

2 Brunel Avenue
Leicester
LE3 9BU

info@structuralcalcs.co.uk

www.structuralcalcs.co.uk

07598895578

325 Leicester Rd, Markfield, LE67 9RH

Front, Rear and Loft Extension

Bhavin Gulab – MSc – BEng – Civil and Structural Engineer

[Google Reviews](#)

Issue 2

Job No 2935

02/02/25

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1. Basis of structural design

All design has been carried out in accordance with the current British Standards (Eurocodes):

BS EN 1990:2002 Eurocode 1: Basis of Structural Design.

BS EN 1991-1-1:2002 Eurocode 1: Actions on structures. General actions – Densities, self-weight, imposed loads

BS EN 1991-1-4:2005 Eurocode 1: Actions on structures. General actions – Wind actions

BS EN 1992-1-1:2004 Eurocode 2: Design of concrete structures

BS EN 1993-1-1:2005 Eurocode 3: Design of steel structures

BS EN 1997-1:2004 Eurocode 7: Geotechnical design

These calculations should be read in conjunction with any relevant architectural details and drawings.

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2. Standard Loadings

Walls		
Internal Stud Wall Studding 2 * Plasterboard	0.20 kN/m ² 0.30 kN/m ²	WL 0.50 kN/m²
100 Lightweight Block Wall (Internal) Blockwork 2 x Plaster	1.00 kN/m ² 0.50 kN/m ²	WL 1.50 kN/m²
140 Lightweight Block Wall (Internal) Blockwork 2 x Plaster	1.40 kN/m ² 0.50 kN/m ²	WL 1.90 kN/m²
External Tile/Stud Wall Studding Plasterboard Battens, Felt and Insulation Tile Hanging	0.10 kN/m ² 0.15 kN/m ² 0.15 kN/m ² 0.60 kN/m ²	WL 1.00 kN/m²
External Brick/Block cavity Wall Brick 100 Lightweight Block Plaster	2.10 kN/m ² 1.00 kN/m ² 0.25 kN/m ²	WL 3.35 kN/m²
215 Dense Block/Brick Wall Blockwork 2 x Plasterboard	4.30 kN/m ² 0.30 kN/m ²	WL 4.60 kN/m²
275 Lightweight Block Cavity Wall 2 x 100 Blockwork 2 x Plasterboard	2.20 kN/m ² 0.30 kN/m ²	WL 2.50 kN/m²

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Roofs and Floors		
Sloping Roofs (on plan)		
Tiles	0.85 kN/m ²	DL 1.05 kN/m ²
Battens and felt	0.10 kN/m ²	LL 0.75 kN/m ²
Rafters	0.10 kN/m ²	WL 1.80 kN/m²
Live Load	0.75 kN/m ²	
Roof Space		
Joists and Insulation	0.10 kN/m ²	DL 0.25 kN/m ²
Soffit	0.15 kN/m ²	LL 0.25 kN/m ²
Live Load	0.25 kN/m ²	WL 0.50 kN/m²
Flat Timber Roofs		
Chippings and Felt	0.40 kN/m ²	DL 1.05 kN/m ²
Boards Joists and Firings	0.45 kN/m ²	LL 0.75 kN/m ²
Soffit and Insulation	0.20 kN/m ²	WL 1.8 kN/m²
Live Load	0.75 kN/m ²	
Timber Floors		
Boards and Joists	0.30 kN/m ²	DL 1.45 kN/m ²
Soffit	0.15 kN/m ²	LL 1.50 kN/m ²
Live Load	1.50 kN/m ²	WL 2.95 kN/m²
Partitions	1.00 kN/m ²	
PC Floors		
150 PCC Slab (200)	3.10 kN/m ²	DL 4.62 kN/m ²
Screed	1.40 kN/m ²	LL 1.50 kN/m ²
Insulation	0.07 kN/m ²	WL 6.12 kN/m²
Finishes	0.05 kN/m ²	
Live Load	1.50 kN/m ²	

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3. Construction notes

3.1 Construction notes

All assumed load bearing walls, foundations and structural elements are to be exposed and checked on site with the Building Control Officer, prior to the commencement of works for suitability. Trial holes are to be excavated prior to commencement of works and are to be inspected by the Building Control Officer/Soils specialist. All new foundations are to be founded off natural virgin ground.

Any deviation from the design assumptions is to be reported to the Architect/Engineer prior to commencement of works.

The existing building is assumed to be in good structural order and any defects are to be reported to the architect/engineer.

The temporary stability of the structure during all stages of the construction work is the responsibility of the contractor.

3.2 New works

All new steelwork to be grade S355, unless noted otherwise in the calculations. Steelwork to be thoroughly wire brushed and painted with two coats of zinc phosphate primer. All damaged areas of painted are to be touched up after erection of the steelwork. All bolts are to be grade 8.8 unless noted otherwise, and all welds are to be 6mm continuous fillet type, unless noted otherwise.

All new timber is assumed to be grade C16 (previously SC3), unless noted otherwise within the calculations.

All structural timber is to be suitably treated against decay and insect attack.

Timber wall plates are to be held down using 30x5mm thick galvanised mild steel vertical restraint straps. 1200mm long with flat return, at 2000mm max centres (and either side of wide windows). The strap should be fixed to the solid masonry wall using 5noxNo.12 screws in plugged holes if existing walls and built into the walls if new.

30x5mm thick galvanised mild steel lateral restraint straps, 1200mm long with 150mm upstand, are to be provided between external walls and floors at 2000mm max centres. Where straps are perpendicular to the span of the floor, they shall be taken across 3no joists, with solid timber noggin between. The straps upstand shall be fixed to the solid masonry wall using 5noxNo12 screws in plugged holes if existing walls and built into the walls if new.

Movement joints are to be provided in accordance with masonry manufacturers recommendations (approximately 6m centres in blockwork and 12m centres in brickwork). Concrete grades are to be noted within the calculations and placed, compacted, and protected in accordance with EC3. As a guide reinforced concrete to be C35 grade; non-reinforced concrete to be C25/C30 grade.

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All new masonry piers/walls are to be built as noted within the calculations and the architects/engineer's specifications.

3.3 General Notes

These calculations and details are to be read in conjunction with all relevant architects and engineers' drawings and specifications.

Full building regulation approval should be obtained prior to the commencement of works on site/before any materials are ordered (i.e. steelwork/timber etc). Any works carried out prior to this are undertaken at the clients/contractor's own risk.

The works are to be carried out to the approval and satisfaction of the building control officer, to accepted good building practice and with full compliance and in accordance with all relevant British Standards and Codes of Practice.

The lengths and spans used in these calculations are for design purposes only and should not be used as a basis for ordering materials/fabrication purposes. Note, the design span in these calculations is not the same as clear span.

All lengths and spans used in these calculations should be verified on site prior to commencement of any construction works. Contractor and/or steel fabricator to take their dimensions on site.

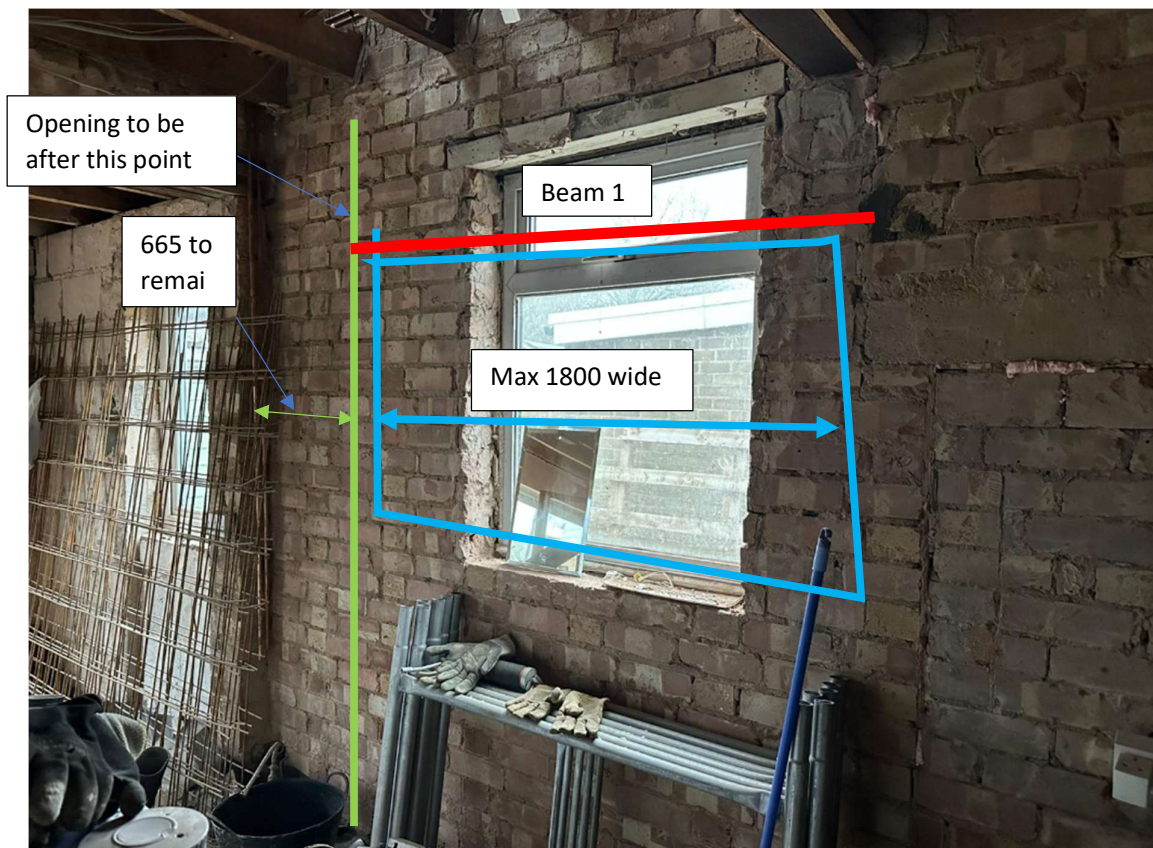
Builder/contractor is to check that the structural engineer's proposal (i.e. location of steel beams/trimmers etc) is feasible and necessary before ordering materials. Open up top of walls etc to determine existing spans & necessity of steels/beams.

