Barratt Associates Ltd.

Structural, Civil and Geotechnical Engineers

Project reference:	22568
Project Address:	101 Caeconna Road, SA5 5HZ
Client:	Kelly Guarino
Description:	Steel UB with UDL and 1 Point Load Design - rear extension knock through
<u>Design Code</u>	

Design for steel is based on:

- Eurocode 3: Design of steel structures
- Eurocode 5: Design of timber structures

Details prepared by:	I Lumby BSc, CEng (Build), IEng, MCABE, MIMechE, MCMI
Details checked by:	Barratt Associates Ltd. Structural & Civil Engineering Consultanc

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Barratt Associates Ltd.	Project	101 Caeconna Road, SA5 5HZ	Made by	Date	Job No
Structural, Civil and Geotechnical Engineers	Client	Kelly Guarino	IL	4-1-23	22568
Tel: 07946 065051	Description	Load breakdown for structural steel	Checked	Revision	Page No
Email: info@barratt-associates.co.uk	Description		BA	A	1

INTRODUCTION

The following document is associated with the construction work to take place at the mentioned address and contains design calculations for structural elements, as well as approximate schematic arrangements of those elements.

IMPORTANT GUIDANCE ON THE USE OF THIS DOCUMENT (TO BE READ BY ALL PARTIES) - The document should be reviewed in its entirety by the builder, architect (if applicable) and client, along with any other relevant documentation, prior to commencement of the work, and any layouts, instructions or recommendations should be followed. Any deviations from the proposals made without the engineer's consent are beyond the scope of this document and the engineer cannot be held liable for any adverse consequences of such deviations. It is the responsibility of the architect (where applicable), client or builder to notify the engineer if any changes have been made. The calculations carried out in this document have been carried out in good faith based on the proposed and existing dimensions and data provided by the client, architect or site visit. Approval of these calculations and drawings by the Local Authority Building Control should be obtained prior to any ordering of material or fabrication. Where information about the existing arrangements of buildings, such effects (red can existing dimensions and data provided by the client, architect or site visit, approval of these calculations and drawings by the Local Authority Building Control should be obtained prior to any ordering or material or fabrication. Where information about the existing arrangements of buildings, such at feast (red can existed the approximate the explorement to make a example compared to make a calculation or any ordering or labout the existing arrangements of buildings.

as floor / roof span orientations or load-bearing wall arrangements, is not available, the engineer will use their judgement to make assumptions. These, generally conservative assumptions will be clearly outlined within the document, and should be confirmed by a suitably qualified individual on site prior to commencement of the work. The engineer is then to be notified of any discrepancies prior to commencement of the work as design changes may be necessary. Where drawings, construction specifications, method statements or additional design calculations are omitted and are not referenced it is because these have not been requested by the client. These can be made available by the engineer at the client's request. IF IN DOUBT: ASK!!

DESIGN STANDARDS - All calculations are carried out using Eurocodes or British Standards

HEALTH AND SAFETY INFORMATION - Any specific risks that are identified either within these calculations / drawings or any related to this document are to be assessed and managed by the builder / contractor or client. No responsibility for risk assessments or means of mitigating these risks will be taken by the Engineer.

GENERAL CONSTRUCTION NOTES

 Any span dimensions shown in this document are for the purpose of calculations only and are not to be used as a final dimension. Suitable end bearings are to be added to the calculated open span.
 All structural work has is to be carried out by a competent builder in accordance with the requirements of The Building Regulations Part A and the recommendations set out in BS8103 Parts 1-3. • All dimensions are to be checked on site by the builder / contractor / fabricator prior to commencement of fabrication / machining /construction. Any discrepancies between the information outlined herein and the dimensions on site are to be reported to the engineer.

All parties are assumed to be aware of their responsibilities under the Construction Design and Management (CDM) Regulations 2015.

. The client must find out whether the work falls under the Party Wall Act. If it does, the Act requires the client to notify all affected neighbours of proposed works

• The architects drawings are to be read in conjunction with this document and any discrepancies reported to the engineer immediately

• Barratt Associates could have not visited site and therefore take no responsibility for the quality of construction nor its compliance with this document, it is the contractors responsibility to ensure that all works comply with the drawings, notes and assumptions made within these calculations.

 The client should be aware that where beams are installed within existing masonry structures it is likely that minor cracking will occur within the masonry above due to the load redistribution.
 For structural elements not covered by this document it is assumed that a design is being prepared / provided by others, if additional calculations / drawings / specifications are required then please contact Barratt Associates and we can provide a fee for their design. • All proprietary (i.e. off-the-shelf) items specified within this document are to be installed in strict accordance with the manufacturer's recommendations

MASONRY NOTES

• At locations where bearing information is provided on the layout generally this will be in a position where load-bearing masonry (with foundations / support) has been assumed. It should be confirmed by a suitably qualified individual that these walls are load-bearing, and the masonry is to be inspected for suitability prior to commencement of the work. • In many instances historic buildings, will have poor quality masonry and degrading mortar capable of sustaining only a limited amount of compressive force. In such cases the engineer should be

notified as the padstone sizes specified may need to be increased in size. • All padstones specified are to be C35 concrete (as specified in the materials section). Where it is not possible to find "off the shelf" padstone sizes it may be necessary to cast in -situ padstones

• Where existing masonry is deemed to be of poor quality, or the mortar has degraded significantly, the brickwork should be either re-pointed or replaced in its entirety as appropriate prior to loading. • Where steel beams bear directly onto masonry (i.e. no padstones) they are to be bedded onto a dry / level mortar bed.

• Horizontal/Vertical restraint strapping to floors and roofs as indicated in the Building Regulations Part A "Lateral support by roofs and floors" [Diagrams 15 and 16] must be provided

• Ties movement joints should be provided at the following maximum centres; 6m centres in blockwork, 12m centres in brickwork. Joint locations should be as per the architects/LABC recommendations.

STEELWORK NOTES

It is the responsibility of the client or builder to check the span of the opening pier to steel fabrication, and suitable end bearing lengths are to be added to the open span.

Where possible beams installed in pairs should be bolted together through the centre of the webs using M12 bolts @ 500mm cent res with spacer tubes in between.
 All beams are to be seated centrally on padstones, columns or masonry posts unless noted otherwise.

Unless noted otherwise in the design or layout information beams are to bear over the full width of any spreader or post.
 All steel beams which are to support a wall above are to be positioned centrally to that wall.

- All steelwork is to be fabricated and erected in accordance with the least edition of the National Structural Steelwork Specification (NSSS) and Building Regulations. Steelwork finish / paint systems shall be in accordance with the recommendations of the Corus / Tata guide; "The Prevention of Corrosion on Structural Steelwork". Fire protection of steelwork is to be specified by the architect, if intumescent painting is required then the paint system should be compatible with the primer / underlying corrosion protection system. Any welded joints should be carried out by a suitably gualified steel fabricator tested in accordance with the relevant British or European standards.
- Where columns / posts are to be set into or flush up against a masonry wall they are to be fixed / tied into the masonry by the method detailed by building regulations.
- Provide 15mm gap to under-side of steelwork at intersecting wall locations where no bearing information is shown so as to prevent unintended load transfer to non load-bearing walls.

TIMBER NOTES

All timbers are to be C24 unless otherwise stated.

- All timbers are to be a noticed and cut in accordance with current Building Regulations.
 All timber shall be factory treated in accordance with BS5268 'code of practice for the preservation treatment of structural timber'.
- Unless fully built into masonry, new timber joists will be supported using hangers with side flanges to prevent rotation, or side flanges full depth blocking should be provided to joist ends.
 Where built into masonry, all timbers are to have an end bearing length of not less than 100mm.

• Where supported by timber posts, the bearing should be the full width of any such supporting post. Where beams are seated on posts they are to be positioned centrally

• At the top and bottom of timber column / post positions a minimum of 2No horizontal restraint straps and noggins should be provided running orthogonally to the joist / rafter to provide restraint. • For columns / posts directly adjacent to existing masonry, these should be resin anchored into the masonry using M12 Hilti-HY70 anchors (or similar) @ 450mm vertical centres.

. Where steelwork is to be installed in loft conversions ensure there is a 25mm gap between the top of the existing ceiling joist and the underside of the steel beam to avoid unintended load transfer.

FOUNDATION & CONCRETING NOTES

• Foundation design calculations will, unless noted otherwise, be based on an assumed bearing capacity of 100kN/m2. For the design to be valid it should be ensured that the formation level bearing stratum is inspected for suitability on site by an LABC officer or other suitably qualified individual prior to commencement of the work. • In certain incidences 100kN/m2 bearing pressure will not be achievable and so ground improvement or piled foundations & suspended substructures will be required, this will require recalculation of the

• Where extra load or an additional storey is to be added onto an existing wall/foundation, the existing foundation is to be exposed and inspected by the Building Inspector to check it is adequate to

Where existing loadbearing walls have been assumed in this calculation document, the foundation support the wall should be exposed and checked/approved by the Building Inspector provided.

commencement of works.

• Unless this is a document specifically intended to calculate required spread footing depths for shrinkable clays with near-by vegetation the foundation depths will not be specified within this document. Any reference to "depths" of footings or pads will likely refer to the minimum thickness of the concrete required. If new pad foundations are required adjacent to existing strip footing or brick spread foundations then the new pad should extend beyond the minimum depth to at least as deep as the existing adjacent footing. If the existing adjacent footing is shallower than the new pad, local underpinning may be required to prevent undermining.

 General minimum depths for strip footings / spread foundations are not less than 450mm for bearing strata other than clay, and not less than 900mm for footings in shrinkable clay with no nearby vegetation. For foundations in shrinkable clays the proximity of nearby vegetation should be carefully considered and the guidance of the engineer and/or LABC officer should be sought. • Where the thickness of concrete specified in spread foundations is not sufficient to reach a suitable bearing stratum the excavation can be filled using either well compacted crushed hardcore or lean mix concrete up to foundation formation level.

