_{Rd} =$		tanco ovocado -	nnllad daal-	n vortical !-
I and another the set of the set	farm E (10 *	-		tance exceeds a	appireu uesigi	n vertical lo
Load combinations derived	-		•	-	wind (0.00()	
Combination 1		Infav + 1 × perm f				
Combination 2		Infav + 1 × perm f	av + 1.5 × 0.7 :	× variable + 1.5 ×	wina (0.471)	
The following output relates	s to combinatio	n 2				
Reduction factor for slender		-	-			
Design bending moment top of	or bottom of colu	IIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIII	$\gamma_{fGv} \times G_k \times e_{Gb}$	+ $\gamma_{fQv} \times Q_k \times e_{Qb}$)	= 0.578 kNm	
Besign bending moment top t	bottom of colum	n N _{idb} = abs($\gamma_{fGv} \times G_k + \gamma_{fQv}$	× Q _k) = 7.92 kN		
Design vertical load at top or l			450 67 mm			
Design vertical load at top or l Initial eccentricity - cl.5.5.1.1		$e_{init} = h_{eff} / 4$				
Design vertical load at top or l Initial eccentricity - cl.5.5.1.1 Conservativley assume mome		oad at the top of t	he column is e		d height	
Design vertical load at top or l Initial eccentricity - cl.5.5.1.1	ntal load	load at the top of t $M_{Edb} = \gamma_{fWh}$	he column is e	× t = 0.557 kNm	d height	

