## **Structural Calculation Itd**



Mobbs Wood Farm, Coventry, CV7 9JN 02476 709222 **10/01/2020** 

# Site AddressBrook House, Cockshutt, Ellesmere SY12 0JR

Structural Engineer	Eng Alessia Masini, PhD
Checking Engineer	Jason Pritchard BEng

### Description of Works

JP1524 Structural steelwork details relating to proposed house remodelling.

### Standards

BS 5268:part 2: 1996 Structural use of timber BS 6399 part 2 - 1995 Wind loads BS 5950 part 1 – 2000 Structural use of steelwork in building BS 8110 part 1 - 1997 Structural use of concrete BS 5628 part 1 - 1992 Code of practice for use of masonry BS 6399 part 1 - 1996 Loading for buildings: dead and imposed BS 6399 part 3 - 1988 Loading for buildings: imposed roof load Wind loads

### Notes

The contractor is responsible for all temporary works involved with the project. Advice may be requested from the engineer, but additional fees may be involved for further designs.

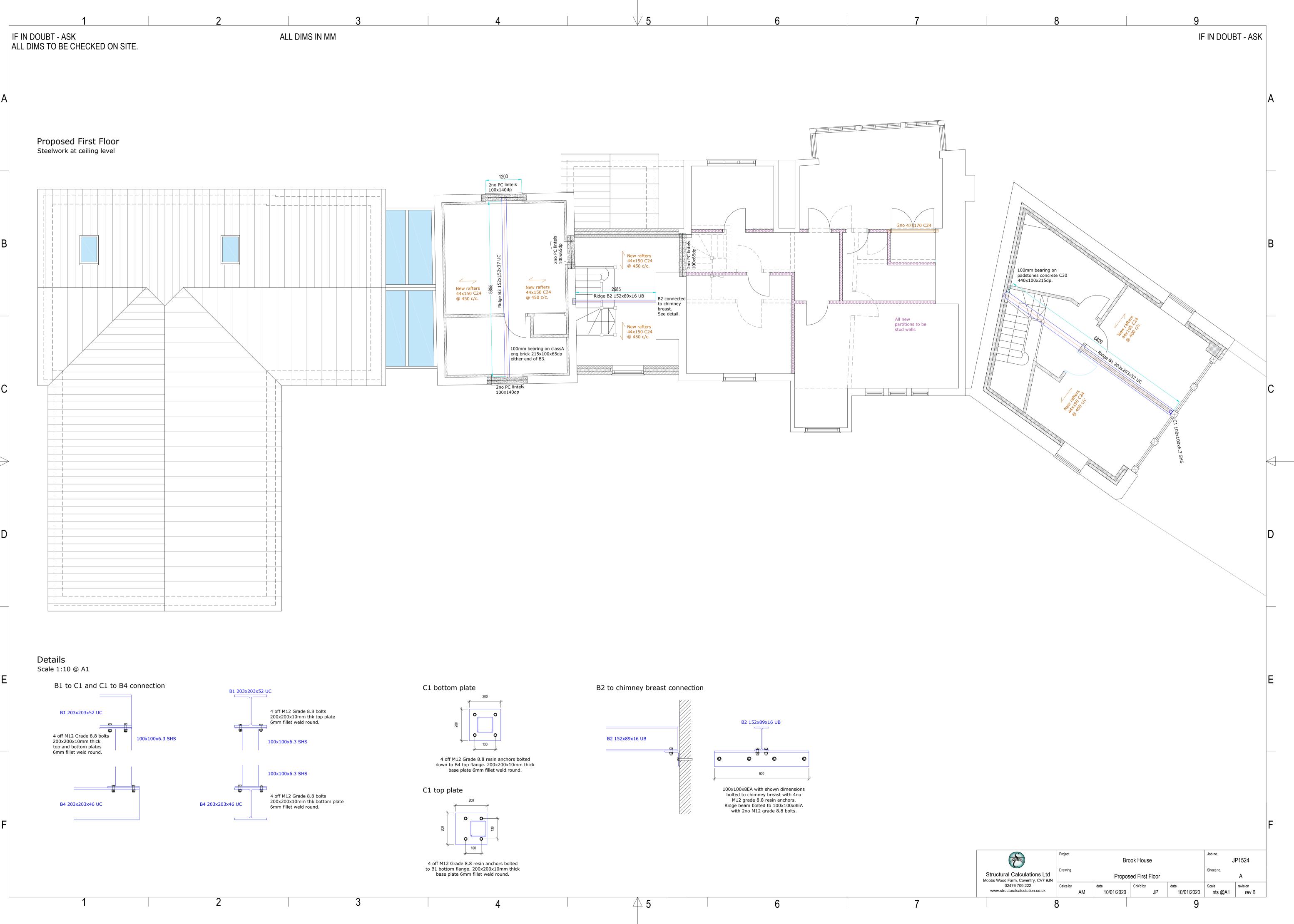
Any and all structural steelwork is now required by law to be CE Marked and must be supplied by an execution class 2 capable fabricator with an externally assessed and approved FPC.

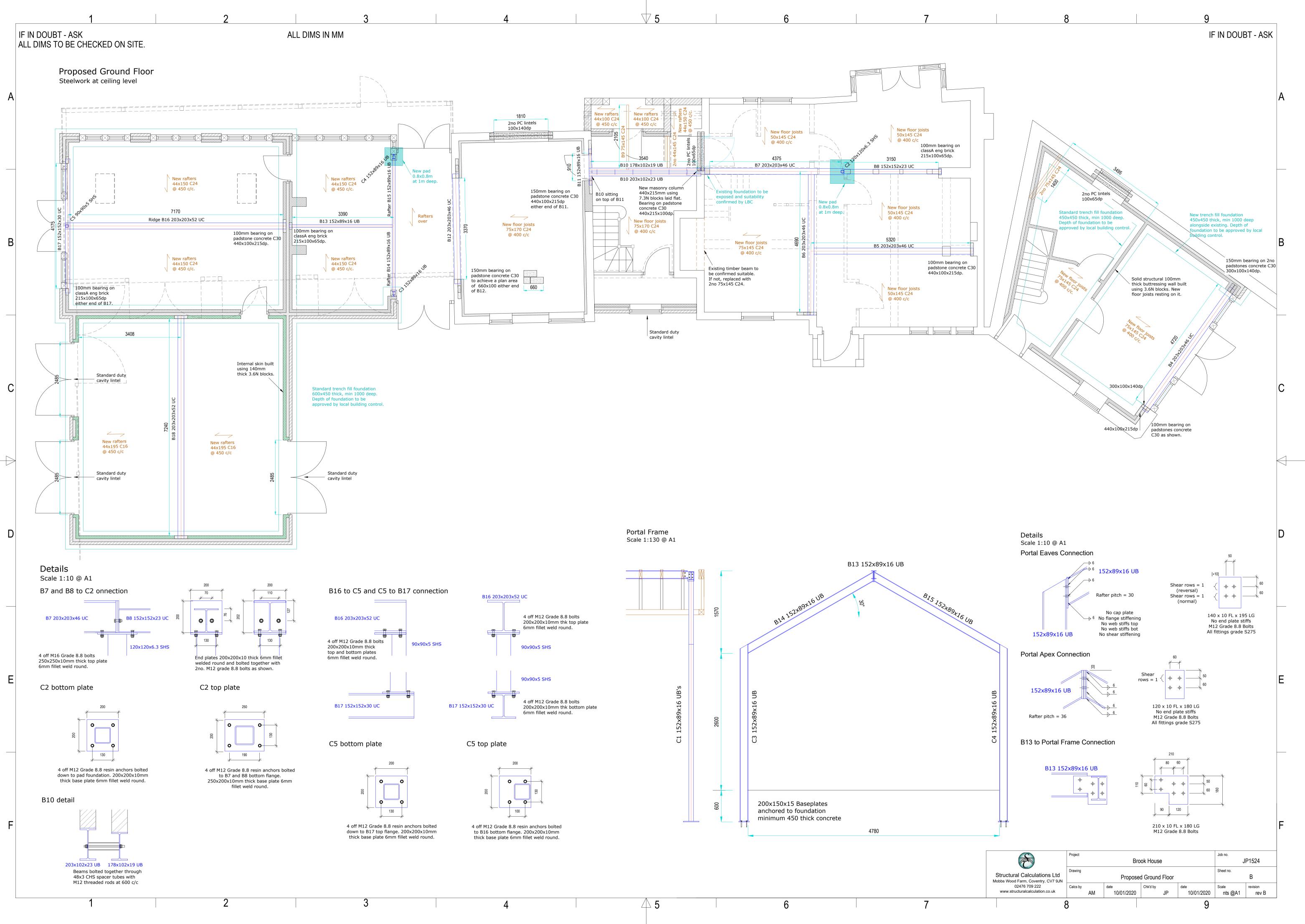
Only the work requested and as contained within this report has been undertaken, no checks on other observations or information gained either on site or from the drawing have been made. Further checks can be carried out but again additional fees may be involved.

In the absence of detailed ground condition information the foundations are assumed satisfactory for ground bearing. This must be verified on site and agreed with L.A. Officer. Further consideration to detail may have to be undertaken at a design stage however written instruction will be required and additional fees may be involved.

These drawings are not architectural and are provided only to indicate the position of the calculated structural elements. Further advice should be sort from a suitably qualified architect to ensure compliance with current building regulations and best practices.

If something is missing from the report or an item is left unresolved or unclear please contact the engineer for clarification prior to carrying out the work.





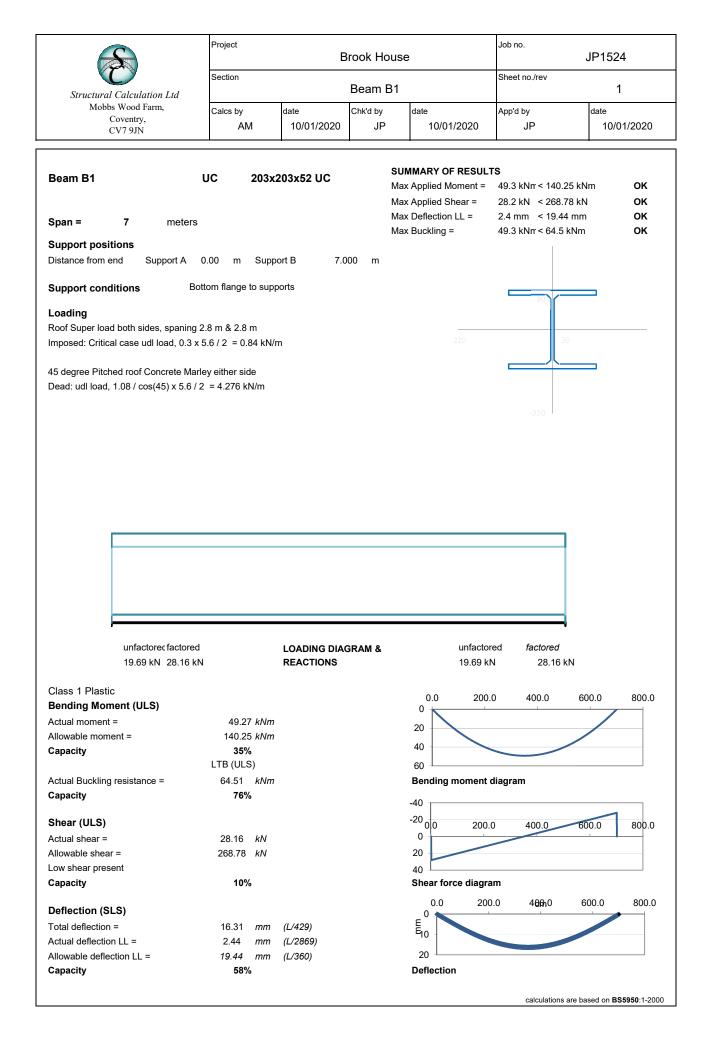


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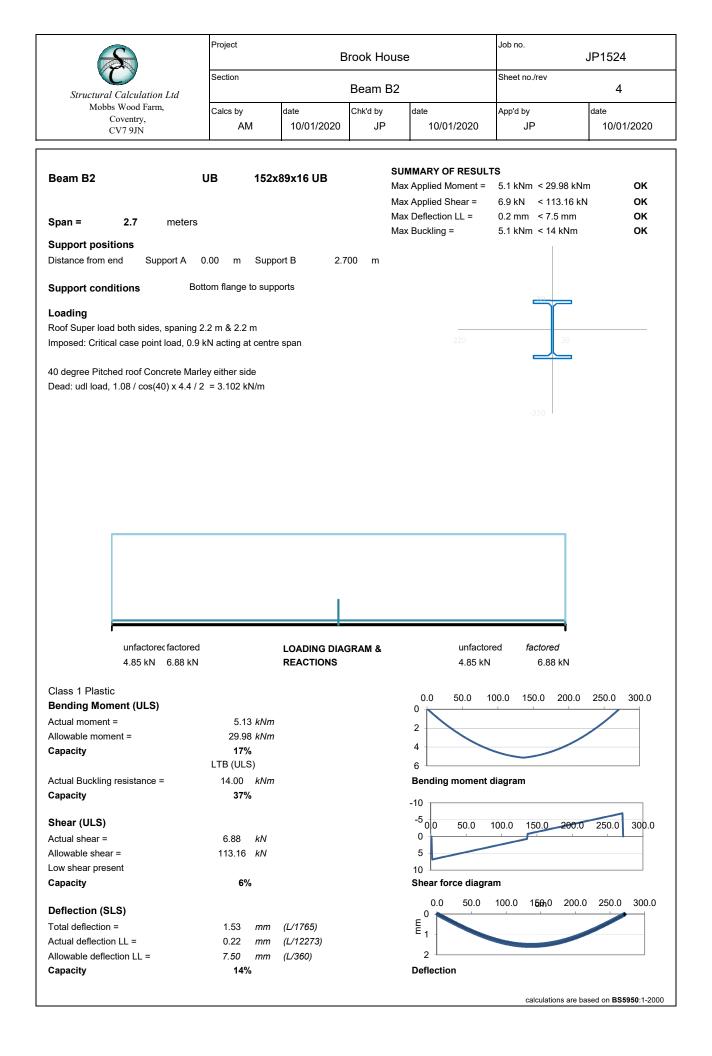
# **Start of supporting Calculations**

General Imposed Load Values		ext	racts from BS	S6399 Part 1
	UDL	Poi		
Domestic and residential		1.5	1.4	
Offices		2.5	2.7	
Factories workshops etc.		5	4.5	
Roof super imposed		0.6	0.9	
Balconies	same as room to w	/hich they give a	ccess.	
		, 0		
General Dead Load Values				
	UDL			
Timber Joist suspended floor		0.5		
Timber Joist with ceiling underneath		0.7		
Concrete slab 150mm thick		3.7		
Concrete slab 200mm thick		4.8		
Mezzanine floor joists + boards		0.4		
Ceiling only		0.2		
Ceiling plus loft storage		1.3		
5mm thick steel sheet		0.4		
General Dead Load Values				
	kN/m3			
Stud wall plasterboard		5.48		
Brick/block solid		19.6		
Brick/block solid + finishes		18.4		
Brick/block 2 skin + sml cavity + finishes		16.6		
Brick/block 2 skin + 100mm cavity +				
finishes		13.1		
Lght weight block solid		7.9		
Lght weight block inner Brick outer + finishes		10.2		
		10.2		
General Roof Load Values				
	kN/m2			
Roof Tiles Concrete		0.69	Slate	0.39
Roof Battens & Felt		0.05	2.410	0.00
Roof Rafters		0.09		
Roof Insulation		0.10		
Roof Plasterboard lining		0.05		
Roof Services		0.10		
Roof and Ceiling Superimposed Load		0.60		
Roof Snow load		0.50		

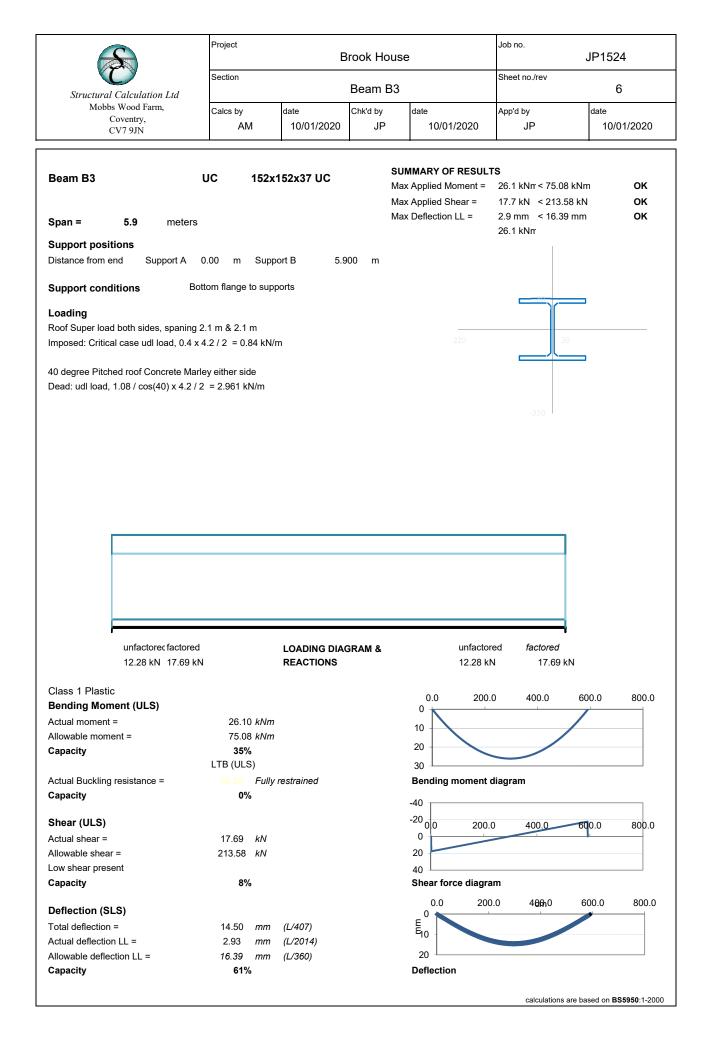


S	Project	B	rook House	e	Job no.	JP1524
Structural Calculation Ltd	Section	Bea	aring Beam	B1	Sheet no./rev	2
Mobbs Wood Farm, Coventry, CV7 9JN	Calcs by AM	date 10/01/2020	Chk'd by JP	<sup>date</sup> 10/01/2020	App'd by JP	date 10/01/2020
Beam End Reaction =	28.16	kN (factored	)	Variable Load Safe Permanent Load S	-	
Masonry						
Masonry type =	3.6N Blo	ckwork				
Characteristic strength of masonry =	2.6	N/mm²				
Width of beam end bearing =	204.3	mm		Bearing factor =	1.25	
Length of beam end bearing =	100	mm		γm =	3.00	
Stress						
Maximum Bearing Stress =	1.08	N/mm²				
Actual Bearing Stress =	1.38	N/mm²				
Capacity	127%		Padsto	ne Required		
Padstone						
A Engineering Brick						
Characteristic strength of padstone =	13.2	N/mm²				
Width of padstone =	100	mm				
Length of padstone =	440	mm				
Maximum bearing stress padstone =	5.50	N/mm²				
Stress under beam =	1.38	N/mm²	_			
Capacity	25%		ОК			
Maximum bearing stress masonry	1.08	N/mm²				
Stress under padstone =	0.64	N/mm²				
Capacity	59%		ок			

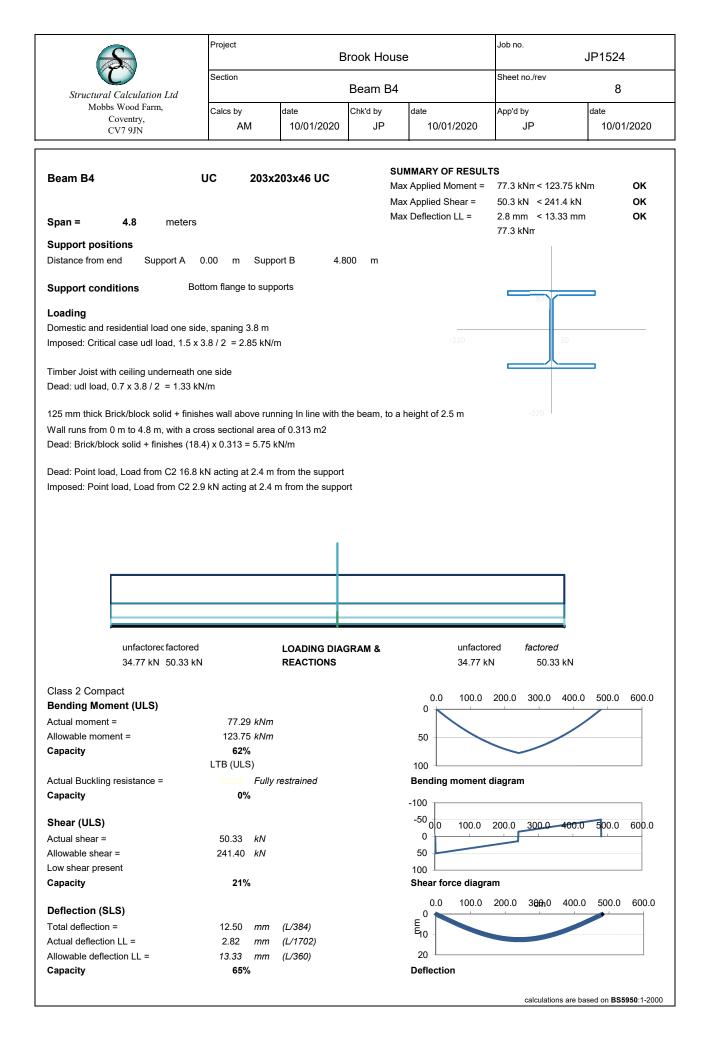
	Project	В	rook House			JP1524
Structural Calculation Ltd	Section	(	Column C1		Sheet no./rev	3
Mobbs Wood Farm,	Calcs by	date	Chk'd by	date	App'd by	date
Coventry, CV7 9JN	ÂM	10/01/2020	JP	10/01/2020	JP	10/01/2020
Column C1 SHS Height = 3.650 meters	100x10	)0x6.3	Compression resi Buckling resistand	length 'L <sub>Ex</sub> / L <sub>Ey</sub> ' <i>[mm]</i> stance: 295	= 438 kN > 29 kN Nm > 9 kNm	0 / 4380 ОК ОК
			Overall check:	0.80	) < 1.00	ОК
End restraint				300		
Top x-x: Not held in position but rest	rained in direc	tion				
Top y-y: Not held in position but rest	rained in direc	tion			at 1 = 29 kN	
Stm. x-x: Effectively held in position a	and restrained	in direction				
Btm. y-y: Effectively held in position a	and restrained	in direction				
Effectice length factors						
For x-x axis ' $L_{Fx-x}$ ' = 1.20						
For y-y axis ' $L_{Fy-y}$ ' = 1.20						
Loading		x-axis				
mposed Direct compression 'PL' [kN	I] =					
Dead Direct compression 'PL' [kN	1] =			- 3.0		
at 1 at 2	at 3	at 4				
Eccentricity 100						
LL Reaction 2.9						
DL Reaction 16.8						
Applied moment about x-x axis 'M <sub>xx</sub> ' <i>[kNi</i>	m] =	4.15				
Applied moment about y-y axis 'M <sub>yy</sub> ' <i>[kNi</i>	m] =					
			Total vertical			
			load:29 kN			
				y-axis		
				-300 1 9-4015		
Section is class 1 plastic			NOTE: Loads a	are factored.		
Steel Grade:	S2	75				
Resultant moment about x-x axis 'M <sub>xx</sub> ' <i>[k</i>	•	37				
Resultant moment about y-y axis 'M <sub>yy</sub> ' <i>[k</i>	: <i>Nm]</i> = 5.1	4				
Compression resistance			Buckling resist			
Reduced design strength 'p <sub>yr</sub> ' [ <i>MPa</i> ] =	N//		Equivalent slende	rness 'λ <sub>LT</sub> ':		48.0
Slenderness for x-x axis ' $\lambda_x$ ' =		5.3	Parameter 'p <sub>E</sub> ' =			877.2
Slenderness for y-y axis $\lambda_y' =$		5.3	Robertson consta			7.0
Strut curve for x-x axis:	a)			t slenderness ' $\lambda_{L0}$ ' =		34.3
Strut curve for y-y axis:	a)		Perry factor ŋ <sub>LT</sub> =			0.1
Parameter 'p <sub>E</sub> ' =		2.3	Parameter $Ø_{LT} =$	<b></b>		618.2
Robertson constant ' $\alpha$ ' =	2.0		Bending strength			242.8
Limiting slenderness 'λ₀' =	17			resistance 'M <sub>bs</sub> ' <i>[kNm]</i>		19.7
Perry factor 'ŋ' =	0.2			about x-x 'M <sub>xx</sub> ' [kNm] =	-	8.4
Parameter 'Ø' =		8.6	Capacity			43%
Compressive strength $p_c' [N/mm^2] =$	12	6.7				
Based on y-y axis calculations			Bending about			10.1
Compression resistance 'P <sub>c</sub> ' [kN] =	29	4.0	Moment of resista	-		18.4
						<b>F</b> 1
Applied total vertical load 'F <sub>c</sub> ' [kN] = Capacity	28 10		Applied moment a	about y-y 'M <sub>yy</sub> ' [kNm] =	•	5.1 <b>28%</b>



S	Project	В	rook Hou	se	Job no.	JP1524
Structural Calculation Ltd	Section	Bea	aring Bea	m B2	Sheet no./rev	5
Mobbs Wood Farm, Coventry, CV7 9JN	Calcs by AM	date 10/01/2020	Chk'd by JP	<sup>date</sup> 10/01/2020	App'd by JP	date 10/01/2020
Beam End Reaction =	6.88	kN (factored	)	Variable Load Safe Permanent Load S		
Masonry						
Masonry type =	Weak Br	ickwork				
Characteristic strength of masonry =	2.8	N/mm²				
Width of beam end bearing =	88.7	mm		Bearing factor =	1.25	
Length of beam end bearing =	100	mm		γm =	3.00	
Stress						
Maximum Bearing Stress =	1.17	N/mm²				
Actual Bearing Stress =	0.78	N/mm²				
Capacity	66%		Pads	tone Not Required		

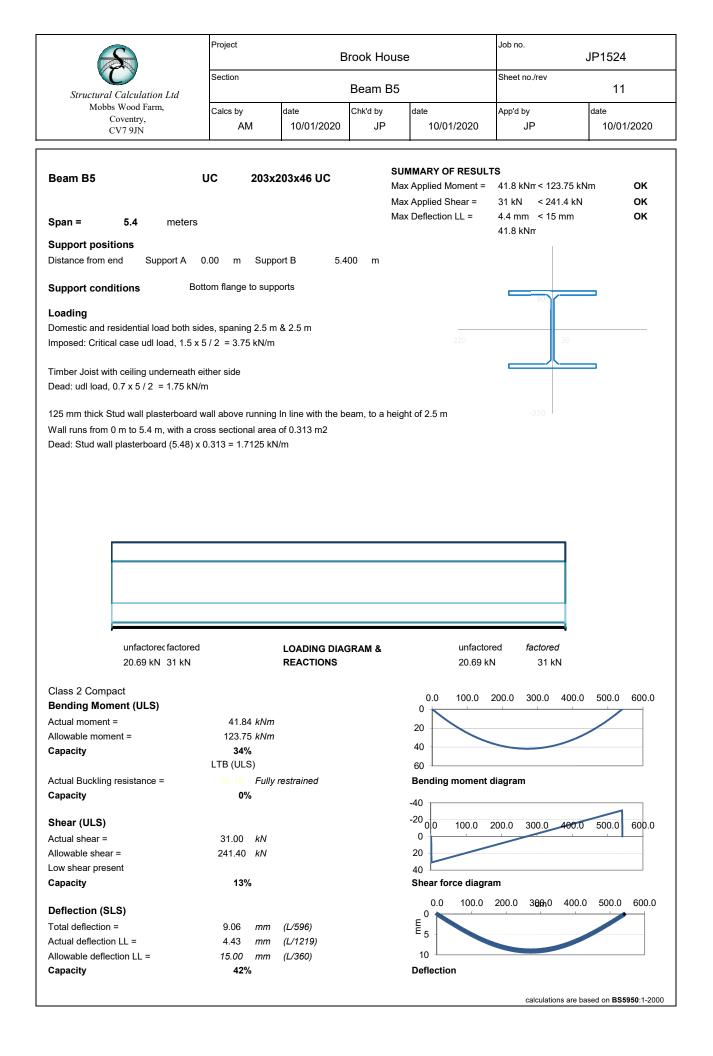


S	Project	В	rook Hou	se	Job no.	JP1524
Structural Calculation Ltd	Section	Bea	aring Bear	m B3	Sheet no./rev	7
Mobbs Wood Farm, Coventry, CV7 9JN	Calcs by AM	date 10/01/2020	Chk'd by JP	date 10/01/2020	App'd by JP	<sup>date</sup> 10/01/2020
Beam End Reaction =	17.69	kN (factored)	)	Variable Load Safe Permanent Load Sa		
Masonry						
Masonry type =	Weak Bri	ckwork				
Characteristic strength of masonry =	2.8	N/mm²				
Width of beam end bearing =	154.4	mm		Bearing factor =	1.25	
Length of beam end bearing =	100	mm		γm =	3.00	
Stress						
Maximum Bearing Stress =	1.17	N/mm²				
Actual Bearing Stress =	1.15	N/mm²				
Capacity	98%		Padst	one Not Required		



S	Project	В	rook House	e	Job no.	JP1524
Structural Calculation Ltd	Section	Bea	aring Beam	B4	Sheet no./rev	9
Mobbs Wood Farm, Coventry, CV7 9JN	Calcs by AM	<sup>date</sup> 10/01/2020	Chk'd by JP	date 10/01/2020	App'd by JP	date 10/01/2020
Beam End Reaction =	50.33	kN (factored	)	Variable Load Safe Permanent Load S		
Masonry						
Masonry type =	3.6N Blo	ckwork				
Characteristic strength of masonry =	2.6	N/mm²				
Width of beam end bearing =	203.6	mm		Bearing factor =	1.25	
Length of beam end bearing =	150	mm		γm =	3.00	
Stress						
Maximum Bearing Stress =	1.08	N/mm²				
Actual Bearing Stress =	1.65	N/mm²				
Capacity	152%		Padsto	ne Required		
Padstone						
A Engineering Brick						
Characteristic strength of padstone =	13.2	N/mm²				
Width of padstone =	200	mm				
Length of padstone =	300	mm				
Maximum bearing stress padstone =	5.50	N/mm²				
Stress under beam =	1.65	N/mm²				
Capacity	30%		OK			
Maximum bearing stress masonry	1.08	N/mm²				
Stress under padstone =	0.84	N/mm²				
Capacity	77%		ок			

S	Project	В	rook Hous	e	Job no.	JP1524
Structural Calculation Ltd	Section	Bea	aring Beam	B4	Sheet no./rev	10
Mobbs Wood Farm, Coventry, CV7 9JN	Calcs by AM	<sup>date</sup> 10/01/2020	Chk'd by JP	date 10/01/2020	App'd by JP	date 10/01/2020
Beam End Reaction =	50.33	kN (factored	)	Variable Load Safe Permanent Load S		
Masonry						
Masonry type =	3.6N Blo	ckwork				
Characteristic strength of masonry =	2.6	N/mm²				
Width of beam end bearing =	203.6	mm		Bearing factor =	1.25	
Length of beam end bearing =	100	mm		γm =	3.00	
Stress						
Maximum Bearing Stress =	1.08	N/mm²				
Actual Bearing Stress =	2.47	N/mm²				
Capacity	228%		Padsto	ne Required		
Padstone						
A Engineering Brick						
Characteristic strength of padstone =	13.2	N/mm²				
Width of padstone =	100	mm				
Length of padstone =	640	mm				
Maximum bearing stress padstone =	5.50	N/mm²				
Stress under beam =	2.47	N/mm²				
Capacity	45%		ОК			
Maximum bearing stress masonry	1.08	N/mm²				
Stress under padstone =	0.79	N/mm²				
Capacity	73%		ок			



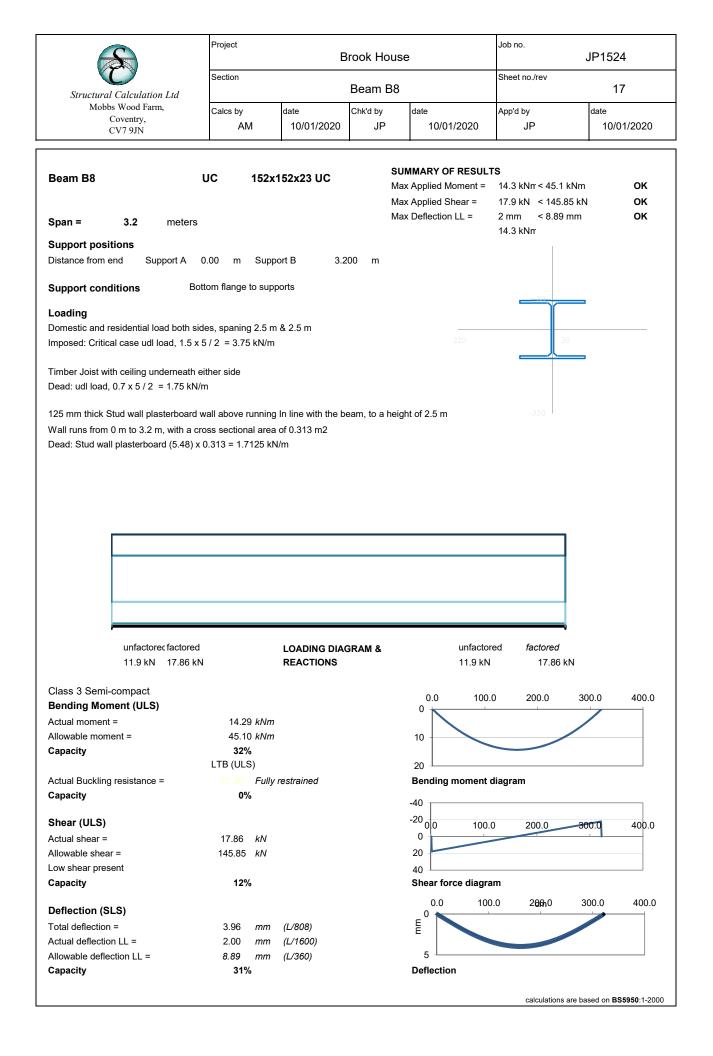
S	Project	В	rook House	e	Job no.	JP1524
Structural Calculation Ltd	Section	Bea	aring Beam	B5	Sheet no./rev	12
Mobbs Wood Farm, Coventry, CV7 9JN	Calcs by AM	<sup>date</sup> 10/01/2020	Chk'd by JP	date 10/01/2020	App'd by JP	date 10/01/2020
Beam End Reaction =	31.00	kN (factored	)	Variable Load Safe Permanent Load S	-	
Masonry						
Masonry type =	Weak Bri	ickwork				
Characteristic strength of masonry =	2.8	N/mm²				
Width of beam end bearing =	203.6	mm		Bearing factor =	1.25	
Length of beam end bearing =	100	mm		γm =	3.00	
Stress						
Maximum Bearing Stress =	1.17	N/mm²				
Actual Bearing Stress =	1.52	N/mm²				
Capacity	130%		Padsto	ne Required		
Padstone						
Concrete C30						
Characteristic strength of padstone =	30	N/mm²				
Width of padstone =	100	mm				
Length of padstone =	440	mm				
Maximum bearing stress padstone =	12.50	N/mm²				
Stress under beam =	1.52	N/mm²				
Capacity	12%		ОК			
Maximum bearing stress masonry	1.17	N/mm²				
Stress under padstone =	0.70	N/mm²				
Capacity	60%		ок			