Where open contactor of the existing walls which may reduce the effective area of the foundations, or where the load is to be focused on a particular area of existing foundations, it is advised that the foundations are inspected for suitability by an LABC officer or other suitably qualified individual prior to commencement of the work.
 For foundations in chemically aggressive soil conditions the guidance in BS8500-1 and BRE Special Digest 1 should be followed, if in doubt chemical testing should be undertaken.

• Any site requirements for Radon / Ground Gas Protection should be made by the Architect / LABC officer and not the Engineer unless requested

Revision Rev Date Checked by



Barratt Associa	ates Ltd.	Project	101 Caeconna	Road, SA5 5HZ		Made by	Date	Job No
Structural, Civil and Geotechr		Client	Kelly Guarino			IL	4-1-23	22568
Tel: 07946 065051		Description	Load broakdow	n for structural s	stool	Checked	Revision	Page No
Email: info@barratt-associates.co	o.uk	Description	LUau Dieakuuw		SIEEI	BA	В	2
Loading								
Load breakdown for pitched roof						Roof pitch =	35	0
			Characteristic	Factor	Design			
Dead	Concrete tile, Timber b	attens, and felt	0.55	1.35	0.74	kN/m²		
		ig and services		1.35	0.20	kN/m²		
	Raft	ers & Insulation		1.35	0.27	kN/m²		
Live		Snow		1.5	0.42	kN/m²		
		w _{1,1} '=		w _{1,1} =	1.64	kN/m²		
	Load on plan	w ₁ '=	1.44	w ₁ =	2.00	kN/m²		
Lood brookdown for flat roof								
Load breakdown for flat roof			Characteristic	Factor	Design			
Dead	Ashphalt an	d waterproofing	0.45	1.35	0.61	kN/m²		
		ig and services		1.35	0.20	kN/m²		
		and Insulation		1.35	0.27	kN/m ²		
Live		Snow		1.5	0.42	kN/m²		
		w _{2,1} '=	1.08	w _{2,1} =	1.50	kN/m²		
	Load on plan	· .		w ₂ =	1.50	kN/m²		
Load breakdown for timber floor			Characteristic	Fastar	Decian			
Deed		Flooring	Characteristic 0.15	Factor 1.35	Design 0.20	kN/m²		
Dead	Coilir	Flooring og and services		1.35	0.20	kN/m²		
		•						
	JOISTS	and Insulation		1.35	0.27	kN/m²		
Live		Domestic		1.5	2.25	kN/m²		
	Load on plan	w _{3,1} '= w ₃ '=		w _{3,1} =	2.93 2.93	kN/m² kN/m²		
	Load on plan	w3 —	2.00	w ₃ =	2.55			
Load breakdown for stud wall			Characteristic	Factor	Design			
Dead	12mi	n ply sheathing	0.08	1.35	0.11	kN/m²		
		imber cladding		1.35	0.47	kN/m ²		
		Ids & insulation		1.35	0.32	kN/m ²		
Live	Plasterboared,			1.55	0.32	kN/m ²		
	, laciono carca, i	w _{4,1} '=		w _{4,1} =	1.13	kN/m²		
	Load on plan			w ₄ =	1.13	kN/m²		
Load breakdown for masonry								
		Density		γ =		19 _{kN/m} ³		
				t =	0	.11 m		
		Thickness						
	Chara	Thickness cteristic load		w ₅ '= γt =	2	.09 kN/m²		
	Chara			_{w5} '= γt = = 1.35w ₅ ' =		.09 kN/m² 215 kN/m²		
Load breakdown for external ston		cteristic load						
		cteristic load	w5=		2.82			
		cteristic load Design load	w5=	= 1.35w ₅ ' =	2.82	215 kN/m²		
	lework	cteristic load Design load Density	w5=	= 1.35w ₅ ' = γ =	2.82	215 kN/m² 24 kN/m ³		

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Barratt Associates Ltd.	Project 101 Caeconna Road, SA5 5HZ				e by Date	Job No
Structural, Civil and Geotechnical Engineers Tel: 07946 065051	Client	Kelly Guarino	n - extension inner skin (L cheek Cheo		22568 n Page No
Email: info@barratt-associates.co.uk	Description	wall)		B		3
1.Loads						
See page 2 for load breakdown Loaded width for pitched roof =	d ₁ =	0.00 m				
Loaded width for flat roof =	d ₁ =				8.20	
Loaded width for timber floor =	d ₃ =		\sim	\sim		\sim
Loaded height for stud wall =	d ₄ =	0.00 m			• • •	
Loaded height for masonry wall =	d ₅ =	= 2.40 m			2.9	
Beam self weight	w _s =	0.19 kN/m	1			
Factored load on beam			w=	∑w _n d _n + 1.3		<mark>.20</mark> kN/m
Unfactored load on beam				w'=≥w' _n d _n +	$W_s = 6$	<mark>.01</mark> kN/m
Beam clear span	L=		*End Bearings	must be add	ed*	
Design bending moment Design shear force	M=wL²/8= V _{Ed} =wL/2=	8.62 kN.n • 11.89 kN	า	-	b	
Design shear force	•Ed-••C/2-	11.03 KN				tf
2.Beam properties	UB 17	'8 × 102 × 19#				•
Depth of section		h=	177.8 mm			
Width of section		b=	101.2 mm	h .	tw	
Web thickness		t _w =	4.8 mm			
Flange thickness		t _f =	7.9 mm		_ r	
Root radius		r=	7.6 mm			
Second moment of area		I _x =	1360 cm ⁴			
Plastic modulus		W _{pl} = A=	171 cm ³ 24.3 cm ²		UB 178 × 102	× 19#
Section area		E=	24.3 Cm ² 210000 N/mm ²			
Young's modulus Steel yield strength		E= f _v =	275 N/mm ²			
Shear area	A _v =A-2b	$t_{f} + (t_{w} + 2r)t_{f} =$	9.89 cm ² (but not les	ss than $\eta h_w t_w$)	
3.Cross-section classification						
		ε=√235/fy=		0.924		
Flange		$c=(b-t_w-2r)/2=$		40.6		
		c/t _f =		5.14 <9ε=	8.320	
		Class 1				
Web		$c=h-2t_f-2r=$		146.8 mm		
		c/t _w =		31 <72ε=	66.56	
Cross section resistance partial safty factor		Class 1 Y _{M0} =		1.00		EU3 Table 5.2 EU3 6.1
4.Shear resistance of cross section						
Design shear resistance		$V_{c,Rd} = A_v(f_y/\sqrt{3})/\gamma_{Mo}$	= 15	57.03 kN	011	EU3 (6.18)
Maximum shear force to shear resistance ratio		$V_{Ed}/V_{C,Rd}=$		0.08 <1	OK	EU3 (6.17)
5.Bending resistance of cross section						
Design moment resistance		$M_{c,Rd} = W_p I \times f_y / \gamma_{M0} =$		47.03 kN.m		EU3 (6.14)
Maximum moment to moment resistance ratio		M/M _{c,Rd=}		0.18 <1	OK	
6.Deflection						
Maximum deflection		u _{max} =5w'L ⁴ /384EI=		1.94 mm		
Allowable deflection						
		u'=L/360= u _{max} /u'=		8.06 mm	ок	