Martin Perry Associates	Project	HAMETETHYI	FARM, BODMI	N	Job no.	3111
Martin Perry Associates	Calcs for MAS	ONRY COLUM	IN BEARING C	СНЕСК	Start page no./F	Revision 18
Parade House, The Parade Liskeard, Cornwall PL14 6AH	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	ÂM	15/06/2023	MWP	15/06/2023		
Eccentricity at top or bottom o	f column - eq 6 5	ein = max(N	Nidh / Nidh + Phh	+ e _{init} , 0.05 × b) =	150.0 mm	
Reduction factor top or bottom			1 - 2 × e _{ib} / b, 0			
Ratio of top and middle mnts of		$\alpha_{mdb} = 1.0$,		
Design bending moment at mi	5		$x = abs(y_{fGv} \times G)$	$\hat{b}_k \times e_{Gb} + \gamma_{fQv} \times Q_k$	(× eob) = 0.57	' 8 kNm
Design vertical load at middle				$_{k} + \gamma_{fGv} \times t \times b \times \gamma$		
Eccentricity due to horizontal I			/ N _{mdb} = 37.2 I			
Eccentricity middle of column						
Eccentricity at middle of colum	nn due to creep	e _{kb} = 0.0 m	m			
Eccentricity at middle of colum	nn - eq.6.6	e _{mkb} = max	(e _{mb} + e _{kb} , 0.05	5 × b) = 82.5 mm		
From eq.G.2		$A_{1b} = 1 - 2$	× e _{mkb} / b = 0.6	25		
Short term secant modulus of	elasticity factor	Ke = 1000				
Modulus of elasticity - cl.3.7.2		$E = K_E \times f_k$	= 4516 N/mm ²			
Slenderness - eq.G.4		λ_b = (h _{eff} / b) × √(f _k / E) = ().216		
From eq.G.3		$u_b = (\lambda_b - 0)$.063) / (0.73 - 1	1.17 × e _{mkb} / b) = 0	0.299	
Reduction factor at middle of o	column - eq.G.1	$\Phi_{mb} = max$	$(A_{1b} \times e_e^{-(u_b \times u_b)})$	^{/2} , 0) = 0.598		
Reduction factor for slenderne	ess and eccentricity	y $\Phi_b = \min(\Phi)$	$(b, \Phi_{mb}) = 0.31$	8		
Reduction factor for slender	ness and eccentr	icity about the	minor axis -	Section 6.1.2.2		
Design bending moment top o		-		$\gamma_{fQv} \times Q_k \times e_{Qt}$ =	0 kNm	
Design vertical load at top or b	oottom of column	N _{idt} = abs(γ	_{fGv} × Gk + γ _{fQv} >	< Q _k) = 7.92 kN		
Initial eccentricity - cl.5.5.1.1			150 = 6.7 mm			
Conservativley assume mome	ent due to wind loa	d at the top of t	he column is e	qual to that at mi	d height	
Design moment due to horizor	ntal load	$M_{Edt} = \gamma_{fWh}$	$\times \alpha \times W_k \times h^2$	k b = 0.557 kNm		
Eccentricity due to horizontal I	oad	$e_{ht} = M_{Edt} /$	N _{idt} = 70.3 mm			
Eccentricity at top or bottom of	f column - eq.6.5	e _{it} = max(N	l _{idt} / N _{idt} + e _{ht} +	$e_{init}, 0.05 \times t) = 77$	7.0 mm	
Reduction factor top or bottom	n of column - eq.6.4	4 $\Phi_{it} = max(1$	- 2 \times e_{it} / t, 0)	= 0.65		
Ratio of top and middle mnts of	due to eccentricity	$\alpha_{mdt} = 1.0$				
Design bending moment at mi	ddle of column	$M_{mdt} = \alpha_{mdt}$	$\times abs(\gamma_{fGv} \times Gk)$	$x \times e_{Gt} + \gamma_{fQv} \times Q_k$	× e _{Qt}) = 0 kNm	ו
Design vertical load at middle	of column	$N_{mdt} = \gamma_{fGv}$	$\langle G_k + \gamma_{fQv} \times Q_k \rangle$	$+ \gamma_{fGv} \times t \times b \times \gamma >$	× h / 2 = 14.9 7	7 7 kN
Eccentricity due to horizontal I			/ N _{mdt} = 37.2 m			
Eccentricity middle of column				e _{init} = 43.8 mm		
Eccentricity at middle of colum		e _{kt} = 0.0 mi				
Eccentricity at middle of colum	nn - eq.6.6			× t) = 43.8 mm		
From eq.G.2			< e _{mkt} / t = 0.80	1		
Short term secant modulus of	elasticity factor	K _E = 1000				
Modulus of elasticity - cl.3.7.2			= 4516 N/mm ²			
Slenderness - eq.G.4			$\times \sqrt{(f_k / E)} = 0.$		240	
From eq.G.3				$.17 \times e_{mkt} / t) = 0.12$	249	
Reduction factor at middle of c	•		$A_{1t} \times e_{e^{-(u_t \times u_t)/2}},$	(0) = 0.776		
Reduction factor for slenderne			$(t, \Phi_{mt}) = 0.65$			
Columns subjected to mainly						
Design value of the vertical loa				N _{mdt}) = 14.977 kl	N	
Design compressive strength	-		= 1.505 N/mm ²			
Vertical resistance of column -	- eq.6.2			< f _d = 92.765 kN		
Utilisation		$N_{Ed} / N_{Rd} =$		tamaa	nnlled dee'	m vortis-11
	ŀ	PASS - Design	vertical resis	tance exceeds a	applied desig	n vertical l