S		В	rook House	e		JP1524
Structural Calculation Ltd	Section		Beam B6		Sheet no./rev	13
Mobbs Wood Farm, Coventry, CV7 9JN	Calcs by AM	date 10/01/2020	Chk'd by JP	date 10/01/2020	App'd by JP	date 10/01/2020
Beam B6 Span = 4.7 meters	UC 203x	203x46 UC	Ma: Ma:	MMARY OF RESUL x Applied Moment = x Applied Shear = x Deflection LL =		ок
Support positions	0.00 m Supp	ort B 4.7	00 m		60.1 kNm	
Support conditions Bo	ttom flange to supp	ports				
Loading Domestic and residential load one sic Imposed: Critical case udl load, 1.5 x		/m		-220	80	
Timber Joist with ceiling underneath o Dead: udl load, 0.7 x 3.5 / 2 = 1.225						_
125 mm thick Stud wall plasterboard Wall runs from 0 m to 4.7 m, with a c Dead: Stud wall plasterboard (5.48) x	ross sectional area	of 0.313 m2	eam, to a heig	ht of 2.5 m	-220	
Dead: Point load, Load from B5 10.6						
Dead: Point load, Load from B5 10.6 Imposed: Point load, Load from B5 10						
Dead: Point load, Load from B5 10.6	0.1 kN acting at 2.6		port	unfacto 25.8 kt		
Dead: Point load, Load from B5 10.6 Imposed: Point load, Load from B5 10 unfactorec factored 23.16 kN 34.54 kł Class 2 Compact	0.1 kN acting at 2.6	5 m from the sup	port	25.8 kł 0.0 100.4	N 38.5 kN	400.0 500.0
Dead: Point load, Load from B5 10.6 Imposed: Point load, Load from B5 10 unfactorec factored 23.16 kN 34.54 kt	0.1 kN acting at 2.6 N 60.13 <i>kNm</i> 123.75 <i>kNm</i> <b>49%</b>	5 m from the sup	port	25.8 kl	N 38.5 kN	400.0 500.0
Dead: Point load, Load from B5 10.6 Imposed: Point load, Load from B5 10 unfactorec factored 23.16 kN 34.54 kt Class 2 Compact Bending Moment (ULS) Actual moment = Allowable moment =	0.1 kN acting at 2.6 0.13 kNm 123.75 kNm <b>49%</b> LTB (ULS)	5 m from the sup	port	25.8 kl	N 38.5 kN 0 200.0 300.0	400.0 500.0
Dead: Point load, Load from B5 10.6 Imposed: Point load, Load from B5 10 unfactorec factored 23.16 kN 34.54 kt Class 2 Compact Bending Moment (ULS) Actual moment = Allowable moment = Capacity Actual Buckling resistance = Capacity Shear (ULS)	0.1 kN acting at 2.6 0.1 kN acting at 2.6 0.13 kNm 123.75 kNm	5 m from the sup	port	25.8 kl 0.0 100.1 50 100 100 Bending moment -50 100.0	N 38.5 kN 0 200.0 300.0	400.0 500.0
Dead: Point load, Load from B5 10.6 mposed: Point load, Load from B5 10 unfactorec factored 23.16 kN 34.54 kt Class 2 Compact Bending Moment (ULS) Actual moment = Allowable moment = Capacity Actual Buckling resistance = Capacity Shear (ULS) Actual shear = Allowable shear =	0.1 kN acting at 2.6 0.13 kNm 123.75 kNm 49% LTB (ULS) 75.98 Fully	5 m from the sup	port	25.8 kl	N 38.5 kN 0 200.0 300.0	
Dead: Point load, Load from B5 10.6 Imposed: Point load, Load from B5 10 unfactorec factored 23.16 kN 34.54 kt Class 2 Compact Bending Moment (ULS) Actual moment = Allowable moment = Capacity Actual Buckling resistance = Capacity Shear (ULS) Actual shear = Allowable shear = Low shear present	N 60.13 <i>kNm</i> 123.75 <i>kNm</i> <b>49%</b> LTB (ULS) <b>75.98</b> <i>Fully</i> <b>0%</b> 38.50 <i>kN</i>	5 m from the sup	port	25.8 kl 0.0 100.1 50 100 100 Bending moment -50 100.0	N 38.5 kN 0 200.0 300.0 t diagram 200.0 300.0	
Dead: Point load, Load from B5 10.6 Imposed: Point load, Load from B5 10 unfactorec factored 23.16 kN 34.54 kf Class 2 Compact Bending Moment (ULS) Actual moment = Allowable moment = Capacity Actual Buckling resistance = Capacity	N 60.13 <i>kNm</i> 123.75 <i>kNm</i> <b>49%</b> LTB (ULS) <b>75.98</b> <i>Fully</i> <b>0%</b> 38.50 <i>kN</i> 241.40 <i>kN</i>	5 m from the sup	port	25.8 kl	N 38.5 kN 0 200.0 300.0 t diagram 200.0 300.0	

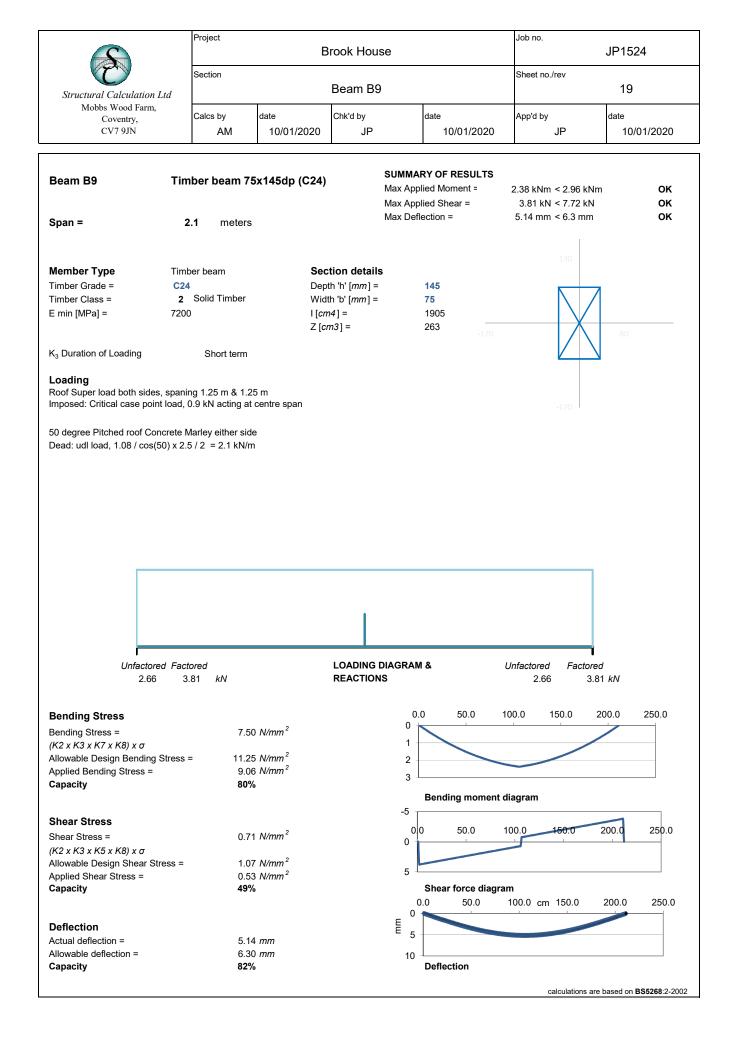
S	Project	В	rook House	e	Job no.	JP1524
Structural Calculation Ltd	Section	Bea	aring Beam	B6	Sheet no./rev	14
Mobbs Wood Farm, Coventry, CV7 9JN	Calcs by AM	date 10/01/2020	Chk'd by JP	date 10/01/2020	App'd by JP	date 10/01/2020
Beam End Reaction =	38.50	kN (factored	)	Variable Load Safe Permanent Load S	-	
Masonry						
Masonry type =	Weak Bri	ickwork				
Characteristic strength of masonry =	2.8	N/mm²				
Width of beam end bearing =	203.6	mm		Bearing factor =	1.25	
Length of beam end bearing =	100	mm		γm =	3.00	
Stress						
Maximum Bearing Stress =	1.17	N/mm²				
Actual Bearing Stress =	1.89	N/mm²				
Capacity	162%		Padsto	ne Required		
Padstone						
Concrete C30						
Characteristic strength of padstone =	30	N/mm²				
Width of padstone =	100	mm				
Length of padstone =	660	mm				
Maximum bearing stress padstone =	12.50	N/mm²				
Stress under beam =	1.89	N/mm²				
Capacity	15%		ОК			
Maximum bearing stress masonry	1.17	N/mm²				
Stress under padstone =	0.58	N/mm²				
Capacity	50%		ок			

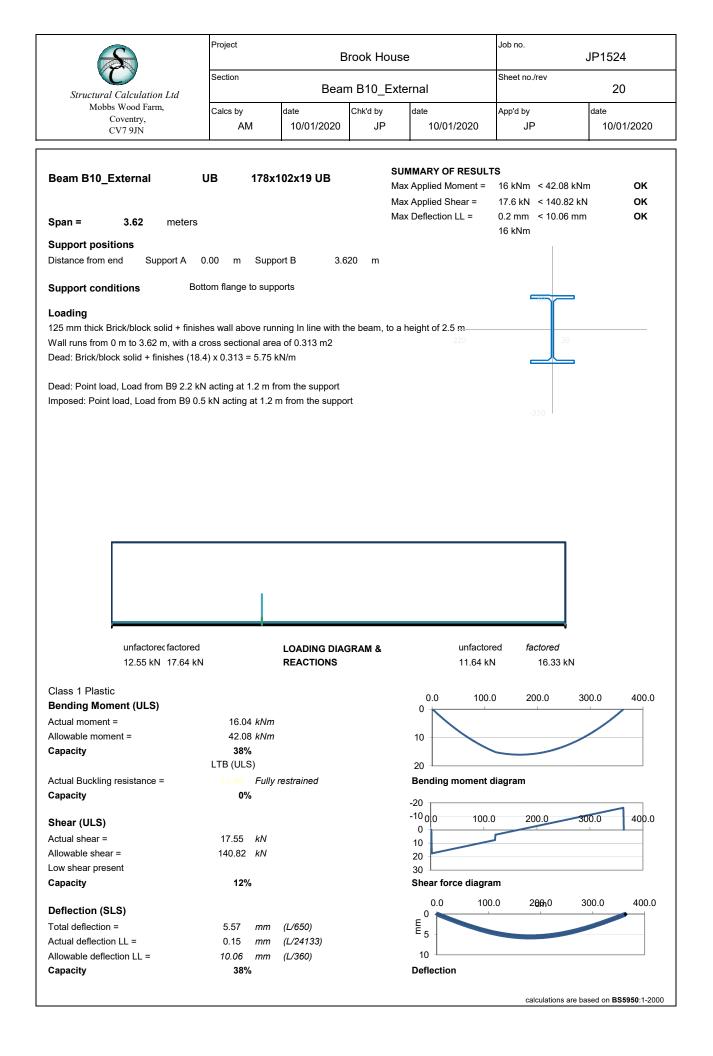
				e			JP1524	
Structural Calculation Ltd	Section		Beam B7		Sheet no./re	v	15	
Mobbs Wood Farm, Coventry, CV7 9JN	Calcs by AM	date 10/01/2020	Chk'd by JP	date 10/01/2020	App'd by JP		date 10/01	/2020
<b>Beam B7</b> S <b>pan = 4.4</b> me	UC 203x20	03x46 UC	Ma: Ma	MMARY OF RESU x Applied Moment = x Applied Shear = x Deflection LL =		241.4 kN		ок ок ок
Support positions	A 0.00 m Suppo	rt B 4.40	00 m					
Support conditions	Bottom flange to suppo	orts					_	
<b>.oading</b> Domestic and residential load on nposed: Critical case udl load, 7		n		-220	80	30		
Timber Joist with ceiling underne Dead: udl load, $0.7 \times 2.3 / 2 = 0.1$						_/(	-	
Vall runs from 0 m to 4.4 m, with Dead: Stud wall plasterboard (5.4								
mposed: UDL load, Domestic ar	nd residential Critical case	e UDL 3.6 kN/m	-					
Dead: UDL load, Timber Joist wi mposed: UDL load, Domestic ar Dead: Point load, Load from B6	nd residential Critical case	e UDL 3.6 kN/m om the support	acting from 3.	4 m to 4.4 m				
mposed: UDL load, Domestic ar	nd residential Critical case 12.6 kN acting at 3.4 m fr	e UDL 3.6 kN/m	acting from 3.			bored 5.7 kN		
mposed: UDL load, Domestic ar Dead: Point load, Load from B6 unfactorec fact 15.91 kN 23.3 Class 2 Compact Bending Moment (ULS) Actual moment =	nd residential Critical case 12.6 kN acting at 3.4 m fr ored 56 kN 40.13 <i>kNm</i>	e UDL 3.6 kN/m om the support	acting from 3.	4 m to 4.4 m	kN 4		400.0	500.0
mposed: UDL load, Domestic ar Dead: Point load, Load from B6 unfactorec fact 15.91 kN 23.4 Class 2 Compact Bending Moment (ULS)	nd residential Critical case 12.6 kN acting at 3.4 m fr ored 56 kN	e UDL 3.6 kN/m om the support	acting from 3.	4 m to 4.4 m unfact 30.69	kN 4	5.7 kN	400.0	500.0
mposed: UDL load, Domestic ar Dead: Point load, Load from B6 unfactorec fact 15.91 kN 23.9 Class 2 Compact Bending Moment (ULS) Actual moment = Nowable moment =	40.13 <i>kNm</i> 12.6 kN acting at 3.4 m fr ored 56 kN 123.75 <i>kNm</i> <b>32%</b> LTB (ULS)	e UDL 3.6 kN/m om the support	acting from 3.	4 m to 4.4 m unfact 30.69	kN 4	5.7 kN	400.0	500.0
mposed: UDL load, Domestic ar Dead: Point load, Load from B6 unfactorec fact 15.91 kN 23.4 Class 2 Compact Bending Moment (ULS) Actual moment = Capacity Actual Buckling resistance =	tored 40.13 <i>kNm</i> 12.6 kN 40.13 <i>kNm</i> 123.75 <i>kNm</i> 32% LTB (ULS) 79.80 <i>Fully re</i>	e UDL 3.6 kN/m om the support	acting from 3.	4 m to 4.4 m unfact 30.69 0.0 100.0 20 40 60 Bending moment -100 -50	kN 4	5.7 kN	/	
mposed: UDL load, Domestic ar Dead: Point load, Load from B6 unfactorec fact 15.91 kN 23.4 Class 2 Compact Bending Moment (ULS) Actual moment = Allowable moment = Capacity Actual Buckling resistance = Capacity Shear (ULS) Actual shear = Allowable shear =	tored 40.13 <i>kNm</i> 12.6 kN 40.13 <i>kNm</i> 123.75 <i>kNm</i> 32% LTB (ULS) 79.80 <i>Fully re</i>	e UDL 3.6 kN/m om the support	acting from 3.	4 m to 4.4 m unfact 30.69 0.0 100.0 20 40 60 Bending momen -100 -50 0.0 100.0 0 100.0	kN 4	5.7 kN	400.0	
mposed: UDL load, Domestic ar Dead: Point load, Load from B6 unfactorec fact 15.91 kN 23.4 Class 2 Compact Bending Moment (ULS) Actual moment = Capacity Actual Buckling resistance = Capacity	40.13 <i>kNm</i> 12.6 kN acting at 3.4 m fr ored 56 kN 40.13 <i>kNm</i> 123.75 <i>kNm</i> <b>32%</b> LTB (ULS) 79.80 <i>Fully re</i> <b>0%</b> 45.70 <i>kN</i>	e UDL 3.6 kN/m om the support	acting from 3.	4 m to 4.4 m unfact 30.69 0.0 100.0 40 60 Bending momen -100 -50 0,0 100.0	kN 4	5.7 kN 300.0	/	500.0
mposed: UDL load, Domestic ar Dead: Point load, Load from B6 unfactorec fact 15.91 kN 23.3 Class 2 Compact Bending Moment (ULS) Actual moment = Allowable moment = Capacity Actual Buckling resistance = Capacity Shear (ULS) Actual shear = Allowable shear = Allowable shear = Allowable shear = Allowable shear = Allowable shear =	ad residential Critical case 12.6 kN acting at 3.4 m fr 12.6 kN acting at 3.4 m fr 12.6 kN 123.75 <i>kNm</i> 123.75 <i>kNm</i> <b>32%</b> LTB (ULS) <b>79.80</b> <i>Fully re</i> <b>0%</b> 45.70 <i>kN</i> 241.40 <i>kN</i> <b>19%</b> 5.44 <i>mm</i>	e UDL 3.6 kN/m om the support	acting from 3.	4 m to 4.4 m unfact 30.69 0.0 100.0 20 40 60 Bending momen -100 -50 0.0 100.0 0 0 100.0	kN 4	5.7 kN 300.0	/	

S	Project	В	rook House	e	Job no.	JP1524
Structural Calculation Ltd	Section	Bea	aring Beam	B7	Sheet no./rev	16
Mobbs Wood Farm, Coventry, CV7 9JN	Calcs by AM	<sup>date</sup> 10/01/2020	Chk'd by JP	date 10/01/2020	App'd by JP	date 10/01/2020
Beam End Reaction =	80.26	kN (factored	)	Variable Load Safe Permanent Load S	-	
Masonry						
Masonry type =	3.6N Blo	ckwork				
Characteristic strength of masonry =	2.6	N/mm²				
Width of beam end bearing =	203.6	mm		Bearing factor =	1.25	
Length of beam end bearing =	100	mm		γm =	3.00	
Stress						
Maximum Bearing Stress =	1.08	N/mm²				
Actual Bearing Stress =	3.94	N/mm²				
Capacity	364%		Padsto	ne Required		
Padstone						
Concrete C30						
Characteristic strength of padstone =	30	N/mm²				
Width of padstone =	215	mm				
Length of padstone =	440	mm				
Maximum bearing stress padstone =	12.50	N/mm²				
Stress under beam =	3.94	N/mm²				
Capacity	32%		ОК			
Maximum bearing stress masonry	1.08	N/mm²				
Stress under padstone =	0.85	N/mm²				
Capacity	78%		ок			



S	Project	В	Brook Hous	e	Job no.	JP1524
Structural Calculation Ltd	Section	Bea	aring Bearr	n B8	Sheet no./rev	18
Mobbs Wood Farm, Coventry, CV7 9JN	Calcs by AM	<sup>date</sup> 10/01/2020	Chk'd by JP	date 10/01/2020	App'd by JP	date 10/01/2020
Beam End Reaction =	17.86	kN (factored	)	Variable Load Safe Permanent Load S		
Masonry						
Masonry type =	Weak Br	ickwork				
Characteristic strength of masonry =	2.8	N/mm²				
Width of beam end bearing =	152.2	mm		Bearing factor =	1.25	
Length of beam end bearing =	100	mm		γm =	3.00	
Stress						
Maximum Bearing Stress =	1.17	N/mm²				
Actual Bearing Stress =	1.17	N/mm²				
Capacity	101%		Padsto	one Required		
Padstone						
A Engineering Brick						
Characteristic strength of padstone =	13.2	N/mm²				
Width of padstone =	100	mm				
Length of padstone =	215	mm				
Maximum bearing stress padstone =	5.50	N/mm²				
Stress under beam =	1.17	N/mm²				
Capacity	21%		ок			
Maximum bearing stress masonry	1.17	N/mm²				
Stress under padstone =	0.83	N/mm²				
Capacity	71%		OK			





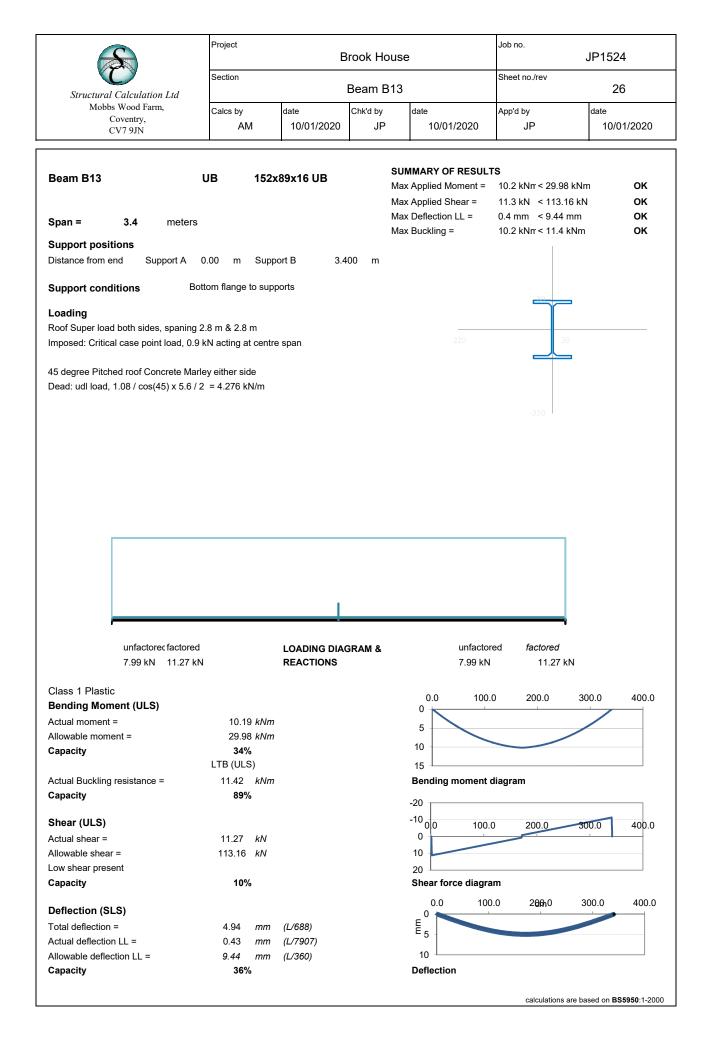
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Structural Ca	lculation Ltd	Section	Bea	ım B5_inter	nal	Sheet no./rev	21	
Cov	'ood Farm, entry, 7 9JN	Calcs by AM	date 10/01/2020	Chk'd by JP	date 10/01/2020	App'd by JP	date 10/01/	2020
•	3.62 meters	IB 203x	102x23 UB	Max Max	MMARY OF RESUL < Applied Moment = < Applied Shear = < Deflection LL =		05 kN	ок ок ок
Support position		00 m Supp	ort B 3.6	20 m				
Support condition	ons Botto	om flange to sup	ports				_	
.oading						80 m		
	ne side, spaning 4.4 ase udl load, 0.3 x 4.		m		-220	3	0	
•	roof Concrete Marle 8 / cos(45) x 4.4 / 2						-	
ead: UDL load, Ti	imber Joist with ceili	ng underneath oi	ne side 1.54 kN/m	n acting from 0	m to 3.62 m			
	imber Joist with ceili	-		-				
mposed: UDL load		-		acting from 0			×Ν	
nposed: UDL load	l, Domestic and resid unfactorec factored 26.85 kN 39.02 kN	-	ISE UDL 3.3 kN/m	acting from 0	m to 3.62 m unfacto 26.85 k 0.0 100	KN 39.02 k	KN 300.0	400.0
nposed: UDL load	I, Domestic and resid unfactorec factored 26.85 kN 39.02 kN It (ULS)	35.32 kNm 56.93 kNm <b>62%</b>	ISE UDL 3.3 kN/m	acting from 0	m to 3.62 m unfacto 26.85 k 0.0 100 20	KN 39.02 k		400.0
nposed: UDL load	I, Domestic and resid unfactorec factored 26.85 kN 39.02 kN It <b>(ULS)</b>	35.32 <i>kNm</i> 56.93 <i>kNm</i> <b>62%</b> LTB (ULS)	ISE UDL 3.3 kN/m	acting from 0	m to 3.62 m unfacto 26.85 k 0.0 100 0	0.0 200.0		400.0
nposed: UDL load	I, Domestic and resid unfactorec factored 26.85 kN 39.02 kN It <b>(ULS)</b>	35.32 kNm 56.93 kNm 62% LTB (ULS) 20.14 Fully	LOADING DIAC	acting from 0	m to 3.62 m unfacto 26.85 k 0.0 100 20 40 Bending moment -50 0 0 100	kN 39.02 k 0.0 200.0		400.0
Class 1 Plastic Bending Moment Actual moment = Allowable moment Capacity Actual Buckling res Capacity Chear (ULS) Actual shear =	I, Domestic and resid unfactorec factored 26.85 kN 39.02 kN It <b>(ULS)</b>	35.32 kNm 56.93 kNm 62% LTB (ULS) 20.14 Fully	LOADING DIAC	acting from 0	m to 3.62 m unfacto 26.85 k 0.0 100 20 40 Bending moment -50	kN 39.02 k 0.0 200.0	300.0	
Class 1 Plastic Bending Moment Actual moment = Allowable moment Capacity Actual Buckling res Capacity Chear (ULS) Actual shear = Allowable shear = Allowable shear = Allowable shear = Allowable shear =	I, Domestic and resid unfactorec factored 26.85 kN 39.02 kN It (ULS) =	35.32 kNm 56.93 kNm 62% LTB (ULS) 20:14 Fully 0% 39.02 kN	LOADING DIAC	acting from 0	m to 3.62 m unfacto 26.85 k 0.0 100 20 40 Bending moment -50 0 0 100	<pre>(N 39.02 k ).0 200.0  diagram .0 200.0</pre>	300.0	
Class 1 Plastic Bending Moment Actual moment = Actual Buckling res Capacity Class 1 Plastic Bending Moment Actual Buckling res Capacity Chear (ULS) Actual shear = Actual s	I, Domestic and resid unfactorec factored 26.85 kN 39.02 kN It (ULS) =	35.32 kNm 56.93 kNm 62% LTB (ULS) 20.14 Fully 0% 39.02 kN 181.05 kN	LOADING DIAC	acting from 0	m to 3.62 m unfacto 26.85 k 0.0 100 20 40 Bending moment -50 0 50 Shear force diagr 0.0 10	<pre>(N 39.02 k ).0 200.0  diagram .0 200.0</pre>	300.0	
mposed: UDL load	I, Domestic and resid	35.32 kNm 56.93 kNm 62% LTB (ULS) 20.14 Fully 0% 39.02 kN 181.05 kN	LOADING DIAC	acting from 0	unfacto 26.85 k 0.0 100 20 40 Bending moment -50 0 100 0 50 Shear force diagr	<pre>(N 39.02 k ).0 200.0  diagram .0 200.0 am</pre>	300.0 300.0	400.0

			В	rook House	9		JP1524
Structural	Calculation Ltd	Section		Beam B11		Sheet no./rev	22
	Wood Farm,	Calcs by	date	Chk'd by	date	App'd by	date
	oventry, V7 9JN	AM	10/01/2020	JP	10/01/2020	JP	10/01/2020
				SUI	MARY OF RESU	LTS	
Beam B11		UB 152	x89x16 UB	Max	Applied Moment =	= 12.1 kNm < 29.98 kN	Nm <b>OK</b>
				Мах	Applied Shear =	42.4 kN < 113.16 k	N <b>OK</b>
ipan =	0.91 meters			Мах	Deflection LL =	0.1 mm < 2.53 mm	n <b>OK</b>
-				Max	Buckling =	12.1 kNrr < 33.7 kNr	m <b>OK</b>
<b>Support positi</b> Distance from en		0.00 m Supp	port B 0.9	10 m			
upport condi	tions Bo	ottom flange to sup	ports				
oading							
omestic and res	sidential load one si	de, spaning 4.2 m					
nposed: Critical	case udl load, 1.5 x	<pre>&lt;4.2 / 2 = 3.15 kN/</pre>	′m			30	
	n ceiling underneath ).7 x 4.2 / 2 = 1.47 k						
	m to 0.91 m, with a k solid + finishes (18				-		
ead. Point load	Load from R10 20	4 kN acting at 0.3	m from the suppor	t			
	, Load from B10 30.	-					
	, Load from B10 30. oad, Load from B10	-					
		-					
		-					
		-					
		-					
		-					
		-					
		-					
	oad, Load from B10	7.2 kN acting at 0.	3 m from the supp	ort			
	unfactorec factorec	7.2 kN acting at 0.	3 m from the supp	ort	unfactu		
	oad, Load from B10	7.2 kN acting at 0.	3 m from the supp	ort	unfact 16.66		
nposed: Point lo	unfactorec factorec	7.2 kN acting at 0.	3 m from the supp	ort	16.66	kN 24.09 kN	80.0 100.0
nposed: Point lo	unfactorec factorec 29.47 kN 42.51 k	7.2 kN acting at 0.	3 m from the supp	ort		kN 24.09 kN	80.0 100.0
Class 1 Plastic Bending Mome Inctual moment =	unfactorec factorec 29.47 kN 42.51 k ent (ULS)	7.2 kN acting at 0. d N 12.13 kNm	3 m from the supp	ort	16.66 0.0 20.0	kN 24.09 kN	80.0 100.0
Class 1 Plastic Bending Mome Actual moment =	unfactorec factorec 29.47 kN 42.51 k ent (ULS)	7.2 kN acting at 0. d N 12.13 kNm 29.98 kNm	3 m from the supp	ort	16.66 0.0 20.0 0	kN 24.09 kN	80.0 100.0
Class 1 Plastic Bending Mome Actual moment =	unfactorec factorec 29.47 kN 42.51 k ent (ULS)	7.2 kN acting at 0. 7.2 kN acting at 0. 12.13 kNm 29.98 kNm 40%	3 m from the supp	ort	0.0 20.0 0 5 10	kN 24.09 kN	80.0 100.0
nposed: Point lo Class 1 Plastic <b>Bending Mome</b> Ictual moment = Ilowable momen <b>Capacity</b>	unfactorec factorec 29.47 kN 42.51 k ent (ULS) : nt =	7.2 kN acting at 0. d N 12.13 kNm 29.98 kNm	3 m from the supp	ort	16.66 0.0 20.0 5 10 15	kN 24.09 kN 40.0 60.0	80.0 100.0
nposed: Point lo Class 1 Plastic Bending Mome Inctual moment = Ilowable moment apacity Inctual Buckling r	unfactorec factorec 29.47 kN 42.51 k ent (ULS) : nt =	7.2 kN acting at 0. 7.2 kN acting at 0. 12.13 kNm 12.13 kNm 29.98 kNm 40% LTB (ULS)	3 m from the supp	ort	0.0 20.0 0 5 10	kN 24.09 kN 40.0 60.0	80.0 100.0
Class 1 Plastic Sending Mome Inctual moment = Illowable moment Capacity Inctual Buckling r Capacity	unfactorec factorec 29.47 kN 42.51 k ent (ULS) : nt =	7.2 kN acting at 0. 7.2 kN acting at 0. 12.13 kNm 29.98 kNm 40% LTB (ULS) 33.71 kNm	3 m from the supp	ort	16.66	kN 24.09 kN 40.0 60.0	80.0 100.0
Class 1 Plastic Bending Mome Inctual moment = Ilowable moment Capacity Inctual Buckling r Capacity Shear (ULS)	unfactorec factorec 29.47 kN 42.51 k ent (ULS) : nt =	7.2 kN acting at 0. 7.2 kN acting at 0. 12.13 kNm 29.98 kNm 40% LTB (ULS) 33.71 kNm	3 m from the supp	ort	16.66	kN 24.09 kN 40.0 60.0 t diagram	
Class 1 Plastic Bending Mome Actual moment = Allowable moment apacity Actual Buckling r Capacity Shear (ULS) Actual shear =	unfactorec factorec 29.47 kN 42.51 k ent (ULS) nt =	7.2 kN acting at 0. 7.2 kN acting at 0. 12.13 kNm 29.98 kNm 40% LTB (ULS) 33.71 kNm 36%	3 m from the supp	ort	16.66	kN 24.09 kN 40.0 60.0 t diagram	
Class 1 Plastic Bending Mome Actual moment = Allowable moment apacity Actual Buckling r Capacity Chear (ULS) Actual shear = Allowable shear =	unfactorec factorec 29.47 kN 42.51 k ent (ULS) nt = resistance =	7.2 kN acting at 0.3 12.13 kNm 29.98 kNm 40% LTB (ULS) 33.71 kNm 36% 42.37 kN 113.16 kN	3 m from the supp	ort	16.66 0.0 20.0 5 10 15 Bending moment -20 0 20 40 60	kN 24.09 kN 40.0 60.0 t diagram	
Class 1 Plastic Bending Mome Actual moment = Illowable moment apacity Actual Buckling r Capacity Chear (ULS) Actual shear = Illowable shear : ow shear present	unfactorec factorec 29.47 kN 42.51 k ent (ULS) nt = resistance =	7.2 kN acting at 0. 7.2 kN acting at 0. 12.13 kNm 29.98 kNm 40% LTB (ULS) 33.71 kNm 36% 42.37 kN	3 m from the supp	ort	16.66 0.0 20.0 5 10 15 Bending moment -20 0 20 40	kN 24.09 kN 40.0 60.0 t diagram	
Class 1 Plastic Bending Mome Actual moment = Actual Buckling r Capacity Actual Buckling r Capacity Shear (ULS) Actual shear = Actual shear =	unfactorec factorec 29.47 kN 42.51 k ent (ULS) nt = resistance = = nt	7.2 kN acting at 0.3 12.13 kNm 29.98 kNm 40% LTB (ULS) 33.71 kNm 36% 42.37 kN 113.16 kN	3 m from the supp	ort	16.66 0.0 20.0 5 10 15 Bending moment -20 0 20 40 60 Shear force diag	kN 24.09 kN 40.0 60.0 t diagram	
Class 1 Plastic Bending Mome Actual moment = Allowable moment Capacity Actual Buckling r Capacity Shear (ULS) Actual shear = Allowable she	unfactorec factorec 29.47 kN 42.51 k ent (ULS) nt = resistance = = nt	7.2 kN acting at 0.3 12.13 kNm 29.98 kNm 40% LTB (ULS) 33.71 kNm 36% 42.37 kN 113.16 kN	3 m from the supp	ort	16.66 0.0 20.0 5 10 15 Bending moment -20 0 20 40 60 Shear force diag	kN 24.09 kN 40.0 60.0 t diagram	80.0 100.0
	unfactorec factorec 29.47 kN 42.51 k ent (ULS) nt = resistance = = nt S)	7.2 kN acting at 0.3 12.13 kNm 29.98 kNm 40% LTB (ULS) 33.71 kNm 36% 42.37 kN 113.16 kN 37%	3 m from the supp	ort	16.66	kN 24.09 kN 40.0 60.0 t diagram	80.0 100.0
Class 1 Plastic Bending Mome Actual moment = Allowable moment Capacity Actual Buckling r Capacity Shear (ULS) Actual shear = Allowable shear = Allowable shear = Capacity Capacity Capacity Capacity Capacity Capacity Capacity Capacity Capacity Capacity Capacity	unfactorec factorec 29.47 kN 42.51 k ent (ULS) = nt = = nt S) = LL =	7.2 kN acting at 0.3 12.13 kNm 29.98 kNm 40% LTB (ULS) 33.71 kNm 36% 42.37 kN 113.16 kN 37% 0.34 mm	3 m from the supp	ort	16.66 0.0 20.0 5 10 15 Bending moment -20 0 20 40 60 Shear force diag	kN 24.09 kN 40.0 60.0 t diagram	80.0 100.0