Ear fait Associates LideProjectInitial constraints SA 5.0 /2Initial constraintsJob NormConstraintConstraintsConstraintsConstraintsConstraintsConstraintsConstraintsConstraintConstraintsCons								
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LaddImage: Construction of the section of the sectin of the section of		Description		n - extension outer ski			ů	
See app2 for load the landow is leaded with for lat root = $4 = 0.00 \text{ m}$ Landow dish for lat root = $4 = 0.00 \text{ m}$ Landow dish for lat root = $4 = 0.00 \text{ m}$ Landow dish for sup wall = $4 = 0.00 \text{ m}$ Landow dish for sup wall = $4 = 0.00 \text{ m}$ Landow dish for sup wall = $4 = 0.00 \text{ m}$ Landow dish for sup wall = $4 = 0.00 \text{ m}$ Landow dish for sup wall = $4 = 0.00 \text{ m}$ Landow dish for sup wall = $4 = 0.00 \text{ m}$ Landow dish for sup wall = 2.9 Beam soft weight $w_{a} = 0.15 \text{ kVm}$ Landow dish for base Design bending moment Design bending moment Design status and base Landow dish for sup wall = 1.00 m Landow dish for latence is the disk $w_{a} = 0.15 \text{ kVm}$ Landow dish for latence is disk of base Design status and base Landow dish for latence is disk of base Design bending moment Design bending moment Design bending moment Design status and base Landow dish for latence is the disk for latence is the disk for latence is disk of the disk for latence is disk for laten			(cneek wall)		BA	A	4	
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Ladded wight for inher from - Ladded high for mascory wall = $d_1 = 0.00 \text{ m}$ $d_2 = 0.00 \text{ m}$ $d_3 = 2.00 \text{ m}$ $d_4 = 2.40 \text{ m}$ $d_5 = 2.40 \text{ m}$ $d_1 = 2.40 \text{ m}$ $d_2 = 0.15 \text{ kNm}$ Factored load on beam Unfactored load on beam Second moment of area Plastic modulus Second moment of area Second moment of area Plastic modulus Second moment of area Second moment of area Plastic modulus Second moment of area Second moment of area Second moment of area Plastic modulus Second moment of area Plastic modulus Second moment of area Second moment of area		d ₁ =	= 0.00 m					
Loaded height for stud wall = $d_{x} = 0.00 \text{ m}$ 2.9 Beam set weight $w_{x} = 0.19 \text{ kNm}$ Factored load on beam Design bending moment $w_{x} = 0.19 \text{ kNm}$ $w_{x} = 0.19 \text{ kNm}$ The description of beam Design bending moment $w_{x} = 0.19 \text{ kNm}$ $w_{x} = 0.19 \text{ kNm}$ $w_{x} = 0.19 \text{ kNm}$ The description of beam Design bending moment $w_{x} = 0.19 \text{ kNm}$ $w_{x} = 0.19 \text{ kNm}$ The description of beam $w_{x} = 0.19 \text{ kNm}$ The description of beam $w_{x} = 0.19 \text{ kNm}$ $w_{x} = 0.19 \text{ kNm}$ The description of beam $w_{x} = 0.19 \text{ kNm}$ $w_{x} = 0.19 \text{ kNm}$ The description of beam $w_{x} = 0.19 \text{ kNm}$ The description of beam $w_{x} = 0.19 \text{ kNm}$ $w_{x} = 0.19 \text{ kNm}$ The description of beam $w_{x} = 0.19 \text{ kNm}$ $w_{x} = 0.19 \text{ kNm}$ The description of beam $w_{x} = 0.19 \text{ kNm}$ $w_{x} = 0.19 \text{ kNm}$ The description of the description of t	Loaded width for flat roof =	d ₂ =	= 0.60 m			7.93		
Laaded height for masony wall = $q_{\pm} = 2.4 \text{ m}$ 2.9 Beam set weight $w_{\pm} = 0.13 \text{ kVm}$ $w_{\pm} = 0.13 \text{ kVm}$ $w_{\pm} \geq w_{\pm} d_{\pm} + 1.35 w_{\pm} = \frac{7.35 \text{ kVm}}{5.35 \text{ kVm}}$ $w_{\pm} \geq w_{\pm} d_{\pm} + 1.35 w_{\pm} = \frac{7.35 \text{ kVm}}{5.35 \text{ kVm}}$ $w_{\pm} \geq w_{\pm} d_{\pm} + 1.35 w_{\pm} = \frac{7.35 \text{ kVm}}{5.35 \text{ kVm}}$ $w_{\pm} \geq w_{\pm} d_{\pm} + 1.35 w_{\pm} = \frac{7.35 \text{ kVm}}{5.35 \text{ kVm}}$ $w_{\pm} \geq w_{\pm} d_{\pm} + 1.35 w_{\pm} = \frac{7.35 \text{ kVm}}{5.35 \text{ kVm}}$ $w_{\pm} \geq w_{\pm} d_{\pm} + 1.35 \text{ kVm}$ $w_{\pm} = 0.12 \text{ kVm}$ $w_{\pm} = 0.12 \text{ kVm}$ $w_{\pm} = 0.13 \text{ kVm}$ $w_{\pm} = 0.32 \text{ kVm}$ $w_{\pm} = 0.33 \text{ kVm}$ $w_{\pm} =$		-		$\frown \frown \frown \frown$	\sim	\sim	\frown	
Beam set weight $w_{e}^{=}$ 0.19 kNmFactored load on beam $w_{e}^{=}w_{e}^{-1}/28$ $w_{e}^{-1}/28$ $w_{e}^{-1}/28$ Beam clear span $w_{e}^{-1}/28$ $w_{e}^{-1}/28$ $w_{e}^{-1}/28$ Design bearding moment $w_{e}^{-1}/28$ $w_{e}^{-1}/28$ $w_{e}^{-1}/28$ Second moment of area $w_{e}^{-1}/28$ $w_{e}^{-1}/28$ $w_{e}^{-1}/28$ Second noment of area $w_{e}^{-1}/28$ $w_{e}^{-1}/28$ $w_{e}^{-1}/28$ Second rates </th <th>-</th> <th></th> <th></th> <th></th> <th></th> <th>2.0</th> <th></th>	-					2.0		
Factored load on beam $w \sum w_{c} d_{w} + 1.35 w_{c} = \frac{7.93}{5.8} NNm$ Beam clear span"End Bearings must be added"Design bending moment $w \sum w_{c} d_{w} + 1.35 w_{c} = \frac{7.93}{5.8} NNm$ End Bearings moment"End Bearings must be added"Design bending moment $w \sum w_{c} d_{w} + 1.35 w_{c} = \frac{7.93}{5.8} NNm$ Colspan="2">Colspan="2">"End Bearings must be added"Design bear forceUB 178 x 102 x 199Design bear forceUB 178 x 102 x 199We take the forcesTotal take the forcesRecord moment of areaUB 178 x 102 x 199Plange thicknesstake 7.8 mmUpung's modulusSation areaA= 210000 Nmm²Sation areaA=A-201; {U_{1} + 2/2}/{L_{1} = 9.89 cm' (but not less than float, to be colspan="2">Colspan= 6.25 forceColspan= 1Colspan= 1Colspan= 1Colspan= 1Colsp	Lodueu neight for masonry war -	u ₅ –	- 2.40 111			2.9		
Unfactored load on beam $\qquad \qquad \qquad$	Beam self weight	w _s =	= 0.19 kN/n				_	
Beam clear span Design shear forceL2)m M = V/28 = 1.50 km*End Bearings must be added2.Beam properties Depth of sectionUB 178 x 102 x 19# h = 177.8 mm b = 1012 mm b = 1012 mm type = 4.8 mm h = 7.9 mm b = 1012 mm type = 4.8 mm transition of areaImage thicknessImage thickness2.Beam properties Depth of sectionUB 178 x 102 x 19# h = 177.8 mm b = 1012 mm type = 4.8 mm the = 177.8 mm b = 1012 mm type = 4.8 mm the = 177.8 mm b = 1012 mm type = 4.8 mm the = 177.8 mm b = 1012 mm type = 4.8 mm the = 177.8 mm b = 1012 mm type = 4.8 mm the = 177.8 mm the = 170 mm type = 4.8 mm the = 171 mm Section area the = 21000 Mmm² Section area the section area the section area the section area the section area the section are				W				
Design bending moment Design shear force $W=W_1Ze_{-}$ $\overline{8.33}$ kLm 11.50 kN2.Beam properties Depth of socionUB 178 x102 x 197 h = 1012 mm V_{eff} = 4.8 mm V_{eff} = 4.8 mm V_{eff} = 7.8 mm/ V_{eff} = 7.8 mm/ V_{eff} = 8.320 V_{eff} = 8.320 V_{eff} = 8.320 V_{eff} = 5.14 < $e^{e_{-}}$ 8.320 V_{eff} = 8.320 V_{eff} = 7.14 < $e^{e_{-}}$ 8.320 V_{eff} = 8.320 V_{eff} = 31 < 72e = 66.56 C_{1000} 1 and V_{eff} = 31 < 72e = 66.56 C_{1000} = 1.3 table 5.2 Cross section resistance partial satify factorEU3 Table 5.2 V_{eff} = 7.03 kN EU3 (6.16) Maximum shear force to shear resistance ratio V_{eff} = 47.03 kN EU3 (6.16) Maximum shear force to shear resistance ratio V_{eff} = 47.03 kN EU3 (6.16) Maximum other tresistance and W_{eff} = 1.88 mm Aux=-2.400 MM_{eff} = 0.07 <1 OK EU3 (6.16) Maximum dollection U_{eff} = 1.88 mm Aux=-2.400 MM_{eff} = 1.88 mm Aux=-2.400 MM_{eff} = 0.18 mm Auximum dollection	Unfactored load on beam				w'=∑w' _n d _n +\	V _s = 5.8	85 kN/m	
Design shear force $V_{eq}=WL/2 = 11.50 \text{ KN}$ 2.8ean propertiesUB 176 × 102 × 19# h = 107.8 mm b = 101.2 mm tydich of section $h = 177.8 mm$ b = 101.2 mm tydich of sectionWeb thickness $t_{w} = 4.8 mm$ $t_{w} = 4.8 mm$ Second moment of area $t_{w} = 7.6 mm$ $t_{w} = 1360 cm^{2}$ Plastic modulus $W_{em} = 171 cm^{2}$ $h = 171 cm^{2}$ Second moment of area $t_{w} = 1360 cm^{2}$ $h = 0.012 mm$ $t_{w} = 4.24 a cm^{2}$ Young's modulus $E = 210000 \text{ N/mm^{2}}$ Section area $A = 243 cm^{2}$ Young's modulus $E = 210000 \text{ N/mm^{2}}$ Stead area $A_{v}=A-2bLt + (t_{w}-2t)t_{e} = 9.89 cm^{2} (but not less than \eta h_{w} t_{w})3.Cross-section classificationt_{p} = 0.924Flangec_{e}(b_{w}-2t)/2 = 40.6c_{k} = -3.14 < ster areaa_{k} = A/2bLt + (t_{w}-2t)/t_{e} = 9.89 cm^{2} (but not less than \eta h_{w} t_{w})Use bc_{k} = -3.14 < ster a.320Class 1C_{k} = -3.14 < ster a.320Webc_{k} = -3.14 < ster a.320Class 1C_{k} = -3.14 < ster a.320Class 1C_{k} = -3.14 < ster a.320Cross section resistance partial safty factorV_{w} = -3.14 < ster a.31 < 72z = 66.55Class 1C_{k} = -3.14 < ster a.31 < 72z = 66.56Class 1C_{k} = -3.14 < ster a.31 < 72z = 66.56Class 1C_{k} = -3.14 < ster a.31 < 72z = 66.56Class 1C_{k} = -3.14 < ster a.31 < 72z = 66.56Class 1C_{k} = -3.14 < ster a.31 < 72z = 66.56Class 1C_{k} = -3.14 < ster a.31 < 72$	-				gs must be adde	d*		
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Depth of sectionh=177.8 mmWidh of sectionb=101.2 mmWidh of sectionb=101.2 mmWidh of sectionb=101.2 mmPlage thicknesst_=7.9 mmFlange thicknesst_=7.9 mmPlage thicknesst_=7.9 mmPlage thicknesst_=7.9 mmPlastic modulus T_{re} 7.6 mmSecond moment of areat_=1360 cm ² Plastic modulus $W_{pl}=$ 171 cm ² Section area $A=$ 24.3 cm ² Young's modulusE=210000 M/mm ² Shear area $A_{v}=A-2bt_{v}t_{v}(w_{v}2)t_{v}=$ 9.89 cm ² (but not less than $\eta h_w t_w)$ 3.Cross-section classification $t_{v}=A-2bt_{v}t_{v}(w_{v}2)t_{v}=$ 9.89 cm ² (but not less than $\eta h_w t_w)$ 3.Cross-section classification $t_{v}=A-2bt_{v}t_{v}(w_{v}2)t_{v}=$ 9.89 cm ² (but not less than $\eta h_w t_w)$ 4.Shear resistance partial safty factor $t_{vav}=0$ 0.924Class 1Class 1EU3 Table 5.2Cross section resistance partial safty factor $v_{wav}=$ 1.00EU3 Table 5.2Shear resistance of cross section $V_{vav}=A_v(t_v/3)v_{two}=$ 157.03 kNEU3 (a.16, 18)Maximum shear force to shear resistance $V_{wav}=W_{v}bt/v_{wav}=$ 47.03 kN.mEU3 (a.16, 18)Maximum shear force to shear resistance ratio $W_{wav}=W_{v}bt/v_{wav}=$ 47.03 kN.mEU3 (a.16, 18)Maximum anter to moment resistance ratio $M_{wav}=W_{v}t_{v}^{1}/384El=$ 1.89 mmAllowable	2.Beam properties	UB 17	78 × 102 × 19#			b		