No opening works were undertaken and therefore any defects hidden or inaccessible could not be reported upon and thus not the engineer's responsibility.

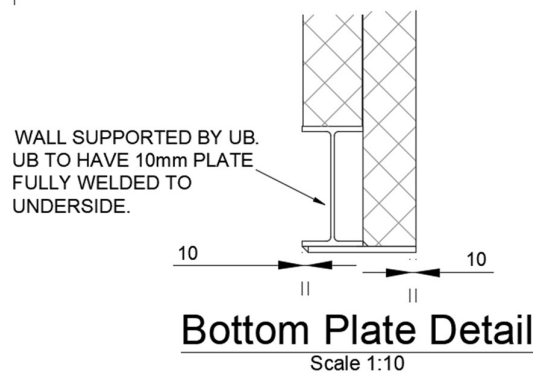
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4. Drawings

Opening in Kitchen



Use 203x102x13 UB with 400x100x200 deep padstone on each end



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Chimney Removal – Gallow Bracket



Gallow Brackets

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Calculations Gallows

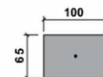
Load on lintel = $2.15\text{kN/m}^2 \times 2 \text{ layers} \times 2\text{m high} \times 1.5 = 13\text{kN/m ULS}$

2no Lintels so load per lintel is say 7 kN/m – **Therefore use 2no 100x65 PC Lintels laid flat between both gallow brackets.**

Lintel Span 0.7m

100x65

STANDARD LENGTHS (MM)	600	750	900	1050	1200	1350	1500	1650
Clear Span (mm)	300	450	600	750	900	1050	1200	1350
Weight (kg/m)	15.3	15.3	15.3	15.3	15.3	15.3	15.3	15.3
UDL (kN/m)	23.9	23.9	13.8	8.9	6.5	5.3	4.1	3.2
RM (kNm)	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8



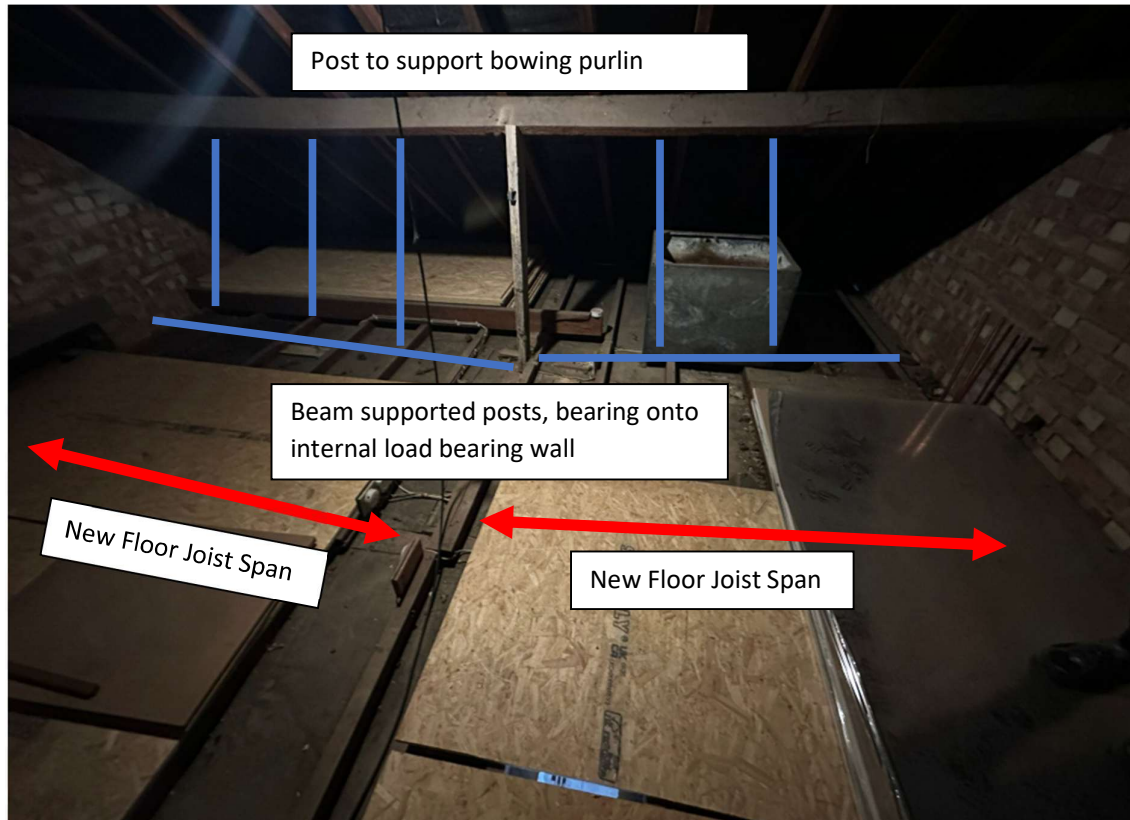
Wall is 2 skins of brickwork work – Each **gallow bracket be as below bolted to wall using 3no M12 x 100mm length** anchor bolts per bracket.

Lintels to span between gallow brackets



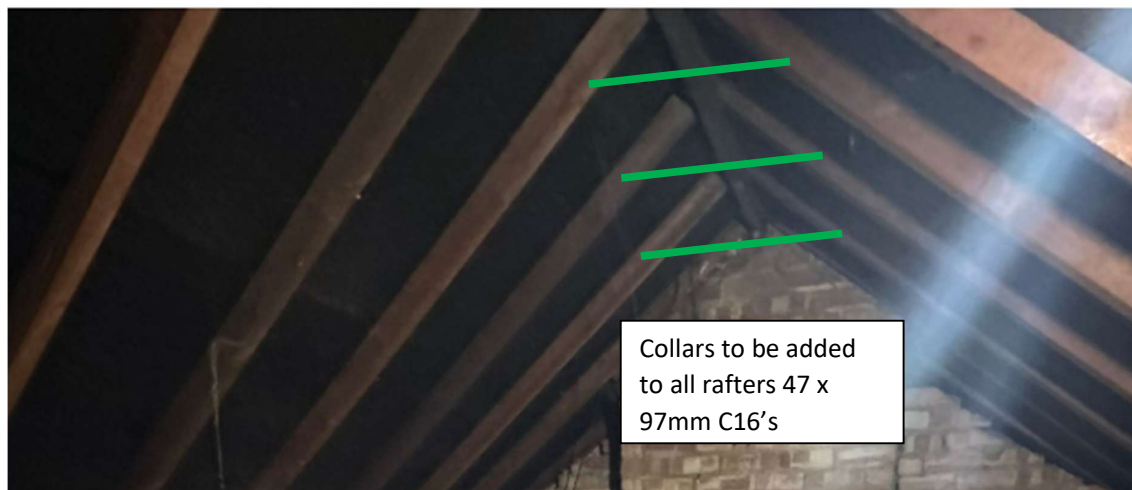
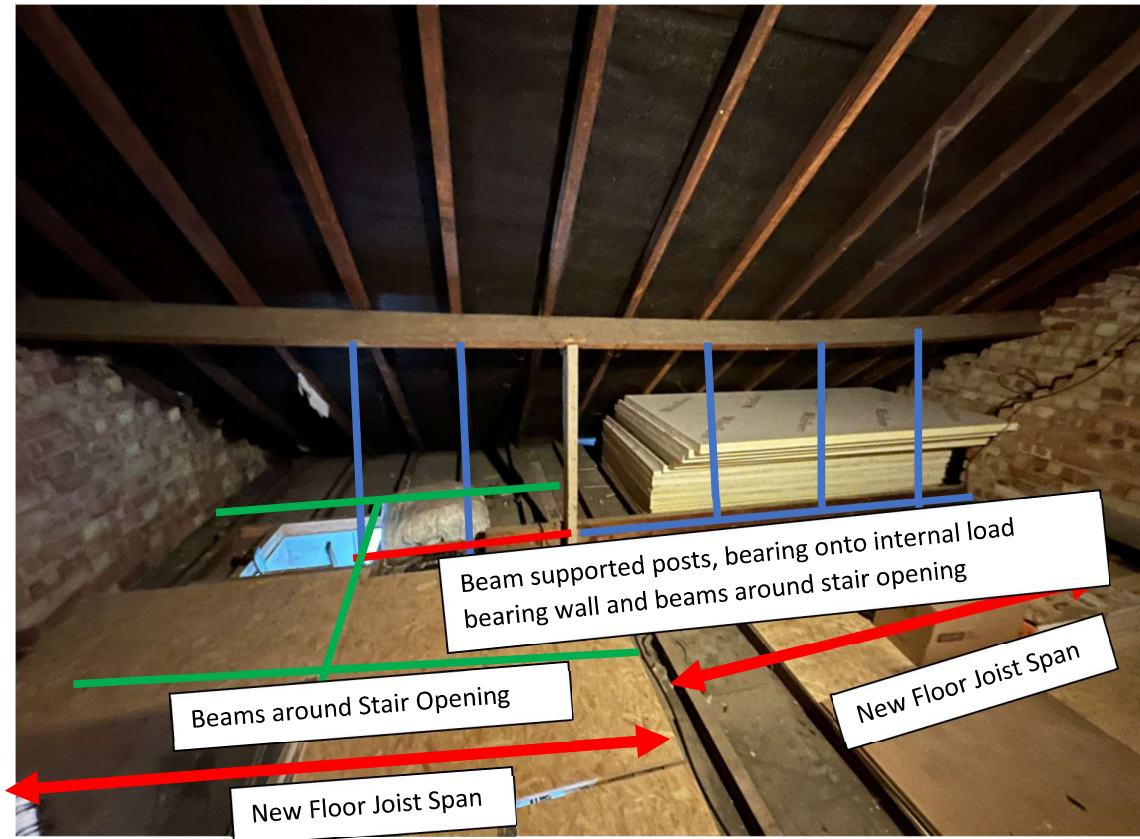
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Loft Conversion