S	Project	В	rook House	e	Job no.	JP1524
Structural Calculation Ltd	Section	Bea	ring Beam	B11	Sheet no./rev	23
Mobbs Wood Farm, Coventry, CV7 9JN	Calcs by AM	date 10/01/2020	Chk'd by JP	date 10/01/2020	App'd by JP	date 10/01/2020
Beam End Reaction =	42.51	kN (factored	)	Variable Load Safe Permanent Load S		
Masonry						
Masonry type =	Weak Br	ickwork				
Characteristic strength of masonry =	2.8	N/mm²				
Width of beam end bearing =	88.7	mm		Bearing factor =	1.25	
Length of beam end bearing =	150	mm		γm =	3.00	
Stress						
Maximum Bearing Stress =	1.17	N/mm²				
Actual Bearing Stress =	3.20	N/mm²				
Capacity	274%		Padsto	ne Required		
Padstone						
A Engineering Brick						
Characteristic strength of padstone =	13.2	N/mm²				
Width of padstone =	100	mm				
Length of padstone =	440	mm				
Maximum bearing stress padstone =	5.50	N/mm²				
Stress under beam =	3.20	N/mm²				
Capacity	58%		ОК			
Maximum bearing stress masonry	1.17	N/mm²				
Stress under padstone =	0.97	N/mm²				
Capacity	83%		ок			

2		Section				Sheet no./rev	JP1524
Structural Cal		Section		Beam B12		Sheet no./rev	24
Mobbs Wo Cove CV7	entry,	Calcs by AM	<sup>date</sup> 10/01/2020	Chk'd by JP	date 10/01/2020	App'd by JP	<sup>date</sup> 10/01/2020
Beam B12	L	JC 203	x203x46 UC	Ma Ma	MMARY OF RESUL x Applied Moment = x Applied Shear =	42.4 kNrr < 123.75 k 49.9 kN < 241.4 kN	ок
ipan = 3	3.4 meters				x Deflection LL = x Buckling =	0.7 mm < 9.44 mm 42.4 kNr < 91.5 kNr	
Support position		.00 m Sup	oport B 3.4			42.4 KWI 3 01.0 KWI	
Support conditio	ns Botte	om flange to su	pports				_
oading						80	
-	e side, spaning 4.2	m					
nposed: Critical ca	use udl load, 0.3 x 4	.2/2 = 0.63 kM	l/m			30	
5 degree Pitched r	oof Concrete Marle	v one side					
-	3 / cos(45) x 4.2 / 2	-					
and UDI load Ti	mbor loist with coili	ng undornoath	ono sido 1 47 kN/m	acting from 0	m to 3.4 m		
	mber Joist with ceili , Domestic and resi	-		-			
mposed: UDL load		-		n acting from		red <i>factored</i>	
nposed: UDL load	, Domestic and resi	-	ase UDL 3.15 kN/r	n acting from	0 m to 3.4 m		
nposed: UDL load	, Domestic and resi nfactorec factored 4.69 kN 49.86 kN	-	ase UDL 3.15 kN/r	n acting from	0 m to 3.4 m unfacto 34.69 k	N 49.86 kN	300.0 400.0
nposed: UDL load	, Domestic and resi nfactorec factored 4.69 kN 49.86 kN	dential Critical d	LOADING DIAG	n acting from	0 m to 3.4 m	N 49.86 kN	300.0 400.0
nposed: UDL load	, Domestic and resi nfactorec factored 4.69 kN 49.86 kN t (ULS)	-	ase UDL 3.15 kN/r LOADING DIAG REACTIONS	n acting from	0 m to 3.4 m unfacto 34.69 k 0.0 100	N 49.86 kN	300.0 400.0
nposed: UDL load	, Domestic and resi nfactorec factored 4.69 kN 49.86 kN t (ULS)	dential Critical d	ase UDL 3.15 kN/r LOADING DIAG REACTIONS	n acting from	0 m to 3.4 m unfacto 34.69 k 0.0 100	N 49.86 kN	300.0 400.0
nposed: UDL load	, Domestic and resi Infactorec factored 4.69 kN 49.86 kN t (ULS) =	42.38 <i>kNr</i> 123.75 <i>kNr</i> <b>34%</b> LTB (ULS)	LOADING DIAG REACTIONS	n acting from	0 m to 3.4 m unfacto 34.69 k 0.0 100 40 60	N 49.86 kN	300.0 400.0
nposed: UDL load	, Domestic and resi Infactorec factored 4.69 kN 49.86 kN t (ULS) =	42.38 <i>kNr</i> 123.75 <i>kNr</i> <b>34%</b> LTB (ULS) 91.46 <i>kNr</i>	LOADING DIAG REACTIONS	n acting from	0 m to 3.4 m unfacto 34.69 k	N 49.86 kN	300.0 400.0
nposed: UDL load	, Domestic and resi Infactorec factored 4.69 kN 49.86 kN t (ULS) =	42.38 <i>kNr</i> 123.75 <i>kNr</i> <b>34%</b> LTB (ULS)	LOADING DIAG REACTIONS	n acting from	0 m to 3.4 m unfacto 34.69 k 0.0 100 20 40 60 Bending moment -100	N 49.86 kN .0 200.0 3 diagram	
nposed: UDL load	, Domestic and resi Infactorec factored 4.69 kN 49.86 kN t (ULS) =	42.38 <i>kNr</i> 123.75 <i>kNr</i> 34% LTB (ULS) 91.46 <i>kNr</i> 46%	LOADING DIAG REACTIONS	n acting from	0 m to 3.4 m unfacto 34.69 k 0.0 100 20 40 60 Bending moment	N 49.86 kN .0 200.0 3 diagram	300.0 400.0
nposed: UDL load	, Domestic and resi Infactorec factored 4.69 kN 49.86 kN t (ULS) =	42.38 <i>kNr</i> 123.75 <i>kNr</i> <b>34%</b> LTB (ULS) 91.46 <i>kNr</i>	LOADING DIAG REACTIONS	n acting from	0 m to 3.4 m unfacto 34.69 k 0.0 100 20 40 60 Bending moment -100 -50 00 100	N 49.86 kN .0 200.0 3 diagram	
nposed: UDL load	, Domestic and resi Infactorec factored 4.69 kN 49.86 kN t (ULS) =	42.38 <i>kNr</i> 123.75 <i>kNr</i> <b>34%</b> LTB (ULS) 91.46 <i>kNr</i> <b>46%</b>	LOADING DIAG REACTIONS	n acting from	0 m to 3.4 m unfacto 34.69 k 0.0 100 20 40 60 Bending moment -100 0 100 0 100	N 49.86 kN .0 200.0 3 diagram	
nposed: UDL load u Class 2 Compact Sending Moment actual moment = Nowable moment = Class 2 Compact Sending Moment actual moment = Nowable moment = Class 2 Compact Sending Moment actual moment = Nowable moment = Class 2 Compact Sending Moment actual shear = Nowable shear = Nowable shear = Nowable shear = Nowable shear =	, Domestic and resi Infactorec factored 4.69 kN 49.86 kN t (ULS) =	42.38 <i>kNr</i> 123.75 <i>kNr</i> <b>34%</b> LTB (ULS) 91.46 <i>kNr</i> <b>46%</b>	LOADING DIAG REACTIONS	n acting from	0 m to 3.4 m unfacto 34.69 k 0.0 100 20 40 60 Bending moment -50 0 0 100 0 100 0 100 0 100 0 100	N 49.86 kN .0 200.0 3 diagram	
Anposed: UDL load	, Domestic and resi Infactorec factored 4.69 kN 49.86 kN t (ULS) =	42.38 <i>kNr</i> 123.75 <i>kNr</i> <b>34%</b> LTB (ULS) 91.46 <i>kNr</i> <b>49</b> .86 <i>kN</i> 241.40 <i>kN</i>	LOADING DIAG REACTIONS	n acting from	0 m to 3.4 m unfacto 34.69 k 0.0 100 20 40 60 Bending moment -100 -50 100 50 100 50 100 50 100 50 100 50 100 50 100 50 100 50 100 50 100 50 100 50 100 50 100 50 100 10	N 49.86 kN .0 200.0 3 diagram	
Class 2 Compact Bending Moment Actual moment = Allowable moment = Actual Buckling res Capacity Chear (ULS) Actual shear = Allowable shear	, Domestic and resi Infactorec factored 4.69 kN 49.86 kN t (ULS) =	42.38 <i>kNr</i> 123.75 <i>kNr</i> <b>34%</b> LTB (ULS) 91.46 <i>kNr</i> <b>49</b> .86 <i>kN</i> 241.40 <i>kN</i>	n n	n acting from	0 m to 3.4 m unfacto 34.69 k 0.0 100 0 40 60 Bending moment -100 -50 0 0 100 0 50 100 0 50 0 0 100 0 50 0 0 100 0 50 0 0 100 0 50 0 0 100 0 50 0 0 100 0 0 100 0 0 0 0 0 0 0 0 0 0 0 0	N 49.86 kN .0 200.0 3 diagram	300.0 400.0
mposed: UDL load	, Domestic and resi infactorec factored 4.69 kN 49.86 kN t (ULS) = istance =	42.38 kNr 123.75 kNr 34% LTB (ULS) 91.46 kNr 49.86 kN 241.40 kN	LOADING DIAG REACTIONS	n acting from	0 m to 3.4 m unfacto 34.69 k 0.0 100 20 40 60 Bending moment -100 -50 100 50 100 50 100 50 100 50 100 50 100 50 100 50 100 50 100 50 100 50 100 50 100 50 100 50 100 10	N 49.86 kN .0 200.0 3 diagram	300.0 400.0
Class 2 Compact Bending Moment Actual moment = Allowable moment = Capacity Actual Buckling res Capacity Shear (ULS) Actual shear = Allowable shear = Capacity Deflection (SLS) Fotal deflection =	, Domestic and resi infactorec factored 4.69 kN 49.86 kN t (ULS) = istance =	42.38 kNr 123.75 kNr 34% LTB (ULS) 91.46 kNr 49.86 kN 241.40 kN 21% 3.79 mm	LOADING DIAG REACTIONS	n acting from	0 m to 3.4 m unfacto 34.69 k 0.0 100 0 40 60 Bending moment -100 -50 0 0 100 0 50 100 0 50 0 0 100 0 50 0 0 100 0 50 0 0 100 0 50 0 0 100 0 50 0 0 100 0 0 100 0 0 0 0 0 0 0 0 0 0 0 0	N 49.86 kN .0 200.0 3 diagram	300.0 400.0

S	Project	В	rook House	e	Job no.	JP1524
Structural Calculation Ltd	Section	Bea	ring Beam	B12	Sheet no./rev	25
Mobbs Wood Farm, Coventry, CV7 9JN	Calcs by AM	date 10/01/2020	Chk'd by JP	date 10/01/2020	App'd by JP	date 10/01/2020
Beam End Reaction =	49.86	kN (factored	)	Variable Load Safe Permanent Load S		
Masonry						
Masonry type =	Weak Br	ickwork				
Characteristic strength of masonry =	2.8	N/mm²				
Width of beam end bearing =	100	mm		Bearing factor =	1.25	
Length of beam end bearing =	150	mm		γm =	3.00	
Stress						
Maximum Bearing Stress =	1.17	N/mm²				
Actual Bearing Stress =	3.32	N/mm²				
Capacity	285%		Padsto	ne Required		
Padstone						
A Engineering Brick						
Characteristic strength of padstone =	13.2	N/mm²				
Width of padstone =	100	mm				
Length of padstone =	660	mm				
Maximum bearing stress padstone =	5.50	N/mm²				
Stress under beam =	3.32	N/mm²				
Capacity	60%		ОК			
Maximum bearing stress masonry	1.17	N/mm²				
Stress under padstone =	0.76	N/mm²				
Capacity	65%		ок			



S	Project	В	rook House	e	Job no.	JP1524
Structural Calculation Ltd	Section	Bea	ring Beam	B13	Sheet no./rev	27
Mobbs Wood Farm, Coventry, CV7 9JN	Calcs by AM	date 10/01/2020	Chk'd by JP	<sup>date</sup> 10/01/2020	App'd by JP	date 10/01/2020
Beam End Reaction =	11.27	kN (factored	)	Variable Load Safe Permanent Load S	-	
Masonry						
Masonry type =	3.6N Blo	ckwork				
Characteristic strength of masonry =	2.6	N/mm²				
Width of beam end bearing =	88.7	mm		Bearing factor =	1.25	
Length of beam end bearing =	100	mm		γm =	3.00	
Stress						
Maximum Bearing Stress =	1.08	N/mm²				
Actual Bearing Stress =	1.27	N/mm²				
Capacity	117%		Padsto	ne Required		
Padstone						
A Engineering Brick						
Characteristic strength of padstone =	13.2	N/mm²				
Width of padstone =	100	mm				
Length of padstone =	215	mm				
Maximum bearing stress padstone =	5.50	N/mm²				
Stress under beam =	1.27	N/mm²				
Capacity	23%		ОК			
Maximum bearing stress masonry	1.08	N/mm²				
Stress under padstone =	0.52	N/mm²				
Capacity	48%		ок			

R	Project	Brook	House		Job no. JP1	524
Structural Calculation Ltd Mobbs Wood Farm	Calcs for	Portal	Frame		Start page no./Re	evision 28
Ansty CV7 9JN	Calcs by AM	Calcs date 10/01/2020	Checked by JP	Checked date 10/01/2020	Approved by JP	Approved date 10/01/2020

### STEEL MEMBER ANALYSIS & DESIGN (EN1993-1-1:2005)

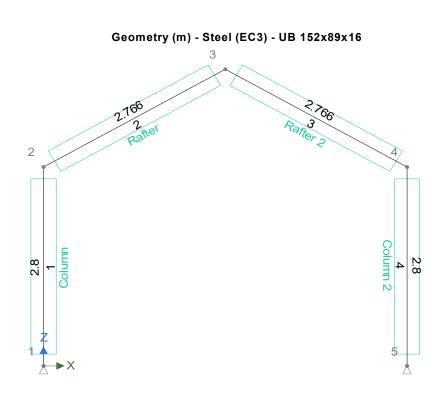
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 4.3.04

### ANALYSIS

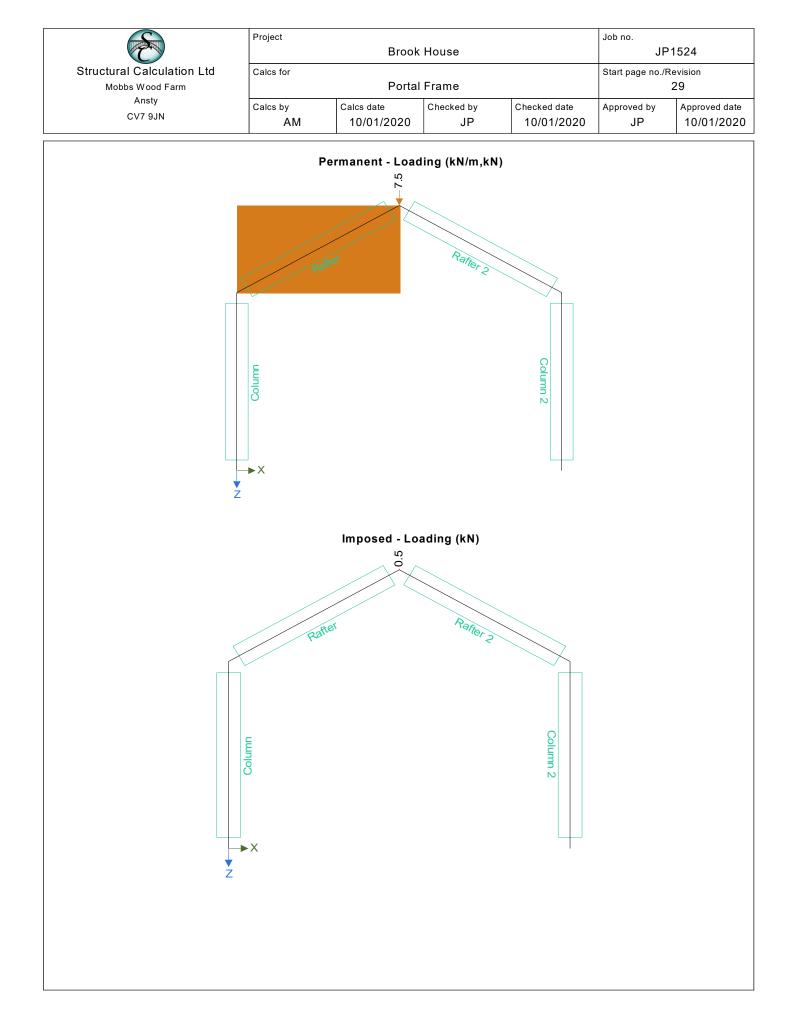
### Geometry

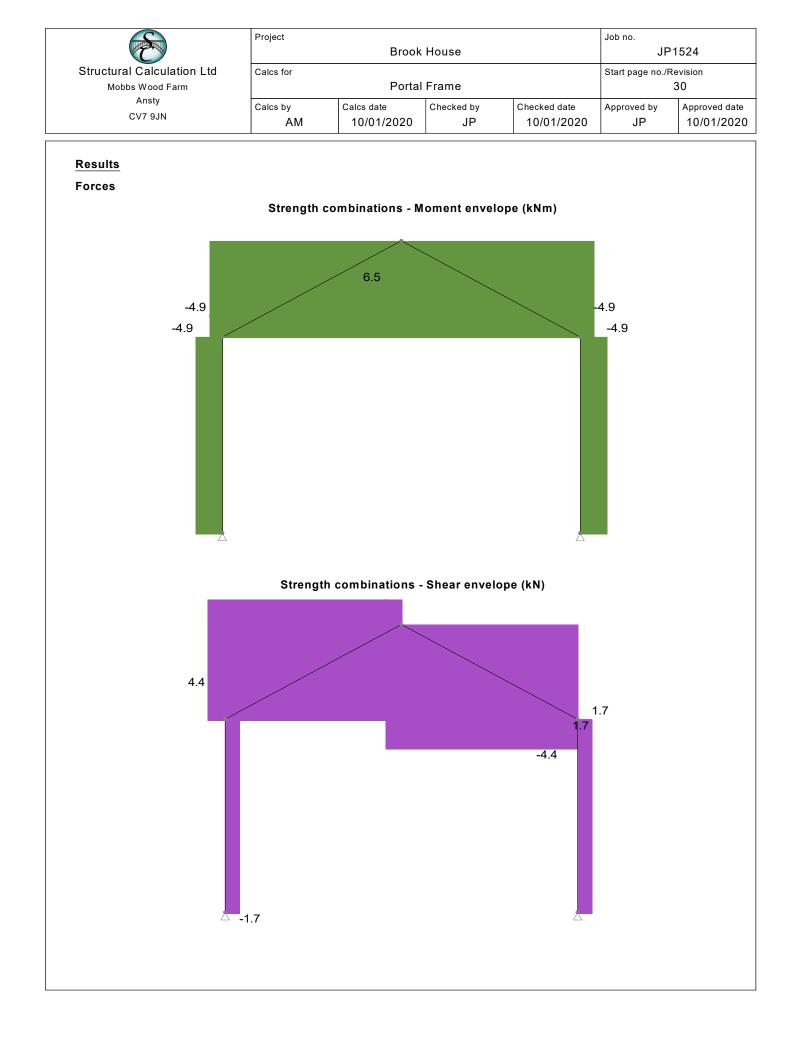
Tedds calculation version 1.0.27

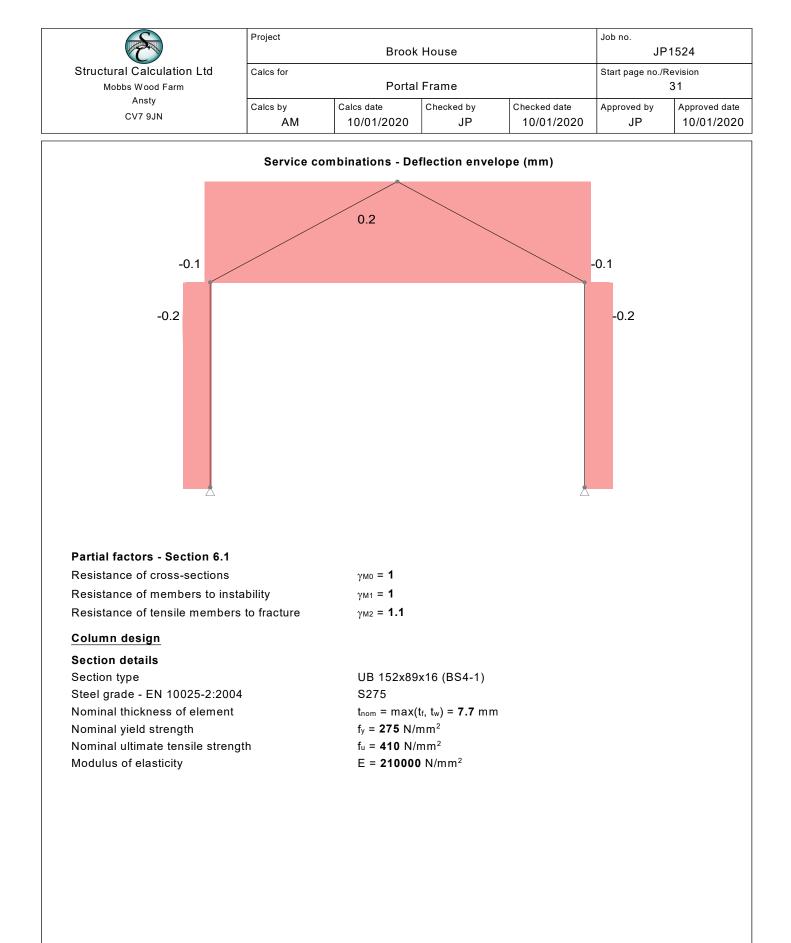


#### Loading

Self weight included







	Project	Brook	House		Job no. J	P1524
Structural Calculation Ltd	Calcs for				Start page no.	
Mobbs Wood Farm		Portal	Frame			32
Ansty CV7 9JN	Calcs by AM	Calcs date 10/01/2020	Checked by JP	Checked date 10/01/2020	Approved by JP	Approved da 10/01/20
2.77						
$\uparrow  \stackrel{\bullet}{\uparrow}  \stackrel{\bullet}{\uparrow}$		Section de	<b>9x16 (BS4-1)</b> epth, h, 152.4 mm			
		Mass of se	eadth, b, 88.7 mm ection, Mass, 16 kg/m			
		Web thick	ckness, t <sub>f</sub> , 7.7 mm ness, t <sub>w</sub> , 4.5 mm			
		Area of se	ls, r, 7.6 mm ction, A, 2032 mm <sup>2</sup>			
152.4-		Radius of	gyration about y-axis, i <sub>y</sub> , 64. gyration about z-axis, i <sub>z</sub> , 21.	016 mm		
Ì		Elastic sec	ction modulus about y-axis, \ ction modulus about z-axis, \	N <sub>el.z</sub> , 20237 mm <sup>3</sup>		
	→ -4	Plastic sec	ction modulus about y-axis, \ ction modulus about z-axis, \	N <sub>pl.z</sub> , 31180 mm <sup>3</sup>		
			oment of area about y-axis, oment of area about z-axis,			
∠:∠ →						
<u>↓</u> <u>+</u>						
	88.7					
Lateral restraint Both flanges have lateral rest	raint at supports	s only				
5		,				
Consider Combination 4 4	250 1 4 50 1 4	EDO (Ctronoth)				
Consider Combination 1 - 1		· - ·				
Consider Combination 1 - 1 Classification of cross sect		5.5	1/m m <sup>2</sup> / f ] = 0 02			
Classification of cross sect	ions - Section s	<b>5.5</b> ε = √[235 Ν	l/mm <sup>2</sup> / fy] = <b>0.92</b>			
Classification of cross sect Internal compression parts	ions - Section s	5.5 ε = √[235 Ν ding and compre	ssion - Table 5.			
Classification of cross sect	ions - Section s	5.5 ε = √[235 Ν ding and compre c = d = 121	ssion - Table 5. .8 mm	2 (sheet 1 of 3)		
Classification of cross sect Internal compression parts	ions - Section s	5.5 ε = √[235 Ν ding and compre c = d = 121 α = min([h	ssion - Table 5.	2 (sheet 1 of 3) × fy) - (t <sub>f</sub> + r)] / c	s, 1) = <b>0.522</b>	
Classification of cross section Internal compression parts Width of section	ions - Section subject to ben	5.5 ε = √[235 Ν ding and compre c = d = 121 α = min([h	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × t <sub>w</sub>	2 (sheet 1 of 3) × fy) - (t <sub>f</sub> + r)] / c	s, 1) = <b>0.522</b>	
Classification of cross sect Internal compression parts	ions - Section subject to ben	5.5 $\varepsilon = \sqrt{235}$ N ding and compre c = d = 121 $\alpha = min([h + c / t_w = 27.7])$	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × t <sub>w</sub>	2 (sheet 1 of 3) × fy) - (tr + r)] / c 96 × ε / (13 × α ·	s, 1) = <b>0.522</b>	
Classification of cross sect Internal compression parts Width of section Outstand flanges - Table 5.2	ions - Section subject to ben	5.5 $\varepsilon = \sqrt{235}$ N ding and compre c = d = 121 $\alpha = min([h])$ $c / t_w = 27.7$ $c = (b - t_w - b)$	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × t <sub>w</sub> 1 = 29.3 × ε <= 3	2 (sheet 1 of 3) × fy) - (t <sub>f</sub> + r)] / c 96 × ε / (13 × α · mm	:, 1) = <b>0.522</b> - 1) Cla:	
Classification of cross sect Internal compression parts Width of section Outstand flanges - Table 5.2	ions - Section subject to ben	5.5 $\varepsilon = \sqrt{235}$ N ding and compre c = d = 121 $\alpha = min([h])$ $c / t_w = 27.7$ $c = (b - t_w - b)$	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × t <sub>w</sub> 1 = 29.3 × ε <= 3 2 × r) / 2 = 34.5	2 (sheet 1 of 3) × fy) - (t <sub>f</sub> + r)] / c 96 × ε / (13 × α · mm	s, 1) = <b>0.522</b> - 1) Clas	ss 1
Classification of cross sect Internal compression parts Width of section Outstand flanges - Table 5.2	ions - Section subject to ben 2 (sheet 2 of 3)	5.5 $\varepsilon = \sqrt{235}$ N ding and compre c = d = 121 $\alpha = min([h])$ $c / t_w = 27.7$ $c = (b - t_w - b)$	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × t <sub>w</sub> 1 = 29.3 × ε <= 3 2 × r) / 2 = 34.5	2 (sheet 1 of 3) × fy) - (t <sub>f</sub> + r)] / c 96 × ε / (13 × α · mm	s, 1) = <b>0.522</b> - 1) Clas	ss 1
Classification of cross sect Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section	ions - Section subject to ben 2 (sheet 2 of 3)	5.5 $\varepsilon = \sqrt{235}$ N ding and compre c = d = 121 $\alpha = min([h])$ $c / t_w = 27.7$ $c = (b - t_w - b)$	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × t <sub>w</sub> 1 = 29.3 × ε <= 3 2 × r) / 2 = <b>34.5</b> = 4.8 × ε <= 9 × ε	2 (sheet 1 of 3) × fy) - (t <sub>f</sub> + r)] / c 96 × ε / (13 × α · mm	s, 1) = <b>0.522</b> - 1) Clas	ss 1
Classification of cross sect Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section	ions - Section 4 subject to ben 2 (sheet 2 of 3) on 6.2.4	5.5 $\varepsilon = \sqrt{235}$ N ding and compre c = d = 121 $\alpha = min([h]$ $c / t_w = 27.2$ $c = (b - t_w - c)$ $c / t_f = 4.5 = 0$ NEd = 6.6 k	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × t <sub>w</sub> 1 = 29.3 × ε <= 3 2 × r) / 2 = <b>34.5</b> = 4.8 × ε <= 9 × ε	2 (sheet 1 of 3) × fy) - (tr + r)] / c 96 × ε / (13 × α · mm Class 1	s, 1) = <b>0.522</b> - 1) Clas	ss 1
Classification of cross sect Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force	ions - Section 4 subject to ben 2 (sheet 2 of 3) on 6.2.4	5.5 $\varepsilon = \sqrt{235}$ N ding and compre c = d = 121 $\alpha = min([h])$ c / tw = 27.2 c = (b - tw - c) c = (b - tw - c) c / tr = 4.5 = 0 NEd = 6.6 k N <sub>c,Rd</sub> = N <sub>pl,F</sub> NEd / N <sub>c,Rd</sub> = 0	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × t <sub>w</sub> 1 = 29.3 × ε <= 3 2 × r) / 2 = 34.5 = 4.8 × ε <= 9 × ε N Rd = A × f <sub>y</sub> / γ <sub>M0</sub> = = 0.012	2 (sheet 1 of 3) × f <sub>y</sub> ) - (t <sub>f</sub> + r)] / c 96 × ε / (13 × α mm Class 1 558.8 kN	s, 1) = <b>0.522</b> - 1) Clas Se	ss 1 ction is clas
Classification of cross sect Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force	ions - Section 4 subject to ben 2 (sheet 2 of 3) on 6.2.4	5.5 $\varepsilon = \sqrt{235}$ N ding and compre c = d = 121 $\alpha = min([h])$ c / tw = 27.2 c = (b - tw - c) c = (b - tw - c) c / tr = 4.5 = 0 NEd = 6.6 k N <sub>c,Rd</sub> = N <sub>pl,F</sub> NEd / N <sub>c,Rd</sub> = 0	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × t <sub>w</sub> 1 = 29.3 × ε <= 3 2 × r) / 2 = 34.5 = 4.8 × ε <= 9 × ε N Rd = A × fy / γmo =	2 (sheet 1 of 3) × f <sub>y</sub> ) - (t <sub>f</sub> + r)] / c 96 × ε / (13 × α mm Class 1 558.8 kN	s, 1) = <b>0.522</b> - 1) Clas Se	ss 1 ction is clas
Classification of cross sect Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force Design resistance of section - Slenderness ratio for y-y ax	ions - Section 4 subject to ben 2 (sheet 2 of 3) on 6.2.4 - eq 6.10	5.5 $\varepsilon = \sqrt{235}$ N ding and compre c = d = 121 $\alpha = min([h])$ c / tw = 27.7 c = (b - tw - c) c = (b - tw - c) c / tr = 4.5 = 0 NEd = 6.6 k N <sub>c,Rd</sub> = N <sub>PI,F</sub> NEd / N <sub>c,Rd</sub> = PASS - Desig kling - Section 6.3	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × t <sub>w</sub> 1 = 29.3 × ε <= 3 2 × r) / 2 = 34.5 = 4.8 × ε <= 9 × ε N Rd = A × fy / γM0 = = 0.012 n compression 3.1.3	2 (sheet 1 of 3) × f <sub>y</sub> ) - (t <sub>f</sub> + r)] / c 96 × ε / (13 × α mm Class 1 558.8 kN	s, 1) = <b>0.522</b> - 1) Clas Se	ss 1 ction is clas
Classification of cross sect Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force Design resistance of section - Slenderness ratio for y-y ax Critical buckling length	ions - Section 4 subject to ben 2 (sheet 2 of 3) on 6.2.4 - eq 6.10	5.5 $\varepsilon = \sqrt{235}$ N ding and compre c = d = 121 $\alpha = min([h - c / tw = 27.2])$ c = (b - tw - c / tw = 27.2] c = (b - tw - c / tw - c / tw = 27.2] c = (b - tw - c / tw - c / tw - c / tw = 27.2] c = (b - tw - c /	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × t <sub>w</sub> 1 = 29.3 × ε <= 3 2 × r) / 2 = 34.5 = 4.8 × ε <= 9 × ε N Rd = A × f <sub>y</sub> / γ <sub>M0</sub> = = 0.012 n compression 3.1.3 1 = 2800 mm	2 (sheet 1 of 3) × fy) - (t <sub>f</sub> + r)] / c 96 × ε / (13 × α mm Class 1 558.8 kN <i>resistance exc</i>	s, 1) = <b>0.522</b> - 1) Clas Se	ss 1 ction is clas
Classification of cross sect Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force Design resistance of section - Slenderness ratio for y-y ax Critical buckling length Critical buckling force	ions - Section 4 subject to ben 2 (sheet 2 of 3) on 6.2.4 - eq 6.10 is flexural bucl	5.5 $\varepsilon = \sqrt{[235 N]}$ ding and compre c = d = 121 $\alpha = min([h]$ $c / t_w = 27.7$ $c = (b - t_w - c) / t_w = 27.7$	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × tw 1 = 29.3 × $\varepsilon$ <= 3 2 × r) / 2 = 34.5 = 4.8 × $\varepsilon$ <= 9 × $\varepsilon$ N Rd = A × fy / $\gamma$ M0 = = 0.012 n compression 3.1.3 1 = 2800 mm E × ly / Lor,y <sup>2</sup> = 22	2 (sheet 1 of 3) × fy) - (tr + r)] / c 96 × ε / (13 × α - mm Class 1 558.8 kN <i>resistance exc</i> 05.5 kN	s, 1) = <b>0.522</b> - 1) Clas Se	ss 1 ction is clas
Classification of cross sect Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force Design resistance of section - Slenderness ratio for y-y ax Critical buckling length Critical buckling force Slenderness ratio for buckling	ions - Section 4 subject to ben 2 (sheet 2 of 3) on 6.2.4 - eq 6.10 is flexural buck	5.5 $\varepsilon = \sqrt{[235 N]}$ ding and compre c = d = 121 $\alpha = min([h]$ c / tw = 27.7 c = (b - tw - c) / tr = 4.5 = 0 NEd = 6.6 k Nc,Rd = NpI,R NEd / Nc,Rd = 0 PASS - Desig kling - Section 6.3 Lor,y = Lm1_s Ncr,y = $\pi^2 \times \overline{\lambda}_y = \sqrt{(A \times b^2)}$	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × tw 1 = 29.3 × $\varepsilon$ <= 3 2 × r) / 2 = 34.5 = 4.8 × $\varepsilon$ <= 9 × $\varepsilon$ N Rd = A × fy / $\gamma$ M0 = = 0.012 n compression 3.1.3 1 = 2800 mm E × ly / Lcr.y <sup>2</sup> = 22 fy / Ncr.y) = 0.503	2 (sheet 1 of 3) × fy) - (tr + r)] / c 96 × ε / (13 × α - mm Class 1 558.8 kN <i>resistance exc</i> 05.5 kN	s, 1) = <b>0.522</b> - 1) Clas Se	ss 1 ction is clas
Classification of cross sect Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force Design resistance of section - Slenderness ratio for y-y ax Critical buckling length Critical buckling force Slenderness ratio for buckling Check y-y axis flexural buck	ions - Section 4 subject to ben 2 (sheet 2 of 3) on 6.2.4 - eq 6.10 is flexural buck	5.5 $\varepsilon = \sqrt{[235 \text{ N}]}$ ding and compre c = d = 121 $\alpha = \min([h])$ c / tw = 27.7 c = (b - tw - c) / tr = 4.5 = 0 NEd = 6.6 k N <sub>c,Rd</sub> = N <sub>pl,F</sub> NEd / N <sub>c,Rd</sub> = $PASS - Desig$ kling - Section 6.3 $L_{cr,y} = L_m1_{-s}$ $N_{cr,y} = \sqrt{(A \times c)}$ $\epsilon - Section 6.3.1.1$	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × tw 1 = 29.3 × $\varepsilon$ <= 3 2 × r) / 2 = 34.5 = 4.8 × $\varepsilon$ <= 9 × $\varepsilon$ N Rd = A × fy / $\gamma$ M0 = = 0.012 n compression 3.1.3 1 = 2800 mm E × ly / Lcr.y <sup>2</sup> = 22 fy / Ncr.y) = 0.503	2 (sheet 1 of 3) × fy) - (tr + r)] / c 96 × ε / (13 × α - mm Class 1 558.8 kN <i>resistance exc</i> 05.5 kN	s, 1) = <b>0.522</b> - 1) Clas Se	ss 1 ction is clas
Classification of cross sect Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force Design resistance of section - Slenderness ratio for y-y ax Critical buckling length Critical buckling force Slenderness ratio for buckling Check y-y axis flexural buck Buckling curve - Table 6.2	ions - Section 4 subject to ben 2 (sheet 2 of 3) on 6.2.4 - eq 6.10 is flexural buck g - eq 6.50 cling resistance	5.5 $\varepsilon = \sqrt{235}$ N ding and compre c = d = 121 $\alpha = min([h])$ c / tw = 27.7 c = (b - tw - c) / tr = 27.7 c = (b - tw - c) / tr = 4.5 = 0 NEd = 6.6 k Nc,Rd = NpI,F NEd / Nc,Rd = 0 PASS - Desig kling - Section 6.3 Lor,y = Lm1_s Nor,y = $\pi^2 \times c$ $\overline{\lambda}y = \sqrt{A} \times c$ e - Section 6.3.1.1 a	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × tw 1 = 29.3 × $\varepsilon$ <= 3 2 × r) / 2 = 34.5 = 4.8 × $\varepsilon$ <= 9 × $\varepsilon$ N Rd = A × fy / $\gamma$ M0 = = 0.012 n compression 3.1.3 1 = 2800 mm E × ly / Lcr.y <sup>2</sup> = 22 fy / Ncr.y) = 0.503	2 (sheet 1 of 3) × fy) - (tr + r)] / c 96 × ε / (13 × α - mm Class 1 558.8 kN <i>resistance exc</i> 05.5 kN	s, 1) = <b>0.522</b> - 1) Clas Se	ss 1 ction is clas
Classification of cross sect Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force Design resistance of section - Slenderness ratio for y-y ax Critical buckling length Critical buckling force Slenderness ratio for buckling Check y-y axis flexural buck Buckling curve - Table 6.2 Imperfection factor - Table 6.2	ions - Section 4 subject to ben 2 (sheet 2 of 3) on 6.2.4 - eq 6.10 is flexural buck g - eq 6.50 cling resistance	5.5 $\varepsilon = \sqrt{235}$ N ding and compre c = d = 121 $\alpha = min([h])$ c / tw = 27.7 c = (b - tw - c) / tr = 4.5 = 0 NEd = 6.6 k Nc,Rd = NpI,F NEd / Nc,Rd = 0 PASS - Desig kling - Section 6.3 Lor,y = $\pi^2 \times \lambda_y = \sqrt{A} \times 2^{-1}$ $\lambda_y = \sqrt{A} \times 2^{-1}$	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × tw 1 = 29.3 × $\varepsilon$ <= 3 2 × r) / 2 = 34.5 = 4.8 × $\varepsilon$ <= 9 × $\varepsilon$ N Rd = A × fy / $\gamma$ M0 = = 0.012 n compression 3.1.3 1 = 2800 mm E × ly / Lor,y <sup>2</sup> = 22 fy / Nor,y) = 0.503	2 (sheet 1 of 3) × f <sub>y</sub> ) - (t <sub>f</sub> + r)] / c 96 × ε / (13 × α - mm Class 1 558.8 kN resistance exc 05.5 kN	eeds desig	ss 1 ction is clas
Classification of cross sect Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force Design resistance of section - Slenderness ratio for y-y ax Critical buckling length Critical buckling force Slenderness ratio for buckling Check y-y axis flexural buck Buckling curve - Table 6.2 Imperfection factor - Table 6.7	ions - Section 4 subject to ben 2 (sheet 2 of 3) on 6.2.4 - eq 6.10 is flexural buck g - eq 6.50 kling resistance	5.5 $\varepsilon = \sqrt{[235 N]}$ ding and compre $c = d = 121$ $\alpha = min([h]$ $c / tw = 27.7$ $c = (b - tw - c) / tr = 4.5 = 0$ $NEd = 6.6 k$ $N_{c,Rd} = N_{pl,F}$ $NEd / N_{c,Rd} = 0$ $PASS - Desig$ kling - Section 6.3 $L_{cr,y} = L_{m1,s}$ $N_{cr,y} = \pi^{2} \times \overline{\lambda}_{y} = \sqrt{(A \times a)}$ e - Section 6.3.1.1 $a$ $\alpha_{y} = 0.21$ $\varphi_{y} = 0.5 \times (a)$	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × tw 1 = 29.3 × $\varepsilon$ <= 3 2 × r) / 2 = 34.5 = 4.8 × $\varepsilon$ <= 9 × $\varepsilon$ N Rd = A × fy / $\gamma$ M0 = = 0.012 n compression 3.1.3 1 = 2800 mm E × ly / Lcr.y <sup>2</sup> = 22 fy / Ncr.y) = 0.503 1 + $\alpha$ y × ( $\overline{\lambda}$ y - 0.2	2 (sheet 1 of 3) × fy) - (tr + r)] / c 96 × $\varepsilon$ / (13 × $\alpha$ - mm Class 1 558.8 kN resistance exc 05.5 kN 2) + $\overline{\lambda}y^2$ ) = 0.65	eeds desig	ss 1 ction is clas
Classification of cross sect Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force Design resistance of section - Slenderness ratio for y-y ax Critical buckling length Critical buckling force Slenderness ratio for buckling Check y-y axis flexural buck Buckling curve - Table 6.2 Imperfection factor - Table 6.2	ions - Section 4 subject to ben 2 (sheet 2 of 3) on 6.2.4 - eq 6.10 is flexural buck g - eq 6.50 kling resistance 1 tion factor 6.49	5.5 $\varepsilon = \sqrt{[235 N]}$ ding and compre $c = d = 121$ $\alpha = min([h])$ $c / tw = 27.7$ $c = (b - tw - c) / tr = 4.5 = 0$ $N_{Ed} = 6.6 k$ $N_{c,Rd} = N_{PI,F}$ $N_{Ed} / N_{c,Rd} = 0$ $PASS - Desig$ kling - Section 6.3 $L_{cr,y} = L_{m1_s}$ $N_{cr,y} = \pi^2 \times \lambda_y = \sqrt{(A \times a)}$ $a$ $\alpha_y = 0.21$ $\varphi_y = 0.5 \times (\lambda_y = min(1))$	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × tw 1 = 29.3 × $\varepsilon$ <= 3 2 × r) / 2 = 34.5 = 4.8 × $\varepsilon$ <= 9 × $\varepsilon$ N Rd = A × fy / $\gamma$ M0 = = 0.012 n compression 3.1.3 1 = 2800 mm E × ly / Lor,y <sup>2</sup> = 22 fy / Nor,y) = 0.503	2 (sheet 1 of 3) × fy) - (tr + r)] / c 96 × $\varepsilon$ / (13 × $\alpha$ - mm Class 1 558.8 kN resistance exc 05.5 kN 2) + $\overline{\lambda}y^2$ ) = 0.65 2) + $\overline{\lambda}y^2$ ) = 0.65	eeds desig	ss 1 ction is clas