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Record moment of area $I_{s} = 7.6 \text{ mm}$ Second moment of area $I_{s} = 1360 \text{ cm}^{2}$ Plastic modulus $W_{p} = 171 \text{ cm}^{2}$ Section area $A = 24.3 \text{ cm}^{2}$ Young's modulus $E = 210000 \text{ N/mm}^{2}$ Steel yield strength $I_{s} = 275 \text{ N/mm}^{2}$ Shear area $A_{s} = A - 2bt_{1} + (t_{w} + 2r)t_{s} = 9.89 \text{ cm}^{2} (but not less than \eta h_{w} t_{w})3.Cross-section classificationE = \sqrt{235/fy} = 0.924Flange e = (b_{w} - 2r)2 = 40.6c h_{s} = 5.14 \text{ < Spt} = 8.320Class 1Web c = h - 2t_{1} - 2r = 146.8 \text{ mm}c I_{w} = 31 < 72r = 66.56Class 1Web c = h - 2t_{1} - 2r = 146.8 \text{ mm}c I_{w} = 31 < 72r = 66.56Class 14.Shear resistance partial safty factor4.Shear resistance of cross sectionDesign shear resistance ratio V_{c,Rd} = 0.07 < 1 OK EU3 (6.18)Maximum shear force to shear resistance ratio V_{c,Rd} = 0.07 < 1 OK EU3 (6.17)5.Bending resistance of cross sectionDesign moment resistance area M_{w,Rd} = M_{w} x _{s} / t_{MW} = 47.03 \text{ kN.m} EU3 (6.14)Maximum moment to moment resistance ratio M_{w,Rd} = 0.18 < 1 OK6.DeflectionMaximum deflection u_{ma} = 5w L^{1/3} / 348 \text{El} = 1.89 \text{ mm}Allowable deflection u = L/280 = 8.06 \text{ mm}$	Web thickness		t _w =	4.8 mm			I	
Root radiusr=7.6 mnSecond moment of areaI,=1360 cm²Plastic modulusWp=171 cm²Section areaA=24.3 cm²Young's modulusE=210000 N/mm²Steel yield strengthI,=275 N/mm²Shear areaA_=A-2btrt(t_w+2r)t_=9.89 cm² (but not less than η h_wt_w)3.Cross-section classification=Flange $c_{=}(25.fy=$ 0.924Class 1Class 1Web $c_{=}(2-1_x-2r)/2=$ 40.6 $c_{+}(2-1_x-2r)/2=$ 146.8 mm $c_{+}(2-1_x-2r)/2=$ 40.6 $c_{+}(2-1_x-2r)/2=$ 10.0EU3 table 5.2Cross sectionDesign shear resistance partial safty factor $V_{+}(1/3)(Y_{+}(-1_x-2r))$ Ashear resistance of cross section $V_{-}(2-1_x-2r)/2=$ Design moment resistance ratio $W_{-}(1/3)(Y_{+}(-1_x-2r))$ Design moment to moment re	Flange thickness		t _f =	7.9 mm	ь	tw		
Plastic modulus Very V_{p} [T 17 cm ³ Section area A= 24.3 cm ³ Young's modulus E= 210000 N/mm ³ Steel yield strength ($y=$ 275 N/mm ² Shear area A _v =A-2btr+(t_w+2r)t _i = 9.89 cm ³ (but not less than $\eta h_w t_w$) 3.Cross-section classification $\frac{e^{-\sqrt{25}fy}}{2} = 0.924$ Flange $e^{-(b^{-t_w}-2r)/2} = 40.6$ $e^{t_w} = 0.924$ $e^{-(b^{-t_w}-2r)/2} = 66.56$ $e^{t_w} = 0.18 \times 10^{-10} K$ EU3 (6.18) Maximum shear force to shear resistance ratio $V_{w,W} = 47.03 kN.m$ EU3 (6.17) Shear the sistance of cross section Design moment resistance ratio $M_{w,Ra^{-t}} = 0.18 \times 10^{-10} K$ EU3 (6.14) Maximum deflection $u_{w,w} = 5w^{t_w} t_{384} El_{w} = 1.89 mm$ Allowable deflection $u_{w} = U_{300} = 8.06 mm$	Root radius			4	"			
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Steel yield strength $t_y =$ 275 N/m² Shear area $A_y=A-2b_1+(t_w+2r)t_l=$ 9.89 cm² (but not less than $\eta h_w t_w)$ 3.Cross-section classification $\varepsilon = \sqrt{235/fy} =$ 0.924 Flange $\varepsilon = \sqrt{235/fy} =$ 0.924 OCIEss 1 0.924 0.924 Web $c_1h_{-2}h_{-2}r_{1/2}r_{-2}$ 14.68 mm $c/t_w =$ 31.<72 $\varepsilon =$ 66.56 Cross section resistance partial safty factor $V_{wo}=$ 1.00 EU3 Table 5.2 Ashear resistance of cross section $EU3$ Table 5.2 $Cross section resistance ratio V_{e,Re}=A_{e}(f_{e}/\sqrt{3})Y_{hw}= 157.03 kN EU3 (6.16) Design shear resistance of cross section V_{e,Re}=A_{e}(f_{e}/\sqrt{3})Y_{hw}= 157.03 kN EU3 (6.17) Stending resistance of cross section V_{e,Re}=A_{e}(f_{e}/\sqrt{3})Y_{hw}= 157.03 kN EU3 (6.17) Stending resistance of cross section V_{e,Re}=A_{e}(f_{e}/\sqrt{3})Y_{hw}= 47.03 kN.m EU3 (6.14) Maximum moment to moment resistance ratio M_{w_{e,Re}} 0.18 <1 OK OK Geneticion u_{windw}=300 = 8.06 mm 0.06 mm 0.06 mm 0.06 mm $								
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3.Cross-section classification $\epsilon = \sqrt{235/fy} = 0.924$ Flange $c = (b + l_w - 27)/2 = 40.6$ $c/t_e = 5.14 < 9\epsilon = 8.320$ $c/t_e = 0.14 < 9\epsilon = 8.320$ Uveb $c = h \cdot 21 + 27 = 146.8 \text{ mm}$ $c/t_w = 31 < 72\epsilon = 66.56$ Class 1 EU3 Table 5.2 Cross section resistance partial safty factor $V_{MO} = 1.00$ EU3 Table 5.2 A.Shear resistance of cross section EU3 Table 5.2 EU3 Table 5.2 Design shear resistance of cross section EU3 (6.18) EU3 (6.18) Maximum shear force to shear resistance ratio $V_{c,Ra} = A_c (f_s/N) / M_{MO} = 157.03 \text{ kN}$ EU3 (6.16) 5.Bending resistance of cross section Eu3 (6.17) EU3 (6.18) EU3 (6.17) 5.Bending resistance of cross section Design moment resistance ratio $M_{w,Rd} = W_p / M_s / Y_{MO} = 47.03 \text{ kN.m}$ EU3 (6.14) Maximum moment to moment resistance ratio MM _{w,Rd} = 0.18 < 1		∆ –Δ-2h	,		less than nh t)			
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Flange $c=(b-l_w^2-r)/2=$ 40.6 $c/t_=$ $5.14 < 9te=$ 8.320 Class 1 $Class 1$ $Class 1$ Web $c+h-2t_r-2r=$ $146.8 mm$ $c/t_w=$ $31 < 72te=$ 66.56 Class 1 $Class 1$ $EU3$ Table 5.2 Cross section resistance partial safty factor $V_{M0}=$ 1.00 $EU3$ (6.18) 4.Shear resistance of cross section $V_{c,Rd}=A_v(t_v/3)/y_{M0}=$ 157.03 kN $EU3$ (6.18) Maximum shear force to shear resistance ratio $V_{e,Rd}=A_v(t_v/3)/y_{M0}=$ $0.07 < 1$ OK $EU3$ (6.18) Design moment resistance of cross section $V_{e,Rd}=M_v[x_1/v_{M0}=$ 47.03 kN.m $EU3$ (6.14) Maximum moment to moment resistance ratio $MM_{e,Rd}=W_p[x_1/v_{M0}=$ $0.18 < 1$ OK 6.Deflection $M_{max}=5wL^4/384El=$ 1.89 mm $Allowable$ deflection $u=L/360=$ 8.06 mm	3.Cross-section classification							
$c_{t\mu}$ $5.14 < 92^{\mu}$ 8.320 Web $c_{th}-2t_{2}2r_{2}$ 146.8 mm c_{tw} = $31 < 72\epsilon$ 66.56 Class 1 c_{tw} = $31 < 72\epsilon$ Cross section resistance partial safty factor Y_{M0} = 1.00 EU3 Table 5.2Cross section resistance of cross section $EU3$ Table 5.2 $EU3$ Table 5.2Design shear resistance of cross section $V_{c,Rd}=A_{v}(t_{v}/3)/v_{M0}$ = 157.03 kN $EU3$ (6.18)Maximum shear force to shear resistance ratio $V_{c,Rd}=A_{v}(t_{v}/3)/v_{M0}$ = $0.07 < 1$ OK $EU3$ (6.17)S.Bending resistance of cross section $V_{c,Rd}=W_{p} xt_{v}/v_{M0}$ = 47.03 kN.m $EU3$ (6.14)Maximum moment resistance ratio $M_{c,Rd}=W_{p} xt_{v}/v_{M0}$ = 47.03 kN.m $EU3$ (6.14)Maximum deflection $u_{max}=5w'L^4/384E $ = 1.89 mm $A wab $ Allowable deflection $u'=L/360$ = 8.06 mm $V_{e'}$			ε=√235/fy=					
WebClass 1 $c=h-2t_2-2r=$ $ct_w=$ 146.8 mm $c_w=$ 146.8 mm $c_w=$ 146.8 mm $c_w=$ 146.8 mm $ct_w=$ 140.3 table 5.2 $ct_w=$ 140.8 mm $ct_w=$ <td>Flange</td> <td></td> <td>c=(b-t_w-2r)/2=</td> <td></td> <td>40.6</td> <td></td> <td></td>	Flange		c=(b-t _w -2r)/2=		40.6			
Web $c=h-2l_{T}-2r=$ $c/l_w=$ 146.8 mm $c/l_w=$ c_{1} de_{1} <td></td> <td></td> <td>c/t_f=</td> <td></td> <td>5.14 <9ε=</td> <td>8.320</td> <td></td>			c/t _f =		5.14 <9ε=	8.320		
$c_{t_w} = 31 < 72\varepsilon = 66.56$ $Cass 1 \\ Cross section resistance partial safty factor PM0 = 1.00$ $c_{Lass 1} \\ M_{M0} = 1.00$ $c_{L33 1} \\ Lass 1 $								
Cross section resistance partial safty factorEU3 Table 5.2 EU3 Table 5.2 EU3 6.14.Shear resistance of cross section Design shear resistance $V_{c,Rd}=A_v(f_y/\sqrt{3})/\gamma_{Mo}=$ 1.00EU3 6.18 EU3 (6.18) EU3 (6.18)Maximum shear force to shear resistance ratio $V_{c,Rd}=V_{c,Rd}=$ 0.07<1	Web				146.8 mm			
Cross section resistance partial safty factor Y_{M0} =1.00EU3 6.1 4.Shear resistance of cross section $V_{c,Rd}=A_v(f_y/\sqrt{3})/Y_{M0}$ =157.03 kNEU3 (6.18)Design shear resistance to shear resistance ratio $V_{c,Rd}=A_v(f_y/\sqrt{3})/Y_{M0}$ =157.03 kNEU3 (6.18) 5.Bending resistance of cross section $V_{c,Rd}=W_p xf_y/Y_{M0}$ =47.03 kN.mEU3 (6.14)Design moment resistance $M_{c,Rd}=W_p xf_y/Y_{M0}$ =47.03 kN.mEU3 (6.14) 6.Deflection $W/M_{c,Rd}$ =0.18 <1OKMaximum deflection $u_{max}=5wL^4/384EI=$ 1.89 mmAllowable deflectionAllowable deflection $u'=L/360=$ 8.06 mm $V_{c,Rd}$					31 <72ε=	66.56		
A.Shear resistance of cross sectionDesign shear resistance $V_{c,Rd}=A_v(f_y/\sqrt{3})/\gamma_{Mo}=$ 157.03 kNEU3 (6.18)Maximum shear force to shear resistance ratio $V_{Ed}/V_{C,Rd}=$ 0.07 <1	Cross section registered partial activity factor				1.00			
Design shear resistance $V_{c,Rd}=A_v(f_y/\sqrt{3})/\gamma_{Mo}=$ 157.03 kNEU3 (6.18)Maximum shear force to shear resistance ratio $V_{c,Rd}=$ $0.07 < 1$ OKEU3 (6.17) 5.Bending resistance of cross section $M_{c,Rd}=W_p xf_y/\gamma_{M0}=$ 47.03 kN.mEU3 (6.14)Design moment resistance $M_{c,Rd}=W_p xf_y/\gamma_{M0}=$ 47.03 kN.mEU3 (6.14)Maximum moment to moment resistance ratio $M/M_{c,Rd}=$ $0.18 < 1$ OK 6.Deflection $u_{max}=5w'L^4/384El=$ 1.89 mmAllowable deflection $u'=L/360=$ 8.06 mm	cross section resistance partial saity factor		Υмо=		1.00		EU3 6.1	
Maximum shear force to shear resistance ratio $V_{Ed}/V_{C,Rd}$ =0.07 <1OKEU3 (6.17) 5.Bending resistance of cross section Design moment resistance $M_{c,Rd}$ = $W_p xf_y / Y_{M0}$ =47.03 kN.mEU3 (6.14)Maximum moment to moment resistance ratio $M/M_{c,Rd}$ =0.18 <1								
5.Bending resistance of cross sectionDesign moment resistance $M_{c,Rd}=W_p xf_y/\gamma_{M0}=$ 47.03 kN.mEU3 (6.14)Maximum moment to moment resistance ratio $M/M_{c,Rd}=$ 0.18 <1	•			=		01/		
Design moment resistance $M_{c,Rd}=W_p xf_y/\gamma_{M0}=$ 47.03 kN.mEU3 (6.14)Maximum moment to moment resistance ratio $M/M_{c,Rd}=$ 0.18 <1	waximum snear force to shear resistance ratio		$v_{Ed}/v_{C,Rd}=$		0.07 <1	ОК	EU3 (6.17)	
Maximum moment to moment resistance ratio M/M _{c,Rd=} 0.18 <1	5.Bending resistance of cross section							
6.Deflection u _{max} =5w'L ⁴ /384El= 1.89 mm Allowable deflection u'=L/360= 8.06 mm	Design moment resistance				47.03 kN.m		EU3 (6.14)	
Maximum deflection u _{max} =5w'L ⁴ /384EI= 1.89 mm Allowable deflection u'=L/360= 8.06 mm	Maximum moment to moment resistance ratio		$M/M_{c,Rd=}$		0.18 <1	OK		
Allowable deflection u'=L/360= 8.06 mm			4					
				:				
Actual to permissible deflection ratio $U_{max}/U=0.23 < 1$ OK						01/		
			u _{max} /u =		0.23 <1	UK		