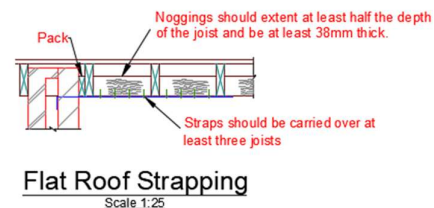
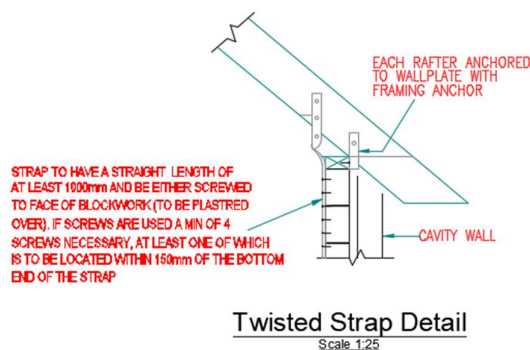
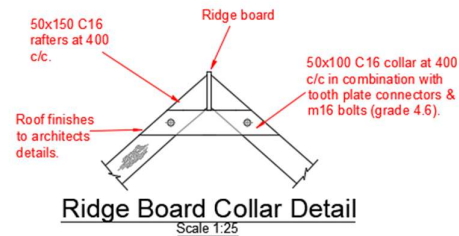
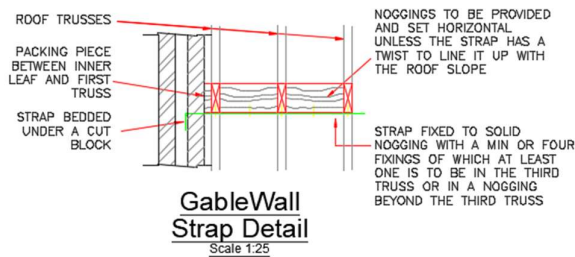
Roof OPENINGS 800 wide Use 5no50x74 C16s around opening

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Wall Strapping Details



S - Provide roof strap fixed to wall and rafter/joist to prevent spreading. 30x5x1000 galvanised metal straps to BS EN845-1 at 2000 approx centers.

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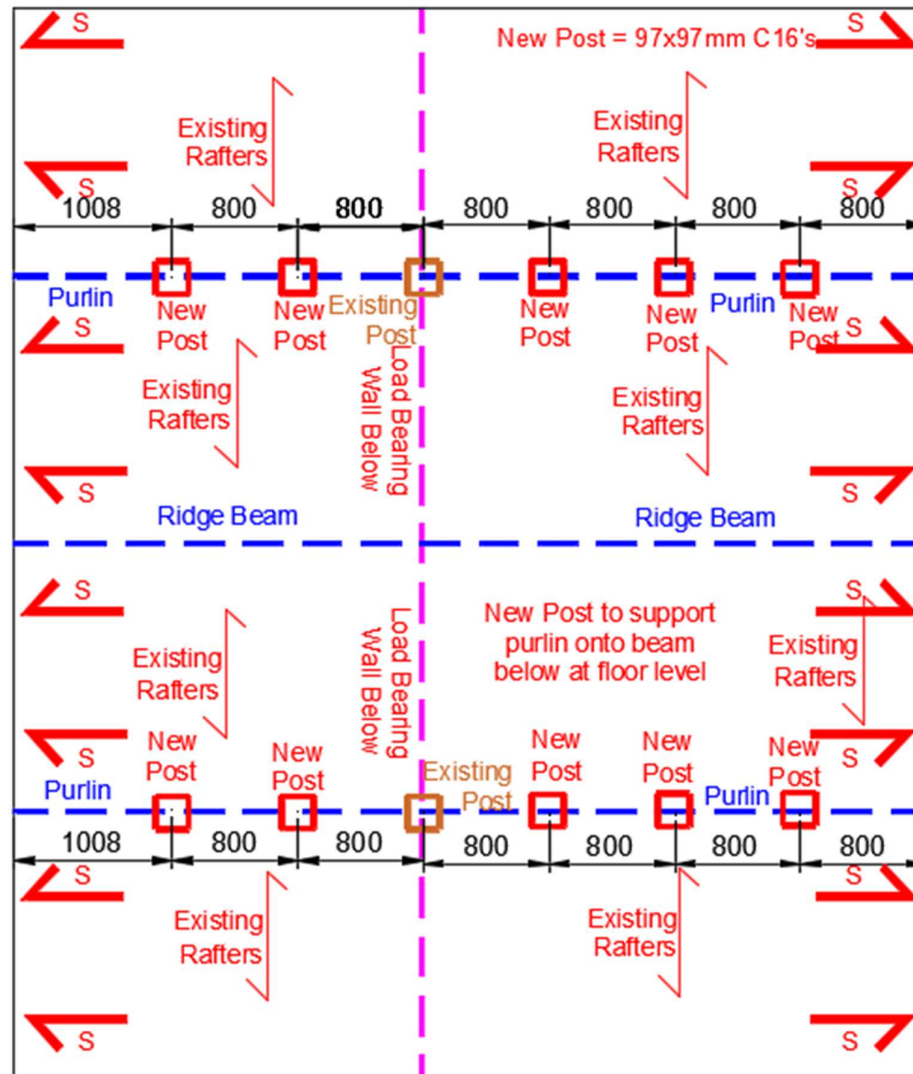


Figure 1 - Roof Plan showing Post 97x97 C16 positions

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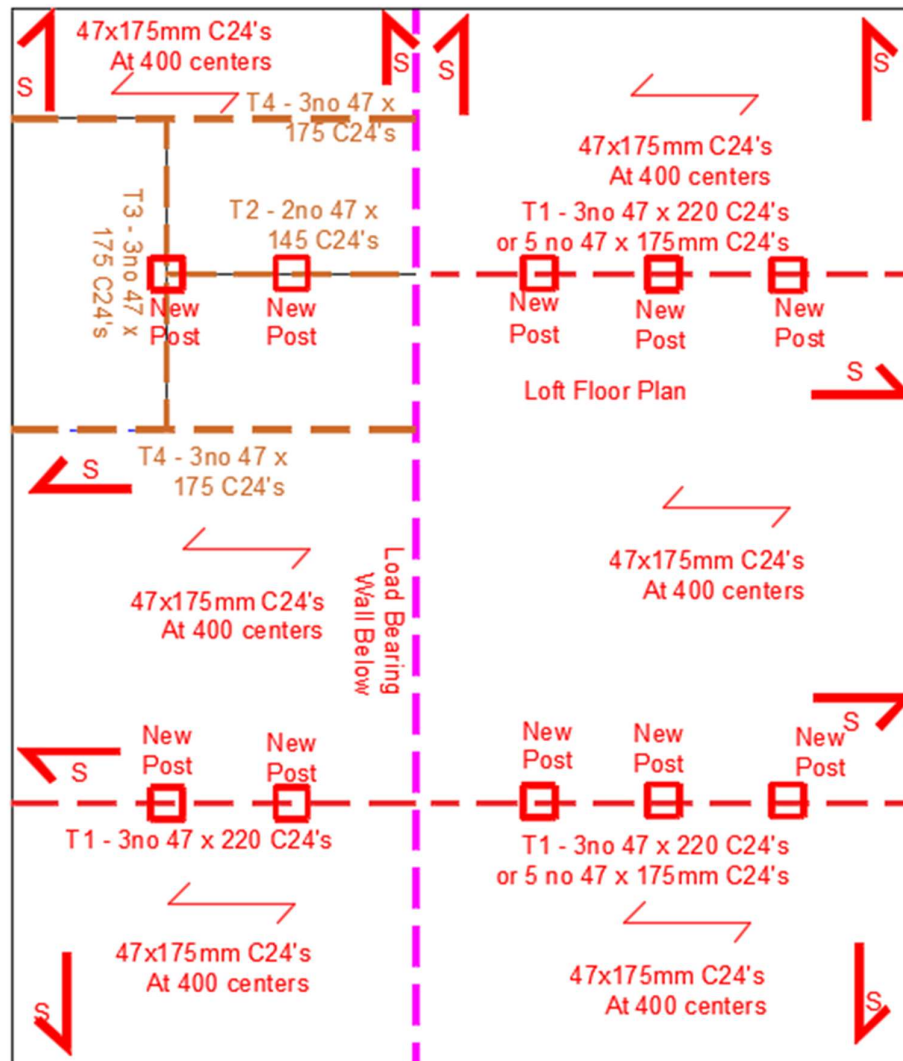