<b>E</b>	Project	Brook	House		Job no. JF	1524		
Structural Calculation Ltd Mobbs Wood Farm	Calcs for	Porto	l Frame		Start page no./F	Revision 33		
Ansty								
CV7 9JN	Calcs by AM	Calcs date 10/01/2020	Checked by JP	Checked date 10/01/2020	Approved by JP	Approved date 10/01/2020		
		PASS - D	esign bucklir	ng resistance exc	eeds design	compression		
Slenderness ratio for z-z axi	s flexural buck	-						
Critical buckling length			s1_seg1 = <b>2800</b> n					
Critical buckling force	0.50		$E \times I_z / L_{cr,z}^2 =$					
Slenderness ratio for buckling	- eq 6.50	$\lambda_z = \sqrt{A \times A}$	t fy / Ncr,z) = <b>1.5</b>	35				
Check z-z axis flexural buck	ling resistance		l					
Buckling curve - Table 6.2		b						
Imperfection factor - Table 6.1		αz = <b>0.34</b>						
Buckling reduction determinat				$(0.2) + \overline{\lambda}z^2 = 1.90$	)4			
Buckling reduction factor - eq		<i>,</i> ,,		$\bar{\lambda}z^{2}$ )), 1) = <b>0.33</b>				
Design buckling resistance - e	q 6.47		$\times$ A $\times$ fy / $\gamma$ M1 =	184.3 kN				
		Ned / Nb,z,Rd <b>PASS - D</b>		ng resistance exc	ceeds design	compression		
Check torsional and torsion	al-flexural buc	kling - Section 6	3.1.4					
Torsional buckling length		-	s1_seg1_R = <b>2800</b>	mm				
Distance from shear centre to	centroid in y ax	is y <sub>0</sub> = <b>0.0</b> m	n					
Distance from shear centre to	centroid in z ax	$z axis z_0 = 0.0 mm$						
Radius of gyration		$i_0 = (i_y^2 + $	$i_0 = \sqrt{(i_y^2 + i_z^2)} = 67.4 \text{ mm}$					
Elastic critical torsional bucklin	ng force	$N_{cr,T} = 1 / i_0$	$N_{cr,T}$ = 1 / $i_0^2 \times (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2)$ = <b>905.6</b> kN					
Torsion factor		β⊤ = 1 - (y₀	$\beta \tau = 1 - (y_0 / i_0)^2 = 1$					
Elastic critical torsional-flexura	al buckling force	1						
No	$T_{,TF} = N_{cr,y} / (2 \times 1)$	$\beta T$ ) × [1 + N <sub>cr,T</sub> / N <sub>c</sub>	cr,y - √[(1 - Ncr,⊤	$/ N_{cr,y})^2 + 4 \times (y_0 / )^2$	io) <sup>2</sup> × N <sub>cr,T</sub> / N <sub>c</sub>	er,y]] = <b>905.6</b> kN		
Elastic critical buckling force			Icr,T, Ncr,TF) = 9					
Slenderness ratio for torsional	buckling - eq 6	.52 $\overline{\lambda}_T = \sqrt{[A \times ]}$	fy / Ncr] = <b>0.78</b>	6				
Design resistance for torsio	nal and torsior		ling - Section	6.3.1.1				
Buckling curve - Table 6.2		b						
Imperfection factor - Table 6.1			$ \begin{aligned} \alpha \tau &= 0.34 \\ \phi \tau &= 0.5 \times (1 + \alpha \tau \times (\overline{\lambda} \tau - 0.2) + \overline{\lambda} \tau^2) = 0.908 \\ \chi \tau &= \min(1 / (\phi \tau + \sqrt{(\phi \tau^2 - \overline{\lambda} \tau^2)}), 1) = 0.733 \end{aligned} $					
Buckling reduction determinat								
Buckling reduction factor - eq								
Design buckling resistance - e	eq 6.47		$\times$ A $\times$ fy / $\gamma$ M1 =	409.8 kN				
		NEd / Nb,T,R		a registeres ex	aada daaigu			
Check design at start of spa	n	7433 - L	esiyii bucklir	ig resistance exc	.eeus aesign	compression		
	<u></u>							
Check shear - Section 6.2.6			4 - 407		00			
Height of web			tf = 137 mm	η = <b>1.0</b>	UU			
		hw / tw = 30	.4 = 32.9 × ε /		realister	on he income		
Dooign choor force		\/ _: _ <b>/ 7</b>	٧N	Shear buckling	resistance c	an de ignored		
Design shear force		$V_{y,Ed} = 1.7$		(† ± 2) v r) v fr	h	8 mm <sup>2</sup>		
Shear area - cl 6.2.6(3)	2 6(2)	-		$(t_w + 2 \times r) \times t_f, \eta \times \eta \times \eta$	-	, 11111		
Design shear resistance - cl 6	.2.0(2)			√(3)) / γ <sub>M0</sub> = <b>129.8</b>				
		V <sub>y,Ed</sub> / V <sub>c,y,F</sub>		loar rosistanco o	vcaade daeir	an shaar force		
		PAS	S - Design sh	near resistance e	xceeds desig	n shear i		

	Project	Brook	House		Job no. JP	1524	
Structural Calculation Ltd	Calcs for	Portal	Frame		Start page no./F	Revision 34	
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CV7 9JN	Calcs by AM	Calcs date 10/01/2020	Checked by JP	Checked date 10/01/2020	Approved by JP	Approved date 10/01/202	
Check design at end of span	<u>l</u>						
Check shear - Section 6.2.6							
Height of web		$h_w = h - 2 \times$	t <sub>f</sub> = <b>137</b> mm	η = <b>1.0</b>	00		
		h <sub>w</sub> / t <sub>w</sub> = 30	.4 = 32.9 × ε /	η < 72 × ε / η			
				Shear buckling	resistance c	an be ignore	
Design shear force		V <sub>y,Ed</sub> = 1.7					
Shear area - cl 6.2.6(3)		-		tw + 2 × r) × tf, $\eta$ ×		1 mm <sup>2</sup>	
Design shear resistance - cl 6.	.2.6(2)			√(3)) / үмо = <b>129.8</b>	kN		
		V <sub>y,Ed</sub> / V <sub>c,y,R</sub>					
		PAS	S - Design sh	ear resistance e	xceeds desig	n shear for	
Check bending moment - Se	ction 6.2.5						
Design bending moment		$M_{y,Ed} = 4.9$	kNm				
Design bending resistance mo	oment - eq 6.13			/ γ <sub>M0</sub> = <b>33.9</b> kNm			
		M <sub>y,Ed</sub> / M <sub>c,y,Rd</sub> = 0.143 Design bending resistance moment exceeds design bending mome					
	PASS - I	Design bendi	ng resistance	moment exceed	s design ber	nding mome	
Slenderness ratio for lateral	torsional buckling						
Correction factor - For cantilev	rer beams	kc = <b>1</b>					
		$C_1 = 1 / k_c^2$	= 1				
Poissons ratio		v = 0.3 G = E / [2 × (1 + v)] = 80769 N/mm <sup>2</sup> L = 1.0 × Lm1_s1_seg1_B = 2800 mm Mcr = C1 × $\pi^2$ × E × Iz / L <sup>2</sup> × $\sqrt{(I_w / I_z + L^2 × G × I_t / (\pi^2 × E × I_z))}$ = 31.3					
Shear modulus							
Unrestrained effective length							
Elastic critical buckling momer	nt						
		kNm					
Slenderness ratio for lateral to	rsional buckling		$d_{v,y} \times f_y / M_{cr}) = f_{v,y}$	1.041			
Limiting slenderness ratio		$\overline{\lambda}_{LT,0} = 0.4$					
			$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0} - La$	teral torsional b	uckling cann	ot be ignore	
Check buckling resistance -	Section 6.3.2.1						
Buckling curve - Table 6.5		b					
-		αlt = <b>0.34</b>					
Imperfection factor - Table 6.3		uli - 0.34					
		β = <b>0.75</b>					
	tions	$\beta = 0.75$	[1 + αιτ × ( λιτ	- λιτ.ο) + β × λιτ	<sup>2</sup> ] = 1.016		
Correction factor for rolled sec LTB reduction determination fa	tions	$\beta = 0.75$ $\phi_{LT} = 0.5 \times$		- λιτ.ο) + β × λιτ - β × λιτ²)], 1, 1 /	-		
Correction factor for rolled sec LTB reduction determination fa	tions	β = <b>0.75</b> φ <sub>LT</sub> = 0.5 × χ <sub>LT</sub> = min(1	/[¢LT + √(¢LT <sup>2</sup>	, ,	λ <sub>LT<sup>2</sup></sub> ) = 0.674	)	
Correction factor for rolled sec LTB reduction determination fa LTB reduction factor - eq 6.57	tions actor	$\beta = 0.75$ $\phi_{LT} = 0.5 \times \chi_{LT} = min(1 - 1)$ f = min(1 - 1)	/[¢LT + √(¢LT <sup>2</sup>	- β × λ̃ιτ²)], 1, 1 / [1 - 2 × (λ̃ιτ - 0.8	λ <sub>LT<sup>2</sup></sub> ) = 0.674	)	
Correction factor for rolled sec LTB reduction determination fa LTB reduction factor - eq 6.57 Modification factor	tions actor - eq 6.58	$\beta = 0.75$ $\phi_{LT} = 0.5 \times \chi_{LT} = min(1 + $	/ [φιτ + √(φιτ² 0.5 × (1 - kc) × n(χιτ / f, 1, 1 /	- β × λ̃ιτ²)], 1, 1 / [1 - 2 × (λ̃ιτ - 0.8	λ <sub>LT<sup>2</sup></sub> ) = 0.674	)	
Correction factor for rolled sec LTB reduction determination fa LTB reduction factor - eq 6.57 Modification factor Modified LTB reduction factor	tions actor - eq 6.58	$\beta = 0.75$ $\phi_{LT} = 0.5 \times \chi_{LT} = min(1 + $	/ [φιτ + √(φιτ <sup>2</sup> ) 0.5 × (1 - kc) × n(χιτ / f, 1, 1 / <sup>c</sup> ,mod × Wply × fy	$-\beta \times \overline{\lambda} [\tau^{2}], 1, 1 / [1 - 2 \times (\overline{\lambda} [\tau - 0.8])]$ $\overline{\lambda} [\tau^{2}] = 0.674$	λ <sub>LT<sup>2</sup></sub> ) = 0.674	)	
Correction factor for rolled sec LTB reduction determination fa LTB reduction factor - eq 6.57 Modification factor Modified LTB reduction factor	tions actor - eq 6.58 oment - eq 6.55	$\beta = 0.75$ $\phi_{LT} = 0.5 \times \chi_{LT} = min(1 - \chi_{LT,mod} = min(1 - \chi_{LT,mod} = min(1 - \chi_{LT,mod} = min(1 - \chi_{LT}))$	/ $[\phi_{LT} + \sqrt{(\phi_{LT}^2 + $	$-\beta \times \overline{\lambda} [\tau^{2}], 1, 1 / [1 - 2 \times (\overline{\lambda} [\tau - 0.8])]$ $\overline{\lambda} [\tau^{2}] = 0.674$	$(\overline{\lambda}_{LT}^2) = 0.674$ $(3)^2], 1) = 1.000$		
Correction factor for rolled sec LTB reduction determination fa LTB reduction factor - eq 6.57 Modification factor Modified LTB reduction factor Design buckling resistance mo	tions actor - eq 6.58 oment - eq 6.55 <b>PASS - L</b>	$\beta = 0.75$ $\phi_{LT} = 0.5 \times \chi_{LT} = min(1 - \chi_{LT,mod} = min(1 - \chi_{LT,mod} = min(1 - \chi_{LT,mod} = min(1 - \chi_{LT}))$	/ $[\phi_{LT} + \sqrt{(\phi_{LT}^2 + $	- β × λ̃ <sub>L</sub> τ <sup>2</sup> )], 1, 1 / [1 - 2 × (λ̃ <sub>L</sub> τ - 0.8 λ̃ <sub>L</sub> τ <sup>2</sup> ) = <b>0.674</b> / γ <sub>M1</sub> = <b>22.9</b> kNm	$(\overline{\lambda}_{LT}^2) = 0.674$ $(3)^2], 1) = 1.000$		
Correction factor for rolled sec LTB reduction determination fa LTB reduction factor - eq 6.57 Modification factor Modified LTB reduction factor	tions actor - eq 6.58 oment - eq 6.55 <i>PASS - L</i> ce - Section 6.2.9	β = 0.75 $φ_{LT} = 0.5 ×$ $χ_{LT} = min(1 - 1)$ $χ_{LT,mod} = mi$ $M_{b,y,Rd} = χ_{LT}$ $M_{y,Ed} / M_{b,y,f}$ <b>Design bucklin</b> 4 Ny,lim = min(1 - 1)	$/ \left[ \phi_{LT} + \sqrt{(\phi_{LT}^2 + $	- β × λ̃ <sub>L</sub> τ <sup>2</sup> )], 1, 1 / [1 - 2 × (λ̃ <sub>L</sub> τ - 0.8 λ̃ <sub>L</sub> τ <sup>2</sup> ) = 0.674 / γ <sub>M1</sub> = 22.9 kNm	λ <sub>LT<sup>2</sup>) = 0.674 (3)<sup>2</sup>], 1) = 1.000 (5 design ber</sub>	nding mome	
Correction factor for rolled sec LTB reduction determination fa LTB reduction factor - eq 6.57 Modification factor Modified LTB reduction factor Design buckling resistance mod	tions actor - eq 6.58 oment - eq 6.55 <i>PASS - L</i> <b>ce - Section 6.2.9</b> - eq.6.33 & eq.6.3	$\beta = 0.75 \\ \phi_{LT} = 0.5 \times \\ \chi_{LT} = min(1 - 1) \\ \chi_{LT,mod} = min(1 - 1) \\ \chi_{LT,mod} = min(1 - 1) \\ M_{b,y,Rd} = \chi_{LT} \\ M_{y,Rd} = \chi_{LT} \\ M_{y,Rd} = M_{y,Rd} = M_{y,Rd} \\ M_{y,Rd} \\ M_{y,Rd} = M_{y,Rd} \\ M_{y,Rd} \\ M_{y,Rd} = M_{y,Rd} \\ M_{y,Rd}$	$/ \left[ \phi_{LT} + \sqrt{(\phi_{LT}^2 + $	$\beta \times \overline{\lambda}_{L}\tau^{2}), 1, 1 / [1 - 2 \times (\overline{\lambda}_{L}\tau - 0.8]), 1, 1 / [1 - 2 \times (\overline{\lambda}_{L}\tau - 0.8]), 1, 1 / [1 - 2 \times (\overline{\lambda}_{L}\tau^{2})] = 0.674$ $/ \gamma_{M1} = 22.9 \text{ kNm}$ <i>moment exceed</i> $0.5 \times h_{W} \times t_{W} \times f_{y} / \gamma$	<sup>1</sup> / <sub>λLT<sup>2</sup></sub> ) = 0.674 <sup>3</sup> / <sub>λLT<sup>2</sup></sub> ) = 1.000 <sup>2</sup> / <sub>λLT<sup>2</sup></sub> ) = 1.000 <sup>3</sup> / <sub>λLT<sup>2</sup></sub> ) = 1.000 <sup>3</sup> / <sub>λLT<sup>2</sup></sub> ) = 84.8 kN <sup>3</sup> / <sub>λLT<sup>2</sup></sub> ) = 84.8 kN	nding mome	
Correction factor for rolled sec LTB reduction determination fa LTB reduction factor - eq 6.57 Modification factor Modified LTB reduction factor Design buckling resistance mod <b>Check bending and axial for</b> Bending and axial force check <b>Allowance need not be m</b>	tions actor - eq 6.58 oment - eq 6.55 <b>PASS - L</b> <b>ce - Section 6.2.9</b> - eq.6.33 & eq.6.3 <b>ade for the effect</b>	$\beta = 0.75$ $\phi_{LT} = 0.5 \times$ $\chi_{LT} = min(1 - 1)$ $\chi_{LT,mod} = mi$ $M_{b,y,Rd} = \chi_{LT}$ $M_{y,Ed} / M_{b,y,f}$ $Design bucklin$ $4 N_{y,lim} = min(1 - 1)$ $N_{Ed} / N_{y,lim} = 0$ of the axial for	$/ \left[ \phi_{LT} + \sqrt{(\phi_{LT}^2 + $	$\beta \times \overline{\lambda}_{L}\tau^{2}), 1, 1 / [1 - 2 \times (\overline{\lambda}_{L}\tau - 0.8]), 1, 1 / [1 - 2 \times (\overline{\lambda}_{L}\tau - 0.8]), 1, 1 / [1 - 2 \times (\overline{\lambda}_{L}\tau^{2})] = 0.674$ $/ \gamma_{M1} = 22.9 \text{ kNm}$ <i>moment exceed</i> $0.5 \times h_{W} \times t_{W} \times f_{y} / \gamma$	<sup>1</sup> / <sub>λLT<sup>2</sup></sub> ) = 0.674 <sup>3</sup> / <sub>λLT<sup>2</sup></sub> ) = 1.000 <sup>2</sup> / <sub>λLT<sup>2</sup></sub> ) = 1.000 <sup>3</sup> / <sub>λLT<sup>2</sup></sub> ) = 1.000 <sup>3</sup> / <sub>λLT<sup>2</sup></sub> ) = 84.8 kN <sup>3</sup> / <sub>λLT<sup>2</sup></sub> ) = 84.8 kN	nding mome	
Correction factor for rolled sec LTB reduction determination fa LTB reduction factor - eq 6.57 Modification factor Modified LTB reduction factor Design buckling resistance mo <b>Check bending and axial for</b> Bending and axial force check	tions actor - eq 6.58 pment - eq 6.55 <b>PASS - L</b> <b>ce - Section 6.2.9</b> - eq.6.33 & eq.6.3 ade for the effect ad compression -	$\beta = 0.75$ $\phi_{LT} = 0.5 \times$ $\chi_{LT} = min(1 - 1)$ $\chi_{LT,mod} = min$ $M_{b,y,Rd} = \chi_{LT}$ $M_{y,Ed} / M_{b,y,I}$ $A N_{y,Iim} = min(1 - 1)$ $N_{Ed} / N_{y,Iim} = min(1 - 1)$	$/ \left[ \phi_{LT} + \sqrt{(\phi_{LT}^2 + $	- $\beta \times \overline{\lambda}_{L}\tau^{2}$ ), 1, 1 / [1 - 2 × ( $\overline{\lambda}_{L}\tau$ - 0.8 $\overline{\lambda}_{L}\tau^{2}$ ) = 0.674 / γ <sub>M1</sub> = 22.9 kNm moment exceed 0.5 × h <sub>w</sub> × t <sub>w</sub> × f <sub>y</sub> / γ	<sup>1</sup> / <sub>λLT<sup>2</sup></sub> ) = 0.674 <sup>3</sup> / <sub>λLT<sup>2</sup></sub> ) = 1.000 <sup>2</sup> / <sub>λLT<sup>2</sup></sub> ) = 1.000 <sup>3</sup> / <sub>λLT<sup>2</sup></sub> ) = 1.000 <sup>3</sup> / <sub>λLT<sup>2</sup></sub> ) = 84.8 kN <sup>3</sup> / <sub>λLT<sup>2</sup></sub> ) = 84.8 kN	nding mome	

	Project	Brook	House		Job no. JF	1524
Structural Calculation Ltd Mobbs Wood Farm	Calcs for	Portal	Frame		Start page no./F	Revision 35
Ansty CV7 9JN	Calcs by	Calcs date	Checked by	Checked date	Approved by Approved da	
	AM	10/01/2020	JP	10/01/2020	JP	10/01/20
		C <sub>my</sub> = max(	0.6 + 0.4 × ψ <sub>γ</sub>	) = 0.600		
			n / -4.856 kNr	,		
		αLT = -2.42	8 kNm / -4.856	6 kNm = <b>0.500</b>		
		C <sub>mLT</sub> = max	$x(0.6 + 0.4 \times \psi)$	LT) = <b>0.600</b>		
Interaction factors k <sub>ij</sub> for me	mbers suscep	tible to torsional	deformations	s - Table B.2		
Characteristic moment resista			<sub>y</sub> × f <sub>y</sub> = <b>33.9</b> ki			
Characteristic moment resistance		Mz,Rk = Wpl.	z × fy = <b>8.6</b> kN	m		
Characteristic resistance to no	ormal force	$N_{Rk} = A \times f_{N}$	, = <b>558.8</b> kN			
Interaction factors		$k_{yy} = C_{my} \times$	$(1 + \min(\overline{\lambda}_y -$	0.2, 0.8) × NEd / ()	(y × <b>N</b> rk / γm1))	= 0.602
				× NEd / ((CmLT - 0.2		
Interaction formulae - eq 6.61	& eq 6.62			$\times$ My,Ed / ( $\chi$ LT $\times$ My,		
				× $M_{y,Ed}$ / ( $\chi_{LT}$ × $M_{y,Fd}$ ),		
				ding and compre		
Consider Combination 2 - 1	.0G + 1.0Q + 1.	0RQ (Service)				
Check design 2217 mm alor	ng span					
Check y-y axis deflection - S Maximum deflection	Section 7.2.1	δ <sub>y</sub> = <b>0.2</b> mn	n			
Allowable deflection			Lm1_s1 / 180 =	15.6 mm		
		δy / δy,Allowable –		13.0 mm		
		Oy 7 Oy,Allowad		wable deflection	exceeds des	ian deflect
Defter design						.g
Rafter design						
Section details						
Section type	1		x16 (BS4-1)			
Steel grade - EN 10025-2:200 Nominal thickness of element		S275	t <sub>f</sub> , t <sub>w</sub> ) = <b>7.7</b> mn	2		
Nominal yield strength		fy = 275 N/i		11		
Nominal ultimate tensile stren	ath	$f_{\rm u} = 410 \text{ N/r}$				
Modulus of elasticity	J	E = 210000				
N						
			9x16 (BS4-1)			
↑			epth, h, 152.4 mm eadth, b, 88.7 mm			
			ection, Mass, 16 kg/m ckness, t <sub>f</sub> , 7.7 mm			
		Web thick	ness, t <sub>w</sub> , 4.5 mm			
		Area of se	s, r, 7.6 mm ction, A, 2032 mm <sup>2</sup>			
4			gyration about y-axis, i , gyration about z-axis, i ,			
25			tion modulus about y-a tion modulus about z-a			
		.5 Plastic sec	tion modulus about y-a	xis, W <sub>pl.y</sub> , 123256 mm <sup>3</sup>		
152.	→ -4		tion modulus about z-a	xis, W <sub>pl.z</sub> , 31180 mm³ axis, I <sub>v</sub> , 8342621 mm <sup>4</sup>		
152.	→ -4		Unient of alea about y-a			
152,		Second m	oment of area about z-a			
+-7.7		Second m				
+ →   + 7.7		Second m				
+ - - - - - - - - - - - - -	→	Second m				
+ → 152.		Second m				

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Structural Calculation Ltd Mobbs Wood Farm	Calcs for	Portal	Frame		Start page no./	Revision 36	
Ansty CV7 9JN	Calcs by AM	Calcs date 10/01/2020	Checked by JP	Checked date 10/01/2020	Approved by JP	Approved 10/01/2	
<b>Lateral restraint</b> Both flanges have lateral restr	raint at support	s only					
Consider Combination 1 - 1.	.35G + 1.5Q + 1	I.5RQ (Strength)					
Classification of cross secti	ons - Section	5.5					
		ε = √[235 Ν	$I/mm^2 / f_y] = 0.$	92			
Internal compression parts	subject to ben			5.2 (sheet 1 of 3)	)		
Width of section		c = d = <b>121</b>					
				$t_{w} \times f_{y}$ ) - ( $t_{f} + r$ )] / (	-		
		c / t <sub>w</sub> = 27.1	= 29.3 × ε <=	= 396 × ε / (13 × α	- 1) Clas	s 1	
Outstand flanges - Table 5.2	(sheet 2 of 3)						
Width of section		``	2 × r) / 2 = <b>34</b>				
		c / t <sub>f</sub> = 4.5 =	= 4.8 × ε <= 9	×ε Class 1			
					Sec	tion is cla	
Check compression - Section	on 6.2.4						
Design compression force		N <sub>Ed</sub> = <b>4.5</b> k					
Design resistance of section -	eq 6.10		$a = A \times f_y / \gamma Mo$	= <b>558.8</b> kN			
		NEd / Nc,Rd :		n raciatanaa awa	ande desier		
Olem de merce se the f	a flan sait i	-	-	on resistance exc	.eeus aesign	compres	
Slenderness ratio for y-y axi	is flexural buc	-					
Critical buckling length Critical buckling force			1 = 2766  mm	2260 1 KN			
Slenderness ratio for buckling	- eg 6 50	$N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 = 2260.1 \text{ kN}$ $\overline{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} = 0.497$					
-							
Check y-y axis flexural buck Buckling curve - Table 6.2	ling resistance						
Imperfection factor - Table 6.1	l	a α <sub>y</sub> = <b>0.21</b>					
Buckling reduction determinat		$\alpha_y = 0.21$ $\phi_y = 0.5 \times (1 + \alpha_y \times (\overline{\lambda}_y - 0.2) + \overline{\lambda}_y^2) = 0.655$					
Buckling reduction factor - eq				$(\overline{\lambda}_{y}^{2}), 1) = 0.925$	-		
Design buckling resistance - e			× A × f <sub>y</sub> / γ <sub>M1</sub> =				
		N <sub>Ed</sub> / N <sub>b,y,Rd</sub>					
				ng resistance exc	ceeds design	compres	
Slenderness ratio for z-z axi	is flexural buc	kling - Section 6.3	3.1.3				
Critical buckling length		$L_{cr,z} = L_{m2_s}$	1_seg1 <b>= 2766</b> n	nm			
Critical buckling force		$N_{cr,z} = \pi^2 \times$	$N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z^2} = 243.1 \text{ kN}$				
Slenderness ratio for buckling	- eq 6.50	$\overline{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} = 1.516$					
Check z-z axis flexural buck Buckling curve - Table 6.2	ling resistanc	e - Section 6.3.1.1 b					
Imperfection factor - Table 6.1		αz = <b>0.34</b>					
Buckling reduction determinat		$\phi_z = 0.5 \times ($	1 + $\alpha_z \times (\overline{\lambda}_z -$	$(0.2) + \overline{\lambda}z^2 = 1.87$	73		
Buckling reduction factor - eq			-	$\bar{\lambda}z^2$ )), 1) = <b>0.336</b>			
Design buckling resistance - e			× A × f <sub>y</sub> / үм1 =				
		NEd / Nb,z,Rd					