Beam is OK

Barratt Associates Ltd.	Project	101 Caeconna Roa	ad, SA5 5HZ	Made	by Date	Job No
Structural, Civil and Geotechnical Engineers	Client	Kelly Guarino	IL	4-1-23	22568	
Tel: 07946 065051			OL and 1no pont load - o			Page No
Email: info@barratt-associates.co.uk	Description	skin to knock throu		BA	В	5
1.Loads						
See page 2 for load breakdown		0.00		Р	h	
Loaded width for pitched roof =	d ₁ = d ₂ =		, <u> </u>		b	
Loaded width for flat roof = Loaded width for timber floor =	d ₂ = d ₃ =			\sim \sim \sim \sim	$(\vee \vee \vee$	
Loaded height for stud wall =	d ₄ =		A			В
Loaded height for masonry wall =	d ₅ =	= 2.40 m		L =	4.7 m	
				a =	1.80 m	
				b =	2.90 m	
Beam self weight	w _s =	= 0.25 kN/n	n			_
Factored load on beam				w=∑w _n d _n + 1.35		2 kN/m
Unfactored load on beam				w'=∑w' _n d _n +	w _s = 8.96	kN/m
Unfactored Point load					P' = 17.00) kN
Factored Point load from					P= 23.00) kN
Beam Clear Span		L=		learings must be	e added*	
Design bending moment	-	$L^{2}/8]+[Pab/L]=$	59.84 kN.m			
	V _{Ed} =(wL/2)+Max[(43.38 kN			
Reactions		R _A = R _B =	43.38 kN 37.99 kN		b	
		K _B =	37.99 KN			tf
2 Poom proportion		54 x 102 x 25#		t		-
2.Beam properties	06.2					
Depth of section Width of section		h=	257.2 mm	h	tw	
		b=	101.9 mm	"		
Web thickness		t _w =	6 mm			
Flange thickness		t _f =	8.4 mm	Ļ		
Root radius Second moment of area		r= I _x =	7.6 mm 3410 cm ⁴			
Plastic modulus		W _{pl} =	306 cm ³		UB 254 x 102 x	25#
					OB 234 X 102 X	20#
Section area		A=	32 cm ²			
Young's modulus		E=	210000 N/mm ²			
Steel yield strength		f _y =	275 N/mm ²			
Shear area	A _v =A	$-2bt_f+(t_w+2r)t_f=$	16.66 cm ² (but not le	ess than ηh _w t _w)		
3.Cross-section classification		100010				
_		ε=√235/fy=		0.033		
Flange		$c=(b-t_w-2r)/2=$	-3	321.75		
		c/t _f =		-1.25 < <mark>9ε=</mark>	0.30	
		Class 1				
web		c=h-2t _f -2r=	-1	1058.4 mm		
		c/t _w =		-10 <72ε=	2.41	
		Class 1		4.00		EU3 Table 5.2
Cross section resistance partial safty factor		γ _{м0} =		1.00		EU3 6.1
4.Shear resistance of cross section						
Design shear resistance		$V_{c,Rd} = A_v (f_y / \sqrt{3}) / \gamma_{Mo} =$	-	264.5 kN		EU3 (6.18)
Maximum shear force to shear resistance ratio		V _{Ed} /V _{C,Rd} =		0.16 <1	OK	EU3 (6.17)
5.Bending resistance of cross section						
Design moment resistance		$M_{c,Rd} = W_p I \times f_v / \gamma_{M0} =$		84.15 kN.m		EU3 (6.14)
Maximum moment to moment resistance ratio		M/M _{c.Rd=}		0.71 <1	ок	_00 (0.14)
				0.11 1	UN UN	
6.Deflection						
Maximum deflection u _{max} =[5w'L ⁴ /384EI]+[((Pab(b+I)/27FI	IL)√(3a(I +b))]=		12.71 mm	(Assumes a>	·b)
Allowable deflection	J (((, ~~(, ~) [) Z []	u'=L/360=		13.06 mm		-1
Actual to permissible deflection ratio		u/u/=		0.97 <1	ок	
		Beem is OK				