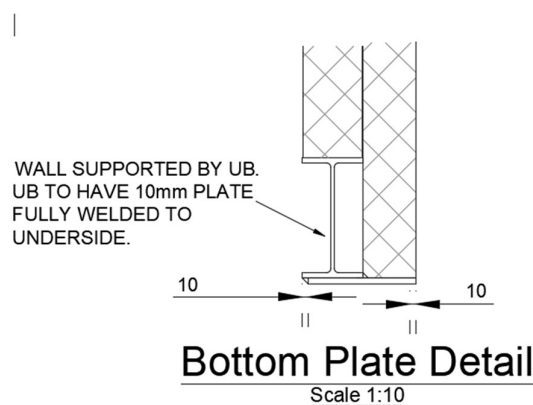
Figure 2 - Loft Floor Plan Showing Timber Sizes and Strapping Locations

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Stairway Lintel Replacement

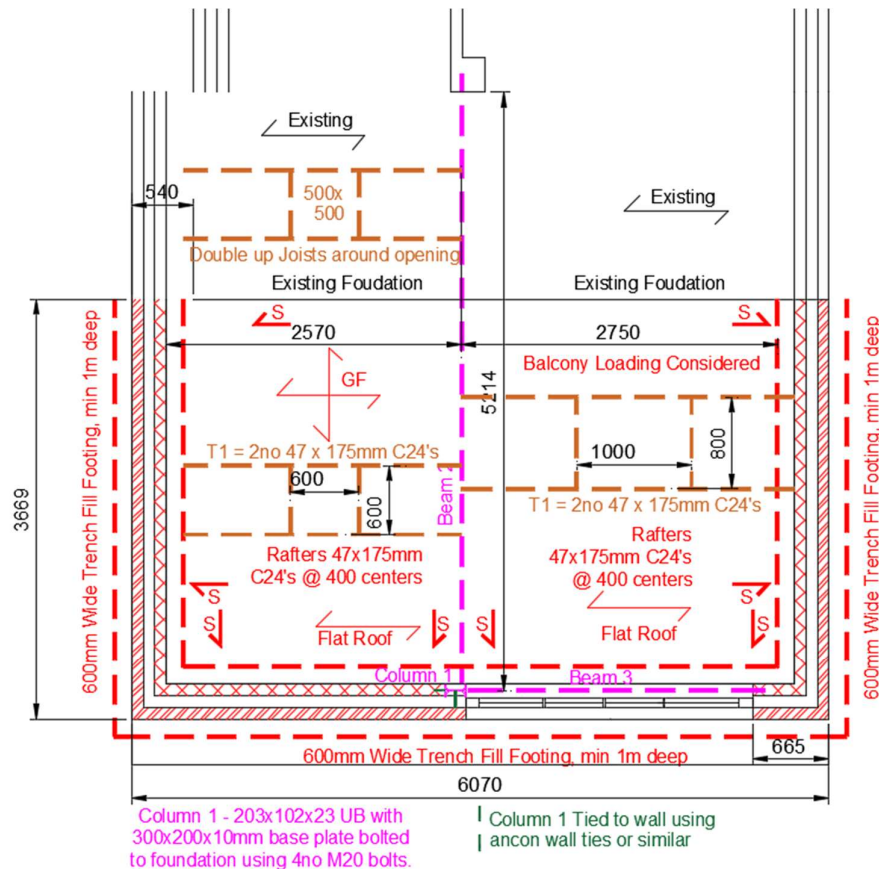


By inspection Use 203x102x13 UB with 200x100x100 deep padstone on each end existing lintel unknown therefore replace due to increased floor loading.



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Rear Extension



Ground Floor

150mm deep C35 reinforced concrete slab

Foundations

F1 - Trench fill footing, 600mm wide C25

Depth to be agreed with Building Control.

Masonry

Unless noted otherwise, provide 3.6 N/mm² 100mm wide blockwork above dpc, provide 7.3 N/mm² 100mm wide blockwork below dpc.

Movement joints

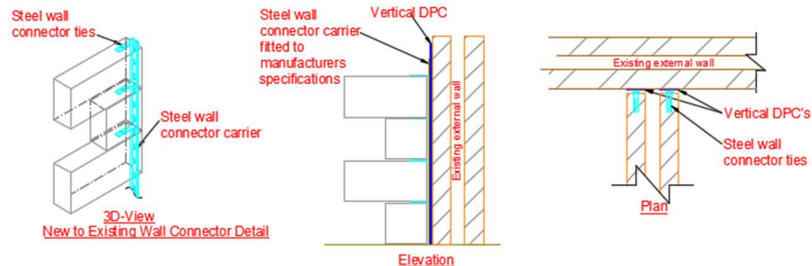
At max 6m centres to blockwork.
At max 12m centres to brickwork.

S - Provide roof strap fixed to wall and rafter/joist to prevent spreading. 30x5x1000 galvanised metal straps to BS EN845-1 at 2000 approx centers.

Beam 2 - 203x133x30 UB onto 300x100x100 padstone on each end

Beam 3 - 203x133x30 UB onto 300x100x100 padstone on each end. UB to have 10mm plate welded to underside also 6mm web stiffeners at 500 centers

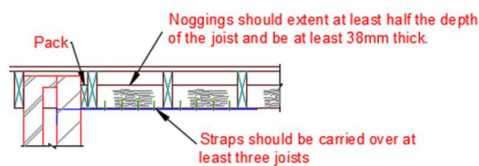
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Wall Starter

Scale 1:25

150mm deep ground bearing grade C35 RC slab (with 1 layer of A193 mesh in top 30mm cover) on 1200g DPM on sand blinded 150mm well compacted hardcore. Screed, insulation & finish to architects details.



Flat Roof Straps

Scale 1:25

Foundations to be centred under walls over.

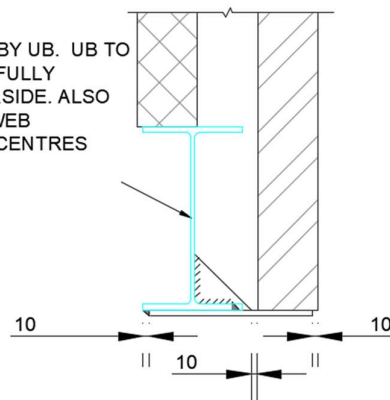
Centre line of wall

NB. Despite min dims shown all foundations to be formed in undisturbed strata. Depths to be agreed on site with the local building inspector.

Foundation Detail

Scale 1:25

WALL SUPPORTED BY UB. UB TO HAVE 10mm PLATE FULLY WELDED TO UNDERSIDE. ALSO FULLY WELD 6mm WEB STIFFNERS 500mm CENTRES



Bottom Plate Detail

Scale 1:10

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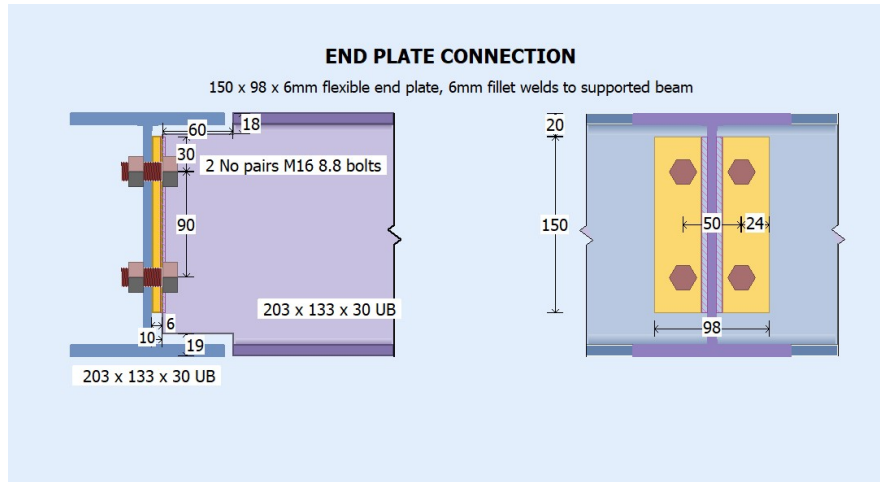