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Structural Calculation Ltd Mobbs Wood Farm	Calcs for	Porta	l Frame		Start page no./F	Revision 37
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		10/01/2020	01	10/01/2020	01	10/01/2
Check torsional and torsional Torsional buckling length	al-flexural buckl	-	3.1.4 a1_seg1_R = 2766	mm		
Distance from shear centre to	controid in v ovia					
Distance from shear centre to	-	-				
Radius of gyration						
Elastic critical torsional bucklin	( -	,	$^{2} \times E \times I_{w} / L_{cr,T}^{2}) =$	912.3 kN		
Torsion factor	ig loloc	βτ = 1 - (yo	•			
Elastic critical torsional-flexura	l buckling force	pr i (jo	, 10)			
	•	·) × [1 + Nor + / No	xx - √[(1 - Nor⊤	/ N <sub>cr,y</sub> ) <sup>2</sup> + 4 × (y <sub>0</sub> /	$i_0)^2 \times N_{cr} \tau / N_c$	
Elastic critical buckling force			lcr,T, Ncr,TF) = 9			
Slenderness ratio for torsional	buckling - eg 6 F		fy / Ncr] = <b>0.78</b>			
		-	-			
Design resistance for torsion	hai and torsiona		ing - Section	0.3.1.1		
Buckling curve - Table 6.2		b ar <b>- 0 24</b>				
Imperfection factor - Table 6.1	on foot	ατ = <b>0.34</b>	4		0.5	
Buckling reduction determinati			•	$(0.2) + \overline{\lambda}\tau^2 = 0.9$	00	
Buckling reduction factor - eq 6				$\bar{\lambda}_{T^2}$ ), 1) = <b>0.735</b>		
Design buckling resistance - e	q 6.4 <i>1</i>		$\times$ A $\times$ fy / $\gamma$ M1 =	410.8 kN		
		NEd / Nb,T,Rd	i = 0.011			
		PASS - D	esign bucklir	ig resistance exc	eeds design	compress
Check design at start of spa	<u>n</u>	PASS - D	esign bucklir	ig resistance exc	ceeds design	compress
<u>Check design at start of spa</u> Check shear - Section 6.2.6	<u>n</u>	PASS - D	esign bucklir	ng resistance exc	ceeds design	compress
	<u>n</u>		e <b>sign bucklir</b> tf = <b>137</b> mm	ng resistance exc η = <b>1.0</b>	-	compress
Check shear - Section 6.2.6	<u>n</u>	hw = h - 2 >	-	η = <b>1.0</b>	-	compress
Check shear - Section 6.2.6	<u>n</u>	hw = h - 2 >	< t <sub>f</sub> = <b>137</b> mm	η = <b>1.0</b>	00	
Check shear - Section 6.2.6	<u>n</u>	hw = h - 2 >	c t <sub>f</sub> = <b>137</b> mm .4 = 32.9 × ε /	η = <b>1.0</b> η < 72 × ε / η	00	
Check shear - Section 6.2.6 Height of web	<u>n</u>	hw = h - 2 → hw / tw = 30 Vy,Ed = <b>4.4</b>	α tr = <b>137</b> mm .4 = 32.9 × ε / kN	η = <b>1.0</b> η < 72 × ε / η	00 resistance c	an be igno
<b>Check shear - Section 6.2.6</b> Height of web Design shear force	_	hw = h - 2 > hw / tw = 30 Vy,Ed = <b>4.4</b> Av = max(A	<ul> <li>c t<sub>f</sub> = <b>137</b> mm</li> <li>.4 = 32.9 × ε /</li> <li>kN</li> <li>A - 2 × b × t<sub>f</sub> + (</li> </ul>	η = <b>1.0</b> η < 72 × ε / η Shear buckling	00 <i>resistance c</i> h <sub>w</sub> × t <sub>w</sub> ) = 818	an be igno
<b>Check shear - Section 6.2.6</b> Height of web Design shear force Shear area - cl 6.2.6(3)	_	$h_{w} = h - 2 >$ $h_{w} / t_{w} = 30$ $V_{y,Ed} = 4.4$ $A_{v} = max(A$ $V_{c,y,Rd} = V_{pl}$ $V_{y,Ed} / V_{c,y,R}$	k tr = 137 mm $.4 = 32.9 \times \epsilon / kN$ $A - 2 \times b \times tr + (k)$ $A - 2 \times b \times tr$ $A - 2 \times b \times tr$ $A - 2 \times b \times tr$ $A - 2 \times$	η = 1.00 η < 72 × ε / η Shear buckling t <sub>w</sub> + 2 × r) × t <sub>f</sub> , η × √(3)) / γ <sub>M0</sub> = 129.8	00 <i>resistance c</i> h <sub>w</sub> × t <sub>w</sub> ) = 818 8 kN	<b>an be igno</b> 8 mm²
<b>Check shear - Section 6.2.6</b> Height of web Design shear force Shear area - cl 6.2.6(3)	_	$h_{w} = h - 2 >$ $h_{w} / t_{w} = 30$ $V_{y,Ed} = 4.4$ $A_{v} = max(A$ $V_{c,y,Rd} = V_{pl}$ $V_{y,Ed} / V_{c,y,R}$	k tr = 137 mm $.4 = 32.9 \times \epsilon / kN$ $A - 2 \times b \times tr + (k)$ $A - 2 \times b \times tr$ $A - 2 \times b \times tr$ $A - 2 \times b \times tr$ $A - 2 \times$	η = <b>1.0</b> η η < 72 × ε / η <b>Shear buckling</b> t <sub>w</sub> + 2 × r) × tr, η ×	00 <i>resistance c</i> h <sub>w</sub> × t <sub>w</sub> ) = 818 8 kN	<b>an be igno</b> 8 mm²
<b>Check shear - Section 6.2.6</b> Height of web Design shear force Shear area - cl 6.2.6(3)	2.6(2)	$h_{w} = h - 2 >$ $h_{w} / t_{w} = 30$ $V_{y,Ed} = 4.4$ $A_{v} = max(A$ $V_{c,y,Rd} = V_{pl}$ $V_{y,Ed} / V_{c,y,R}$	k tr = 137 mm $.4 = 32.9 \times \epsilon / kN$ $A - 2 \times b \times tr + (k)$ $A - 2 \times b \times tr$ $A - 2 \times b \times tr$ $A - 2 \times b \times tr$ $A - 2 \times$	η = 1.00 η < 72 × ε / η Shear buckling t <sub>w</sub> + 2 × r) × t <sub>f</sub> , η × √(3)) / γ <sub>M0</sub> = 129.8	00 <i>resistance c</i> h <sub>w</sub> × t <sub>w</sub> ) = 818 8 kN	<b>an be igno</b> 8 mm²
<b>Check shear - Section 6.2.6</b> Height of web Design shear force Shear area - cl 6.2.6(3) Design shear resistance - cl 6.	2.6(2)	$h_{w} = h - 2 >$ $h_{w} / t_{w} = 30$ $V_{y,Ed} = 4.4$ $A_{v} = max(A$ $V_{c,y,Rd} = V_{pl}$ $V_{y,Ed} / V_{c,y,R}$	k tr = 137 mm $.4 = 32.9 \times \epsilon / kN$ $A - 2 \times b \times tr + (k)$ $A - 2 \times b \times tr + k$ $A - 2 \times b \times tr + k$ A	η = 1.00 η < 72 × ε / η Shear buckling t <sub>w</sub> + 2 × r) × t <sub>f</sub> , η × √(3)) / γ <sub>M0</sub> = 129.8	00 <i>resistance c</i> h <sub>w</sub> × t <sub>w</sub> ) = 818 8 kN	<b>an be igno</b> 8 mm²
Check shear - Section 6.2.6 Height of web Design shear force Shear area - cl 6.2.6(3) Design shear resistance - cl 6. Check bending moment - Se	– 2.6(2) ction 6.2.5	$h_w = h - 2 >$ $h_w / t_w = 30$ $V_{y,Ed} = 4.4$ $A_v = max(A$ $V_{c,y,Rd} = V_{pl}$ $V_{y,Ed} / V_{c,y,R}$ <i>PAS</i> $M_{y,Ed} = 4.9$	x tr = 137 mm .4 = 32.9 × ε / kN $X - 2 × b × tr + ( xy,Rd = A_V × (fy / d) = 0.034S - Design structure for the second se$	η = 1.00 η < 72 × ε / η Shear buckling t <sub>w</sub> + 2 × r) × t <sub>f</sub> , η × √(3)) / γ <sub>M0</sub> = 129.8	00 resistance c h <sub>w</sub> × t <sub>w</sub> ) = 818 kN xceeds desig	<b>an be igno</b> 8 mm²
Check shear - Section 6.2.6 Height of web Design shear force Shear area - cl 6.2.6(3) Design shear resistance - cl 6. Check bending moment - Se Design bending moment	– 2.6(2) ction 6.2.5	$h_w = h - 2 >$ $h_w / t_w = 30$ $V_{y,Ed} = 4.4$ $A_v = max(A$ $V_{c,y,Rd} = V_{pl}$ $V_{y,Ed} / V_{c,y,R}$ <i>PAS</i> $M_{y,Ed} = 4.9$	x tr = 137 mm .4 = 32.9 × ε / kN x - 2 × b × tr + ( y,Rd = Av × (fy / d) = 0.034 $x - Design shows kNm bly,Rd = W_{pl,y} × f$	η = 1.00 η < 72 × ε / η Shear buckling $t_w + 2 × r) × t_f, η ×$ $\sqrt{(3)} / γ_{M0} = 129.8$ the ar resistance e.	00 resistance c h <sub>w</sub> × t <sub>w</sub> ) = 818 kN xceeds desig	<b>an be igno</b> 8 mm²
Check shear - Section 6.2.6 Height of web Design shear force Shear area - cl 6.2.6(3) Design shear resistance - cl 6. Check bending moment - Se Design bending moment	– 2.6(2) <b>ction 6.2.5</b> ment - eq 6.13	hw = h - 2 > hw / tw = 30 Vy,Ed = <b>4.4</b> Av = max(A Vc,y,Rd = Vpl Vy,Ed / Vc,y,R <b>PAS</b> My,Ed = <b>4.9</b> Mc,y,Rd = Mp My,Ed / Mc,y,	k tr = 137 mm $.4 = 32.9 \times \epsilon / kN$ $A - 2 \times b \times tr + (k)$ $A - 2 \times tr + (k)$ $A - 2 \times tr + (k)$ $A - 2 \times tr + (k)$ A	η = 1.00 η < 72 × ε / η Shear buckling $t_w + 2 × r) × t_f, η ×$ $\sqrt{(3)} / γ_{M0} = 129.8$ the ar resistance e.	00 resistance c h <sub>w</sub> × t <sub>w</sub> ) = 818 8 kN xceeds desig	an be igno 3 mm² 9n shear fo
Check shear - Section 6.2.6 Height of web Design shear force Shear area - cl 6.2.6(3) Design shear resistance - cl 6. Check bending moment - Se Design bending moment	– 2.6(2) ction 6.2.5 ment - eq 6.13 <i>PASS</i>	hw = h - 2 > hw / tw = 30 Vy,Ed = 4.4 Av = max(A Vc,y,Rd = Vpl Vy,Ed / Vc,y,R <b>PAS</b> My,Ed = 4.9 Mc,y,Rd = Mp My,Ed / Mc,y,	k tr = 137 mm $.4 = 32.9 \times \epsilon / kN$ $A - 2 \times b \times tr + (k)$ $A - 2 \times tr + (k)$ $A - 2 \times tr + (k)$ $A - 2 \times tr + (k)$ A	η = 1.0 η < 72 × ε / η Shear buckling $t_w + 2 × r) × t_r, η ×$ $\sqrt{(3)} / γ_{M0} = 129.8$ pear resistance e. $q / γ_{M0} = 33.9$ kNm	00 resistance c h <sub>w</sub> × t <sub>w</sub> ) = 818 8 kN xceeds desig	an be igno 3 mm² 9n shear fo
Check shear - Section 6.2.6 Height of web Design shear force Shear area - cl 6.2.6(3) Design shear resistance - cl 6. Check bending moment - Se Design bending moment Design bending resistance mo	– 2.6(2) ction 6.2.5 ment - eq 6.13 <i>PASS</i> torsional buckli	hw = h - 2 > hw / tw = 30 Vy,Ed = 4.4 Av = max(A Vc,y,Rd = Vpl Vy,Ed / Vc,y,R <b>PAS</b> My,Ed = 4.9 Mc,y,Rd = Mp My,Ed / Mc,y,	k tr = 137 mm $.4 = 32.9 \times \epsilon / kN$ $A - 2 \times b \times tr + (k)$ $A - 2 \times tr + (k)$ $A - 2 \times tr + (k)$ $A - 2 \times tr + (k)$ A	η = 1.0 η < 72 × ε / η Shear buckling $t_w + 2 × r) × t_r, η ×$ $\sqrt{(3)} / γ_{M0} = 129.8$ pear resistance e. $q / γ_{M0} = 33.9$ kNm	00 resistance c h <sub>w</sub> × t <sub>w</sub> ) = 818 8 kN xceeds desig	an be igno 3 mm² 9n shear fo
Check shear - Section 6.2.6 Height of web Design shear force Shear area - cl 6.2.6(3) Design shear resistance - cl 6. Check bending moment - Se Design bending moment Design bending resistance mo Slenderness ratio for lateral	– 2.6(2) ction 6.2.5 ment - eq 6.13 <i>PASS</i> torsional buckli	hw = h - 2 > hw / tw = 30 Vy,Ed = 4.4 Av = max(A Vc,y,Rd = Vpl Vy,Ed / Vc,y,R <b>PAS</b> My,Ed = 4.9 Mc,y,Rd = Mp My,Ed / Mc,y, - Design bendi	$k tr = 137 mm$ $.4 = 32.9 \times \epsilon /$ $kN$ $A - 2 \times b \times tr + (i)$ $A - 2 \times tr + $	η = 1.0 η < 72 × ε / η Shear buckling $t_w + 2 × r) × t_r, η ×$ $\sqrt{(3)} / γ_{M0} = 129.8$ pear resistance e. $q / γ_{M0} = 33.9$ kNm	00 resistance c h <sub>w</sub> × t <sub>w</sub> ) = 818 8 kN xceeds desig	an be igno 3 mm² 9n shear fo
Check shear - Section 6.2.6 Height of web Design shear force Shear area - cl 6.2.6(3) Design shear resistance - cl 6. Check bending moment - Se Design bending moment Design bending resistance mo Slenderness ratio for lateral	– 2.6(2) ction 6.2.5 ment - eq 6.13 <i>PASS</i> torsional buckli	hw = h - 2 > hw / tw = 30 Vy,Ed = 4.4 Av = max(A Vc,y,Rd = Vpl Vy,Ed / Vc,y,R PAS My,Ed = 4.9 Mc,y,Rd = Mp My,Ed / Mc,y, - Design bendition kc = 1	$k tr = 137 mm$ $.4 = 32.9 \times \epsilon /$ $kN$ $A - 2 \times b \times tr + (i)$ $A - 2 \times tr + $	η = 1.0 η < 72 × ε / η Shear buckling $t_w + 2 × r) × t_r, η ×$ $\sqrt{(3)} / γ_{M0} = 129.8$ pear resistance e. $q / γ_{M0} = 33.9$ kNm	00 resistance c h <sub>w</sub> × t <sub>w</sub> ) = 818 8 kN xceeds desig	an be igno 3 mm² 9n shear fo
Check shear - Section 6.2.6 Height of web Design shear force Shear area - cl 6.2.6(3) Design shear resistance - cl 6. Check bending moment - Se Design bending moment Design bending resistance mo Slenderness ratio for lateral Correction factor - For cantilev	– 2.6(2) ction 6.2.5 ment - eq 6.13 <i>PASS</i> torsional buckli	$h_{w} = h - 2 >$ $h_{w} / t_{w} = 30$ $V_{y,Ed} = 4.4$ $A_{v} = max(A$ $V_{c,y,Rd} = V_{pl}$ $V_{y,Ed} / V_{c,y,R}$ $PAS$ $M_{y,Ed} = 4.9$ $M_{c,y,Rd} = M_{f}$ $M_{y,Ed} / M_{c,y}$ - Design bendixing $k_{c} = 1$ $C_{1} = 1 / k_{c}^{2}$ $v = 0.3$	$k tr = 137 mm$ $.4 = 32.9 \times \epsilon /$ $kN$ $A - 2 \times b \times tr + (i)$ $A - 2 \times tr + $	η = 1.00 η < 72 × ε / η Shear buckling $t_w + 2 × r) × t_f, η × 129.8$ rear resistance e. $q / γ_{M0} = 33.9$ kNm moment exceed	00 resistance c h <sub>w</sub> × t <sub>w</sub> ) = 818 8 kN xceeds desig	an be igno 3 mm² 9n shear fo
Check shear - Section 6.2.6 Height of web Design shear force Shear area - cl 6.2.6(3) Design shear resistance - cl 6. Check bending moment - Se Design bending moment Design bending resistance mo Slenderness ratio for lateral Correction factor - For cantilev Poissons ratio	– 2.6(2) ction 6.2.5 ment - eq 6.13 <i>PASS</i> torsional buckli	$h_{w} = h - 2 >$ $h_{w} / t_{w} = 30$ $V_{y,Ed} = 4.4$ $A_{v} = max(A$ $V_{c,y,Rd} = V_{pl}$ $V_{y,Ed} / V_{c,y,R}$ $PAS$ $M_{y,Ed} = 4.9$ $M_{c,y,Rd} = M_{F}$ $M_{y,Ed} / M_{c,y,l}$ - Design bending $k_{c} = 1$ $C_{1} = 1 / kc^{2}$ $v = 0.3$ $G = E / [2 > 100000000000000000000000000000000000$	k tr = 137 mm $A = 32.9 \times \epsilon / kN$ $A - 2 \times b \times tr + (k)$ $A - 2 \times tr + ($	η = 1.0 η < 72 × ε / η Shear buckling $t_w + 2 × r) × t_r, η ×$ $\sqrt{(3)} / γ_{M0} = 129.8$ pear resistance e. $q / γ_{M0} = 33.9$ kNm moment exceed 769 N/mm <sup>2</sup>	00 resistance c h <sub>w</sub> × t <sub>w</sub> ) = 818 8 kN xceeds desig	an be igno 3 mm² 9n shear fo
Check shear - Section 6.2.6 Height of web Design shear force Shear area - cl 6.2.6(3) Design shear resistance - cl 6. Check bending moment Design bending moment Design bending resistance mo Slenderness ratio for lateral Correction factor - For cantilev Poissons ratio Shear modulus	2.6(2) ction 6.2.5 ment - eq 6.13 <i>PASS</i> torsional buckli er beams	$h_{w} = h - 2 >$ $h_{w} / t_{w} = 30$ $V_{y,Ed} = 4.4$ $A_{v} = max(A$ $V_{c,y,Rd} = V_{pl}$ $V_{y,Ed} / V_{c,y,R}$ $PAS$ $M_{y,Ed} = 4.9$ $M_{c,y,Rd} = M_{f}$ $M_{y,Ed} / M_{c,y}$ $- Design bending$ $k_{c} = 1$ $C_{1} = 1 / k_{c}^{2}$ $v = 0.3$ $G = E / [2 >$ $L = 1.0 \times L$	$x tr = 137 mm$ $.4 = 32.9 \times \varepsilon /$ $kN$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$	η = 1.0 η < 72 × ε / η Shear buckling $t_w + 2 × r) × t_r, η ×$ $\sqrt{(3)} / γ_{M0} = 129.8$ pear resistance e. $q / γ_{M0} = 33.9$ kNm moment exceed 769 N/mm <sup>2</sup>	00 resistance c h <sub>w</sub> × t <sub>w</sub> ) = 818 kN xceeds desig	an be igno 3 mm² In shear fo
Check shear - Section 6.2.6 Height of web Design shear force Shear area - cl 6.2.6(3) Design shear resistance - cl 6. Check bending moment - Se Design bending moment Design bending resistance mo Slenderness ratio for lateral Correction factor - For cantilev Poissons ratio Shear modulus Unrestrained effective length	2.6(2) ction 6.2.5 ment - eq 6.13 <i>PASS</i> torsional buckli er beams	$h_{w} = h - 2 >$ $h_{w} / t_{w} = 30$ $V_{y,Ed} = 4.4$ $A_{v} = max(A$ $V_{c,y,Rd} = V_{pl}$ $V_{y,Ed} / V_{c,y,R}$ $PAS$ $M_{y,Ed} = 4.9$ $M_{c,y,Rd} = M_{f}$ $M_{y,Ed} / M_{c,y}$ $- Design bending$ $k_{c} = 1$ $C_{1} = 1 / k_{c}^{2}$ $v = 0.3$ $G = E / [2 >$ $L = 1.0 \times L$	$x tr = 137 mm$ $.4 = 32.9 \times \varepsilon /$ $kN$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$	η = 1.0 η < 72 × ε / η Shear buckling $t_w + 2 × r) × t_f, η ×$ $\sqrt{(3)} / γ_{M0} = 129.8$ pear resistance e. $q / γ_{M0} = 33.9$ kNm moment exceed 769 N/mm <sup>2</sup> 766 mm	00 resistance c h <sub>w</sub> × t <sub>w</sub> ) = 818 kN xceeds desig	an be igno 3 mm² In shear fo
Check shear - Section 6.2.6 Height of web Design shear force Shear area - cl 6.2.6(3) Design shear resistance - cl 6. Check bending moment - Se Design bending moment Design bending resistance mo Slenderness ratio for lateral Correction factor - For cantilev Poissons ratio Shear modulus Unrestrained effective length	2.6(2) ction 6.2.5 ment - eq 6.13 <i>PASS</i> torsional buckli er beams	$h_{w} = h - 2 >$ $h_{w} / t_{w} = 30$ $V_{y,Ed} = 4.4$ $A_{v} = max(A$ $V_{c,y,Rd} = V_{pl}$ $V_{y,Ed} / V_{c,y,R}$ $PAS$ $M_{y,Ed} = 4.9$ $M_{c,y,Rd} = M_{f}$ $M_{y,Ed} / M_{c,y}$ - Design bendixing $k_{c} = 1$ $C_{1} = 1 / kc^{2}$ $v = 0.3$ $G = E / [2 >$ $L = 1.0 \times L$ $M_{cr} = C_{1} \times T$ $kNm$	$x tr = 137 mm$ $.4 = 32.9 \times \varepsilon /$ $kN$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$ $x - 2 \times b \times tr + (t) / t = 0.034$	η = 1.0 η < 72 × ε / η Shear buckling $t_w + 2 × r) × t_r, η ×$ $\sqrt{(3)} / γ_{M0} = 129.8$ hear resistance e. $q / γ_{M0} = 33.9 \text{ kNm}$ moment exceed $769 \text{ N/mm}^2$ 766  mm $× \sqrt{(I_w / I_z + L^2 × G)}$	00 resistance c h <sub>w</sub> × t <sub>w</sub> ) = 818 kN xceeds desig	an be igno 3 mm² In shear fo

	Project	Brook	House		Job no.	1524			
Structural Calculation Ltd Mobbs Wood Farm	Calcs for	Portal	Frame		Start page no./F	Revision 38			
Ansty	Calcs by	Calcs date	Checked by	Checked date	Approved by Approved da				
CV7 9JN	AM	10/01/2020	JP	10/01/2020	JP	10/01/202			
			$\overline{a}_{LT} > \overline{\lambda}_{LT,0} - La$	ateral torsional b	uckling cann	ot be ignor			
Check buckling resistance -	Section 6.3.2.1								
Buckling curve - Table 6.5		b							
Imperfection factor - Table 6.3		αιτ = 0.34							
Correction factor for rolled sect		β <b>= 0.75</b>		<del>.</del>	2				
LTB reduction determination fa	ctor			$T - \overline{\lambda}LT,0) + \beta \times \overline{\lambda}LT$					
LTB reduction factor - eq 6.57				- β × λ̃μτ²)], 1, 1 /					
Modification factor				[1 - 2 × ( λ̃LT - 0.8	3) <sup>2</sup> ], 1) = <b>1.000</b>	)			
Modified LTB reduction factor -	-			λ <sub>LT<sup>2</sup></sub> ) = 0.679					
Design buckling resistance mo	ment - eq 6.55			y / γ <sub>M1</sub> = <b>23</b> kNm					
	<b>D</b> 1 0 0	My,Ed / Mb,y,F			la da sis d				
	PASS - I	Jesign bucklii	ig resistance	moment exceed	s aesign ben	aing mom			
Check bending and axial for									
Bending and axial force check	- eq.6.33 & eq.6.3			$0.5 \times h_w \times t_w \times f_y / \gamma$	үмо) = <b>84.8</b> kN				
		N <sub>Ed</sub> / N <sub>y,lim</sub> = 0.053							
Allowance need not be ma	ade for the effect	of the axial fo	rce on the pl	astic resistance	moment aboi	ut the y-y a			
Check combined bending an	d compression -	Section 6.3.3							
Equivalent uniform moment fac	ctors - Table B.3		kNm / 6.51 kl						
			kNm / 6.51 kN						
			$0.2 + 0.8 \times \alpha_y$ ,						
		ψιτ = -4.85	6 kNm / 6.51 k	«Nm = <b>-0.746</b>					
		αlt = 1.002	kNm / 6.51 kl	Nm = <b>0.154</b>					
		C <sub>mLT</sub> = max	$(0.2 + 0.8 \times \alpha)$	LT, 0.4) = <b>0.400</b>					
Interaction factors k <sub>ij</sub> for mer	nbers susceptibl	e to torsional	deformations	a - Table B.2					
Characteristic moment resistar	ice	$M_{y,Rk} = W_{pl.}$	∕ × fy = <b>33.9</b> kN	Nm					
Characteristic moment resistar	nce	$M_{z,Rk} = W_{pl.z} \times f_y = 8.6 \text{ kNm}$							
Characteristic resistance to no	rmal force	$N_{Rk} = A \times f_y$	= <b>558.8</b> kN						
Interaction factors		$k_{yy} = C_{my} \times$	(1 + min( $\overline{\lambda}_y$ -	0.2, 0.8) $\times$ N <sub>Ed</sub> / ( $\gamma$	(y × Nrk / γm1))	= 0.401			
			_		25) × χz × Nrk /	γ <sub>M1</sub> ) = <b>0.98</b>			
		$K_{zy} = 1 - 0.1$	$\times$ min(1, $\lambda$ z)	× NEd / ((CmLT - 0.2	Ned / ( $\chi_y \times N_{Rk}$ / $\gamma_{M1}$ ) + k <sub>yy</sub> $\times$ M <sub>y,Ed</sub> / ( $\chi_{LT} \times M_{y,Rk}$ / $\gamma_{M1}$ ) = 0.093				
Interaction formulae - eq 6.61 8	& eq 6.62				rк / γм1) = <b>0.09</b>	3			
Interaction formulae - eq 6.61 8	& eq 6.62	NEd / ( $\chi_y \times 1$	<b>I</b> Rk / γм1 <b>) + k</b> yy						
Interaction formulae - eq 6.61 8	& eq 6.62	NEd / ( $\chi_y  imes N$ Ed / ( $\chi_z  imes N$ Ed / ( $\chi_z  imes N$	Jrk / γм1) + kyy Jrk / γм1) + kzy	$\times$ My,Ed / ( $\chi$ LT $\times$ My,	<sub>Rk</sub> / γм1) = <b>0.2</b> 3	31			
Interaction formulae - eq 6.61 a		NEd / ( $\chi_y  imes N$ Ed / ( $\chi_z  imes N$ Ed / ( $\chi_z  imes N$	Jrk / γм1) + kyy Jrk / γм1) + kzy	$ \times \ M_{y,\text{Ed}} \ / \ (\chi_{\text{LT}} \times \ M_{y}, \\ \times \ M_{y,\text{Ed}} \ / \ (\chi_{\text{LT}} \times \ M_{y}, \\ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ $	<sub>Rk</sub> / γм1) = <b>0.2</b> 3	31			
		NEd / ( $\chi_y  imes N$ Ed / ( $\chi_z  imes N$ Ed / ( $\chi_z  imes N$	Jrk / γм1) + kyy Jrk / γм1) + kzy	$ \times \ M_{y,\text{Ed}} \ / \ (\chi_{\text{LT}} \times \ M_{y}, \\ \times \ M_{y,\text{Ed}} \ / \ (\chi_{\text{LT}} \times \ M_{y}, \\ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ $	<sub>Rk</sub> / γм1) = <b>0.2</b> 3	31			
Check design at end of span		Νed / (χy × Ν Νed / (χz × Ν <i>PASS - Co</i>	Jrk / γм1) + kyy Jrk / γм1) + kzy	$ \times \ M_{y,\text{Ed}} \ / \ (\chi_{\text{LT}} \times \ M_{y}, \\ \times \ M_{y,\text{Ed}} \ / \ (\chi_{\text{LT}} \times \ M_{y}, \\ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ \ $	<sub>Rk</sub> / γ <sub>M1</sub> ) = 0.23 ssion checks	31			
<u>Check design at end of span</u> Check shear - Section 6.2.6		Ν <sub>Ed</sub> / (χ <sub>y</sub> × Ν Ν <sub>Ed</sub> / (χ <sub>z</sub> × Ν <b>PASS - Co</b> h <sub>w</sub> = h - 2 ×	Jrk / γм1) + kyy Nrk / γм1) + kzy ombined ben	× My,Ed / $(\chi LT × My, X My,Ed / (\chi LT × My, X My,Ed / (\chi LT × My, M))$ ding and compres	<sub>Rk</sub> / γ <sub>M1</sub> ) = 0.23 ssion checks	31			
<u>Check design at end of span</u> Check shear - Section 6.2.6		Ν <sub>Ed</sub> / (χ <sub>y</sub> × Ν Ν <sub>Ed</sub> / (χ <sub>z</sub> × Ν <b>PASS - Co</b> h <sub>w</sub> = h - 2 ×	JRκ / γΜ1) + kyy JRk / γΜ1) + kzy ombined ben tr = <b>137</b> mm	× My,Ed / $(\chi LT × My, X My,Ed / (\chi LT × My, X My,Ed / (\chi LT × My, M))$ ding and compres	<sub>Rk</sub> / γ <sub>M1</sub> ) = 0.23 <i>ssion check</i> s	81 s are satisfi			
<u>Check design at end of span</u> Check shear - Section 6.2.6		Ν <sub>Ed</sub> / (χ <sub>y</sub> × Ν Ν <sub>Ed</sub> / (χ <sub>z</sub> × Ν <b>PASS - Co</b> h <sub>w</sub> = h - 2 ×	$J_{Rk} / \gamma_{M1} + k_{yy}$ $J_{Rk} / \gamma_{M1} + k_{zy}$ <i>ombined ben</i> $t_{f} = 137 \text{ mm}$ $4 = 32.9 \times \varepsilon / \delta$	× My,Ed / ( $\chi$ LT × My, × My,Ed / ( $\chi$ LT × My, ding and compres $\eta = 1.0$ $\eta < 72 \times \varepsilon / \eta$	<sub>Rk</sub> / γ <sub>M1</sub> ) = 0.23 <i>ssion check</i> s	31 s are satisf			
Check design at end of span Check shear - Section 6.2.6 Height of web		NEd / $(\chi y \times N)$ NEd / $(\chi z \times N)$ <b>PASS - Co</b> hw = h - 2 × hw / tw = 30. Vy,Ed = <b>3.9</b> H	$J_{Rk} / \gamma_{M1} + k_{yy}$ $J_{Rk} / \gamma_{M1} + k_{zy}$ $fr = 137 \text{ mm}$ $4 = 32.9 \times \varepsilon /$ $KN$	× My,Ed / ( $\chi$ LT × My, × My,Ed / ( $\chi$ LT × My, ding and compres $\eta = 1.0$ $\eta < 72 \times \varepsilon / \eta$	<sub>Rk</sub> / γ <sub>M1</sub> ) = 0.23 ssion checks 00 resistance c	31 s are satisfi an be ignoi			
<u>Check design at end of span</u> Check shear - Section 6.2.6 Height of web Design shear force		NEd / $(\chi y \times N)$ NEd / $(\chi z \times N)$ <b>PASS - Co</b> hw = h - 2 × hw / tw = 30. Vy,Ed = <b>3.9</b> H Av = max(A)	$J_{Rk} / \gamma_{M1} + k_{yy}$ $J_{Rk} / \gamma_{M1} + k_{zy}$ <i>ombined bend</i> $t_{f} = 137 \text{ mm}$ $4 = 32.9 \times \varepsilon / k_{N}$ $- 2 \times b \times t_{f} + (k_{N})$	× My,Ed / ( $\chi$ LT × My, × My,Ed / ( $\chi$ LT × My, ding and compres $\eta = 1.0$ $\eta < 72 \times \varepsilon / \eta$ Shear buckling	<sub>Rk</sub> / γ <sub>M1</sub> ) = 0.23 ssion checks 00 resistance c h <sub>w</sub> × t <sub>w</sub> ) = 818	31 s are satisfi an be ignor			
<u>Check design at end of span</u> Check shear - Section 6.2.6 Height of web Design shear force Shear area - cl 6.2.6(3)		NEd / $(\chi y \times N)$ NEd / $(\chi z \times N)$ <b>PASS - Co</b> hw = h - 2 × hw / tw = 30. Vy,Ed = <b>3.9</b> H Av = max(A)	$J_{Rk} / \gamma_{M1} + k_{yy}$ $J_{Rk} / \gamma_{M1} + k_{zy}$ $combined bence$ $t_{f} = 137 \text{ mm}$ $4 = 32.9 \times \varepsilon /$ $4 = 32.9 \times \varepsilon /$ $CN$ $- 2 \times b \times t_{f} + (y_{Rd} = A_{v} \times (f_{y} / y_{Rd}))$	× My,Ed / ( $\chi$ LT × My, × My,Ed / ( $\chi$ LT × My, ding and compres $\eta = 1.0$ $\eta < 72 \times \varepsilon / \eta$ Shear buckling	<sub>Rk</sub> / γ <sub>M1</sub> ) = 0.23 ssion checks 00 resistance c h <sub>w</sub> × t <sub>w</sub> ) = 818	31 s are satisfi an be ignor			

	Project	Brook	House		Job no.	1524		
Structural Calculation Ltd Mobbs Wood Farm	Calcs for	Porta	l Frame		Start page no./F	Revision 39		
Ansty	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved da		
CV7 9JN	AM	10/01/2020	JP	10/01/2020	JP	10/01/20		
Check bending moment - Se	ection 6.2.5							
Design bending moment		M <sub>y,Ed</sub> = 6.5	kNm					
Design bending resistance mo	oment - eq 6.13	$M_{c,y,Rd} = M_{f}$	$p_{\text{l,y,Rd}} = W_{\text{pl.y}} \times f$	<sup>у</sup> / γмо = <b>33.9</b> kNm				
		My,Ed / Mc,y,	Rd = 0.192					
	PAS	S - Design bendi	ng resistance	moment exceed	ls design ber	nding mome		
Slenderness ratio for lateral	torsional bucl	kling						
Correction factor - For cantile	ver beams	kc = <b>1</b>						
		$C_1 = 1 / k_c^2$	= 1					
Poissons ratio		v = 0.3						
Shear modulus		G = E / [2 :	<pre>&lt; (1 + v)] = 807</pre>	769 N/mm²				
Unrestrained effective length		L = 1.0 × L	m2_s1_seg1_T = 2	<b>766</b> mm				
Elastic critical buckling mome	nt	$M_{cr} = C_1 \times$	$\pi^2 \times E \times I_z / L^2$	$\times \sqrt{(I_w / I_z + L^2 \times G)}$	$\times$ It / ( $\pi^2 \times$ E $\times$	(Iz)) = <b>31.8</b>		
		kNm						
Slenderness ratio for lateral to	orsional buckling	$\overline{\lambda}_{LT} = \sqrt{W}$	$pl.y \times f_y / M_{cr}) =$	1.033				
Limiting slenderness ratio		$\overline{\lambda}_{LT,0} = 0.4$						
			$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0} - L_{0}$	ateral torsional b	uckling cann	ot be ignoi		
Check buckling resistance -	Section 6.3.2.	1						
Buckling curve - Table 6.5		b						
Imperfection factor - Table 6.3	3	αLT = <b>0.34</b>						
Correction factor for rolled see	ctions	β = <b>0.75</b>						
LTB reduction determination f	actor	φιτ = 0.5 ×	[1 + αlt × ( λ L	τ - λιτ,ο) + β × λιτ	<sup>2</sup> ] = 1.008			
LTB reduction factor - eq 6.57	,	χιτ = min(1	- / [φ∟τ + √(φ∟τ <sup>2</sup>	- β × λ̃ιτ²)], 1, 1 /	$\frac{1}{\lambda}LT^{2}$ ) = 0.679			
Modification factor				[1 - 2 × ( λ LT - 0.8				
Modified LTB reduction factor	- eq 6.58			λ <sub>LT<sup>2</sup></sub> ) = 0.679	, . ,			
Design buckling resistance m	•			y / γ <sub>M1</sub> = <b>23</b> kNm				
0 0		My,Ed / Mb,y,						
	PASS			moment exceed	ls design ber	nding mome		
Check combined bending a	nd compressio	n - Section 6.3.3						
Equivalent uniform moment fa	-		3 kNm / 6.51 k	Nm = <b>-0.746</b>				
			kNm / 6.51 kN					
			$(0.2 + 0.8 \times \alpha_y)$					
			-	-				
		ψιτ = -4.856 kNm / 6.51 kNm = <b>-0.746</b> αιτ = 1.002 kNm / 6.51 kNm = <b>0.154</b>						
		$C_{mLT} = max(0.2 + 0.8 \times \alpha LT, 0.4) = 0.400$						
Interaction factors k <sub>ij</sub> for me	mbers suscen	tible to torsional	deformations	- Table B 2				
Characteristic moment resista	-		.y × fy = <b>33.9</b> kl					
Characteristic moment resista			.z × fy = 8.6 kN					
Characteristic resistance to no			<sub>y</sub> = 558.8 kN					
Interaction factors				0.2, 0.8) × NEd / ()	V × NRk / VM1))	= 0.401		
				× Ned / ((Cmlt - 0.2				
Interaction formulae - eq 6.61	& eg 6 62			× Νεα / ((Cml1 - 0.2 × My,ed / (χlt × My,				
moraotion ionnulae - eq 0.01	a og 0.02	INEU / (XY X	TINK / YIVI J T KYY	~ IVIY,EU / (LEI × IVIY,	$(\mathbf{x}, \mathbf{y}, \mathbf{y}) = 0 \cdot 1 2$	- 1		
		NET / MARY		$\times$ My,Ed / ( $\chi$ LT $\times$ My,	DK / MMA) - 0 20	D1		