Beam is OK

Project 101 Caeconna Road, SAS 5HZMade byDateJob NoStructural, Civil and Geotechnical Engineers to 0734 oddshClientKelly GuarinoLMade byDateJob NoClientKelly GuarinoLMade byDateJob NoClientKelly GuarinoLMade byDateJob NoLoaddClientKelly GuarinoLMade byDateJob NoLoadd with for pitcher for a Loadd with for macron wall =Structural Civic and the fair colspan="2">Structural Civic fair colspan="2"Loadd with for pitcher for a Loadd with for macron wall =Client Kally Cuvic fair colspan="2">Structural Civic fair colspan="2">Structural Civic fair colspan="2"Client Kally Cuvic fair colspan="2"Client Kally Cuvic fair colspan="2"Loadd with for pitcher for a Loadd with for macron wall and fair colspan="2"Structural Civic
Structural, Civil and Geotechnical Engineers Tei: 07346 065051 East info@Barrat-associates.co.ukClientkelly GuannoIL4.1-2322568 Page No BA2000 BA2
Enablish for Gibbarratt-associates.coukDescriptionSteel Beam Design - inner skin to knock throughBAA611.oadsSee page 210 load breakdownLoaded width for lither ford =Loaded width for lither ford =Loaded width for lither ford =Loaded width for itar col =Loaded on beamUnactered load on beamUnactered load on beamDesign bending momentDesign bending momentDesign bending momentDesign bending momentDesign bending momentDesign bending moment of areaPlaste modulusSection areaYoung's modulusSection areaYoung's modulusSteel yield strengthSteel yield strengthShear areaAque area
1 LoadsSee page 2 for load breakdownLoaded width for pitched root =Loaded width for flat root =Loaded width for flat root =Loaded width for flat root =Loaded width for stud wall =Loaded height for stud wall =Loaded width for stud wall =Loade width fo
Loaded width for pitched root = $d_1 = 3.70 \text{ m}$ Loaded width for flat root = $d_2 = 0.00 \text{ m}$ Loaded width for flat root = $d_2 = 0.00 \text{ m}$ Loaded height for stud wall = $d_2 = 0.00 \text{ m}$ Loaded height for stud wall = $d_3 = 0.19 \text{ kN/m}$ Factored load on beam Beam self weight = $d_3 = 0.19 \text{ kN/m}$ Factored load on beam Beam loader span Design bending moment Design bending moment Design bending moment Design bending moment Design bending moment Design shear force $UE 254 \times 102 \times 254$ Depth of section $UE 254 \times 102 \times 254$ Depth of section $UE 254 \times 102 \times 254$ Design shear force $UE 254 \times 102 \times 254$ Section area $A = 32 \text{ cm}^3$ Young's modulus $E = 210000 \text{ Nmm}^3$ Shear area $A_{a} = 32 \text{ cm}^3$ Design shear $A_{a} = 32 \text{ cm}^3$ Shear area $A_{a} = 4.20 \text{ cm}^2$ Shear area $A_{a} = 0.924$ Flange $c_{0}(+_{a}27)Z_{a} = 4.30 \text{ shear} = 8.320$ Class 1
Loaded width for flat roof = $d_2 = 0.00 \text{ m}$ $d_3 = 1.40 \text{ m}$ Loaded height for stud wall = $d_4 = 0.00 \text{ m}$ $d_5 = 1.40 \text{ m}$ Loaded height for stud wall = $d_4 = 0.00 \text{ m}$ $d_5 = 2.40 \text{ m}$ Beam self weight $w_5 = 0.19 \text{ kN/m}$ Factored load on beam $w_{plactored load on beam}$ Beam clear span Design bending moment Design shear force $W_{Eg} = WL/2 = 43.49 \text{ kN}$ $w_{Eg} = 18.51 \text{ kN/m}$ 2.8em properties $UB 254 \times 102 \times 254$ Depth of section $be = 101.9 \text{ mm}$ $b = 101.9 \text{ mm}$ Width of section $b = 101.9 \text{ mm}$ $b = 101.9 \text{ mm}$ Second moment of area $l_{g} = 3.40 \text{ cm}^2$ $W_{pl} = 306 \text{ cm}^3$ $UB 254 \times 102 \times 254$ Section area $A = 32 \text{ cm}^3$ Young's modulus $E = 210000 \text{ Nmm}^3$ Section area $A = 32 \text{ cm}^3$ Young's modulus $E = 210000 \text{ Nmm}^3$ Section area $A = 32 \text{ cm}^3$ Young's modulus $E = 210000 \text{ Nmm}^3$ Shear area $A = 32 \text{ cm}^3$ Schear area $A = 32 cm$
Loaded width for timber floor = Loaded height for masonry wall = $d_{a} = 1.40 \text{ m}$ $d_{a} = 0.00 \text{ m}$ $d_{a} = 2.40 \text{ m}$ $u_{a} = 2.40 \text{ m}$ $u_{a} = 0.19 \text{ kN/m}$ Factored load on beam Unfactored load on beam Unfactored load on beam Unfactored load on beam Beam celar span Design beam force 2.80 m 2.80 m M=wL/Re Still KNm Veget wL/Ze 4.102 x 25t ^t Depth of section We bit kness I leage thickness I leag
Loaded height for stud wall = $d_{s} = 0.00 \text{ m}$ Laded height for masony wall = $d_{s} = 2.40 \text{ m}$ 4.7 Beam self weight $u_{s} = 0.19 \text{ kN/m}$ Factore load on beam Beam clear span Design bending moment Design shear force $U_{Ed} = \frac{4.1}{3.34} \text{ kN/m}$ 2.Beam properties $U_{Ed} = \frac{4.1}{3.34} \text{ kN/m}$ 2.Beam properties $U_{Ed} = \frac{4.1}{3.34} \text{ kN/m}$ 2.Beam properties $U_{Ed} = \frac{4.1}{3.34} \text{ kN/m}$ $U_{Ed} = \frac{2.1}{3.34} \text{ kN/m}$ $U_{Ed} = \frac{2.1}{3.34} \text{ kN/m}$ $U_{Ed} = \frac{2.1}{3.34} \text{ kN/m}$ $U_{Ed} = 2.1 \text{ kOM} \text{ kM/m}^2$ $U_{Ed} = 0.10 \text{ kM/m}^2$ $U_{Ed} = 0.10 \text{ kM/m}^2$ $U_{Ed} = 0.10 k$
Leaded height for masonry wall = $d_{s} = 2.40 \text{ m}$ 4.7 Beam self weight $w_{s} = 0.19 \text{ kN/m}$ Factored load on beam Unfactored load on beam Design beam force $w_{s} = 0.19 \text{ kN/m}$ Factored load on beam Unfactored load on beam Design beam force $w_{s} = 0.19 \text{ kN/m}$ The dearings must be added $w_{w_{s}} = 0.19 \text{ kN/m}$ The dearings must be added $w_{w_{s}} = 0.19 \text{ kN/m}$ The dearings must be added $w_{w_{s}} = 0.19 \text{ kN/m}$ The dearings must be added $w_{w_{s}} = 0.19 \text{ kN/m}$ The dearings must be added $w_{w_{s}} = 0.19 \text{ kN/m}$ The dearings must be added $w_{w_{s}} = 0.19 \text{ kN/m}$ The dearings must be added $w_{w_{s}} = 0.19 \text{ kN/m}$ The dearings must be added $w_{w_{s}} = 0.19 \text{ kN/m}$ The dearings must be added $w_{w_{s}} = 0.19 \text{ kN/m}$ The dearings must be added $w_{w_{s}} = 0.19 \text{ kN/m}$ The dearings must be added $w_{w_{s}} = 0.101.9 \text{ mm}$ We bitchness $w_{s} = 0.19 \text{ mm}$ Record moment of area $w_{s} = 0.19 \text{ mm}$ Section area Nounds $w_{pl} = 306 \text{ cm}^{3}$ Section area A = 32 \text{ cm}^{3} Subley lied is the grift $w_{s} = 210000 \text{ N/mm}^{3}$ Shear area A $w_{s} = A-2bt_{s} + (t_{w}+2t)t_{s} = 16.66 \text{ cm}^{2} (but not less than \eta h_{w} t_{w})3. Cross-section classificationFinagew_{s} = 0.326 \text{ m}^{3}w_{s} = 0.326 \text{ m}^{3}$
Factored load on beam $w = \sum w_n d_n + 1.35 w_s = 18.51 \text{ kV/m}$ Beam clear span $w = \sum w_n d_n + 1.35 w_s = 18.51 \text{ kV/m}$ Beam clear span $w = \sum w_n d_n + 1.35 w_s = 18.51 \text{ kV/m}$ Beam clear span $w = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $w = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $v = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $v = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $v = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $v = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $v = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $v = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $v = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $v = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $v = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $v = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $v = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ We clear span $v = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $v = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $v = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $v = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $v = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $w = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $w = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $w = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $w = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $w = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $w = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $w = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $w = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $w = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $w = 2W_n d_n + w_s = 13.34 \text{ kV/m}$ Beam clear span $w = 2W_n d_n + W_n = 13.34 \text{ kV/m}$ Beam clear span $w = 2W_n d_n + W_n + W_n$