Figure 3 - Figure 3 - Beam 2 and Beam 3

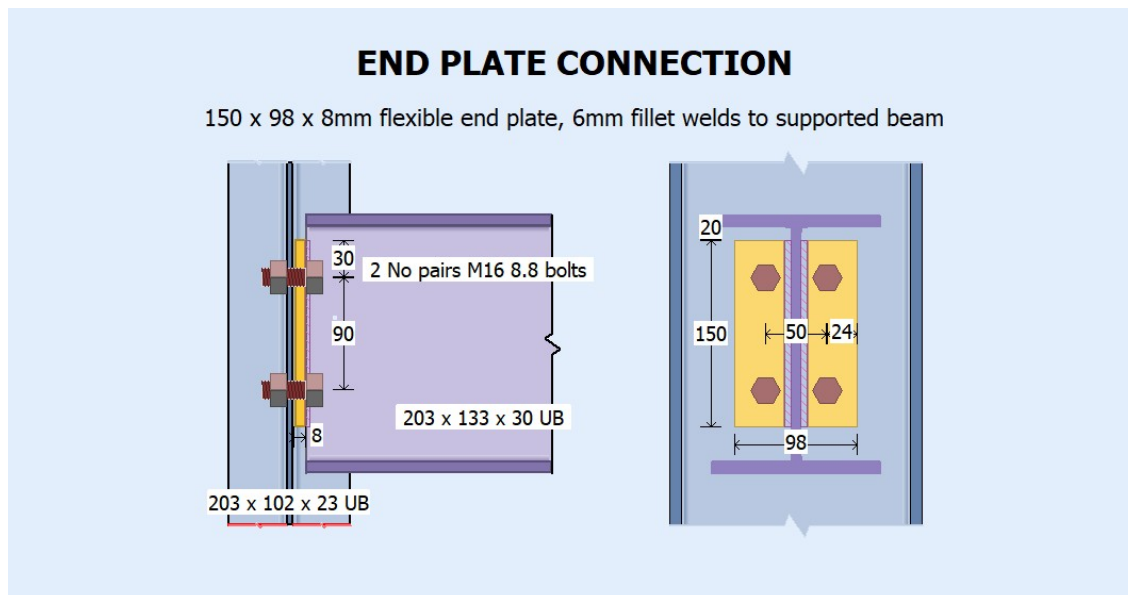


Figure 4 - Column 1 and Beam 2 if required if steel line up with centre of column at edge of door if not then offset column so that beam 2 goes into beam 3 and beam 3 is connected to column 1.

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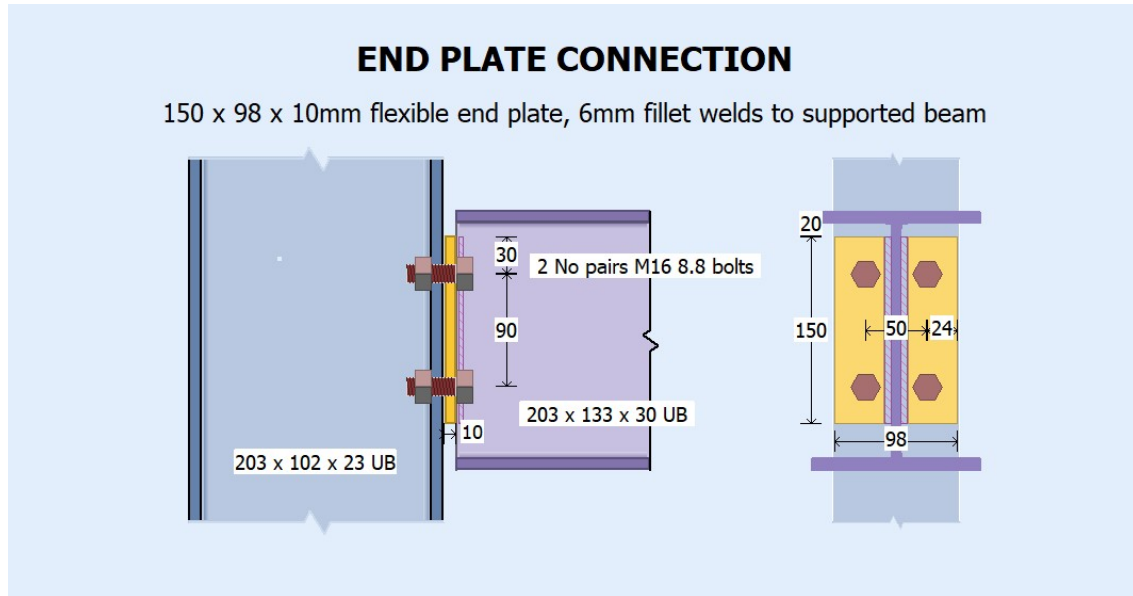
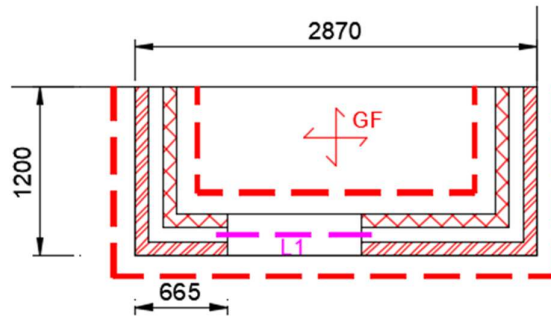


Figure 5 - Column 1 and Beam 3

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Front Extension



Ground Floor Plan

Scale: 1:50

Ground Floor

150mm deep C35 reinforced concrete slab

Foundations

F1 - Trench fill footing, 600mm wide C25

Depth to be agreed with Building Control.

Lintels to be Catnic CG 90 / 100 - by inspection

Masonry

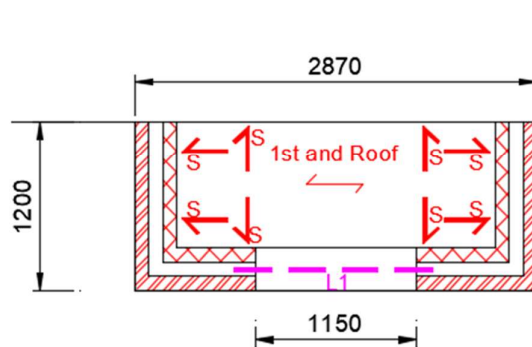
Unless noted otherwise, provide 3.6 N/mm² 100mm wide blockwork above dpc, provide 7.3 N/mm² 100mm wide blockwork below dpc.

Movement joints

At max 6m centres to blockwork.

At max 12m centres to brickwork.

S - Provide roof strap fixed to wall and rafter/joist to prevent spreading. 30x5x1000 galvanised metal straps to BS EN845-1 at 2000 approx centers.



1st/Roof Floor Plan

Scale: 1:50

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5. Calculations

Beam: Beam 1-1 Span: 2.6 m.

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U	G	o.w.	0.25	0	L	0.32	0.32	
U	G	Wall	2.13x2.3	0	L	6.37	6.37	
U	G	1st Floor	1.5x0.4	0	L	0.78	0.78	
U	QA	1st Floor	1.5x0.4	0	L	0.78	0.78	
U	G	Loft Space	0.25x2.5	0	L	0.81	0.81	
U	QH	Loft Space	0.25x2.5	0	L	0.81	0.81	
U	G	Rafters	0	0	L	0.00	0.00	
U	QH	Rafers	0	0	L	0.00	0.00	
Total load (unfactored): 19.76 kN				9.88	9.88			
Dead/Permanent (unfactored): 16.57 kN				8.29	8.29			
Live/Variable (unfactored): 3.18 kN				1.59	1.59			
(6.10): 27.15 kN				13.58	13.58			

Load types: U:UDL; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed; QA: Residential; QH: Roof

Maximum B.M. = 8.82 kNm (6.10) at 1.30 m. from R1

Maximum S.F. = 13.58 kN (6.10) at R1

Mid-span deflections: Dead: $3.79 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $0.73 \times 10^8 / EI$

Total: $4.52 \times 10^8 / EI$

EuroBeam 3.19j 301051 2 Feb 2025

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Beam: Beam 1 Span: 1.8 m.

			Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U	G		o.w.	0.25	0	L	0.22	0.22	
U	G		Wall	4.25x6	0	L	22.95	22.95	
U	G		1st Floor	1.5x1.3	0	L	1.75	1.75	
U	QA		1st Floor	1.5x1.3	0	L	1.75	1.75	
U	G		Loft Space	0.25x0.4	0	L	0.09	0.09	
U	QH		Loft Space	0.25x0.4	0	L	0.09	0.09	
U	G		Rafters	1.1x1.7	0	L	1.68	1.68	
U	QH		Rafers	0.8x1.7	0	L	1.22	1.22	
P	G		Beam: Beam 1-1 : R2	8.29	0.3		6.91	1.38	
P	QA		Beam: Beam 1-1 : R2	0.78	0.3		0.65	0.13	
P	QH		Beam: Beam 1-1 : R2	0.81	0.3		0.68	0.14	
Total load (unfactored): 69.42 kN					38.01	31.42			
Dead/Permanent (unfactored): 61.70 kN					33.61	28.08			
Live/Variable (unfactored): 7.73 kN					4.39	3.33			
(6.10): 94.88 kN					51.97	42.92			

Load types: U:UDL; P:Point load; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed; QA: Residential; QH: Roof

Maximum B.M. = 20.39 kNm (6.10) at 0.84 m. from R1

Maximum S.F. = 51.97 kN (6.10) at R1

Mid-span deflections: Dead: $4.54 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $0.56 \times 10^8 / EI$

Total: $5.10 \times 10^8 / EI$

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : 203 x 102 x 23 UB S355 (Class 1, plastic)