C		Brook	House		JF	P1524
Structural Calculation Ltd	Calcs for				Start page no./	Revision
Mobbs Wood Farm		Porta	l Frame		40	
Ansty CV7 9JN AM		Calcs date 10/01/2020	Checked by JP	Checked date 10/01/2020	Approved by JP	Approved o 10/01/2
Consider Combination 2 - 1.	0G + 1.0Q + 1.0I	RQ (Service)				
Check design at end of span	1					
Check y-y axis deflection - S	ection 7.2.1					
Maximum deflection		δ <sub>y</sub> = <b>0.2</b> mr	n			
Allowable deflection		$\delta_{y,Allowable}$ =	L <sub>m2_s1</sub> / 180 = ·	<b>15.4</b> mm		
		$\delta_{y}  /  \delta_{y, Allowab}$	le = <b>0.014</b>			
			PASS - Allow	wable deflection	exceeds des	sign deflect
Column 2 design						
Section details						
Section type		UB 152x89	x16 (BS4-1)			
Steel grade - EN 10025-2:200-	4	S275	. ,			
Nominal thickness of element		t <sub>nom</sub> = max(	(tf, tw) = <b>7.7</b> mm	ı		
Nominal yield strength		fy = <b>275</b> N/				
Nominal ultimate tensile streng	gth	f <sub>u</sub> = <b>410</b> N/				
Modulus of elasticity		E = 21000	UN/mm²			
<u>►</u>		UB 152x8	9x16 (BS4-1)			
<b>∓</b>		Section de	epth, h, 152.4 mm readth, b, 88.7 mm			
		Mass of se	ection, Mass, 16 kg/m			
		Web thick	ckness, t <sub>f</sub> , 7.7 mm ness, t <sub>w</sub> , 4.5 mm			
			us, r, 7.6 mm ection, A, 2032 mm <sup>2</sup>			
1524			gyration about y-axis, i <sub>y</sub> gyration about z-axis, i <sub>z</sub>			
4		Elastic se	ction modulus about y-ax	kis, W <sub>el.y</sub> , 109483 mm <sup>3</sup>		
	→ ←4.5	Plastic se	ction modulus about y-ax ction modulus about z-ax	kis, W <sub>pl.v</sub> , 123256 mm <sup>3</sup>		
		Second m	oment of area about y-a	xis, I <sub>y</sub> , 8342621 mm <sup>4</sup>		
2.7		Second m	oment of area about z-a	xis, 1 <sub>z</sub> , 897506 mm <sup>-</sup>		
Ť		.1				
	<b>⊲</b> 88.7	₽				
Lateral restraint	oint of august and	- nh/				
Both flanges have lateral restr	aint at supports o	oniy				
Consider Combination 1 - 1.	35G + 1.5Q + 1.5	5RQ (Strength)				
Classification of cross section	ons - Section 5.					
		ε = √[235 Ν	$1/mm^2 / f_y] = 0.$	92		
Internal compression parts s	subject to bendi	ing and compre	ssion - Table	5.2 (sheet 1 of 3	)	
Width of section		c = d = <b>12</b> 1	I <b>.8</b> mm			
				$t_w  \times  f_y)$ - (t_f + r)] /	-	
		c / t <sub>w</sub> = 27.	1 = 29.3 × ε <=	$396  imes \epsilon$ / (13 $ imes \alpha$	- 1) Clas	s 1
Outstand flanges - Table 5.2	(sheet 2 of 3)					
Width of section	-	c = (b - t <sub>w</sub> -	2 × r) / 2 = <b>34</b>	. <b>5</b> mm		
		o/t = 45	= 4.8 × ε <= 9 :	×ε Class <sup>2</sup>	I	
		C/II = 4.5	- 4.0 × 8 ~ - 9	x E Glass		

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Structural Calculation Ltd Mobbs Wood Farm	Calcs for	Porta	l Frame		Start page no./	Revision 41		
Ansty CV7 9JN	Calcs by AM	Calcs date 10/01/2020	Checked by JP	Checked date 10/01/2020	Approved by JP	Approved d 10/01/20		
Check compression - Section	6.2.4							
Design compression force		N <sub>Ed</sub> = <b>6</b> kN						
Design resistance of section - e	q 6.10	$N_{c,Rd} = N_{pl,F}$	$Rd = A \times f_y / \gamma MO$	= <b>558.8</b> kN				
		NEd / Nc,Rd	= 0.011					
		PASS - Desig	n compressio	on resistance exc	ceeds design	compress		
Slenderness ratio for y-y axis	flexural buck	ling - Section 6.	3.1.3					
Critical buckling length		$L_{cr,y} = L_{m3_s}$	1 = <b>2800</b> mm					
Critical buckling force		$N_{cr,y} = \pi^2 \times$	$E \times I_y / L_{cr,y^2} =$	2205.5 kN				
Slenderness ratio for buckling -	eq 6.50	$\overline{\lambda}_y = \sqrt{A \times A}$	fy / Ncr,y) = 0.5	03				
Check y-y axis flexural buckli	ng resistance	- Section 6.3.1.1	I					
Buckling curve - Table 6.2		а						
Imperfection factor - Table 6.1		α <sub>y</sub> = <b>0.21</b>						
Buckling reduction determinatio	n factor	$\phi_{\rm Y} = 0.5 \times ($	$1 + \alpha_y \times (\overline{\lambda}_y -$	$(0.2) + \overline{\lambda}y^2) = 0.65$	59			
Buckling reduction factor - eq 6				$\bar{\lambda}_{y}^{2})), 1) = 0.923$				
Design buckling resistance - eq			$\times \mathbf{A} \times \mathbf{f}_{y} / \gamma_{M1} =$					
		NEd / Nb,y,Rd	•					
				ig resistance exc	eeds design	compress		
Slenderness ratio for z-z axis	flexural buck	ling - Section 6.3	3.1.3	-	-	-		
Critical buckling length		-	1_seg1 = <b>2800</b> m	nm				
Critical buckling force			$E \times I_z / L_{cr,z^2} =$					
Slenderness ratio for buckling -	eq 6.50		fy / Ncr,z) = 1.5					
Check z-z axis flexural buckli	-	- Section 6 3 1 1						
Buckling curve - Table 6.2		b						
Imperfection factor - Table 6.1		$\alpha_z = 0.34$						
Buckling reduction determinatio	n factor	$\phi_z = 0.5 \times ($	$1 + \alpha_z \times (\overline{\lambda}_z -$	$(0.2) + \overline{\lambda}z^2 = 1.90$	)4			
Buckling reduction factor - eq 6.			$\phi_z = 0.5 \times (1 + \alpha_z \times (\overline{\lambda}_z - 0.2) + \overline{\lambda}_z^2) = 1.904$ $\chi_z = \min(1 / (\phi_z + \sqrt{(\phi_z^2 - \overline{\lambda}_z^2)}), 1) = 0.33$					
Design buckling resistance - eq			$\times A \times f_y / \gamma_{M1} =$					
Deelgii Daelainig reeletariee oq	0.11	NEd / Nb,z,Rd						
				ig resistance exc	eeds design	compress		
Check torsional and torsional	-flexural buck		-		J			
Torsional buckling length		-	s1_seg1_R = <b>2800</b>	mm				
Distance from shear centre to c	entroid in y ax							
Distance from shear centre to c	-	•						
Radius of gyration		$i_0 = \sqrt{(i_y^2 + i_y^2)}$	iz <sup>2</sup> ) = <b>67.4</b> mm					
Elastic critical torsional buckling	force	1.	$N_{cr,T} = 1 / i0^2 \times (G \times It + \pi^2 \times E \times Iw / L_{cr,T}^2) = 905.6 \text{ kN}$					
Torsion factor			$\beta \tau = 1 - (y_0 / i_0)^2 = 1$					
Elastic critical torsional-flexural	buckling force	1 (3						
	-		cr,y - √[(1 - Ncr,⊤	$(N_{cr,y})^2 + 4 \times (y_0 / $	io) <sup>2</sup> × N <sub>cr,T</sub> / N	cr,y]] = 905.6		
Elastic critical buckling force			Icr,T, Ncr,TF) = 90					
Slenderness ratio for torsional b	ouckling - eq 6	.52 $\overline{\lambda}_{T} = \sqrt{[A \times ]}$	fy / Ncr] = <b>0.78</b>	6				
Design resistance for torsion	al and torsion	al-flexural buck	ling - Section	6.3.1.1				
Design resistance for torsion			J					
-		b						
Buckling curve - Table 6.2 Imperfection factor - Table 6.1		b ατ = <b>0.34</b>						

	Project	Brook	House		Job no. JF	21524
Structural Calculation Ltd Mobbs Wood Farm	Calcs for	Porta	l Frame		Start page no./I	Revision 42
Ansty	O a la a la a			Observed date	A	
CV7 9JN	Calcs by AM	Calcs date 10/01/2020	Checked by JP	Checked date 10/01/2020	Approved by JP	Approved of 10/01/2
Buckling reduction factor - eq 6	.49	χ⊤ = min(1	/ (φ⊤ + √(φ⊤² -	λ <sub>⊤</sub> <sup>2</sup> )), 1) = <b>0.733</b>		
Design buckling resistance - eq	6.47	$N_{b,T,Rd} = \chi_T$	$\times$ A $\times$ fy / $\gamma_{M1}$ =	• <b>409.8</b> kN		
		NEd / Nb,T,R				
		PASS - D	esign bucklin	ng resistance exc	ceeds design	compress
Check design at start of span	<u>l</u>					
Check shear - Section 6.2.6						
Height of web			< tf = <b>137</b> mm	η = <b>1.0</b>	00	
		h <sub>w</sub> / t <sub>w</sub> = 30	.4 = 32.9 × ε /	• •		
Desire the f			1.5.1	Shear buckling	resistance c	an be igno
Design shear force		$V_{y,Ed} = 1.7$		(+ + ) () (+ +	h+ ) = 044	
Shear area - cl 6.2.6(3)	0.6(0)			$(t_w + 2 \times r) \times t_f, \eta \times t_{f, \eta}$		<b>o</b> mm²
Design shear resistance - cl 6.2	2.0(2)		-	√(3)) / <sub>Умо</sub> = <b>129.8</b>	<b>K</b> N	
		Vy,Ed / Vc,y,R DAS		near resistance e	vcaade dasii	nn shoar fr
		7 43	is - Design si		xceeus uesi	gii Sileai it
Check bending moment - Sec	tion 6.2.5	M <sub>y,Ed</sub> = <b>4.9</b>	kNm			
Design bending moment	ment og 6 13	-		<sub>y</sub> / γ <sub>M0</sub> = <b>33.9</b> kNm		
Design bending resistance moment - eq 6.13				y/γmu - 33.3 κinin		
		Murd / Mau	na = 0 143			
	PASS	M <sub>y,Ed</sub> / M <sub>c,y,</sub> - Design bendi		e moment exceed	ls design ber	nding mon
Slenderness ratio for lateral t		- Design bendi		e moment exceed	ls design ber	nding mon
Slenderness ratio for lateral t Correction factor - For cantileve	orsional buck	- Design bendi		e moment exceed	ls design ber	nding mon
	orsional buck	: - Design bendi ling	ng resistance	e moment exceed	ls design ber	nding mon
	orsional buck	5 - Design bendi ling kc = 1	ng resistance	e moment exceed	ls design ber	nding mon
Correction factor - For cantileve	orsional buck	$k_{c} = 1$ $k_{c} = 1$ $C_{1} = 1 / k_{c}^{2}$ $v = 0.3$	ng resistance		ls design ber	nding mon
Correction factor - For cantileve Poissons ratio	orsional buck	<b>5 - Design bendi</b> ling $k_c = 1$ $C_1 = 1 / k_c^2$ v = 0.3 G = E / [2 > 1]	ng resistance := 1	<b>769</b> N/mm²	ls design ber	nding mon
Correction factor - For cantileve Poissons ratio Shear modulus	orsional buck er beams	<b>b</b> - Design bendi ling $k_c = 1$ $C_1 = 1 / k_c^2$ v = 0.3 G = E / [2 + 2] $L = 1.0 \times L$	ng resistance = 1 < (1 + ν)] = 803 m3_s1_seg1_B = 2	<b>769</b> N/mm²	-	-
Correction factor - For cantileve Poissons ratio Shear modulus Unrestrained effective length	<b>orsional buck</b> er beams t	<b>5 - Design bendi</b> <b>ling</b> $k_c = 1$ $C_1 = 1 / k_c^2$ v = 0.3 G = E / [2 > 2 > 2 > 2 > 2 > 2 > 2 > 2 > 2 > 2	ng resistance = 1 < (1 + ν)] = 803 m3_s1_seg1_B = 2	769 N/mm² 800 mm × √(Iw / Iz + L² × G	-	-
Correction factor - For cantileve Poissons ratio Shear modulus Unrestrained effective length Elastic critical buckling moment	<b>orsional buck</b> er beams t	<b>5 - Design bendi</b> <b>ling</b> $k_c = 1$ $C_1 = 1 / k_c^2$ v = 0.3 G = E / [2 > 2 > 2 > 2 > 2 > 2 > 2 > 2 > 2 > 2	ng resistance f = 1 (1 + v)] = 807 $m_{3_{s1_{seg1_B}}} = 2$ $\pi^2 \times E \times I_z / L^2$ $p_{Ly} \times f_y / M_{cr}) = 1$	769 N/mm² 800 mm × √(Iw / Iz + L² × G	-	-
Correction factor - For cantileve Poissons ratio Shear modulus Unrestrained effective length Elastic critical buckling moment Slenderness ratio for lateral tors	<b>orsional buck</b> er beams t	$\begin{aligned} \mathbf{k}_{c} &= 1 \\ \mathbf{k}_{c} &= 1 \\ \mathbf{C}_{1} &= 1 / \mathbf{k}_{c}^{2} \\ \mathbf{v} &= 0.3 \\ \mathbf{G} &= \mathbf{E} / [2 \times \mathbf{L} \\ \mathbf{L} &= 1.0 \times \mathbf{L} \\ \mathbf{M}_{cr} &= \mathbf{C}_{1} \times \mathbf{L} \\ \mathbf{M}_{cr} &= \mathbf{C}_{1} \times \mathbf{L} \\ \mathbf{K} \mathbf{N} \\ \overline{\lambda}_{LT} &= \sqrt{(W)} \\ \overline{\lambda}_{LT,0} &= 0.4 \end{aligned}$	$mg \ resistance$ $f = 1$ $((1 + v)) = 807$ $m_{3_{s1_{seg1_B}}} = 2$ $\pi^2 \times E \times I_z / L^2$ $p_{ly} \times f_y / M_{cr}) = 0$	769 N/mm² 800 mm × √(Iw / Iz + L² × G	× It / ( $\pi^2$ × E >	< lz)) = <b>31.3</b>
Correction factor - For cantileve Poissons ratio Shear modulus Unrestrained effective length Elastic critical buckling moment Slenderness ratio for lateral tors	<b>orsional buck</b> er beams t sional buckling	$k_{c} = 1$ $C_{1} = 1 / k_{c}^{2}$ $v = 0.3$ $G = E / [2 \times L]$ $M_{cr} = C_{1} \times KNm$ $\overline{\lambda}_{LT} = \sqrt{(W]}$ $\overline{\lambda}_{LT,0} = 0.4$	$mg \ resistance$ $f = 1$ $((1 + v)) = 807$ $m_{3_{s1_{seg1_B}}} = 2$ $\pi^2 \times E \times I_z / L^2$ $p_{ly} \times f_y / M_{cr}) = 0$	769 N/mm² 800 mm × √(I <sub>w</sub> / Iz + L² × G 1.041	× It / ( $\pi^2$ × E >	< lz)) = <b>31.3</b>
Correction factor - For cantileve Poissons ratio Shear modulus Unrestrained effective length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio	<b>orsional buck</b> er beams t sional buckling	$k_{c} = 1$ $C_{1} = 1 / k_{c}^{2}$ $v = 0.3$ $G = E / [2 \times L]$ $M_{cr} = C_{1} \times KNm$ $\overline{\lambda}_{LT} = \sqrt{(W]}$ $\overline{\lambda}_{LT,0} = 0.4$	$mg \ resistance$ $f = 1$ $((1 + v)) = 807$ $m_{3_{s1_{seg1_B}}} = 2$ $\pi^2 \times E \times I_z / L^2$ $p_{ly} \times f_y / M_{cr}) = 0$	769 N/mm² 800 mm × √(I <sub>w</sub> / Iz + L² × G 1.041	× It / ( $\pi^2$ × E >	< lz)) = <b>31.3</b>
Correction factor - For cantileve Poissons ratio Shear modulus Unrestrained effective length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio Check buckling resistance - S	<b>orsional buck</b> er beams t sional buckling	$k_{c} = 1$ $k_{c} = 1$ $C_{1} = 1 / k_{c}^{2}$ $v = 0.3$ $G = E / [2 \times L]$ $L = 1.0 \times L$ $M_{cr} = C_{1} \times L$ $kNm$ $\overline{\lambda}_{LT} = \sqrt{(W)}$ $\overline{\lambda}_{LT,0} = 0.4$	$mg \ resistance$ $f = 1$ $((1 + v)) = 807$ $m_{3_{s1_{seg1_B}}} = 2$ $\pi^2 \times E \times I_z / L^2$ $p_{ly} \times f_y / M_{cr}) = 0$	769 N/mm² 800 mm × √(I <sub>w</sub> / Iz + L² × G 1.041	× It / ( $\pi^2$ × E >	< lz)) = <b>31.3</b>
Correction factor - For cantilever Poissons ratio Shear modulus Unrestrained effective length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio <b>Check buckling resistance - S</b> Buckling curve - Table 6.5	orsional buck er beams t sional buckling Section 6.3.2.1	b <b>b</b> <b>b</b> <b>b</b> <b>b</b> <b>b</b> <b>b</b> <b>b</b> <b>b</b>	$mg \ resistance$ $f = 1$ $((1 + v)) = 807$ $m_{3_{s1_{seg1_B}}} = 2$ $\pi^2 \times E \times I_z / L^2$ $p_{ly} \times f_y / M_{cr}) = 0$	769 N/mm² 800 mm × √(I <sub>w</sub> / Iz + L² × G 1.041	× It / ( $\pi^2$ × E >	< lz)) = <b>31.3</b>
Correction factor - For cantilever Poissons ratio Shear modulus Unrestrained effective length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio <b>Check buckling resistance - S</b> Buckling curve - Table 6.5 Imperfection factor - Table 6.3	orsional buck er beams t sional buckling Section 6.3.2.1	$\begin{aligned} \mathbf{k} & \mathbf{c} = 1 \\ \mathbf{k} & \mathbf{c} = 1 \\ \mathbf{C}_1 &= 1 \ / \ \mathbf{k} \mathbf{c}^2 \\ \mathbf{v} &= 0.3 \\ \mathbf{G} &= \mathbf{E} \ / \ [2 \ \mathbf{z} \\ \mathbf{L} &= 1.0 \ \mathbf{x} \ \mathbf{L} \\ \mathbf{M} & \mathbf{c} = \mathbf{C}_1 \ \mathbf{x} \\ \mathbf{k} & \mathbf{N} \\ \overline{\lambda} & \mathbf{L} \mathbf{T} &= \sqrt{(\mathbf{W})} \\ \overline{\lambda} & \mathbf{L} \mathbf{T}, 0 &= 0.4 \\ \mathbf{b} \\ \mathbf{\alpha} & \mathbf{L} \mathbf{T} &= 0.34 \\ \mathbf{\beta} &= 0.75 \end{aligned}$	ng resistance $f = 1$ $(1 + v)] = 807$ $m_{3_{s1_{seg1_{B}}}} = 2$ $\pi^{2} \times E \times I_{z} / L^{2}$ $p_{l,y} \times f_{y} / M_{cr}) = \frac{1}{\lambda L\tau} > \lambda L\tau, o - L_{s}$	769 N/mm² 800 mm × √(I <sub>w</sub> / Iz + L² × G 1.041	× It / (π <sup>2</sup> × E >	< lz)) = <b>31.3</b>
Correction factor - For cantilever Poissons ratio Shear modulus Unrestrained effective length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio <b>Check buckling resistance - S</b> Buckling curve - Table 6.5 Imperfection factor - Table 6.3 Correction factor for rolled section	orsional buck er beams t sional buckling Section 6.3.2.1	S - Design bendi ling kc = 1 C1 = 1 / kc <sup>2</sup> v = 0.3 G = E / [2 ⇒ L = 1.0 × L Mor = C1 × kNm $\overline{\lambda}$ LT = √(W $\overline{\lambda}$ LT,0 = 0.4 b αLT = 0.34 β = 0.75 φLT = 0.5 ×	ng resistance $f = 1$ $(1 + v) = 807$ $m_{3_{s1_{seg1_{B}}}} = 2$ $\pi^{2} \times E \times I_{z} / L^{2}$ $p_{Ly} \times f_{y} / M_{cr} = \frac{1}{\lambda_{LT}} \times \lambda_{LT,0} - L_{cr}$ $[1 + \alpha_{LT} \times (\overline{\lambda}_{LT})]$	769 N/mm² 800 mm × √(lʷ / lz + L² × G 1.041 ateral torsional b	× $lt / (\pi^2 \times E \times Duckling cannot c$	(Iz)) = 31.3
Correction factor - For cantilever Poissons ratio Shear modulus Unrestrained effective length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio <b>Check buckling resistance - S</b> Buckling curve - Table 6.5 Imperfection factor - Table 6.3 Correction factor for rolled section LTB reduction determination factor	orsional buck er beams t sional buckling Section 6.3.2.1	<b>b</b> <b>b</b> <b>b</b> <b>b</b> <b>c</b> = <b>1</b> <b>c</b> = 1 / kc <sup>2</sup> <b>v</b> = <b>0.3</b> <b>c</b> = E / [2 × L <b>c</b> = 1.0 × L <b>c</b> = 1.0 × L <b>c</b> = C1 × kNm $\bar{\lambda}$ LT = $\sqrt{(W)}$ $\bar{\lambda}$ LT = $\sqrt{(W)}$ $\bar{\lambda}$ LT = <b>0.4</b> <b>b</b> <b>c</b> = <b>0.34</b> <b>b</b> <b>c</b> = <b>0.75</b> φLT = 0.5 × χLT = min(1)	ng resistance $f = 1$ $(1 + v) = 807$ $m_{3_{s1_{seg1_{B}}}} = 2$ $\pi^{2} \times E \times I_{z} / L^{2}$ $p_{Ly} \times f_{y} / M_{cr}) = \frac{1}{\lambda L\tau} > \lambda L\tau, o - L_{s}$ $[1 + \alpha LT \times (\lambda L)]$ $[1 + \alpha LT + \sqrt{(\alpha L)^{2}}]$	769 N/mm² 800 mm × √(lw / lz + L² × G 1.041 ateral torsional b τ - λ̄∟τ.ο) + β × λ̄∟τ	× lt / $(\pi^2 \times E >$ buckling can $r^2] = 1.016$ $\bar{\lambda} L \tau^2) = 0.674$	< lz)) = 31.3 not be igno
Correction factor - For cantilever Poissons ratio Shear modulus Unrestrained effective length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio <b>Check buckling resistance - S</b> Buckling curve - Table 6.5 Imperfection factor - Table 6.3 Correction factor for rolled secti LTB reduction determination fac	orsional buck er beams t sional buckling Section 6.3.2.1 ions ctor	$\begin{aligned} \mathbf{k} &= 1 \\ \mathbf{k}_{c} &= 1 \\ \mathbf{C}_{1} &= 1 / \mathbf{k}_{c}^{2} \\ \mathbf{v} &= 0.3 \\ \mathbf{G} &= \mathbf{E} / [2 \times \mathbf{L} \\ \mathbf{L} &= 1.0 \times \mathbf{L} \\ \mathbf{M}_{cr} &= \mathbf{C}_{1} \times \mathbf{L} \\ \mathbf{M}_{cr} &= \mathbf{C}_{1} \times \mathbf{L} \\ \mathbf{k} \mathbf{N} \\ \mathbf{k} \\ \mathbf{L} \\ \mathbf{L} \\ \mathbf{L} \\ \mathbf{J} \\ \mathbf$	ng resistance $f = 1$ $(1 + v)] = 807$ $m^{3}s^{1}seg^{1}B = 2$ $\pi^{2} \times E \times I_{z} / L^{2}$ $ply \times f_{y} / Mcr) = \frac{1}{\lambda L \tau} \times \frac{1}{\lambda L \tau} - \frac{1}{\lambda L}$ $[1 + \alpha L \tau \times (-\lambda L)]$ $[1 + \alpha L \tau \times (-\lambda L)]$ $(-\lambda L) + \sqrt{(-\lambda L)}$	769 N/mm <sup>2</sup> 800 mm × √(I <sub>w</sub> / I <sub>z</sub> + L <sup>2</sup> × G 1.041 ateral torsional b T - $\bar{\lambda}$ LT,0) + β × $\bar{\lambda}$ LT - β × $\bar{\lambda}$ LT <sup>2</sup> )], 1, 1 /	× lt / $(\pi^2 \times E >$ buckling can $r^2] = 1.016$ $\bar{\lambda} L \tau^2) = 0.674$	< lz)) = 31.3 not be igno
Correction factor - For cantilever Poissons ratio Shear modulus Unrestrained effective length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio <b>Check buckling resistance - S</b> Buckling curve - Table 6.5 Imperfection factor - Table 6.3 Correction factor for rolled secti LTB reduction determination fac LTB reduction factor - eq 6.57 Modification factor	orsional buck er beams t sional buckling Section 6.3.2.1 ions ctor eq 6.58	<b>b</b> <b>b</b> <b>b</b> <b>b</b> <b>c</b> − <b>0.3</b> <b>c</b> − <b>1</b> / kc <sup>2</sup> <b>v</b> = <b>0.3</b> <b>c</b> − <b>c</b> − <b>i</b> / kc <sup>2</sup> <b>v</b> = <b>0.3</b> <b>c</b> − <b>c</b> − <b>i</b> / [2 × 2] <b>c</b> − <b>c</b> − <b>i</b> × − <b>k</b> <b>k</b> Nm $\bar{\lambda}$ LT = $\sqrt{(W)}$ $\bar{\lambda}$ LT = $\sqrt{(W)}$ $\bar{\lambda}$ LT,0 = <b>0.4</b> <b>b</b> <b>c</b> − <b>c</b> − <b>3.4</b> $\beta$ = <b>0.75</b> $\phi$ LT = 0.5 × $\chi$ LT = min(1 <b>f</b> = min(1 - $\chi$ LT,mod = m	ng resistance $f = 1$ $(1 + v) = 807$ $m_{3_{s_{1_{seg_{1_{B}}}}} = 2$ $\pi^{2} \times E \times I_{z} / L^{2}$ $p_{Ly} \times f_{y} / M_{cr} = \frac{1}{\lambda L\tau} > \frac{1}{\lambda L\tau} - \frac{1}{\lambda L\tau}$ $[1 + \alpha L\tau \times (-\lambda L)$ $(-1) = \frac{1}{\lambda L\tau} + \sqrt{(-1)} + \sqrt{(-1)}$ $(-1) = \frac{1}{\lambda L\tau} + \sqrt{(-1)} + \sqrt{(-1)}$ $(-1) = \frac{1}{\lambda L\tau} + \sqrt{(-1)}$	769 N/mm <sup>2</sup> 800 mm × √(I <sub>w</sub> / I <sub>z</sub> + L <sup>2</sup> × G 1.041 ateral torsional b T - $\overline{\lambda}$ LT,0) + β × $\overline{\lambda}$ LT - β × $\overline{\lambda}$ LT <sup>2</sup> )], 1, 1 / E [1 - 2 × ( $\overline{\lambda}$ LT - 0.8	× lt / $(\pi^2 \times E \times$ buckling cannels $\pi^2] = 1.016$ $\bar{\lambda}LT^2) = 0.674$ $3)^2], 1) = 1.000$	< lz)) = 31.3 not be igno
Correction factor - For cantilever Poissons ratio Shear modulus Unrestrained effective length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio <b>Check buckling resistance - S</b> Buckling curve - Table 6.5 Imperfection factor - Table 6.3 Correction factor for rolled secti LTB reduction determination fac LTB reduction factor - eq 6.57 Modification factor Modified LTB reduction factor -	orsional buck er beams t sional buckling Section 6.3.2.1 ions ctor eq 6.58 ment - eq 6.55	S - Design bendi ling $k_c = 1$ $C_1 = 1 / k_c^2$ v = 0.3 G = E / [2 > 2 > 2 = 1.0 × L $M_{cr} = C_1 × T = \sqrt{W}$ $\overline{\lambda}_{LT} = \sqrt{W}$ $\overline{\lambda}_{LT} = \sqrt{W}$ $\overline{\lambda}_{LT,0} = 0.4$ b $\alpha_{LT} = 0.34$ $\beta = 0.75$ $\phi_{LT} = 0.5 × \chi_{LT} = min(1 = min(1 - \chi_{LT,mod} = m M_{b,y,Rd} = \chi_{L} M_{y,Ed} / M_{b,y})$	ng resistance $f = 1$ $((1 + v)) = 807$ $m_{3_{s1_{seg1_{B}}} = 2$ $\pi^{2} \times E \times I_{z} / L^{2}$ $p_{Ly} \times f_{y} / M_{cr}) = \frac{1}{\lambda L \tau} \times \frac{1}{\lambda L \tau} - L_{cr}$ $[1 + \alpha_{LT} \times (-\lambda_{L}) + \frac{1}{\lambda L \tau} - L_{cr}]$ $[1 + \alpha_{LT} \times (-\lambda_{L}) + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $($	769 N/mm <sup>2</sup> 800 mm × √(I <sub>w</sub> / I <sub>z</sub> + L <sup>2</sup> × G 1.041 ateral torsional b T - $\overline{\lambda}$ LT,0) + β × $\overline{\lambda}$ LT - β × $\overline{\lambda}$ LT <sup>2</sup> )], 1, 1 / $(1 - 2 × (\overline{\lambda}$ LT - 0.8 $\overline{\lambda}$ LT <sup>2</sup> ) = 0.674 y / γM1 = 22.9 kNm	× lt / $(\pi^2 \times E >$ buckling cannels $\pi^2] = 1.016$ $\bar{\lambda}_{LT^2}) = 0.674$ $3^2], 1) = 1.000$	( z)) = 31.3 not be igno
Correction factor - For cantilever Poissons ratio Shear modulus Unrestrained effective length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio <b>Check buckling resistance - S</b> Buckling curve - Table 6.5 Imperfection factor - Table 6.3 Correction factor for rolled secti LTB reduction determination fac LTB reduction factor - eq 6.57 Modification factor Modified LTB reduction factor -	orsional buck er beams t sional buckling Section 6.3.2.1 ions ctor eq 6.58 ment - eq 6.55	S - Design bendi ling $k_c = 1$ $C_1 = 1 / k_c^2$ v = 0.3 G = E / [2 > 2 > 2 = 1.0 × L $M_{cr} = C_1 × T = \sqrt{W}$ $\overline{\lambda}_{LT} = \sqrt{W}$ $\overline{\lambda}_{LT} = \sqrt{W}$ $\overline{\lambda}_{LT,0} = 0.4$ b $\alpha_{LT} = 0.34$ $\beta = 0.75$ $\phi_{LT} = 0.5 × \chi_{LT} = min(1 = min(1 - \chi_{LT,mod} = m M_{b,y,Rd} = \chi_{L} M_{y,Ed} / M_{b,y})$	ng resistance $f = 1$ $((1 + v)) = 807$ $m_{3_{s1_{seg1_{B}}} = 2$ $\pi^{2} \times E \times I_{z} / L^{2}$ $p_{Ly} \times f_{y} / M_{cr}) = \frac{1}{\lambda L \tau} \times \frac{1}{\lambda L \tau} - L_{cr}$ $[1 + \alpha_{LT} \times (-\lambda_{L}) + \frac{1}{\lambda L \tau} - L_{cr}]$ $[1 + \alpha_{LT} \times (-\lambda_{L}) + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $($	769 N/mm <sup>2</sup> 800 mm × $\sqrt{(I_w / I_z + L^2 \times G)}$ 1.041 ateral torsional b T - $\overline{\lambda}_{LT,0}$ ) + $\beta \times \overline{\lambda}_{LT}$ - $\beta \times \overline{\lambda}_{LT}$ )], 1, 1 / $i [1 - 2 \times (\overline{\lambda}_{LT} - 0.8)]$	× lt / $(\pi^2 \times E >$ buckling cannels $\pi^2] = 1.016$ $\bar{\lambda}_{LT^2}) = 0.674$ $3^2], 1) = 1.000$	( z)) = 31.3 not be igno
Correction factor - For cantilever Poissons ratio Shear modulus Unrestrained effective length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio <b>Check buckling resistance - S</b> Buckling curve - Table 6.5 Imperfection factor - Table 6.3 Correction factor for rolled secti LTB reduction determination fac LTB reduction factor - eq 6.57 Modification factor Modified LTB reduction factor - Design buckling resistance mor	orsional buck er beams t sional buckling Section 6.3.2.1 ions ctor eq 6.58 ment - eq 6.55 PASS re - Section 6.2	S - Design bendi ling $k_c = 1$ $C_1 = 1 / k_c^2$ v = 0.3 G = E / [2 > 2 > 2 = 1.0 × L $M_{or} = C_1 × 1 × 1.0 × L$ $M_{or} = C_1 × 1.0 × L$ $M_{or} = C_1 × 1.0 × L$ $M_{or} = 0.1 × 1.0 × 1$	ng resistance $f = 1$ $((1 + v)) = 807$ $m_{3_{s1_{seg1_{B}}} = 2$ $\pi^{2} \times E \times I_{z} / L^{2}$ $p_{Ly} \times f_{y} / M_{cr}) = \frac{1}{\lambda L \tau} \times \frac{1}{\lambda L \tau} - L_{cr}$ $[1 + \alpha_{LT} \times (-\lambda_{L}) + \frac{1}{\lambda L \tau} - L_{cr}]$ $[1 + \alpha_{LT} \times (-\lambda_{L}) + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $(1 + \alpha_{LT} + \sqrt{(\phi_{L})} + \frac{1}{\lambda L \tau} - L_{cr}]$ $($	769 N/mm <sup>2</sup> 800 mm × √(I <sub>w</sub> / I <sub>z</sub> + L <sup>2</sup> × G 1.041 ateral torsional b T - $\overline{\lambda}$ LT,0) + β × $\overline{\lambda}$ LT - β × $\overline{\lambda}$ LT <sup>2</sup> )], 1, 1 / $(1 - 2 × (\overline{\lambda}$ LT - 0.8 $\overline{\lambda}$ LT <sup>2</sup> ) = 0.674 y / γM1 = 22.9 kNm	× lt / $(\pi^2 \times E >$ buckling cannels $\pi^2] = 1.016$ $\bar{\lambda}_{LT^2}) = 0.674$ $3^2], 1) = 1.000$	( z)) = 31.3 not be igno
Correction factor - For cantilever Poissons ratio Shear modulus Unrestrained effective length Elastic critical buckling moment Slenderness ratio for lateral tors Limiting slenderness ratio <b>Check buckling resistance - S</b> Buckling curve - Table 6.5 Imperfection factor - Table 6.3 Correction factor for rolled secti LTB reduction determination fac LTB reduction factor - eq 6.57 Modification factor Modified LTB reduction factor - Design buckling resistance mor	orsional buck er beams t sional buckling Section 6.3.2.1 ions ctor eq 6.58 ment - eq 6.55 PASS re - Section 6.2	S - Design bendi ling $k_c = 1$ $C_1 = 1 / k_c^2$ v = 0.3 G = E / [2 > 2 > 2 = 1.0 × L $M_{or} = C_1 × 1 × 1.0 × L$ $M_{or} = C_1 × 1.0 × L$ $M_{or} = C_1 × 1.0 × L$ $M_{or} = 0.1 × 1.0 × 1$	ng resistance $f = 1$ $((1 + v)) = 807$ $m_{3_{s1_{seg1_{B}}} = 2$ $\pi^{2} \times E \times I_{z} / L^{2}$ $p_{Ly} \times f_{y} / M_{cr}) = \frac{1}{\lambda_{LT}} \times \frac{1}{\lambda_{LT}} - L_{r}$ $[1 + \alpha_{LT} \times (-\lambda_{LT}) + \sqrt{(\phi_{LT})^{2}}$ $(-\lambda_{LT}) + \sqrt{(\phi_{LT})^{2}}$ $(-\lambda_{LT})$	769 N/mm <sup>2</sup> 800 mm × √(I <sub>w</sub> / I <sub>z</sub> + L <sup>2</sup> × G 1.041 ateral torsional b T - $\overline{\lambda}$ LT,0) + β × $\overline{\lambda}$ LT - β × $\overline{\lambda}$ LT <sup>2</sup> )], 1, 1 / $(1 - 2 × (\overline{\lambda}$ LT - 0.8 $\overline{\lambda}$ LT <sup>2</sup> ) = 0.674 y / γM1 = 22.9 kNm	× $lt / (\pi^2 \times E >$ buckling cannels for the second secon	ot be igno