Martin Perry Associates	Project	НАМЕТЕТНҮ Р	N	Job no.	3111		
Structural Engineering & Surveying Martin Perry Associates Parade House, The Parade	Calcs for MAS	ONRY COLUM	N BEARING C	HECK	Start page no./I	Start page no./Revision 19	
Liskeard, Cornwall PL14 6AH	Calcs by AM	Calcs date 15/06/2023	Checked by MWP	Checked date 15/06/2023	Approved by	Approved date	
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	fmasonry	$f_d = f_k / \gamma_{Mc} =$	= 1.505 N/mm ²				
Design vertical compressive str	ess	σ _d = min(γ _{fG} N/mm²	$_{ih} \times (G_k + S_{wt}) /$	$(t \times b)$, $0.15 \times mi$	$n(\Phi_{t}, \Phi_{b}) imes f_{d})$	= 0.033	
Design flex masonry strength p	arallel to bed joint	s $f_{xd1} = f_{xk1} / \gamma$	_{Mc} = 0.05 N/mr	n²			
Apparent design flex strength p	arallel to bed join	ts $f_{xd1,app} = f_{xd1}$	+ σ_d = 0.083 N	J/mm ²			
Column subject to lateral load	ding about the m	ajor axis - Seo	ction 6.3				
Elastic section modulus of colu	•	-	6 = 14197333	mm ³			
Moment of resist parallel to bed	l joints - eq.6.15	$M_{Rd1b} = f_{xd1}$	$_{app} \times Z_{b} = 1.18^{\circ}$	1 kNm			
Bending moment coefficient		α = 0.125					
Design moment in column		$M_{Edb} = \gamma_{fWh}$	$\times \alpha \times W_k \times h^2$	< t = 0.557 kNm			
		M _{Edb} / M _{Rd1k}	o = 0.471				
		PASS - Mo	oment resistai	nce greater than	design mon	nent in colun	
Column subject to lateral load	ding about the m	inor axis - Se	ction 6.3				
Elastic section modulus of colu	mn	$Z_t = b \times t^2 /$	6 = 14197333	mm ³			
Moment of resist parallel to bed	l joints - eq.6.15	$M_{Rd1t} = f_{xd1,a}$	$M_{Rd1t} = f_{xd1,app} \times Z_t = 1.181 \text{ kNm}$				
Bending moment coefficient		α = 0.125					
Design moment in column		Medt = γfwh > Medt / Mrd1t		b = 0.557 kNm			

PASS - Moment resistance greater than design moment in column



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PROJECT	HAMETETHY	FARM BODMIN		
Designed by	Checked by MWP	Date JUNE 2023	Project no. 23111	Sheet no. 2.D
		CALCULATIONS	-01.	
		WALL OFFICK		
$R_{00} = 0.1$ $IL = 0.6$	bolm ² Supporte	l'indth = 0.5m	baching th =	0.05 KS/m
1L = 0.6	ENTm2 11) IL =	0.3 by Im
Edds Output				
,		- ta D.		
LSD	mm 7N Blockwork	wall PASS		

Martin Perry Associates	Project	HAMETETHY	Job no. 23111			
Martin Perry Associates Martin Perry Associates Parade House. The Parade	Calcs for	EXISTING MASONRY WALL PANEL CHECK				Revision 21
Liskeard, Cornwall PL14 6AH	Calcs by AM	Calcs date 15/06/2023	Checked by MWP	Checked date 15/06/2023	Approved by	Approved date

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} = ρ ₂ × h = 3000 m	m							



Martin Perry Associates	Project	Ν	Job no. 23111						
Structural Engineering & Surveying	Calcs for		Start page no./Revision						
Parade House, The Parade	EXIST	ING MASONRY	WALL PANE		22				
Liskeard, Cornwall PL14 6AH	Calcs by AM	Calcs date 15/06/2023	Checked by MWP	Checked date 15/06/2023	Approved by	Approved d			
		10/00/2020		10/00/2020					
1									
•									
Masonry details									
Masonry type		Aggregate	concrete - G	roup 1					
Compressive strength of mas	sonry	fc = 7.3 N/r	nm²						
Height of unit		h _u = 215 m	m						
Width of unit		w _u = 150 m	ım						
Conditioning factor		k = 1.0	0						
- Conditioning to the air dry of	condition in accord		.2						
Shape factor - Table A.1	anoth of mononers	d _{sf} = 1.28	d 0.244 N/						
Norm. mean compressive structure	ength of masonry		d _{sf} = 9.344 N/r	mm²					
Density of masonry		•	γ = 18 kN/m³ M4 - General purpose mortar						
Mortar type Compressive strength of masonry mortar			$f_m = 4 \text{ N/mm}^2$						
Compressive strength factor	•	K = 0.75	11						
Characteristic compressive s									
· ·	5		⁷ × fm ^{0.3} = 5.43 3	3 N/mm ²					
Characteristic flexural strengt	th of masonry havi			the bed joints - T	able NA.6				
Characteristic flowural strong	th of moconny boy	$f_{xk1} = 0.217$		where the had in	nto Toblo N	Λ ζ			
Characteristic flexural strengt	in or masonry navi	f _{xk2} = 0.517		ulai to the bed joi		4.0			
Lateral loading details		1262 - 0.017	N/11111						
Characteristic wind load on p	anel	W _k = 0.750) kN/m²						
Vertical loading details									
Permanent load on top of wall		$G_k = 0.05 \ k$	$G_k = 0.05 \text{ kN/m}$ at an eccentricity of 25 mm						
Variable load on top of wall	Q _k = 0.3 kM	Q _k = 0.3 kN/m at an eccentricity of 25 mm							
Partial factors for material	strength								
Category of manufacturing co	ontrol	Category I	I						
Class of execution control		Class 2							
Partial factor for masonry in c	compressive flexure	e γ _{Mc} = 3.00							
Partial factor for masonry in te	ensile flexure	$\gamma_{Mt}=~\textbf{2.70}$							
,	shear	$\gamma_{Mv}=2.50$							
Partial factor for masonry in s		6614							
Partial factor for masonry in s Slenderness ratio of mason	ry walls - Section								
Partial factor for masonry in s Slenderness ratio of mason Allowable slenderness ratio	ry walls - Section	SR _{all} = 27							
Partial factor for masonry in s Slenderness ratio of mason	ry walls - Section	SR _{all} = 27 SR = h _{ef} / t		rness ratio is les					