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$D=203.2 \text{ mm}$ $B=101.8 \text{ mm}$ $t=5.4 \text{ mm}$ $T=9.3 \text{ mm}$ $I_y=2,100 \text{ cm}^4$ $i_z=2.36 \text{ cm}$ $W_{pl,y}=234 \text{ cm}^3$
 $W_{el,y}=207 \text{ cm}^3$

Classification: Flange: $c/t = 40.6/9.3 = 4.37 \leq 9e$ (7.32): Class 1, plastic

EC3 Table 5.2 Web: $c/t = 169.4/5.4 = 31.4 \leq 72e$ (58.6): Class 1, plastic

Shear

Design shear force, $V_{Ed} = 52.0 \text{ kN}$

Shear area, $A_v = A - 2bt_f + (t_w + 2r)t_f = 29.4 \times 100 - 2 \times 102 \times 9.30 + (5.40 + 2 \times 7.60) \times 9.30 = 1,238 \text{ mm}^2$ [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd} = A_v \cdot (f_y / \sqrt{3}) / g_{M0} = 1,238 \times (355 / \sqrt{3}) / (1.0 \times 1000) = 254 \text{ kN}$ (≥ 52.0) OK [EC3 6.2.6]

Shear buckling: $h_w/t_w = 184.6/5.4 = 34.19 \leq 72e$ (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, $M_{Ed} = 20.39 \text{ kNm}$

Moment resistance, $M_{c,y,Rd} = f_y \cdot W_{pl,y} = 355 \times 234 / 1000 = 83.1 \text{ kNm}$ OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

Support conditions R1/R2: Compression flange laterally restrained. Nominal torsional restraint. Both flanges free to rotate on plan (1.0L)

Design buckling resistance moment, $M_{b,Rd} = c_{LT,mod} \cdot M_{c,Rd}$

$c_{LT,mod} = c_{LT}/f$ (but $\leq 1/\bar{\lambda}_{LT}^2$ and ≤ 1.0) [Eq.6.58]

$f = 1 - 0.5(1 - k_c)[1 - 2(\bar{\lambda}_{LT} - 0.8)^2]$ 6.3.2.3(2) $k_c = 1/\bar{\phi}_{C1}$ [NA2.18]

Use buckling curve b: $a = 0.340$ [EC3 Tables 6.3/6.4 NA 2.17]

$c_{LT} = 1/[f_{LT} + \bar{\phi}(F_{LT}^2 - b\bar{\lambda}_{LT}^2)]$ [EC3 (6.57)]

$F_{LT} = 0.5[1 + a_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + b\bar{\lambda}_{LT}^2]$

$\bar{\lambda}_{LT,0} = 0.4$ $b = 0.75$ [EC3 UK NA 2.17]

$\bar{\lambda}_{LT} = \bar{\phi}(f_y \cdot W_{pl,y} / M_{cr})$

$M_{cr} = C_1(p^2 E I_z / L_e^2) \bar{\phi}(I_w / I_z + L_e^2 G I_T / p^2 E I_z)$ [SN003: k and k_w taken as 1.0, conservative]

$W_y = 234.0 \text{ cm}^3$ $I_w = 0.015 \text{ dm}^6$ $I_T = 7.02 \text{ cm}^4$ $I_z = 164 \text{ cm}^4$ $G = 81,000 \text{ N/mm}^2$

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Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{\alpha}_z$	$\bar{\alpha}_{LT}$	f_{LT}	C_{LT}	$C_{LT, mod}$	M
c_{Rd}	$M_{b,Rd}$										
0.00-1.80	1.80	20.4	1.12*	0.95	142.9	0.998	0.763	0.780	0.837	0.861	
	83.1	71.5	OK								

* $C_1 = 4M_{max}/\ddot{O}(M_{max}^2 + 4M_a^2 + 7M_b^2 + 4M_c^2) \leq 2.5$ [$M_{a,b,c}$ quarter pt moments] [Wong & Driver, AISC Eng. Journal, Q1 2010]

Combined bending and shear

$V_{Ed} \leq 0.5 V_{c,Rd}$: Check for bending/shear interaction not required [EC3 6.2.8(2)]

Web capacity at bearings

Resistance of web to transverse forces, $F_{Rd} = f_{yw} \cdot L_{eff} \cdot t_w / g_{M1}$

$$f_{yw} = 355 \text{ N/mm}^2 \quad L_{eff} = C_F l_y$$

$$C_F = 0.5/\bar{\lambda}_F \leq 1.0 \quad \bar{\lambda}_F = \ddot{O}(l_y \cdot t_w \cdot f_{yw} / F_{cr})$$

$$F_{cr} = 0.9 k_F \cdot E \cdot (t_w^3 / h_w) \quad k_F = 2 + 6((S_s + c)/h_w)^2 \leq 6$$

Type (c) load application assumed:

Effective loaded length, $l_y = \min S_s + 2t_f(1 + \ddot{O}(m_1 + m_2))$ [EC3-1-5 Eq.6.10],

$$l_e + t_f \ddot{O}(m_1/2 + (l_e/t_f)^2 + m_2) \text{ [Eq.6.11]}$$

$$\text{or } l_e + t_f \ddot{O}(m_1 + m_2) \text{ [Eq.6.12]}$$

$$l_e = k_F \cdot E \cdot t_w^2 / (2 \cdot f_{yw} \cdot h_w) \leq S_s + c \text{ [Eq.6.13]}$$

$$m_1 = f_{yf} \cdot b_f / (f_{yw} \cdot t_w) \quad m_2 = 0.02(h_w/t_f)^2 \text{ if } \bar{\lambda}_F > 0.5 \text{ else } 0.0$$

Reaction R1: 51.97 kN

Required minimum stiff bearing length, $S_s = 0 \text{ mm}$

c (end of beam to stiff bearing) taken as 0.0

$$m_1 = 18.9 \quad m_2 = 0.00 \quad F_{cr} = 322 \text{ kN} \quad k_F = 2.00 \quad l_e = 0.00 \quad l_y = 28.55 \quad \bar{\lambda}_F = 0.412 \quad C_F = 1.00 \quad L_{eff} = 28.55$$

Resistance of web to transverse forces, $F_{R,d} = 355 \times 28.55 \times 5.4 / (1000 \times 1.0) = 54.7 \text{ kN OK}$

Reaction R2: 42.92 kN

Required minimum stiff bearing length, $S_s = 0 \text{ mm}$

c (end of beam to stiff bearing) taken as 0.0

$$m_1 = 18.9 \quad m_2 = 0.00 \quad F_{cr} = 322 \text{ kN} \quad k_F = 2.00 \quad l_e = 0.00 \quad l_y = 28.55 \quad \bar{\lambda}_F = 0.412 \quad C_F = 1.00 \quad L_{eff} = 28.55$$

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Resistance of web to transverse forces, $F_{R,d} = 355 \times 28.55 \times 5.4 / (1000 \times 1.0) = 54.7 \text{ kN OK}$

Deflection

LL deflection = $0.56 \times 1e8 / (210,000 \times 2,100) = 0.1 \text{ mm OK}$

TL deflection = $5.10 \times 1e8 / (210,000 \times 2,100) = 1.2 \text{ mm (L/1557)}$

Bearings

203x102x23 UB stiff bearing length, $b_1 = t + 1.6r + 2T = 36.2 \text{ mm}$

Design masonry strength = 1.50 N/mm^2 (User-entered value)

R1 (51.97 kN) : 350 x 100 x 100h mm padstone

Stress under padstone = $51.97 \times 1000 / 350 \times 100 = 1.48 \text{ N/mm}^2 \text{ OK}$

R2 (42.92 kN) : 350 x 100 x 100h mm padstone

Stress under padstone = $42.92 \times 1000 / 350 \times 100 = 1.23 \text{ N/mm}^2 \text{ OK}$

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Beam: Loft Floor Span: 3.2 m.

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp
							R2comp

U	G	Floor dead	1.5x0.4	0	L	0.96	0.96
---	---	------------	---------	---	---	------	------

U	QH	Floor Live	1.5x0.4	0	L	0.96	0.96
---	----	------------	---------	---	---	------	------

Total load (unfactored): 3.84 kN ~~1.92~~ ~~1.92~~

Dead/Permanent (unfactored): 1.92 kN 0.96 0.96

Live/Variable (unfactored): 1.92 kN 0.96 0.96

(6.10): 5.47 kN 2.74 2.74

Load types: U:UDL; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed: QH: Roof

Maximum B.M. = 2.19 kNm (6.10) at 1.60 m. from R1

Maximum S.F. = -2.74 kN (6.10) at R2

Mid-span deflections: Dead: $0.82 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $0.82 \times 10^8 / EI$

Total: $1.64 \times 10^8 / EI$

Timber beam calculation to BS EN1995-1-1 using C24 timber

Use 47 x 170 C24 3.4 kg/m approx

$W_{el,y} = 226.4 \text{ cm}^3$ $I_y = 1,924 \text{ cm}^4$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.6$ (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.1$ [EC5 6.6(2)]

Deflection modification factor, $k_{def} = 0.60$ (Service class 1, solid timber/glulam/LVL) [EC5 Table 3.2]

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Bending

Height factor, $k_h = 1.0$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_m = 24.0 \times 0.60 \times 1.00 \times 1.1 / 1.30 = 12.18 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 2.19 \times 1000 / 226.4 = 9.67 \text{ N/mm}^2$ OK

Bending resistance = $12.18 \times 226.4 / 1000 = 2.76 \text{ kNm}$

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 47 = 31.5 \text{ mm}$. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / g_m = 4.00 \times 0.60 \times 1.1 / 1.30 = 2.03 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 2.74 \times 1000 \times (3/2) / (31.5 \times 170) = 0.77 \text{ N/mm}^2$ OK

Shear resistance = $2.03 \times 31.5 \times 170 \times (2/3) / 1000 = 7.25 \text{ kN}$

Deflection

Final deflection limit = $L / 250 = 12.80 \text{ mm}$

Final deflection, $u_{fin} = S u_{inst} (1 + \gamma_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where ($G = E / 16$)

$$u_{inst,v} = 1.2 \times 1.54 \times 10^6 / ((11,000 / 16) \times 47 \times 170) = 0.34 \text{ mm}$$

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 10^8 / EI$				Inst. mm	γ_2	k_{def}	Fin. mm
Dead:	0.82	3.87	1.00	0.60	6.19			
Live QH:	0.82	3.87	0.00	0.60	3.87			
Shear deflection:			0.34			0.44		
Total mm.			<u>8.08</u>		<u>10.50</u>			OK

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Beam: Trimmer 1 Span: 3.2 m.