	Project	Brook	House		Job no. JP1	524
Structural Calculation Ltd Mobbs Wood Farm	Calcs for	Portal	Frame		Start page no./Revision 43	
Ansty CV7 9JN	Calcs by AM	Calcs date 10/01/2020	Checked by JP	Checked date 10/01/2020	Approved by JP	Approved date 10/01/2020

Allowance need not be made for the effect	of the axial force on the plastic resistance moment about the y-y axis
Check combined bending and compression -	Section 6.3.3
Equivalent uniform moment factors - Table B.3	ψy = 0 kNm / -4.856 kNm = <b>0.000</b>
	α <sub>y</sub> = -2.428 kNm / -4.856 kNm = <b>0.500</b>
	$C_{my} = max(0.6 + 0.4 \times \psi_y) = 0.600$
	ψιτ = 0 kNm / -4.856 kNm = <b>0.000</b>
	α <sub>LT</sub> = -2.428 kNm / -4.856 kNm = <b>0.500</b>
	$C_{mLT} = max(0.6 + 0.4 \times \psi_{LT}) = 0.600$
Interaction factors k <sub>ij</sub> for members susceptible	e to torsional deformations - Table B.2
Characteristic moment resistance	$M_{y,Rk} = W_{pl.y} \times f_y = 33.9 \text{ kNm}$
Characteristic moment resistance	$M_{z,Rk}$ = $W_{pl.z} \times f_y$ = <b>8.6</b> kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y = 558.8 \text{ kN}$
Interaction factors	$k_{yy} = C_{my} \times (1 + min(\overline{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) = 0.602$
	$k_{zy} = 1 - 0.1 \times min(1, \ \overline{\lambda}z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) = 0.991$
Interaction formulae - eq 6.61 & eq 6.62	Ned / ( $\chi_y \times N_{Rk}$ / $\gamma_{M1}$ ) + $k_{yy} \times M_{y,Ed}$ / ( $\chi_{LT} \times M_{y,Rk}$ / $\gamma_{M1}$ ) = 0.14
	Ned / ( $\chi_z \times N_{Rk}$ / $\gamma_{M1}$ ) + $k_{zy} \times M_{y,Ed}$ / ( $\chi_{LT} \times M_{y,Rk}$ / $\gamma_{M1}$ ) = 0.243
	PASS - Combined bending and compression checks are satisfied

# Consider Combination 2 - 1.0G + 1.0Q + 1.0RQ (Service)

# Check design 583 mm along span

**Check y-y axis deflection - Section 7.2.1** Maximum deflection Allowable deflection

$$\begin{split} \delta_y &= 0.2 \text{ mm} \\ \delta_{y,\text{Allowable}} &= L_{\text{m3\_s1}} \ / \ 180 = 15.6 \text{ mm} \\ \delta_y \ / \ \delta_{y,\text{Allowable}} &= 0.01 \end{split}$$

PASS - Allowable deflection exceeds design deflection

# Rafter 2 design

Section details	
Section type	UB 152x89x16 (BS4-1)
Steel grade - EN 10025-2:2004	S275
Nominal thickness of element	t <sub>nom</sub> = max(t <sub>f</sub> , t <sub>w</sub> ) = <b>7.7</b> mm
Nominal yield strength	f <sub>y</sub> = <b>275</b> N/mm <sup>2</sup>
Nominal ultimate tensile strength	fu = <b>410</b> N/mm <sup>2</sup>
Modulus of elasticity	E = <b>210000</b> N/mm <sup>2</sup>

	Project	Brook	House		Job no.	P1524	
Structural Calculation Ltd	Calcs for	Brook	Tiouse		-		
Mobbs Wood Farm		Portal	Frame		Start page no./Revision 44		
Ansty CV7 9JN	Calcs by AM	Calcs date 10/01/2020	Checked by JP	Checked date 10/01/2020	Approved by JP	Approved da 10/01/20	
2.7							
			<b>9x16 (BS4-1)</b> epth, h, 152.4 mm				
47 <sup>SI</sup> ↓ <sup>12</sup> ↓ ↓ ↓ ↓ ↓ ↓ ↓ ↓ ↓ ↓	→ <b>+</b> 4.	Mass of se Flange thi Web thick Root radiu Area of se Radius of Elastic sec Elastic sec Plastic sec Plastic sec Second m	eadth, b, 88.7 mm sction, Mass, 16 kg/m ckness, t <sub>n</sub> , 7.7 mm ness, t <sub>w</sub> , 4.5 mm is, r, 7.6 mm ction, A, 2032 mm <sup>2</sup> gyration about y-axis, i <sub>y</sub> , 64. gyration about y-axis, i <sub>z</sub> , 21. ction modulus about y-axis, v ction modulus about y-axis, v ction modulus about y-axis, v oment of area about y-axis, oment of area about z-axis,	.016 mm M <sub>el.y</sub> , 109483 mm <sup>3</sup> M <sub>el.z</sub> , 20237 mm <sup>3</sup> M <sub>pl.y</sub> , 123256 mm <sup>3</sup> M <sub>pl.z</sub> , 31180 mm <sup>3</sup> I <sub>y</sub> , 8342621 mm <sup>4</sup>			
Both flanges have lateral rest	raint at supports	only					
Consider Combination 1 - 1.	.35G + 1.5Q + 1	.5RQ (Strength)					
Consider Combination 1 - 1. Classification of cross secti							
		5.5	l/mm² / fy] = <b>0.92</b>				
	ions - Section &	<b>5.5</b> ε = √[235 Ν	-				
Classification of cross secti	ions - Section &	5.5 ε = √[235 Ν ding and compre c = d = 121	ssion - Table 5.: I.8 mm	2 (sheet 1 of 3)			
Classification of cross secti Internal compression parts	ions - Section &	$ε = \sqrt{235}$ N ding and compre c = d = 121 α = min([h])	<b>ssion - Table 5.</b> I. <b>8</b> mm / 2 + N <sub>Ed</sub> / (2 × t <sub>w</sub>	2 (sheet 1 of 3) y × fy) - (t <sub>f</sub> + r)] / c	c, 1) = <b>0.515</b>		
Classification of cross secti Internal compression parts Width of section	ions - Section & subject to ben	$ε = \sqrt{235}$ N ding and compre c = d = 121 α = min([h])	ssion - Table 5.: I.8 mm	2 (sheet 1 of 3) y × fy) - (t <sub>f</sub> + r)] / c	c, 1) = <b>0.515</b>		
Classification of cross secti Internal compression parts Width of section Outstand flanges - Table 5.2	ions - Section & subject to ben	$ε = \sqrt{235}$ N ding and compre c = d = 121 α = min([h + c + fw]) c + fw = 27.1	ssion - Table 5 I.8 mm / 2 + Ν <sub>Ed</sub> / (2 × t <sub>w</sub> 1 = 29.3 × ε <= 39	2 (sheet 1 of 3) • × fy) - (tr + r)] / c 96 × ε / (13 × α •	c, 1) = <b>0.515</b>		
Classification of cross secti Internal compression parts Width of section	ions - Section & subject to ben	$ε = \sqrt{235}$ N ding and compre c = d = 121 $α = min([h + c + t_w - 27.1])$ $c = (b - t_w - 27.1]$	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × tw 1 = 29.3 × ε <= 39 2 × r) / 2 = 34.5	2 (sheet 1 of 3) • × fy) - (tr + r)] / c 96 × ε / (13 × α • mm	e, 1) = <b>0.515</b> - 1) Clas		
Classification of cross secti Internal compression parts Width of section Outstand flanges - Table 5.2	ions - Section & subject to ben	$ε = \sqrt{235}$ N ding and compre c = d = 121 $α = min([h + c + t_w - 27.1])$ $c = (b - t_w - 27.1]$	ssion - Table 5 I.8 mm / 2 + Ν <sub>Ed</sub> / (2 × t <sub>w</sub> 1 = 29.3 × ε <= 39	2 (sheet 1 of 3) • × fy) - (tr + r)] / c 96 × ε / (13 × α • mm	s, 1) = <b>0.515</b> - 1) Clas	ss 1	
Classification of cross secti Internal compression parts Width of section Outstand flanges - Table 5.2	ions - Section & subject to bend ? (sheet 2 of 3)	$ε = \sqrt{235}$ N ding and compre c = d = 121 $α = min([h + c + t_w - 27.1])$ $c = (b - t_w - 27.1]$	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × tw 1 = 29.3 × ε <= 39 2 × r) / 2 = 34.5	2 (sheet 1 of 3) • × fy) - (tr + r)] / c 96 × ε / (13 × α • mm	s, 1) = <b>0.515</b> - 1) Clas	ss 1	
Classification of cross secti Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section	ions - Section & subject to bend ? (sheet 2 of 3)	$ε = \sqrt{235}$ N ding and compre c = d = 121 $α = min([h + c + t_w - 27.1])$ $c = (b - t_w - 27.1]$	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × t <sub>w</sub> 1 = 29.3 × ε <= 39 2 × r) / 2 = 34.5 = 4.8 × ε <= 9 × ε	2 (sheet 1 of 3) • × fy) - (tr + r)] / c 96 × ε / (13 × α • mm	s, 1) = <b>0.515</b> - 1) Clas	ss 1	
Classification of cross secti Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Sectio	ions - Section & subject to bend ? (sheet 2 of 3) on 6.2.4	$ε = \sqrt{235} \text{ N}$ $c = d = 121$ $α = min([h + a])$ $c = (b - tw - a)$ $c = (b - tw - b)$ $c + tf = 4.5 = 0$ $N_{Ed} = 4.2 \text{ k}$	ssion - Table 5. .8 mm / 2 + N <sub>Ed</sub> / (2 × t <sub>w</sub> 1 = 29.3 × ε <= 39 2 × r) / 2 = 34.5 = 4.8 × ε <= 9 × ε	2 (sheet 1 of 3) • × fy) - (tr + r)] / c 96 × ε / (13 × α • mm : Class 1	s, 1) = <b>0.515</b> - 1) Clas	ss 1	
Classification of cross secti Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force	ions - Section & subject to bend ? (sheet 2 of 3) on 6.2.4	5.5 $\varepsilon = \sqrt{235}$ N ding and compre c = d = 121 $\alpha = min([h + a + b])$ $c / t_w = 27.1$ $c = (b - t_w - b)$ $c / t_f = 4.5 = 0$ NEd = 4.2 k N <sub>c,Rd</sub> = N <sub>PI,R</sub> NEd / N <sub>c,Rd</sub> = N <sub>PI,R</sub>	ssion - Table 5. 8 mm / 2 + N <sub>Ed</sub> / (2 × tw 1 = 29.3 × ε <= 39 2 × r) / 2 = 34.5 = 4.8 × ε <= 9 × ε N Rd = A × fy / γmo = = 0.008	2 (sheet 1 of 3) × fy) - (tr + r)] / c 96 × ε / (13 × α - mm Class 1 558.8 kN	s, 1) = <b>0.515</b> - 1) Clas Se	ss 1 ction is clas	
Classification of cross secti Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force Design resistance of section -	ions - Section & subject to bend ? (sheet 2 of 3) on 6.2.4 eq 6.10	5.5 $\varepsilon = \sqrt{235}$ N ding and compre c = d = 121 $\alpha = min([h + c + c + c + c + c + c + c + c + c + $	ssion - Table 5. 8 mm / 2 + N <sub>Ed</sub> / (2 × tw 1 = 29.3 × ε <= 39 2 × r) / 2 = 34.5 = 4.8 × ε <= 9 × ε N Rd = A × fy / γM0 = = 0.008 n compression	2 (sheet 1 of 3) × fy) - (tr + r)] / c 96 × ε / (13 × α - mm Class 1 558.8 kN	s, 1) = <b>0.515</b> - 1) Clas Se	ss 1 ction is clas	
Classification of cross secti Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force Design resistance of section - Slenderness ratio for y-y axi	ions - Section & subject to bend ? (sheet 2 of 3) on 6.2.4 eq 6.10	5.5 $ε = \sqrt{235}$ N ding and compre c = d = 121 α = min([h + a + b]) c + tw = 27.1 c = (b - tw - b) c + tw = 27.1 c = (b - tw - b) c + tw = 27.1 c = (b - tw - b) c + tw = 27.1 c = (b - tw - b) c + tw = 27.1 c = (b - tw - b) c + tw = 27.1 c = (b - tw - b) c + tw = 27.1 c = (b - tw - b) c + tw = 27.1 c = (b - tw - b) c + tw = 27.1 c = (b - tw - b) c + tw = 27.1 c = (b - tw - b) c + tw = 27.1 c = (b - tw - b) c + tw = 27.1 c = (b - tw - b) c + tw - b c + tw = 27.1 c = (b - tw - b) c + tw = 27.1 c = (b - tw - b) c + tw = 27.1 c = (b - tw - b) c + tw - b c + tw - b	ssion - Table 5. 8 mm / 2 + N <sub>Ed</sub> / (2 × tw 1 = 29.3 × ε <= 39 2 × r) / 2 = 34.5 = 4.8 × ε <= 9 × ε N Rd = A × fy / γM0 = = 0.008 n compression 3.1.3	2 (sheet 1 of 3) × fy) - (tr + r)] / c 96 × ε / (13 × α - mm Class 1 558.8 kN	s, 1) = <b>0.515</b> - 1) Clas Se	ss 1 ction is clas	
Classification of cross secti Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force Design resistance of section - Slenderness ratio for y-y axi Critical buckling length	ions - Section & subject to bend ? (sheet 2 of 3) on 6.2.4 eq 6.10	5.5 $\varepsilon = \sqrt{235}$ N ding and compre c = d = 121 $\alpha = min([h], c - tw = 27.1]$ c = (b - tw - c - tr = 4.5 = -1) NEd = 4.2 k Nc,Rd = NpI,R NEd / Nc,Rd = -1) PASS - Design Cling - Section 6.3 Lor,y = Lm4_s	ssion - Table 5. 8 mm / 2 + N <sub>Ed</sub> / (2 × tw 1 = 29.3 × ε <= 39 2 × r) / 2 = 34.5 = 4.8 × ε <= 9 × ε N Rd = A × fy / γM0 = = 0.008 n compression 3.1.3 1 = 2766 mm	2 (sheet 1 of 3) × fy) - (tr + r)] / c 96 × ε / (13 × α mm : Class 1 558.8 kN <i>resistance exc</i>	s, 1) = <b>0.515</b> - 1) Clas Se	ss 1 ction is clas	
Classification of cross secti Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force Design resistance of section - Slenderness ratio for y-y axi Critical buckling length Critical buckling force	ions - Section & subject to bend ? (sheet 2 of 3) on 6.2.4 eq 6.10 is flexural buck	5.5 $\varepsilon = \sqrt{235}$ N ding and compre c = d = 121 $\alpha = min([h, c - tw = 27.1])$ c = (b - tw - c - tw = 27.1] c = (b - tw = 27.1] c = (	ssion - Table 5.: 1.8 mm $/ 2 + N_{Ed} / (2 \times t_w)$ $1 = 29.3 \times \varepsilon <= 39$ $2 \times r) / 2 = 34.5$ $= 4.8 \times \varepsilon <= 9 \times \varepsilon$ N Rd = A × fy / $\gamma_{MO} =$ = 0.008 n compression 3.1.3 1 = 2766 mm $E \times ly / L_{cr,y^2} = 22$	2 (sheet 1 of 3) 9 × fy) - (tr + r)] / c 96 × ε / (13 × α - mm Class 1 558.8 kN resistance exc 260.1 kN	s, 1) = <b>0.515</b> - 1) Clas Se	ss 1 ction is clas	
Classification of cross secti Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force Design resistance of section - Slenderness ratio for y-y axi Critical buckling length Critical buckling force Slenderness ratio for buckling	ions - Section & subject to bend ? (sheet 2 of 3) on 6.2.4 eq 6.10 is flexural buck	5.5 $\varepsilon = \sqrt{[235 N]}$ ding and compre $c = d = 121$ $\alpha = min([h])$ $c / tw = 27.1$ $c = (b - tw - c) / tr = 4.5 = 0$ $N_{Ed} = 4.2 k$ $N_{c,Rd} = N_{pI,R}$ $N_{Ed} / N_{c,Rd} = 0$ $PASS - Designed Comparison (Comparison $	ssion - Table 5.: 1.8 mm $/ 2 + N_{Ed} / (2 \times t_w)$ $1 = 29.3 \times \varepsilon <= 39$ $2 \times r) / 2 = 34.5$ $= 4.8 \times \varepsilon <= 9 \times \varepsilon$ N Rd = A × fy / $\gamma_{MO} =$ = 0.008 $n \ compression$ 3.1.3 1 = 2766 mm $E \times ly / L_{cr,y}^2 = 222$ fy / N <sub>cr,y</sub> ) = 0.497	2 (sheet 1 of 3) 9 × fy) - (tr + r)] / c 96 × ε / (13 × α - mm Class 1 558.8 kN resistance exc 260.1 kN	s, 1) = <b>0.515</b> - 1) Clas Se	ss 1 ction is clas	
Classification of cross secti Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force Design resistance of section - Slenderness ratio for y-y axi Critical buckling length Critical buckling force Slenderness ratio for buckling Check y-y axis flexural buck	ions - Section & subject to bend ? (sheet 2 of 3) on 6.2.4 eq 6.10 is flexural buck	5.5 $\varepsilon = \sqrt{235}$ N ding and compre c = d = 121 $\alpha = min([h], c / tw = 27.1]$ c = (b - tw - c / tw = 27.1] c = (b - tw - c / tr = 4.5 = 0.5] NEd = 4.2 k N <sub>c,Rd</sub> = N <sub>pl,R</sub> NEd / N <sub>c,Rd</sub> = PASS - Design Kling - Section 6.3 L <sub>cr,y</sub> = L <sub>m4_s</sub> N <sub>cr,y</sub> = $\pi^2 \times \overline{\lambda}_y = \sqrt{(A \times b^2)}$	ssion - Table 5.: 1.8 mm $/ 2 + N_{Ed} / (2 \times t_w)$ $1 = 29.3 \times \varepsilon <= 39$ $2 \times r) / 2 = 34.5$ $= 4.8 \times \varepsilon <= 9 \times \varepsilon$ N Rd = A × fy / $\gamma_{MO} =$ = 0.008 $n \ compression$ 3.1.3 1 = 2766 mm $E \times ly / L_{cr,y}^2 = 222$ fy / N <sub>cr,y</sub> ) = 0.497	2 (sheet 1 of 3) 9 × fy) - (tr + r)] / c 96 × ε / (13 × α - mm Class 1 558.8 kN resistance exc 260.1 kN	s, 1) = <b>0.515</b> - 1) Clas Se	ss 1 ction is clas	
Classification of cross secti Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force Design resistance of section - Slenderness ratio for y-y axi Critical buckling length Critical buckling force Slenderness ratio for buckling Check y-y axis flexural buck Buckling curve - Table 6.2	ions - Section & subject to bend ? (sheet 2 of 3) on 6.2.4 eq 6.10 is flexural buck - eq 6.50 cling resistance	5.5 $\varepsilon = \sqrt{235}$ N ding and compre c = d = 121 $\alpha = min([h, c - tw = 27.1])$ c = (b - tw - c - tw = 27.1] c = (b - tw - c - tw = 27.1] c = (b - tw - c - tw = 27.1] c = (b - tw - c - tw = 27.1] c = (b - tw - c - tw = 27.1] c = (b - tw - c - tw = 27.1] c = (b - tw - c - tw = 27.1] c = (b - tw - c - tw = 27.1] c = (b - tw - c - tw = 27.1] c = (b - tw - c - tw = 27.1] c = (b - tw - c - tw = 27.1] c = (b - tw - c - tw = 27.1] c = (b - tw - c - tw = 27.1] c = (b - tw - c - tw = 27.1] c = (b - tw - c - tw = 27.1] c = (b - tw - c - tw - c - tw = 27.1] c = (b - tw - c - tw - c - tw - tw = 27.1] c = (b - tw - c - tw - c - tw - tw = 27.1] c = (b - tw - c - tw - tw - tw = 27.1] c = (b - tw - c - tw - tw - tw - tw = 27.1] c = (b - tw - c - tw - tw - tw - tw - tw - tw	ssion - Table 5.: 1.8 mm $/ 2 + N_{Ed} / (2 \times t_w)$ $1 = 29.3 \times \varepsilon <= 39$ $2 \times r) / 2 = 34.5$ $= 4.8 \times \varepsilon <= 9 \times \varepsilon$ N Rd = A × fy / $\gamma_{MO} =$ = 0.008 $n \ compression$ 3.1.3 1 = 2766 mm $E \times ly / L_{cr,y}^2 = 222$ fy / N <sub>cr,y</sub> ) = 0.497	2 (sheet 1 of 3) 9 × fy) - (tr + r)] / c 96 × ε / (13 × α - mm Class 1 558.8 kN resistance exc 260.1 kN	s, 1) = <b>0.515</b> - 1) Clas Se	ss 1 ction is clas	
Classification of cross secti Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force Design resistance of section - Slenderness ratio for y-y axi Critical buckling length Critical buckling force Slenderness ratio for buckling Check y-y axis flexural buck Buckling curve - Table 6.2 Imperfection factor - Table 6.1	ions - Section & subject to bend 2 (sheet 2 of 3) on 6.2.4 eq 6.10 is flexural buck - eq 6.50 kling resistance	5.5 $\varepsilon = \sqrt{[235 N]}$ ding and compre $c = d = 121$ $\alpha = min([h], \alpha)$ $c / tw = 27.1$ $c = (b - tw - \alpha)$ $c = (b - tw - \alpha)$ $c / tr = 4.5 = 0$ NEd = 4.2 k Nc,Rd = NpI,R NEd / Nc,Rd = 0 $PASS - Designering = 0$ cling - Section 6.3 Lor,y = $\pi^2 \times \alpha$ $\overline{\lambda}_y = \sqrt{(A \times a)}$ $\alpha_y = 0.21$	ssion - Table 5.: 1.8 mm $/ 2 + N_{Ed} / (2 \times t_w)$ $1 = 29.3 \times \varepsilon <= 39$ $2 \times r) / 2 = 34.5$ $= 4.8 \times \varepsilon <= 9 \times \varepsilon$ N Red = A × fy / YMO = = 0.008 n compression 3.1.3 1 = 2766 mm $E \times l_y / L_{cr,y}^2 = 22$ fy / Ncr,y) = 0.497	2 (sheet 1 of 3) • × fy) - (tr + r)] / c 96 × ε / (13 × α + mm • Class 1 558.8 kN resistance exc 60.1 kN	e, 1) = 0.515 - 1) Clas Se	ss 1 ction is clas	
Classification of cross section Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force Design resistance of section - Slenderness ratio for y-y axi Critical buckling length Critical buckling force Slenderness ratio for buckling Check y-y axis flexural buck Buckling curve - Table 6.2 Imperfection factor - Table 6.1	ions - Section & subject to bend 2 (sheet 2 of 3) on 6.2.4 eq 6.10 is flexural buck a - eq 6.50 cling resistance	5.5 $\varepsilon = \sqrt{[235 N]}$ ding and compre $c = d = 121$ $\alpha = min([h], c / tw = 27.1]$ $c = (b - tw - c / tr = 4.5 = 0.5]$ $N_{Ed} = 4.2 k$ $N_{c,Rd} = N_{pl,R}$ $N_{Ed} / N_{c,Rd} = 0.5$ $L_{cr,y} = L_{m4_s}$ $N_{cr,y} = \pi^2 \times \overline{\lambda}_y = \sqrt{(A \times b^2)}$ $A_y = 0.21$ $\varphi_y = 0.5 \times (0.5)$	ssion - Table 5.: 1.8 mm $/ 2 + N_{Ed} / (2 \times t_w)$ $1 = 29.3 \times \varepsilon <= 39$ $2 \times r) / 2 = 34.5$ $= 4.8 \times \varepsilon <= 9 \times \varepsilon$ N Rd = A × fy / $\gamma_{M0} =$ = 0.008 $n \ compression$ 3.1.3 1 = 2766 mm $E \times ly / L_{cr,y}^2 = 222$ fy / N <sub>cr,y</sub> ) = 0.497 1 $1 + \alpha_y \times (\overline{\lambda}_y - 0.2)$	2 (sheet 1 of 3) $x  imes f_y) - (t_f + r)] / c_y$ 96 × $\varepsilon$ / (13 × $\alpha$ - $mm$ c Class 1 558.8 kN resistance exc 260.1 kN (2) + $\overline{\lambda}y^2$ ) = 0.65	e, 1) = 0.515 - 1) Clas Se	ss 1 ction is clas	
Classification of cross secti Internal compression parts Width of section Outstand flanges - Table 5.2 Width of section Check compression - Section Design compression force Design resistance of section - Slenderness ratio for y-y axi Critical buckling length Critical buckling force Slenderness ratio for buckling Check y-y axis flexural buck Buckling curve - Table 6.2 Imperfection factor - Table 6.1	ions - Section & subject to bend ? (sheet 2 of 3) on 6.2.4 eq 6.10 is flexural buck l - eq 6.50 (ling resistance l tion factor 6.49	5.5 $\varepsilon = \sqrt{[235 \text{ N}]}$ ding and compre $c = d = 121$ $\alpha = \min([h], \alpha = 12]$ $c = (b - tw - 12)$ $c = (b$	ssion - Table 5.: 1.8 mm $/ 2 + N_{Ed} / (2 \times t_w)$ $1 = 29.3 \times \varepsilon <= 39$ $2 \times r) / 2 = 34.5$ $= 4.8 \times \varepsilon <= 9 \times \varepsilon$ N Red = A × fy / YMO = = 0.008 n compression 3.1.3 1 = 2766 mm $E \times l_y / L_{cr,y}^2 = 22$ fy / Ncr,y) = 0.497	2 (sheet 1 of 3) $x  imes f_y) - (t_f + r)] / c_y^2$ $96  imes \epsilon / (13  imes \alpha + c_y^2)$ mm 558.8 kN resistance exc 260.1 kN (2) + $\overline{\lambda}y^2$ ) = 0.65 (2) + $\overline{\lambda}y^2$ ) = 0.65	e, 1) = 0.515 - 1) Clas Se	ss 1 ction is clas	

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Structural Calculation Ltd Mobbs Wood Farm	Calcs for	Portal	Frame		Start page no./F	Revision 45			
Ansty CV7 9JN	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved d			
	AM	10/01/2020	JP	10/01/2020	JP	10/01/20			
		PASS - D	esign bucklin	g resistance exc	ceeds design	compress			
Slenderness ratio for z-z axi	s flexural buc	kling - Section 6.3	3.1.3						
Critical buckling length		$L_{cr,z} = L_{m4_s}$	1_seg1 = <b>2766</b> m	ım					
Critical buckling force		$N_{cr,z} = \pi^2 \times$	$E \times I_z / L_{cr,z^2} =$	<b>243.1</b> kN					
Slenderness ratio for buckling	- eq 6.50	$\overline{\lambda}_z = \sqrt{(A \times A)^2}$	fy / Ncr,z) = 1.5	16					
Check z-z axis flexural buck	ling resistanc	e - Section 6.3.1.1							
Buckling curve - Table 6.2		b							
Imperfection factor - Table 6.1		αz = <b>0.34</b>							
Buckling reduction determinat	ion factor			$(0.2) + \overline{\lambda}z^2) = 1.87$	73				
Buckling reduction factor - eq	6.49	χz = min(1 /	$/(\phi_z + \sqrt{\phi_z^2} - 2)$	λ̄z²)), 1) = <b>0.336</b>					
Design buckling resistance - e	q 6.47	$N_{b,z,Rd} = \chi_z$	$\times$ A $\times$ fy / $\gamma_{M1}$ =	188 kN					
		NEd / Nb,z,Rd							
		PASS - D	esign bucklin	g resistance exc	eeds design:	compress			
Check torsional and torsion	al-flexural buc	-							
Torsional buckling length			1_seg1_R = 2766	mm					
Distance from shear centre to	•	•							
Distance from shear centre to	centroid in z ax								
Radius of gyration		( <sup>1</sup>	$i_0 = \sqrt{(i_y^2 + i_z^2)} = 67.4 \text{ mm}$ N <sub>cr,T</sub> = 1 / $i_0^2 \times (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) = 912.3 \text{ kN}$						
Elastic critical torsional bucklir	ng force			$\times E \times I_w / L_{cr,T^2} =$	912.3 KN				
Torsion factor		β <sub>T</sub> = 1 - (y <sub>0</sub>	$(10)^2 = 1$						
Elastic critical torsional-flexura	-		~/[(1 N -	$(\mathbf{N})^2 + \mathbf{A} + \mathbf{A}$	$i_{\rm A}$ ) <sup>2</sup> · · N = / N	11 - 012 3			
Elastic critical buckling force	,1F – INcr,y / (Z ×	$\beta$ T) × [1 + N <sub>cr,T</sub> / N <sub>c</sub>	r,y - V[(I - Ncr,I) Icr,T, Ncr,TF) = 91		$10) \times INCr, 1 / INC$	or,y]] – 912.3			
Slenderness ratio for torsional	buckling - eg f	•	$f_y / N_{cr} = 0.78$						
		-	-						
<b>Design resistance for torsio</b> Buckling curve - Table 6.2	nai and torsio	b	ing - Section	0.3.1.1					
Imperfection factor - Table 6.1		ατ = <b>0.34</b>							
Buckling reduction determinat	on factor		1 + ατ × ( λ̄τ -	$(0.2) + \overline{\lambda} \tau^2) = 0.9$	05				
Buckling reduction factor - eq				$\bar{\lambda}_{T^2}$ ), 1) = <b>0.735</b>					
Design buckling resistance - e			$\times A \times f_y / \gamma_{M1} =$						
6 6		NEd / Nb,T,Rd							
		PASS - D	esign bucklin	g resistance exc	eeds design:	compress			
Check design at start of spa	n								
Check shear - Section 6.2.6	_								
Height of web		h <sub>w</sub> = h - 2 ×	tf = <b>137</b> mm	η = <b>1.0</b>	00				
J			.4 = 32.9 × ε /	•					
				Shear buckling	resistance c	an be iqnc			
Design shear force		V <sub>y,Ed</sub> = 3.9	kN	3		5			
Shear area - cl 6.2.6(3)				t <sub>w</sub> + 2 × r) × t <sub>f</sub> , η ×	h <sub>w</sub> × t <sub>w</sub> ) = <b>818</b>	<b>3</b> mm <sup>2</sup>			
Design shear resistance - cl 6	.2.6(2)		-	√(3)) / γ <sub>M0</sub> = <b>129.8</b>	-				
		V <sub>y,Ed</sub> / V <sub>c,y,R</sub>							
		PAS	S - Design sh	ear resistance e	xceeds desig	gn shear fo			
Check bending moment - Se	ction 6.2.5								
Design bending moment		M <sub>y,Ed</sub> = 6.5	kNm						

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Structural Calculation Ltd	Calcs for				Start page no./R	Revision			
Mobbs Wood Farm		Portal	Frame			46			
Ansty CV7 9JN	Calcs by AM	Calcs date 10/01/2020	Checked by JP	Checked date 10/01/2020	Approved by JP	Approved dat 10/01/202			
Design bending resistance mo	ment - eg 6.13	$M_{c.v.Rd} = M_{p}$	$I_{v,Rd} = W_{pl,v} \times f_{v}$	/ γ <sub>M0</sub> = <b>33.9</b> kNm					
0 0		My,Ed / Mc,y,F							
	PASS -	Design bendi	ng resistance	moment exceed	s design ben	ding mome			
Slenderness ratio for lateral	torsional bucklin	g							
Correction factor - For cantilev		kc = <b>1</b>							
		$C_1 = 1 / k_c^2$	= 1						
Poissons ratio		v = 0.3							
Shear modulus		G = E / [2 >	(1 + v)] = <b>807</b>	<b>69</b> N/mm²					
Unrestrained effective length		L = 1.0 × L	m4_s1_seg1_T = <b>27</b>	<b>66</b> mm					
Elastic critical buckling momer	nt	$M_{cr} = C_1 \times T_2$	$\tau^2 \times E \times I_z / L^2 >$	$<\sqrt{(I_w / I_z + L^2 \times G)}$	$\times$ It / ( $\pi^2$ $\times$ E $\times$	lz)) = <b>31.8</b>			
		kNm							
Slenderness ratio for lateral to	rsional buckling	$\overline{\lambda}_{LT} = \sqrt{W}$	$p_{I.y} \times f_y / M_{cr}) = $	1.033					
Limiting slenderness ratio		$\overline{\lambda}_{LT,0} = 0.4$							
			$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0} - La$	teral torsional b	uckling cann	ot be ignoi			
Check buckling resistance -	Section 6.3.2.1								
Buckling curve - Table 6.5		b							
Imperfection factor - Table 6.3	αLT = <b>0.34</b>	αLT = <b>0.34</b>							
Correction factor for rolled sec	tions	β = <b>0.75</b>							
LTB reduction determination fa	actor	$\phi_{LT} = 0.5 \times$	[1 + αιτ × ( λιτ	- λιτ,ο) + β × λιτ	<sup>2</sup> ] = <b>1.008</b>				
LTB reduction factor - eq 6.57		χ∟⊤ = min(1	/ [φLT + √(φLT <sup>2</sup> ·	$\cdot \beta \times \overline{\lambda} LT^2$ ], 1, 1 /	$\bar{\lambda}_{LT^2}$ ) = 0.679				
Modification factor		f = min(1 -	0.5 × (1 - kc) ×	[1 - 2 × ( λ̃LT - 0.8	B) <sup>2</sup> ], 1) = <b>1.000</b>	)			
Modified LTB reduction factor	- eq 6.58	χ <sub>LT,mod</sub> = mi	n(χ <sub>LT</sub> / f, 1, 1 /	$\bar{\lambda}_{LT^2}$ ) = 0.679					
Design buckling resistance mo	oment - eq 6.55	$M_{b,y,Rd} = \chi_L$	F,mod × W pl.y × $f_y$	/ γ <sub>M1</sub> = <b>23</b> kNm					
		My,Ed / Mb,y,I	Rd = <b>0.283</b>						
	PASS - I	Design buckli	ng resistance	moment exceed	s design ben	ding mome			
Check bending and axial for	ce - Section 6.2.9								
Bending and axial force check	- eq.6.33 & eq.6.3	4 N <sub>y,lim</sub> = min	$(0.25 \times N_{pl,Rd}, 0)$	$0.5  imes h_w  imes t_w  imes f_y$ / $\gamma$	<sub>(M0</sub> ) = <b>84.8</b> kN				
		NEd / Ny,lim =	= 0.05						
Allowance need not be m	ade for the effect	of the axial fo	rce on the pla	stic resistance i	moment abou	it the y-y a			
Check combined bending an	d compression -	Section 6.3.3							
Equivalent uniform moment fac	ctors - Table B.3	ψy = -4.856	kNm / 6.51 kN	lm = <b>-0.746</b>					
		α <sub>y</sub> = 1.002	kNm / 6.51 kN	m = <b>0.154</b>					
		C <sub>my</sub> = max(	$0.2 + 0.8 \times \alpha_{y}$ ,	0.4) = <b>0.400</b>					
		ψLT <b>= -4</b> .85	6 kNm / 6.51 k	Nm = <b>-0.746</b>					
		αlt = 1.002	kNm / 6.51 kN	lm = <b>0.154</b>					
		C <sub>mLT</sub> = max	(0.2 + 0.8 × αL	T, 0.4) = <b>0.400</b>					
				Table B 2					
Interaction factors k <sub>ij</sub> for mei	mbers susceptibl	e to torsional	deformations	- Table D.2					
Interaction factors k <sub>ij</sub> for mei Characteristic moment resistar			deformations <sub>y</sub> × f <sub>y</sub> = 33.9 kN						
	nce	$M_{y,Rk} = W_{pl}$		m					
Characteristic moment resista	nce nce	$M_{y,Rk} = W_{pl}$ $M_{z,Rk} = W_{pl}$	<sub>y</sub> × f <sub>y</sub> = <b>33.9</b> kN	m					
Characteristic moment resista	nce nce	$M_{y,Rk} = W_{pl.}$ $M_{z,Rk} = W_{pl.}$ $N_{Rk} = A \times f_{y}$	y × fy = <b>33.9</b> kN z × fy = <b>8.6</b> kNn y = <b>558.8</b> kN	m	у × <b>N</b> rk / үм1))	= 0.401			
Characteristic moment resistan Characteristic moment resistan Characteristic resistance to no	nce nce	$M_{y,Rk} = W_{pl.}$ $M_{z,Rk} = W_{pl.}$ $N_{Rk} = A \times f_{y}$ $k_{yy} = C_{my} \times$	y × fy = <b>33.9</b> kN z × fy = <b>8.6</b> kNn z = <b>558.8</b> kN (1 + min( λ̄y - 0	m n					
Characteristic moment resistan Characteristic moment resistan Characteristic resistance to no	nce nce rmal force	$M_{y,Rk} = W_{pl.}$ $M_{z,Rk} = W_{pl.}$ $N_{Rk} = A \times f_{y}$ $k_{yy} = C_{my} \times$ $k_{zy} = 1 - 0.1$	$y \times f_y = 33.9 \text{ kN}$ z × fy = 8.6 kNn z = 558.8 kN (1 + min( $\overline{\lambda}y - (1 + min(1, \overline{\lambda}z))$	m n D.2, 0.8) × Nεd / (χ	25) × χz × Nrk /	γм1) = <b>0.98</b>			