Martin Perry Associates	Project	Project HAMETETHY FARM, BODMIN				Job no. 23111			
Structural Engineering & Surveying	Calcs for				Start page no./Revision				
Parade House, The Parade	EXIS	STING MASONR	Y WALL PANE	L CHECK		23			
Liskeard, Cornwall PL14 6AH	Calcs by AM	Calcs date 15/06/2023	Checked by MWP	Checked date 15/06/2023	Approved by	Approved da			
Partial safety factors for de	sign loads								
Partial safety factor for perma	anent load	$\gamma_{fG} = 1.35$							
Partial safety factor for variab	le imposed load	$\gamma_{fQ} = 1.5$							
Partial safety factor for variab	le wind load	$\gamma_{fW}~=~0.75$							
Check vertical loads									
Reduction factor for slende	rness and ecce	ntricity - Section	6.1.2.2						
Vertical load at bottom of wal	I	$N_{id} = \gamma_{fG} \times$	$(\mathbf{G}_k + \gamma \times \mathbf{t} \times \mathbf{h})$	+ $\gamma_{fQ} \times Q_k = 11.45$	3 kN/m				
Moment at bottom of wall due	e to vertical load	$M_{id} = \gamma_{fG} \times$	$G_k \times e_G + \gamma_{fQ} \times$	Q _k × e _Q = 0.013 k	Nm/m				
Initial eccentricity - cl.5.5.1.1	$e_{init} = h_{ef} / A_{ef}$	450 = 6.7 mm							
Moment at bottom of wall due	e to horizontal loa	ad M _{Eid} = 0.12	2 9 kNm/m						
Eccentricity at bottom of wall	due to horizonta	I load $e_h = M_{Eid} /$	N _{id} = 11.3 mm						
Eccentricity at bottom of wall	- eq.6.5	ei = max(N	$N_{id} / N_{id} + e_h + e$	init, $0.05 \times t$) = 19.	1 mm				
Reduction factor at bottom of	wall - eq.6.4	$\Phi_i = max(2)$	1 - 2 × 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	$M_{md} = \gamma_{fG} >$	$K G_k \times e_G + \gamma_{fQ} \times$	Q _k × e _Q = 0.013	<nm m<="" td=""><td></td></nm>				
Moment at middle of wall due	to horizontal loa	ad M _{Emd} = 0.1	29 kNm/m						
Eccentricity at middle of wall	due to horizonta	load $e_{hm} = M_{Emm}$	$_{\rm i}$ / N _{md} = 21.6 m	ım					
Eccentricity at middle of wall	due to loads - ec	1.6.7 e _m = M _{md} /	N_{md} + e_{hm} + e_{in}	it = 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	× e _{mk} / t = 0.594	ł					
Short term secant modulus of elasticity factor		K _E = 1000	K _E = 1000						
Modulus of elasticity - cl.3.7.2	2		= 5433 N/mm ²						
Slenderness - eq.G.4			$f_{\rm f}$) × $\sqrt{(f_{\rm k} / \rm E)} = 0$						
From eq.G.3				$17 \times e_{mk} / t) = 1.1$	56				
Reduction factor at middle of wall - eq.G.1			$\Phi_{\rm m} = \max(A_1 \times e_{\rm e}^{-(u \times u)/2}, 0) = 0.305$						
Reduction factor for slendern	ess and eccentri	city $\Phi = \min(\Phi)$	$(\Phi_{m}) = 0.305$						
Verification of unreinforced	-	-	-	-	6.1.2				
Design value of the vertical lo			$(N_{id}, N_{md}) = 11.$						
Design compressive strength	•		= 1.811 N/mm ²						
Vertical resistance of wall - e	q.6.2		t × f _d = 82.735		mulle del d				
Uproinforced managements "		-		tance exceeds a	ppilea desig	n vertical lo			
Unreinforced masonry walls	-	ateral loading - S	Section 6.3						
Partial safety factors for de	-								
Partial safety factor for perma		$\gamma_{\rm fG} = 1$							
Partial safety factor for variab		$\gamma_{fQ} = 0$							
Partial safety factor for variab		$\gamma_{\rm fW} = 1.5$	lon the arm t		A				
Limiting height and length to	o inickness rat			adility limit state	e - Annex F				
Length to thickness ratio Limiting height to thickness ra	atio - Figuro E 2	L / t = 26.6 80	/ 00						
Height to thickness ratio	and rigule 1.5	60 h / t = 20							
			PASS - Limitin	ng height to thick	ness ratio is	not exceed			
Design moments of resista	nce in panels								
2									