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp
		R2comp					
U	G	o.w.	0.2 0	L	0.32 0.32		
U	G	Floor dead	1.5x0.4 0	L	0.96 0.96		
U	QH	Floor Live	1.5x0.4 0	L	0.96 0.96		
U	G	Wall	1x0.5 0	L	0.80 0.80		
U	G	Rafters	1.1x2.5 0	L	4.40 4.40		
U	QH	Rafters	0.8x2.5 0	L	3.20 3.20		
Total load (unfactored): 21.28 kN				10.64	10.64		
Dead/Permanent (unfactored): 12.96 kN				6.48	6.48		
Live/Variable (unfactored): 8.32 kN				4.16	4.16		
(6.10): 29.98 kN				14.99	14.99		

Load types: U:UDL; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed: QH: Roof

Maximum B.M. = 11.99 kNm (6.10) at 1.60 m. from R1

Maximum S.F. = 14.99 kN (6.10) at R1

Mid-span deflections: Dead: $5.53 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $3.55 \times 10^8 / EI$

Total: $9.08 \times 10^8 / EI$

Timber beam calculation to BS EN1995-1-1 using C24 timber

Use 3no 47 x 220 C24 13.0 kg/m approx

$W_{el,y} = 1,137 \text{ cm}^3$ $I_y = 12,511 \text{ cm}^4$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

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Loading modification factor, $k_{mod} = 0.6$ (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.1$ [EC5 6.6(2)]

Deflection modification factor, $k_{def} = 0.60$ (Service class 1, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = 1.0$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_m = 24.0 \times 0.60 \times 1.00 \times 1.1 / 1.30 = 12.18 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 11.99 \times 1000 / 1137 = 10.54 \text{ N/mm}^2$ OK

Bending resistance = $12.18 \times 1137 / 1000 = 13.86 \text{ kNm}$

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 141 = 94.5 \text{ mm}$. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / g_m = 4.00 \times 0.60 \times 1.1 / 1.30 = 2.03 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 14.99 \times 1000 \times (3/2) / (94.5 \times 220) = 1.08 \text{ N/mm}^2$ OK

Shear resistance = $2.03 \times 94.5 \times 220 \times (2/3) / 1000 = 28.14 \text{ kN}$

Deflection

Final deflection limit = $L / 250 = 12.80 \text{ mm}$

Final deflection, $u_{fin} = Su_{inst}(1 + \gamma_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2M_{unf} / (G_{mean}A)$ where $(G = E/16)$

$$u_{inst,v} = 1.2 \times 8.51 \times 10^6 / ((11,000/16) \times 141 \times 220) = 0.48 \text{ mm}$$

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 1e^8 / EI$				Inst. mm	γ_2	k_{def}	Fin. mm
Dead:	5.53	4.02	1.00	0.60	6.43			
Live QH:		3.55	2.58	0.00	0.60	2.58		
Shear deflection:				0.48			0.65	
Total mm.			<u>7.08</u>		<u>9.66</u>	OK		

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Beam: Purlin 1 Span: 1.0 m.

			Load name	Loading w1	Start x1	Loading w2	End x2	R1comp
								R2comp
U	G		o.w.	0.1	0	L	0.05	0.05
U	G		Roof	1.1x2.5	0	L	1.38	1.38
U	QH		Roof	0.8x2.5	0	L	1.00	1.00
Total load (unfactored): 4.85 kN 2.42 2.42								
Dead/Permanent (unfactored): 2.85 kN 1.42 1.42								
Live/Variable (unfactored): 2.00 kN 1.00 1.00								
(6.10): 6.85 kN 3.42 3.42								

Load types: U:UDL; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed: QH: Roof

Maximum B.M. = 0.86 kNm (6.10) at 0.50 m. from R1

Maximum S.F. = 3.42 kN (6.10) at R1

Mid-span deflections: Dead: $0.04 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $0.03 \times 10^8 / EI$

Total: $0.06 \times 10^8 / EI$

Timber beam calculation to BS EN1995-1-1 using C24 timber

Use 2no 47 x 72 C24 2.8 kg/m approx

$W_{el,y} = 81.22 \text{ cm}^3$ $I_y = 292 \text{ cm}^4$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.6$ (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.1$ [EC5 6.6(2)]

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Deflection modification factor, $k_{def} = 0.60$ (Service class 1, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = \min((150/h)^{0.2}, 1.3) = (150/72)^{0.2} = 1.16$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_m = 24.0 \times 0.60 \times 1.16 \times 1.1 / 1.30 = 14.11 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 0.86 \times 1000 / 81.2 = 10.54 \text{ N/mm}^2$ OK

Bending resistance = $14.11 \times 81.2 / 1000 = 1.15 \text{ kNm}$

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 94 = 63 \text{ mm}$. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / g_m = 4.00 \times 0.60 \times 1.1 / 1.30 = 2.03 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 3.42 \times 1000 \times (3/2) / (63.0 \times 72) = 1.13 \text{ N/mm}^2$ OK

Shear resistance = $2.03 \times 63 \times 72 \times (2/3) / 1000 = 6.14 \text{ kN}$

Deflection

Final deflection limit = $L / 250 = 4.00 \text{ mm}$

Final deflection, $u_{fin} = S u_{inst} (1 + y_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where ($G = E / 16$)

$$u_{inst,v} = 1.2 \times 0.606 \times 10^6 / ((11,000 / 16) \times 94 \times 72) = 0.16 \text{ mm}$$

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 10^8 / EI$			Inst. mm	y_2	k_{def}	Fin. mm
Dead:	0.04	1.15	1.00	0.60	1.85		
Live QH:	0.03	0.81	0.00	0.60	0.81		
Shear deflection:			0.16		0.21		
Total mm.		<u>2.12</u>		<u>2.87</u>	OK		

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Beam: Purlin 2 Span: 0.8 m.

			Load name	Loading w1	Start x1	Loading w2	End x2	R1comp
								R2comp
U	G		o.w.	0.1 0	L	0.04 0.04		
U	G		Roof	1.1x2.5 0	L	1.10 1.10		
U	QH		Roof	0.8x2.5 0	L	0.80 0.80		

Total load (unfactored): 3.88 kN ~~1.94~~ ~~1.94~~

Dead/Permanent (unfactored): 2.28 kN 1.14 1.14

Live/Variable (unfactored): 1.60 kN 0.80 0.80

(6.10): 5.48 kN 2.74 2.74

Load types: U:UDL; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed: QH: Roof

Maximum B.M. = 0.55 kNm (6.10) at 0.40 m. from R1

Maximum S.F. = 2.74 kN (6.10) at R1

Mid-span deflections: Dead: $0.02 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $0.01 \times 10^8 / EI$

Total: $0.03 \times 10^8 / EI$

Timber beam calculation to BS EN1995-1-1 using C24 timber

Use 2no 47 x 72 C24 2.8 kg/m approx

$W_{el,y} = 81.22 \text{ cm}^3$ $I_y = 292 \text{ cm}^4$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.6$ (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.1$ [EC5 6.6(2)]

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Deflection modification factor, $k_{def} = 0.60$ (Service class 1, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = \min((150/h)^{0.2}, 1.3) = (150/72)^{0.2} = 1.16$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_m = 24.0 \times 0.60 \times 1.16 \times 1.1 / 1.30 = 14.11 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 0.55 \times 1000 / 81.2 = 6.74 \text{ N/mm}^2$ OK

Bending resistance = $14.11 \times 81.2 / 1000 = 1.15 \text{ kNm}$

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 94 = 63 \text{ mm}$. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / g_m = 4.00 \times 0.60 \times 1.1 / 1.30 = 2.03 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 2.74 \times 1000 \times (3/2) / (63.0 \times 72) = 0.91 \text{ N/mm}^2$ OK

Shear resistance = $2.03 \times 63 \times 72 \times (2/3) / 1000 = 6.14 \text{ kN}$

Deflection

Final deflection limit = $L / 250 = 3.20 \text{ mm}$

Final deflection, $u_{fin} = S u_{inst} (1 + y_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where $(G = E / 16)$

$$u_{inst,v} = 1.2 \times 0.388 \times 10^6 / ((11,000 / 16) \times 94 \times 72) = 0.10 \text{ mm}$$

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 10^8 / EI$				Inst. mm	y_2	k_{def}	Fin. mm
Dead:	0.02	0.47	1.00	0.60	0.76			
Live QH:		0.01	0.33	0.00	0.60	0.33		
Shear deflection:				0.10			0.14	
Total mm.			<u>0.90</u>		<u>1.22</u>	OK		

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Beam: Trimmer 2 Span: 1.6 m.