Unfactored load on beam $w= 2w^{2}m^{2}n^{2}m^{2}w^{2}m^{2}m^{2}m^{2}m^{2}m^{2}m^{2}m^{2}m$
Beam clear spanLef4.7 m"End Bearings must be added"Design shear forceM=wL/38= V_{Ed} =WL/2=51.11 kN.m 43.49 kN"End Bearings must be added"2.Beam propertiesUB 254 x 102 x 25#Image: Constraint of the sectionImage: Constraint of the sectionImage: Constraint of the sectionWidth of sectionb=101.9 mm twist be addedImage: Constraint of the sectionImage: Constraint of the sectionImage: Constraint of the sectionWeb thicknesstype=6 mm type=Image: Constraint of the sectionImage: Constraint of the sectionImage: Constraint of the sectionSecond moment of areatype=306 cm³UB 254 x 102 x 25#Platic modulusWppi=306 cm³UB 254 x 102 x 25#Section areaA=32 cm²Young's modulusE=210000 N/mm²Steel yield strengthfy=275 N/mm²Shear areaA_p=A-2btr+(t_w+2r)t_F=16.66 cm² (but not less than nhwtw)Acres-section classificationE=0.924Flangec=0.924Class 1c=8.320
Design bending moment Design shear force $M=wL^2/8=$ $V_{Ed}=wL/2=$ 51.11 kN.m 43.49 kN 2.Beam properties Depth of sectionUB 254 x 102 x 25# b=101.9 mm the=101.9 mm the=Width of sectionb=101.9 mm the=101.9 mm the=We thicknesstu=8.4 mm r=7.6 mm 3.06 cm³100.2 x 25#Flange thicknesstu=8.4 mm the=100.2 x 25#Voor radiusr=7.6 mm the=100.2 x 25#Section area $A=$ 3.2 cm²Young's modulus $W_{pl}=$ 306 cm³UB 254 x 102 x 25#Section area $A=$ 3.2 cm²Young's modulus $E=$ 210000 N/mm² ty=275 N/mm²Shear area $A_v=A-2bt_l+(t_w+2r)t_r=$ 16.66 cm² (but not less than $\eta h_w t_w$)3.Cross-section classification $\epsilon=\sqrt{235/ty=}$ 0.924 c=(b+t_w-2r)/2=Flange $c=(b+t_w-2r)/2=$ 40.35 c/t=6.320 class 1
Design shear force V_{Ed} =WL/2=43.49 kN2.Beam propertiesUB 254 x 102 x 25#Depth of sectionh=2.57.2 mmh=Web thicknesstw=Flange thicknesstw=Flange thicknesstr=Root radiusr=7.6 mmtw=Second moment of areatw=1 k=3410 cm ⁴ Plastic modulusWpi=306 cm3UB 254 x 102 x 25#Section areaA=2 s2 cm2Young's modulusE=2 theel yield strengthfy=2 shear areaA_=A-2bt_1+(t_w+2r) t_t=1 6.66 cm2 (but not less than $\eta h_w t_w$)Section classificationE= $\sqrt{235}/hy=$ 0.924Class 1
2.Beam propertiesUB 254 x 102 x 25#Depth of sectionh=Width of sectionb=Width of sectionb=We thicknesstw=Flange thicknesstu=Root radiusr=Root radiusr=Second moment of arealx=Plastic modulusWpl=Section areaA=Young's modulusE=Steel yield strengthfy=Av=A-2btr+(tw+2r)tr=16.66 cm² (but not less than η hw,tw)Scross-section classificationFlange $c=(b-tw-2r)/2=$ Flange $c=(b-tw-2r)/2=$ 4.80 <9e=
Interpretended Depth of sectionh=257.2 mmWidth of sectionb=101.9 mmhWeb thicknesstw=6 mmhFlange thicknesstv=8.4 mmRoot radiusr=7.6 mmSecond moment of area1x=3410 cm ⁴ Plastic modulusWpi=306 cm ³ UB 254 x 102 x 25#Section areaA=32 cm ² Young's modulusE=210000 N/mm ² Steel yield strengthfy=275 N/mm ² Shear areaAv=A-2btr+(tw+2r)tr=16.66 cm ² (but not less than $\eta h_w t_w$)3.Cross-section classificationFlange $c=\sqrt{235/fy=}$ 0.924Class 1 $c=1b+tw^2r)/2=$ 40.35c/t=4.80 <9t=
Width of section $b = 101.9 \text{ mm}$ $Web thickness t_w = 6 \text{ mm}Flange thickness t_r = 8.4 \text{ mm}Root radius r = 7.6 \text{ mm}Second moment of area l_x = 3410 \text{ cm}^4Plastic modulus W_{pl} = 306 \text{ cm}^3Section area A = 32 \text{ cm}^2Young's modulus E = 210000 \text{ N/mm}^2Steel yield strength f_y = 275 \text{ N/mm}^2Shear area A_v = A - 2bt_r + (t_w + 2r)t_r = 16.66 \text{ cm}^2 (but not less than \eta h_w t_w)3.Cross-section classificationFlange e = \sqrt{235/fy} = 0.924Flange e = (b - t_w - 2r)/2 = 40.35c/t_r = 4.80 < 9e = 8.320Class 1$
Web thickness $t_w=$ 6 mmh $t_w=$ f mmFlange thickness $t_r=$ 8.4 mmh $t_r=$ 7.6 mmRoot radius $r=$ 7.6 mm $t_x=$ 3410 cm ⁴ $UB 254 \times 102 \times 25\#$ Second moment of area $A=$ 32 cm ² $UB 254 \times 102 \times 25\#$ Plastic modulus $E=$ 210000 N/mm ² $UB 254 \times 102 \times 25\#$ Section area $A=$ 32 cm ² Young's modulus $E=$ 210000 N/mm ² Steel yield strength $f_y=$ 275 N/mm ² Shear area $A_y=A-2Dt_f+(t_w+2r)t_f=$ 16.66 cm ² (but not less than $\eta h_w t_w$)Scross-section classificationE= $\sqrt{235/fy=}$ 0.924Flange $c=(b-t_w-2r)/2=$ $c/t_f=$ $4.80 < 9e=$ 8.320 Class 1
Web thicknesstw=6 mm1Flange thicknesstr=8.4 mmRoot radiusr=7.6 mmSecond moment of area $l_x=$ 3410 cm ⁴ Plastic modulus $W_{pl}=$ 306 cm ³ UB 254 x 102 x 25#Section areaA=32 cm ² Young's modulusE=210000 N/mm ² Steel yield strength $f_y=$ 275 N/mm ² Shear area $A_v=A-2bt_f+(t_w+2r)t_f=$ 16.66 cm ² (but not less than $\eta h_w t_w$)3.Cross-section classificationFlange $c=(b-t_w-2r)/2=$ 40.35 $c/t_f=$ 4.80 <9 ϵ =8.320Class 1Class 11
Rot radiusr=7.6 mmSecond moment of areaIx=3410 cm ⁴ Plastic modulusWpl=306 cm ³ Section areaA=32 cm ² Young's modulusE=210000 N/mm ² Steel yield strengthfy=275 N/mm ² Shear areaAy=A-2btr+(tw+2r)tr=16.66 cm ² (but not less than $\eta h_w t_w$)3.Cross-section classificationFlange $\epsilon=\sqrt{235/fy=}$ 0.924c=(b-tw-2r)/2=40.35c/tr=4.80 <9ε=
Second moment of area $I_x = 3410 \text{ cm}^4$ Plastic modulus $W_{pl} = 306 \text{ cm}^3$ UB 254 x 102 x 25# Section area $A = 32 \text{ cm}^2$ Young's modulus $E = 210000 \text{ N/mm}^2$ Steel yield strength $f_y = 275 \text{ N/mm}^2$ Shear area $A_y = A - 2bt_f + (t_w + 2r)t_f = 16.66 \text{ cm}^2 (but not less than \eta h_w t_w)$ 3.Cross-section classification $\frac{\epsilon = \sqrt{235/fy} = 0.924}{C = (b - t_w - 2r)/2 = 40.35}$ Flange $\frac{c(t_p = 4.80 < 9\epsilon = 8.320}{Class 1}$
Plastic modulus W July 200 cm³ UB 254 x 102 x 25# Section area A= 32 cm² Young's modulus E= 210000 N/mm² Steel yield strength $f_y=$ 275 N/mm² Shear area Av=A-2btr+(tw+2r)tr= 16.66 cm² (but not less than $\eta h_w t_w$) 3.Cross-section classification $\epsilon=\sqrt{235/fy=}$ 0.924 Flange $c=(b-t_w-2r)/2=$ 40.35 $c/t_F=$ 4.80 <9 ϵ = 8.320 Class 1 Lass 1 Lass 1
Section areaA=32 cm²Young's modulusE=210000 N/mm²Steel yield strength $f_y=$ 275 N/mm²Shear area $A_v=A-2bt_f+(t_w+2r)t_f=$ 16.66 cm² (but not less than $\eta h_w t_w$)3.Cross-section classificationE= $\sqrt{235/fy=}$ 0.924Flange $c=(b-t_w-2r)/2=$ 40.35 $c/t_f=$ 4.80 <9 ϵ =8.320Class 1
Young's modulusE=210000 N/mm²Steel yield strength $f_y=$ 275 N/mm²Shear area $A_v=A-2bt_f+(t_w+2r)t_f=$ 16.66 cm² (but not less than $\eta h_w t_w$)3.Cross-section classificationE= $\sqrt{235/fy=}$ 0.924Flange $c=(b-t_w-2r)/2=$ 40.35 $c/t_f=$ 4.80 <9 ϵ =8.320Class 1Class 11
Steel yield strength $f_y =$ 275 N/mm²Shear area $A_v = A - 2bt_f + (t_w + 2r)t_f =$ 16.66 cm² (but not less than $\eta h_w t_w$) 3.Cross-section classification $\epsilon = \sqrt{235/fy} =$ 0.924C = (b - t_w - 2r)/2 =0.924C = (b - t_w - 2r)/2 =0.924C - (b - t_w - 2r)/2 =0.926C - (b - t_w - 2r
Shear area $A_v = A - 2bt_r + (t_w + 2r)t_r =$ 16.66 cm² (but not less than $\eta h_w t_w$) 3.Cross-section classification $\epsilon = \sqrt{235/fy} =$ 0.924 Flange $c = (b - t_w - 2r)/2 =$ 40.35 $c/t_r =$ 4.80 <9 ϵ = 8.320 Class 1 Class 1 Class 1
$\epsilon = \sqrt{235/fy} =$ 0.924 Flange $c = (b \cdot t_w - 2r)/2 =$ 40.35 $c/t_r =$ 4.80 <9 ϵ = 8.320 Class 1 Class 1 1
$\epsilon = \sqrt{235/fy} =$ 0.924 Flange $c = (b \cdot t_w - 2r)/2 =$ 40.35 $c/t_r =$ 4.80 <9 ϵ = 8.320 Class 1 Class 1 1
Flange c=(b-t _w -2r)/2= 40.35 c/t _f = 4.80 <9ε=
c/t _t = 4.80 <9ε= 8.320 Class 1
Class 1
Web c=h-2t _r -2r= 225.2 mm
c/t _w = 38 <72ε= 66.56
Class 1 EU3 Table 5.2
Cross section resistance partial safty factor γ_{M0} = 1.00 EU3 6.1
4.Shear resistance of cross section
Design shear resistance $V_{c,Rd}=A_v(f_v/\sqrt{3})/\gamma_{Mo}=$ 264.54 kN EU3 (6.18)
Maximum shear force to shear resistance ratio $V_{Ed}/V_{C,Rd}$ = 0.16 <1 OK EU3 (6.17)
5.Bending resistance of cross section
$\label{eq:moment} \text{Design moment resistance} \qquad M_{c,Rd} = W_p l \times f_y / \gamma_{M0} = \qquad 84.15 \text{ kN.m} \qquad \text{EU3 (6.14)}$
Maximum moment to moment resistance ratio $M/M_{c,Rd=}$ 0.61 <1 OK
6.Deflection
Maximum deflection u _{max} =5w'L ⁴ /384EI= 11.83 mm
Allowable deflection u'=L/360= 13.06 mm
Actual to permissible deflection ratio $u_{max}/u'=$ 0.91 <1 OK