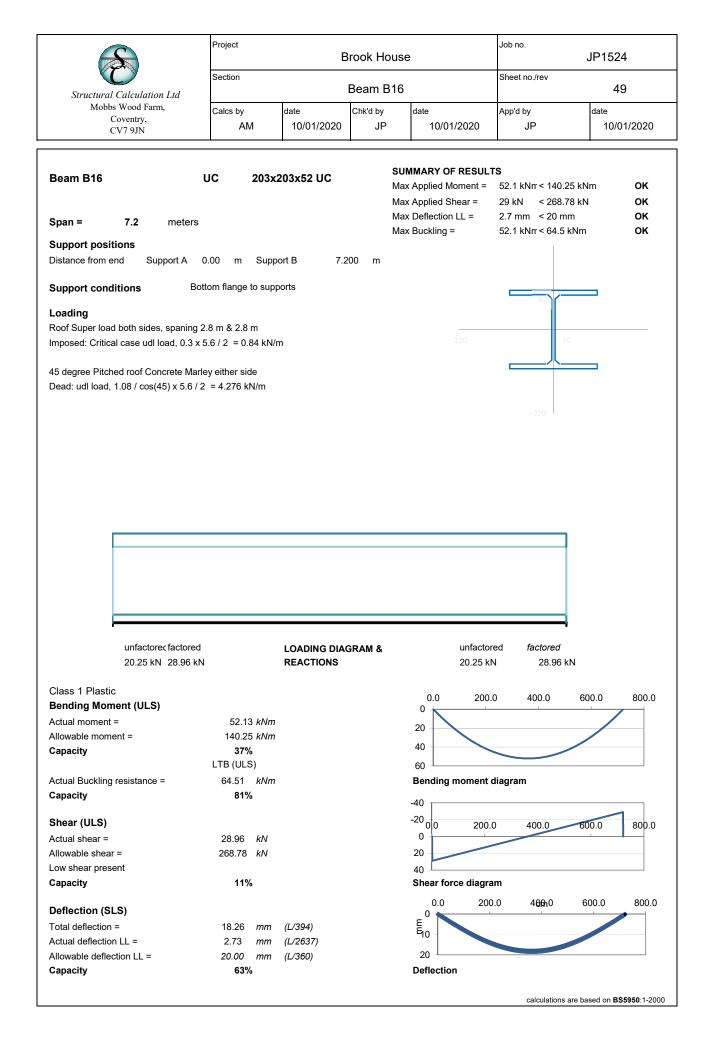
	Project	Brook	House		Job no. JP	1524	
Structural Calculation Ltd	Calcs for				Start page no./F	Revision	
Mobbs Wood Farm		Portal	Frame			47	
Ansty CV7 9JN	Calcs by AM	Calcs date 10/01/2020	Checked by JP	Checked date 10/01/2020	Approved by JP	Approved da	
		PASS - C	ombined ben	ding and compre	ssion check	s are satisfi	
Check design at end of span							
Check shear - Section 6.2.6							
Height of web			t <sub>f</sub> = <b>137</b> mm	η = <b>1.0</b>	00		
		h <sub>w</sub> / t <sub>w</sub> = 30	.4 = 32.9 × ε /	η < 72 × ε / η			
				Shear buckling	resistance c	an be ignor	
Design shear force		V <sub>y,Ed</sub> = <b>4.4</b>				2	
Shear area - cl 6.2.6(3)			$(t_w + 2 \times r) \times t_f, \eta \times$		s mm²		
Design shear resistance - cl 6.2.	0(2)			√(3)) / γ <sub>M0</sub> = <b>129.8</b>	KN		
		Vy,Ed / Vc,y,R PAS		near resistance e	vcoode docie	in choar fai	
Obsels benefities and the second	i	FAS	o - Design Sl	ical lesistalle e	กระระบอ นิยอไยู่	,,, silear 101	
Check bending moment - Sect	ion 6.2.5	M <sub>v.Ed</sub> = <b>4.9</b>	kNm				
Design bending moment Design bending resistance mom	opt og 6 12			- / 33 9 kNm			
Design bending resistance mom	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 33.9 \text{ kNm}$ $M_{y,Ed} / M_{c,y,Rd} = 0.143$						
	PASS			e moment exceed	s desian ber	ndina mome	
Slandarnass ratio for lateral to		-	5			3	
Slenderness ratio for lateral to Correction factor - For cantilever		kc = 1					
	boumo	$C_1 = 1 / k_c^2$	= 1				
Poissons ratio		v = 0.3					
Shear modulus		G = E / [2 >	(1 + v)] = <b>80</b>	<b>769</b> N/mm²			
Unrestrained effective length		-	m4_s1_seg1_B = 2				
Elastic critical buckling moment				$\times \sqrt{(I_w / I_z + L^2 \times G)}$	× It / ( $\pi^2$ × E ×	(Iz)) = <b>31.8</b>	
C C		kNm		,	,	,,	
Slenderness ratio for lateral tors	ional buckling	$\overline{\lambda}_{LT} = \sqrt{W}$	$p_{I.y} \times f_y / M_{cr}) =$	1.033			
Limiting slenderness ratio		$\overline{\lambda}_{LT,0} = 0.4$					
			$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0} - L$	ateral torsional b	uckling cann	ot be ignoi	
Check buckling resistance - S	ection 6.3.2.1						
Buckling curve - Table 6.5		b					
Imperfection factor - Table 6.3		αLT = <b>0.34</b>					
Correction factor for rolled section	ons	$\beta = 0.75$					
LTB reduction determination fac	tor	$\phi_{LT} = 0.5 \times$	[1 + αlt × ( λ̃l	T - $\overline{\lambda}$ LT,0) + β × $\overline{\lambda}$ LT	<sup>2</sup> ] = 1.008		
LTB reduction factor - eq 6.57		χ∟⊤ = min(1	/ [φlt + √(φlt²	- $\beta \times \overline{\lambda} LT^2$ ], 1, 1 /	λ <sub>LT<sup>2</sup></sub> ) = 0.679		
Modification factor		f = min(1 -	0.5 × (1 - k₀) ×	$1 - 2 \times (\overline{\lambda}_{LT} - 0.8)$	3) <sup>2</sup> ], 1) = <b>1.00</b>	)	
Modified LTB reduction factor - e	eq 6.58	$\chi_{LT,mod} = mi$	n(χ∟⊤ / f, 1, 1 /	λ <sub>LT</sub> <sup>2</sup> ) = <b>0.679</b>			
Design buckling resistance mom	ent - eq 6.55	$M_{b,y,Rd} = \chi_L$	$\Gamma_{mod} \times W_{pl.y} \times f_{pl.y}$	y / γ <sub>M1</sub> = <b>23</b> kNm			
		My,Ed / Mb,y,	Rd = 0.211				
	PASS	Design buckli	ng resistance	e moment exceed	s design ber	nding mom	
Check combined bending and	compression	- Section 6.3.3					
	ors - Table B.3	ψy = -4.856	6 kNm / 6.51 k	Nm = <b>-0.746</b>			
Equivalent uniform moment factor							
Equivalent uniform moment facto		α <sub>y</sub> = 1.002	kNm / 6.51 kN	lm = <b>0.154</b>			
Equivalent uniform moment facto			kNm / 6.51 kN 0.2 + 0.8 × $\alpha_{y_1}$				

	Project		Job no.			
C		Brook	House	JP1524		
Structural Calculation Ltd	Calcs for				Start page no./F	Revision
Mobbs Wood Farm		Porta	Frame			48
Ansty	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
CV7 9JN	AM	10/01/2020	JP	10/01/2020	JP	10/01/202
		αιτ = 1.002	2 kNm / 6.51 k	Nm = 0.154		
				LT, 0.4) = <b>0.400</b>		
nteraction factors k <sub>ij</sub> for m	embers suscep	tible to torsional	deformations	s - Table B.2		
Characteristic moment resist	ance	$M_{y,Rk} = W_{pl}$	y × fy = <b>33.9</b> kl	Nm		
Characteristic moment resist	ance	Mz,Rk = Wpl	z × fy = <b>8.6</b> kN	m		
Characteristic resistance to n	ormal force	$N_{Rk} = A \times f_{f}$	, = <b>558.8</b> kN			
nteraction factors		$k_{yy} = C_{my} \times$	(1 + min( λ <sub>y</sub> -	0.2, 0.8) $\times$ NEd / ( $\chi$	у × <b>N</b> rk / γм1))	= 0.401
		k <sub>zy</sub> = 1 - 0.1	$1 \times \min(1, \overline{\lambda}_z)$	× NEd / ((CmLT - 0.2	5) × χz × Nrk /	γ <sub>M1</sub> ) = <b>0.984</b>
nteraction formulae - eq 6.61	& eq 6.62	NEd / ( $\chi_y \times  $	Nrk / γм1) + kyy	$\times$ My,Ed / ( $\chi$ LT $\times$ My,F	кк / γм1) = <b>0.09</b>	3
		NEd / ( $\chi_z$ ×	Nrk / γm1) + kzy	$\times$ My,Ed / ( $\chi$ LT $\times$ My,F	к / γм1) = <b>0.23</b>	81
		PASS - C	ombined ben	ding and compre	ssion checks	s are satisfie

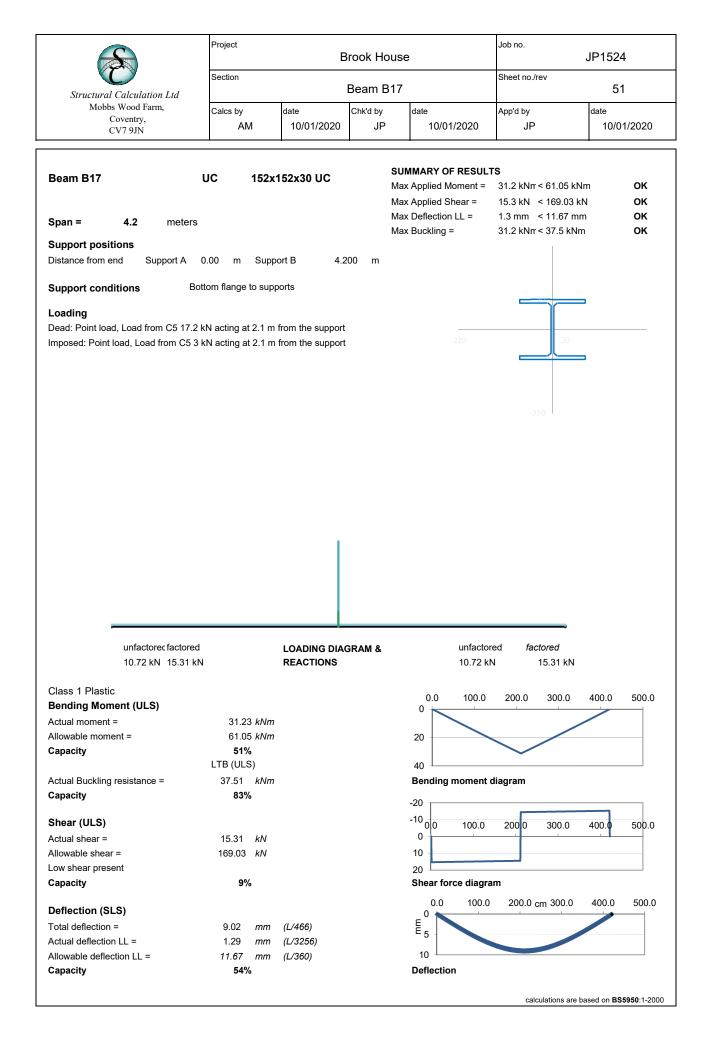
Check y-y axis deflection - Section 7.2.1 Maximum deflection

Allowable deflection

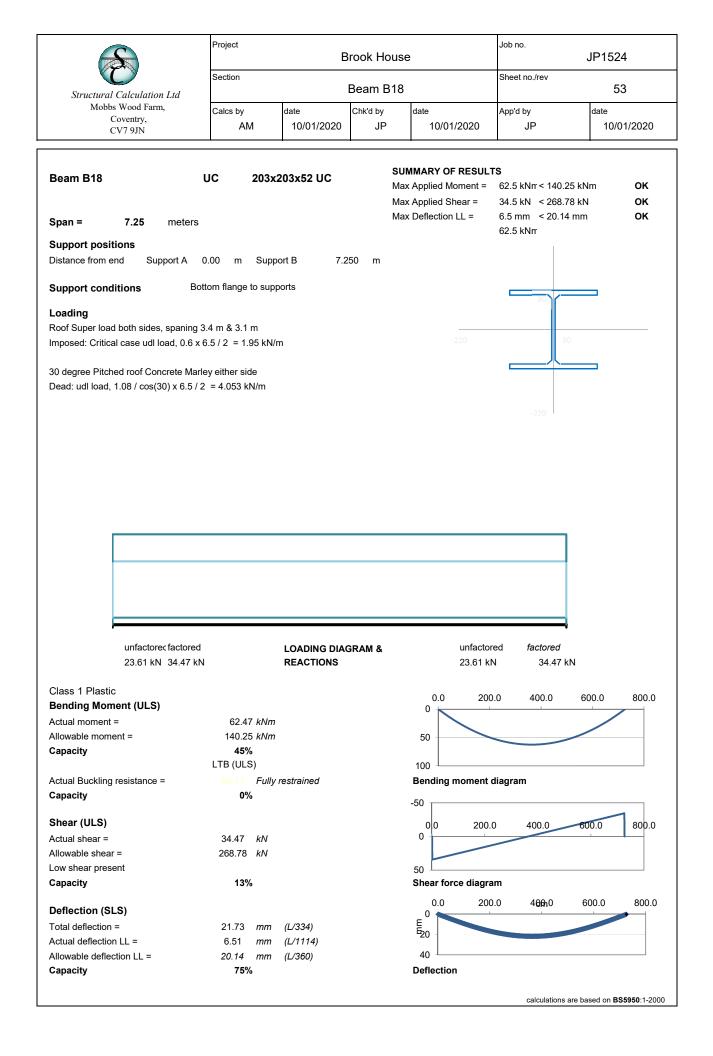
$$\begin{split} \delta_y &= \textbf{0.2 mm} \\ \delta_{y,\text{Allowable}} &= L_{\text{m4\_s1}} \ / \ 180 = \textbf{15.4 mm} \\ \delta_y \ / \ \delta_{y,\text{Allowable}} &= \textbf{0.014} \\ & \textbf{PASS - Allowable deflection exceeds design deflection} \end{split}$$



S	Project	В	rook House	e	Job no.	JP1524
Structural Calculation Ltd	Section	Bea	Sheet no./rev	50		
Mobbs Wood Farm, Coventry, CV7 9JN	Vood Farm, ventry, Calcs by date			<sup>date</sup> 10/01/2020	App'd by JP	date 10/01/2020
Beam End Reaction =	28.96	kN (factored	)	Variable Load Safe Permanent Load S	-	
Masonry						
Masonry type =	3.6N Blo	ckwork				
Characteristic strength of masonry =	2.6	N/mm²				
Width of beam end bearing =	204.3	mm		Bearing factor =	1.25	
Length of beam end bearing =	100	mm		γm =	3.00	
Stress						
Maximum Bearing Stress =	1.08	N/mm²				
Actual Bearing Stress =	1.42	N/mm²				
Capacity	131%		Padsto	ne Required		
Padstone						
A Engineering Brick						
Characteristic strength of padstone =	13.2	N/mm²				
Width of padstone =	100	mm				
Length of padstone =	440	mm				
Maximum bearing stress padstone =	5.50	N/mm²				
Stress under beam =	1.42	N/mm²				
Capacity	26%		ОК			
Maximum bearing stress masonry	1.08	N/mm²				
Stress under padstone =	0.66	N/mm²				
Capacity	61%		ок			



S	Project	В	rook House	e	Job no.	JP1524		
Structural Calculation Ltd	Section	Bea	ring Beam	B17	Sheet no./rev	52		
Mobbs Wood Farm, Coventry, CV7 9JN			Chk'd by JP	date 10/01/2020	App'd by JP	date 10/01/2020		
Beam End Reaction =	15.31	kN (factored	)	Variable Load Safe Permanent Load S	-			
Masonry								
Masonry type =	3.6N Blo	ckwork						
Characteristic strength of masonry =	2.6	N/mm²						
Width of beam end bearing =	100	mm		Bearing factor =	1.25			
Length of beam end bearing =	100	mm		γm =	3.00			
Stress								
Maximum Bearing Stress =	1.08	N/mm²						
Actual Bearing Stress =	1.53	N/mm²						
Capacity	141%		Padsto	ne Required				
Padstone								
A Engineering Brick								
Characteristic strength of padstone =	13.2	N/mm²						
Width of padstone =	100	mm						
Length of padstone =	215	mm						
Maximum bearing stress padstone =	5.50	N/mm²						
Stress under beam =	1.53	N/mm²						
Capacity	28%		ОК					
Maximum bearing stress masonry	1.08	N/mm²						
Stress under padstone =	0.71	N/mm²						
Capacity	66%		ок					



	Project	В	rook House	9	Job no.	JP1524
Structural Calculation Ltd	Section	Bea	Sheet no./rev	54		
Mobbs Wood Farm, Coventry, CV7 9JN	Calcs by AM			date 10/01/2020	App'd by JP	date 10/01/2020
Beam End Reaction =	34.47	kN (factored)	)	Variable Load Safe Permanent Load S	-	
Masonry						
Masonry type =	3.6N Blo	ckwork				
Characteristic strength of masonry =	2.6	N/mm²				
Width of beam end bearing =	204.3	mm		Bearing factor =	1.25	
Length of beam end bearing =	100	mm		γm =	3.00	
Stress						
Maximum Bearing Stress =	1.08	N/mm²				
Actual Bearing Stress =	1.69	N/mm²				
Capacity	156%		Padsto	ne Required		
Padstone						
A Engineering Brick						
Characteristic strength of padstone =	13.2	N/mm²				
Width of padstone =	100	mm				
Length of padstone =	440	mm				
Maximum bearing stress padstone =	5.50	N/mm²				
Stress under beam =	1.69	N/mm²				
Capacity	31%		ОК			
Maximum bearing stress masonry	1.08	N/mm²				
Stress under padstone =	0.78	N/mm <sup>2</sup>				
Capacity	72%		ок			

S	Project	Brook House					JP1524		
Structural Calculation Ltd	Section		(	Column C2		Sheet no./rev	Sheet no./rev 55		
Mobbs Wood Farm,	Calcs by	date		Chk'd by	date	App'd by	date		
Coventry, CV7 9JN	AM		1/2020	JP	10/01/2020	JP	10/01/2020		
Column C2         SHS         120x120x6.3           Height =         2.500         meters				SUMMARY OF RESULTS         Nominal effective length ' $L_{Ex}$ / $L_{Ey}$ ' [mm] =       3000 / 3000         Compression resistance:       655 kN > 64 kN       OI         Buckling resistance:       33 kNm > 5 kNm       OI         Overall check:       0.52 < 1.00					
and venturing									
End restraint	atrained in dire	otion			300				
Fop x-x: Not held in position but res						at 1 = 46 kN			
Top y-y: Not held in position but res Btm. x-x: Effectively held in position						at 1 – 40 ki			
Btm. y-y: Effectively held in position									
Sun, y-y. Encouvery neid in position									
Effectice length factors									
For x-x axis 'L <sub>Fx-x</sub> ' = $1.20$									
For y-y axis 'L <sub>Fy-y</sub> ' = 1.20									
						n			
_oading			x-axis						
mposed Direct compression 'PL' [k	N] =								
Dead Direct compression 'PL' [k	N] =				65 D				
at 1 at 2	at 3	at 4				U			
Eccentricity 100	100								
L Reaction 13.6	6								
DL Reaction 17	5.9								
Applied moment about x-x axis 'M <sub>xx</sub> ' [ <i>k</i> /	-								
Applied moment about y-y axis 'M <sub>yy</sub> ' <i>[kl</i>	vmj =								
				Total vertical		at 3 = 18 kN			
				load:64 kN		at 5 – 10 ki			
					<sub>-300</sub> ∣y-axis	;			
Section is class 1 plastic				NOTE: Loads	s are factored.				
Steel Grade:	s	275		HOTE: Loud					
Resultant moment about x-x axis 'M <sub>xx</sub> '		.43							
Resultant moment about y-y axis 'M <sub>yy</sub> ' /		.93							
Compression resistance				Buckling resi	stance moment				
Reduced design strength 'p <sub>vr</sub> ' <i>[MPa]</i> =	Ν	I/A		Equivalent slen			27.1		
Slenderness for x-x axis ' $\lambda_x$ ' =		4.9		Parameter 'p <sub>E</sub> ' :			2763.9		
Slenderness for y-y axis 'λ <sub>y</sub> ' =		4.9		Robertson cons			7.0		
Strut curve for x-x axis:	а	)		Limiting equival	ent slenderness ' $\lambda_{L0}$ ' =		34.3		
Strut curve for y-y axis:	а	)		Perry factor $\eta_{LT}$	=		0.0		
Parameter 'p <sub>E</sub> ' =	4	79.8		Parameter $Ø_{LT}$			1519.4		
Robertson constant 'α' =	2	.0			h 'p <sub>b</sub> ' <i>[N/mm²]</i> =		275.0		
_imiting slenderness 'λ₀' =		7.2			g resistance 'M <sub>bs</sub> ' <i>[kNr</i>		33.0		
Perry factor 'ŋ' =		.10			t about x-x 'M <sub>xx</sub> ' <i>[kNm]</i>	=	4.4		
Parameter 'Ø' =		00.3		Capacity			13%		
Compressive strength ' $p_c$ ' [N/mm <sup>2</sup> ] =	2	32.1		<b>.</b>					
Based on y-y axis calculations	-	<b>F</b> 4 4		Bending abou			07.5		
Compression resistance $P_c'[kN] =$		54.4 2.4			tance 'M <sub>Ry</sub> ' <i>[kNm]</i> =	_	27.5		
Applied total vertical load 'F <sub>c</sub> ' [kN] =		3.4 ••4			t about y-y 'M <sub>yy</sub> ' <i>[kNm]</i>	-	7.9 20%		
Capacity	1	0%		Capacity			29%		

**Overall check** based on equation for 'simple structures'  $F_c / Pc + M_x / M_{bs} + M_y / (p_y * Z_y) = 64 / 655 + 5 / 33 + 8 / (275 * 100) = 0.10 + 0.13 + 0.29 = 0.10 + 0.29$ 

	Project	В	rook House		JP1524		
Structural Calculation Ltd	Section	(	Column C5		Sheet no./rev	56	
Mobbs Wood Farm,	Calcs by	date	Chk'd by	date	App'd by	date	
Coventry, CV7 9JN	AM	10/01/2020	JP	10/01/2020	JP	10/01/2020	
			SUMMARY OF R	ESULTS			
Column C5 SHS	90x90x	5	Nominal effective	length 'L <sub>Ex</sub> / L <sub>Ey</sub> ' [mm]	= 180	00 / 1800	
			Compression resi	stance: 416	kN > 29 kN	ок	
Height = 1.500 meters			Buckling resistance		Nm > 5 kNm	ОК	
			Overall check:	0.54	l < 1.00	OK	
End restraint				300			
op x-x: Not held in position but rest							
Fop y-y: Not held in position but rest Stm. x-x: Effectively held in position a					at 1 = 29 kN		
Btm. x-x:     Effectively held in position a       Btm. y-y:     Effectively held in position a							
in y-y. Encouvery held in position e		in direction					
Effectice length factors							
For x-x axis 'L <sub>Fx-x</sub> ' = 1.20							
For y-y axis $L_{Fy-y}$ = 1.20							
.oading							
mposed Direct compression 'PL' [kN	J] =	x-axis					
Dead Direct compression 'PL' [kN	-			-50			
at 1 at 2	at 3	at 4					
Eccentricity 100							
L Reaction 3							
DL Reaction 17.2							
Applied moment about x-x axis 'M <sub>xx</sub> ' <i>[kN</i> Applied moment about y-y axis 'M <sub>vv</sub> ' <i>[kN</i>							
	mg –						
			Total vertical				
			load:29 kN	y-axis			
				-200			
Section is class 1 plastic			NOTE: Loads a	are factored.			
Steel Grade: Resultant moment about x x axis 'M <i>' //</i> /	( <i>Nm</i> ] = 4.1						
Resultant moment about x-x axis 'M <sub>xx</sub> ' <i>[k</i> Resultant moment about y-y axis 'M <sub>w</sub> ' <i>[k</i>							
toounant moment about y y and myy [							
Compression resistance			Buckling resist			o	
Reduced design strength 'p <sub>yr</sub> ' [MPa] =	N/A		Equivalent slende	rness 'λ <sub>LT</sub> ':		21.7	
Slenderness for x-x axis 'λ <sub>x</sub> ' = Slenderness for y-y axis 'λ <sub>v</sub> ' =	52. 52.		Parameter 'p <sub>E</sub> ' = Robertson consta	nta –		4281.2 7.0	
Strut curve for x-x axis:	52. a)	2		t slenderness ' $\lambda_{L0}$ ' =		34.3	
Strut curve for y-y axis:	a) a)		Perry factor $\eta_{LT} =$			0.0	
Parameter 'p <sub>E</sub> ' =	743	3.3	Parameter $Ø_{LT}$ =			2278.1	
Robertson constant 'α' =	2.0		Bending strength	p <sub>b</sub> ' [N/mm <sup>2</sup> ] =		275.0	
imiting slenderness ' $\lambda_0$ ' =	17.	2		resistance 'M <sub>bs</sub> ' <i>[kNm</i> ]	1 =	14.6	
Perry factor 'ŋ' =	0.0			bout x-x 'M <sub>xx</sub> ' [kNm] =	:	4.2	
Parameter 'Ø' =	535		Capacity			29%	
Compressive strength 'p <sub>c</sub> ' [ <i>N/mm<sup>2</sup></i> ] =	248	3.8	Dending 1	V V			
Based on y-y axis calculations Compression resistance 'P <sub>c</sub> ' [kN] =	A 4 6	5	Bending about			10.1	
	415		Moment of resista	nce w <sub>Ry</sub> [kivm] =		12.1	
Applied total vertical load $F_c'$ [kN] =	28.	9	Applied moment of	bout y-y 'M <sub>yy</sub> ' [kNm] =	=	2.2	

**Overall check** based on equation for 'simple structures'  $F_c / Pc + M_x / M_{bs} + M_y / (p_y * Z_y) = 29 / 416 + 5 / 15 + 3 / (275 * 44) = 0.07 + 0.29 + 0.18 = 0.07 + 0.29 +$ 

S	Project	Broo	ok House		Job no.	JP1524	
Structural Calculation Ltd	Section	Masonry Colu	mn under B7		Sheet no./rev 57		
Malla Waad Fame	Calcs by AM	date 10/01/2020	Chk'd by JP	<sup>date</sup> 10/01/2020	App'd by JP	date 10/01/2020	

#### MASONRY DATA:

Masonry type:	Solid aggregate concrete b	locks laid flat
Compressive unit strength 'punit	' [ <i>N/mm</i> <sup>2</sup> ] =	7.3
As laid height of laid flat blocks	h <sub>unit</sub> ' [ <i>mm</i> ] =	100.0
Least horizontal dimension of b	locks 'l <sub>unit</sub> ' [ <i>mm</i> ] =	N/A
Mortar strength class/designati	on =	M4 (iii)
Characteristic compressive stre	ngth 'f <sub>k</sub> ' [ <i>N/mm</i> <sup>2</sup> ] =	4.10

## COLUMN DATA:

Column height 'H' [m] =	2.50
Column width 'b' [ <i>mm</i> ] =	440.0
Column thickness 't' [mm ] =	215.0

### LOADING:

Applied design vertical load on column 'PL' [kN] =	
Load eccentricity along column width $e_{x-b}$ [mm] =	0.0
Load eccentricity along column thickness $e_{x-t}$ [mm] =	0.0
Ratio $e_{x-b} / b =$	0.00
Ratio $e_{x-t} / t =$	0.00

### SLENDERNESS CHECK

Column slenderness in minor axis ' $\lambda_{minor}$ ' =	11.63
Column slenderness in major axis ' $\lambda_{major}$ ' =	5.68
Maximum slenderness 'λ <sub>max</sub> ' =	11.63
Maximum allowable slenderness ' $\lambda_{allowable}$ ' =	27.00
SLENDERNESS: OK	

#### MINOR AXIS

Eccentricity due to slenderness 'e <sub>a_minor</sub> ' =	8.89
Design eccentricity in the mid-height of column ${\rm 'e_{t\_minor'}}$ =	15.34
Total design eccentricity 'e <sub>m_minor</sub> ' =	15.34
Capacity reduction factor ' $\beta_{minor}$ ' =	0.94
Cap for capacity reduction factor $'\beta_{cap\_minor}' =$	1.00
Final capacity reduction factor $'\beta_{final_{minor}}' =$	0.94

#### MAJOR AXIS

Eccentricity due to slenderness 'e <sub>a_major</sub> ' =	
Design eccentricity in the mid-height of column ${\rm 'e_{t\_major'}}$ =	
Total design eccentricity 'e <sub>m_major</sub> ' =	13.20
Capacity reduction factor ' $\beta_{major}$ ' =	
Cap for capacity reduction factor $'\beta_{cap_major'}$ =	1.00
Final capacity reduction factor ' $\beta_{\text{final}_{\text{major}}}$ ' =	1.00

## DESIGN LOAD RESISTANCE

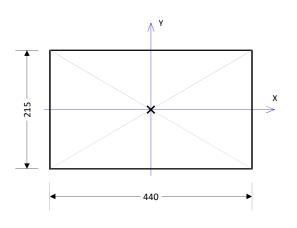
Characteristic compressive strength ' $f_k$ ' [ <i>N/mm</i> <sup>2</sup> ] =	4.10	
Partial safety factor for compression ' $\gamma_m$ ' =	3.50	
Minimum capacity reduction factor ' $\beta_{min}$ ' =	0.94	
Reduction factor for small plan area column ' $\gamma_{area}$ ' =	0.84	
Design vertical load resistance 'R <sub>d</sub> ' [kN] =	87.98	
Applied design vertical load on column 'PL' [kN ] =	56.70	
COLUMN DESIGN LOAD RESISTANCE: OK		
NOTE: all eccentricities are in 'mm'		

SUMMARY OF RESULTS:	
Slenderness check: 11.63 < 27.00	ОК
Column design vertical load resistance: 56.7 < 88.0	ОК

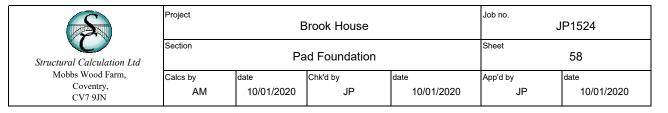
# PARTIAL SAFETY FACTORS:

Category II	Category of masonry units:
Normal	Category of construction control:

#### DIAGRAM



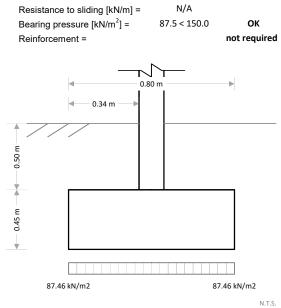
calculations are based on BS 5628-1: 2005



Foundation details				
Pad foundation type:		unre	inford	ed
Foundation width =		0.8	m	
Foundation thickness =		0.45	m	
Soil overburden depth =		0.5	m	
Load width =		0.12	m	
Load eccentricity =		0	m	
Concrete density =		24	kN/m	1 <sup>3</sup>
Characteristic strength of conci	rete =	30	N/mr	n²
Soil density =		20.5	kN/m	1 <sup>3</sup>
Loading	dead	impo	sed	wind
Partial safety factor ' $\gamma_{f}$ ' =	1.40	1.60		
	0.00			

Partial safety factor ' $\gamma_{f}$ ' =	1.40	1.60
Surcharge 'UDL' [kN/m <sup>2</sup> ] =	0.00	0.00
Axial load 'N' [kN] =	22.90	19.60
Horizontal load 'H' [kN] =	0.00	0.00
Moment load 'M' [kNm] =	0.00	0.00
Foundation s/w $[kN/m^2]$ =	10.80	-
Soil s/w $[kN/m^2] =$	10.25	-

# SUMMARY OF RESULTS Restoring moment [kNm/m] = Resistance to sliding [kN/m] =



N/A

Allowable bearing pressure =

<b>150</b> kN/m²	
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Unreinforced concrete

Reinfor	cement not required		
Min. un	reinforced foundation depth =	0.34	т
Average	pressure on the right =	87.46	kN/m²
Average	pressure on the left =	87.46	kN/m²

#### Overturning ~ erall factor

overtaining	
Overall factor of safety =	1.50
Overturning moment =	N/A
Restoring moment (foundation loads) =	N/A
Restoring moment (axial loads) =	N/A
Total restoring moment =	N/A

# Restoring moment: N/A

Sliding

onanig			
Overall factor of safety =	1.50		
Passive pressure coefficient =	N/A		
Total horizontal load =	N/A		
Resistance due to base friction =	N/A		
Resistance due to passive force =	N/A		
Total resistance to sliding =	N/A		
Sliding: N/A			
Bearing pressure			
Total base reaction =	87.46		
Effective eccentricity =	0		
Maximum base pressure =	87.46		
Minimum base pressure =	87.46		
Base reaction acts within middle third of base			
Bearing pressure: OK			

calculations are based on BS8110-1: 1997