VPA	Project				Job no.	Job no.		
Martin Perry Associates		HAMETETHY FARM, BODMIN				23111		
Martin Perry Associates	Calcs for				Start page no./Revision 24			
Parade House, The Parade Liskeard, Cornwall PL14 6AH	EXIS	STING MASONRY	WALL PANEL	CHECK				
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved da		
	AM	15/06/2023	MWP	15/06/2023				
Design compressive strength	of masonry	$f_d = f_k / \gamma_{Mc}$ =	= 1.811 N/mm ²					
Design vertical compressive stress		$\sigma_{d} = min(\gamma_{fG})$	$s \times (G_k + S_{wt}) / t$, $0.15 \times \Phi \times f_d$) =	0 N/mm ²			
Design flexural strength of m	asonry parallel to	o bed joints						
		$f_{xd1} = f_{xk1} / \gamma$	_{Mt} = 0.08 N/mm	1 ²				
Apparent design flexural stre	ngth of masonry	parallel to bed joir	nts					
		$f_{xd1,app} = f_{xd1}$	+ σ _d = 0.081 N	l/mm²				
Design flexural strength of m	asonry perpendic	cular to bed joints						
		$f_{xd2} = f_{xk2} / \gamma$	_{Mt} = 0.191 N/m	m²				
Elastic section modulus of wall		$Z = t^2 / 6 = 0$	Z = t ² / 6 = 3750000 mm ³ /m					
Moment of resistance paralle	l to bed joints - e	q.6.15						
	$M_{Rd1} = f_{xd1,app} \times Z = 0.302 \text{ kNm/m}$							
Moment of resistance perper	dicular to bed jo	ints - eq.6.15						
		$M_{Rd2} = f_{xd2}$	< Z = 0.718 kNr	m/m				
Design moment in panels								
Orthogonal strength ratio	$\mu = f_{xd1,app}$ /	$\mu = f_{xd1,app} / f_{xd2} = 0.42$						
Using yield line analysis to	calculate bendi	ng moment coef	ficient					
Bending moment coefficient		α = 0.038						
Design moment in wall		$M_{Ed} = \gamma_{fW} \times \alpha \times W_k \times L^2 = 0.686 \text{ kNm/m}$						
				sistance momei				