Beam is OK

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Tel: 07946 065051	Description	Padstone Design - knock through	Checked	Revision	Page No
Email: info@barratt-associates.co.uk	Description	Faustone Design - Knock (mough	BA	А	7

4.2

N/mm²

Padstone Calculator

Beam End Reaction =	86.00	kN (factored)
Factored Load at End of Beam		Variable Load Safety Factor = 1.5 Permanent Load Safety Factor = 1.35

Characteristic strength of masonry =

(Brickwork usually = 4.5 N/mm²)
(3.6N Blockwork usually = 2.6 N/mm²)
(A Engineering Brick = 13.2 N/mm²)
(B Engineering Brick = 10.5 N/mm²)
(Weak Brickwork = approx 2.8 N/mm²)
(7.3N Blockwork usually = 4.2 N/mm²)
(10.4N Blockwork usually = 5.4 N/mm²)

Width of beam end bearing =100mmLength of beam end bearing =150mm

$$\gamma m = 3.0$$

Bearing Factor = 1.25

Results

Maximum Bearing Stress = 1.75 N/mm² Actual Bearing Stress = 5.73 N/mm²

Padstone Required

Padstone Results

Characteristic strength of Padstone =	40.0	N/mm²	(A Engineering Brick = 13.2 N/mm²) (B Engineering Brick = 10.5 N/mm²)
Width of Padstone = 215	mm		(Concrete C15 = 15 N/mm^2)
Length of Padstone = 330	mm		(Concrete C30 = 30 N/mm ²)
Depth of Padstone = 215	mm		(Concrete C40 = 40 N/mm ²)
			(Steel Plate = 275 N/mm ²)
Allowable padstone stress =	16.67	N/mm ²	
Stress under beam end bearing =	5.73	N/mm ²	Therefore Padstone Stress OK
Allowable masonry stress =	1.75	N/mm ²	
Stress under padstone =	1.21	N/mm ²	Therefore Masonry Stress OK



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Contact Pressure Check on Concrete Ground Slab/Foundation

Contact Pressure Check on Concrete Ground Slab/Foundation			Person
Nib Contact Length Nib Contact Width Minimum depth of foundations required Pad Length (using 45°rule) Pad Width (using 45°rule)	L= Β= Τ _f = L _f = Β _f = γ=	780 mm 300 mm 200 mm 1180 mm 600 mm 24 kN/m ³	P _{f, app}
Unit weight of concrete Weight of foundation	y – W = LBDy =	3.3984 kN	
Axial load from colummn Weight of foundation Self-weight of wall above concrete Total applied presure Pad length Pad width Base area of pad foundation	W = LBDY = W = LBDY = W = Pf,app= L _f = B _f = A=	86.00 kN 3.3984 kN 3 kN 92.40 kN 1.18 m 0.6 m 0.708 m ²	B _f
Contact pressure on ground	q=N _{Ed} /A=	130.51 kN/m ²	
Assumed ground bearing capacity	q _{allowable}	150 kN/m ²	
Factor of safety	$F.O.S = q/q_{allowable} =$	1.15 >1	ок

Bearing capacity is OK

D=

200 mm

Minimum depth of existing foundations required

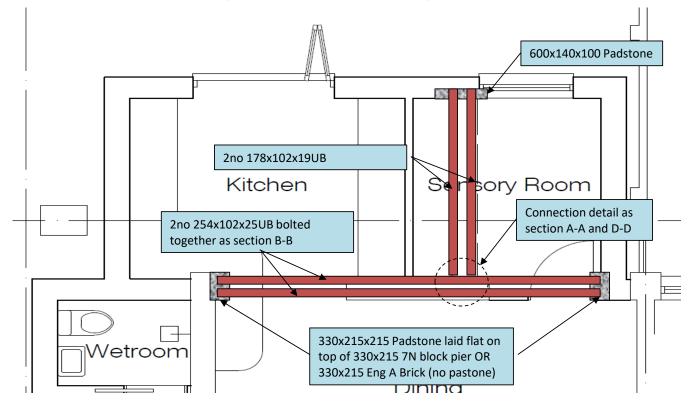
The design is assuming:

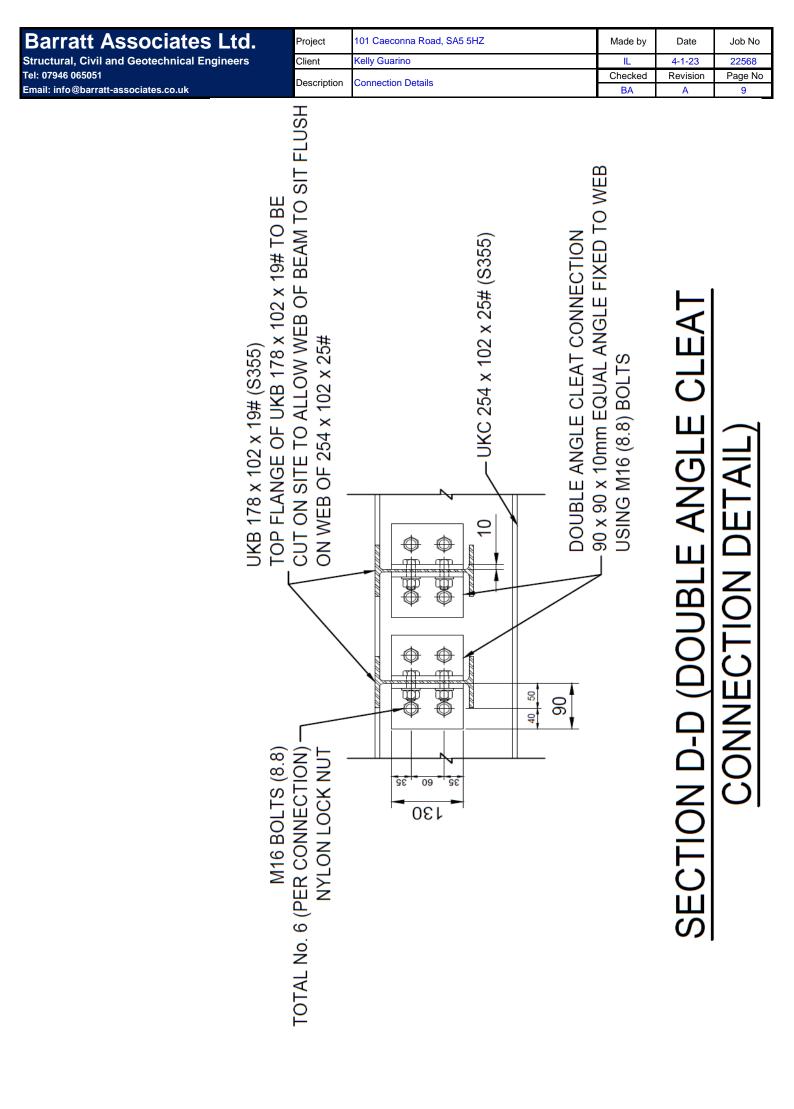
780mm min long masonry nib wall return either side of end bearing (ignore in extension)

Masonry is in good condition and 7Nmm2

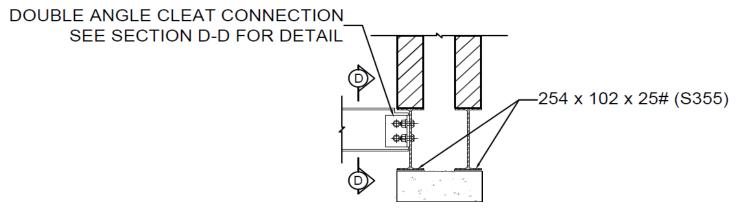
Cavity wall is suitable tied

Foundations are present, adequately designed and on a standard load bearing strata of 150kN/m2

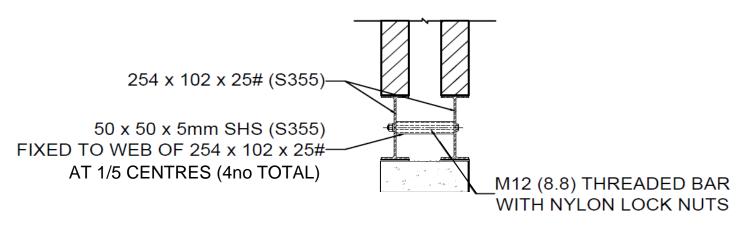




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SECTION A-A



SECTION B-B