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp
		R2comp					
U	G	o.w.	0.2	0	L	0.16	0.16
U	G	Floor dead	1.5x0.4	0	L	0.48	0.48
U	QH	Floor Live	1.5x0.4	0	L	0.48	0.48
U	G	Wall	1x0.5	0	L	0.40	0.40
U	G	Rafters	1.1x2.5	0	L	2.20	2.20
U	QH	Rafters	0.8x2.5	0	L	1.60	1.60
Total load (unfactored): 10.64 kN				5.32	5.32		
Dead/Permanent (unfactored): 6.48 kN				3.24	3.24		
Live/Variable (unfactored): 4.16 kN				2.08	2.08		
(6.10): 14.99 kN				7.49	7.49		

Load types: U:UDL; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed: QH: Roof

Maximum B.M. = 3.00 kNm (6.10) at 0.80 m. from R1

Maximum S.F. = 7.49 kN (6.10) at R1

Mid-span deflections: Dead: $0.35 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $0.22 \times 10^8 / EI$

Total: $0.57 \times 10^8 / EI$

Timber beam calculation to BS EN1995-1-1 using C24 timber

Use 2no 47 x 147 C24 5.8 kg/m approx

$W_{el,y} = 338.5 \text{ cm}^3$ $I_y = 2,488 \text{ cm}^4$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

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Loading modification factor, $k_{mod} = 0.6$ (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.1$ [EC5 6.6(2)]

Deflection modification factor, $k_{def} = 0.60$ (Service class 1, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = \min((150/h)^{0.2}, 1.3) = (150/147)^{0.2} = 1.00$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_m = 24.0 \times 0.60 \times 1.00 \times 1.1 / 1.30 = 12.23 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 3.00 \times 1000 / 339 = 8.85 \text{ N/mm}^2$ OK

Bending resistance = $12.23 \times 339 / 1000 = 4.14 \text{ kNm}$

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 94 = 63 \text{ mm}$. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / g_m = 4.00 \times 0.60 \times 1.1 / 1.30 = 2.03 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 7.49 \times 1000 \times (3/2) / (63.0 \times 147) = 1.21 \text{ N/mm}^2$ OK

Shear resistance = $2.03 \times 63 \times 147 \times (2/3) / 1000 = 12.53 \text{ kN}$

Deflection

Final deflection limit = $L / 250 = 6.40 \text{ mm}$

Final deflection, $u_{fin} = S u_{inst} (1 + \gamma_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where $(G = E / 16)$

$$u_{inst,v} = 1.2 \times 2.13 \times 10^6 / ((11,000 / 16) \times 94 \times 147) = 0.27 \text{ mm}$$

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 1e^8 / EI$			Inst. mm	γ_2	k_{def}	Fin. mm
Dead:	0.35	1.26	1.00	0.60	2.02		
Live QH:		0.22	0.81	0.00	0.60	0.81	
Shear deflection:			0.27		0.37		
Total mm.			<u>2.34</u>		<u>3.20</u>	OK	

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Beam: Trimmer 2 Span: 1.6 m.

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp
							R2comp
U	G	o.w.	0.2	0	L	0.16	0.16
U	G	Floor dead	1.5x0.4	0	L	0.48	0.48
U	QH	Floor Live	1.5x0.4	0	L	0.48	0.48
U	G	Wall	1x0.5	0	L	0.40	0.40
P	G	Beam: Purlin 1 : R1	1.42	0.8		0.71	0.71
P	QH	Beam: Purlin 1 : R1	1.00	0.8		0.50	0.50
P	G	Beam: Purlin 2 : R1	1.14	0.8		0.57	0.57
P	QH	Beam: Purlin 2 : R1	0.80	0.8		0.40	0.40

Total load (unfactored): 7.40 kN ~~3.70~~ **3.70**

Dead/Permanent (unfactored): 4.64 kN 2.32 2.32

Live/Variable (unfactored): 2.76 kN 1.38 1.38

(6.10): 10.40 kN 5.20 5.20

Load types: U:UDL; P:Point load; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed; QH: Roof

Maximum B.M. = 3.31 kNm (6.10) at 0.80 m. from R1

Maximum S.F. = 5.20 kN (6.10) at R1

Mid-span deflections: Dead: $0.33 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $0.20 \times 10^8 / EI$

Total: $0.53 \times 10^8 / EI$

Timber beam calculation to BS EN1995-1-1 using C24 timber

Use 2no 47 x 147 C24 5.8 kg/m approx

$W_{el,y} = 338.5 \text{ cm}^3$ $I_y = 2,488 \text{ cm}^4$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

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$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,\text{mean}} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{\text{mod}} = 0.6$ (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{\text{sys}} = 1.1$ [EC5 6.6(2)]

Deflection modification factor, $k_{\text{def}} = 0.60$ (Service class 1, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = \min((150/h)^{0.2}, 1.3) = (150/147)^{0.2} = 1.00$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{\text{mod}} \cdot k_h \cdot k_{\text{sys}} / g_m = 24.0 \times 0.60 \times 1.00 \times 1.1 / 1.30 = 12.23 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 3.31 \times 1000 / 339 = 9.78 \text{ N/mm}^2$ OK

Bending resistance = $12.23 \times 339 / 1000 = 4.14 \text{ kNm}$

Shear

Effective width for shear, $b_{\text{ef}} = k_{\text{cr}} \cdot b = 0.67 \times 94 = 63 \text{ mm}$. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{\text{mod}} \cdot k_{\text{sys}} / g_m = 4.00 \times 0.60 \times 1.1 / 1.30 = 2.03 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 5.20 \times 1000 \times (3/2) / (63.0 \times 147) = 0.84 \text{ N/mm}^2$ OK

Shear resistance = $2.03 \times 63 \times 147 \times (2/3) / 1000 = 12.53 \text{ kN}$

Deflection

Final deflection limit = $L/250 = 6.40 \text{ mm}$

Final deflection, $u_{\text{fin}} = S u_{\text{inst}} (1 + \gamma_2 \cdot k_{\text{def}})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{\text{inst},v} = 1.2 M_{\text{unf}} / (G_{\text{mean}} A)$ where ($G = E/16$)

$u_{\text{inst},v} = 1.2 \times 2.35 \times 10^6 / ((11,000/16) \times 94 \times 147) = 0.30 \text{ mm}$

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 10^8 / EI$				Inst. mm	γ_2	k_{def}	Fin. mm
Dead:	0.33	1.20	1.00	0.60	1.93			
Live QH:		0.20	0.75	0.00	0.60	0.75		
Shear deflection:				0.30			0.41	
Total mm.			<u>2.25</u>		<u>3.08</u>			OK

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Beam: Trimmer 3 Span: 2.0 m.

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U	G	o.w.	0.2	0	L	0.20	0.20	
U	G	Floor dead	1.5x0.8	0	L	1.20	1.20	
U	QH	Floor Live	1.5x0.8	0	L	1.20	1.20	
P	G	Beam: Purlin 1 : R1	1.42	1		0.71	0.71	
P	QH	Beam: Purlin 1 : R1	1.00	1		0.50	0.50	
P	G	Beam: Purlin 2 : R1	1.14	1		0.57	0.57	
P	QH	Beam: Purlin 2 : R1	0.80	1		0.40	0.40	
P	G	Beam: Trimmer 2 : R1	2.32	1		1.16	1.16	
P	QH	Beam: Trimmer 2 : R1	1.38	1		0.69	0.69	
Total load (unfactored): 13.26 kN				6.63	6.63			
Dead/Permanent (unfactored): 7.68 kN			3.84	3.84				
Live/Variable (unfactored): 5.58 kN			2.79	2.79				
(6.10): 18.74 kN			9.37	9.37				

Load types: U:UDL; P:Point load; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed: QH: Roof

Maximum B.M. = 7.52 kNm (6.10) at 1.00 m. from R1

Maximum S.F. = 9.37 kN (6.10) at R1

Mid-span deflections: Dead: $1.11 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $0.78 \times 10^8 / EI$

Total: $1.88 \times 10^8 / EI$

Timber beam calculation to BS EN1995-1-1 using C24 timber

Use 3no 47 x 170 C24 10.1 kg/m approx

$W_{el,y} = 679.2 \text{ cm}^3$ $I_y = 5,773 \text{ cm}^4$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

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Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,\text{mean}} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{\text{mod}} = 0.6$ (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{\text{sys}} = 1.1$ [EC5 6.6(2)]

Deflection modification factor, $k_{\text{def}} = 0.60$ (Service class 1, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = 1.0$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{\text{mod}} \cdot k_h \cdot k_{\text{sys}} / g_m = 24.0 \times 0.60 \times 1.00 \times 1.1 / 1.30 = 12.18 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 7.52 \times 1000 / 679 = 11.08 \text{ N/mm}^2$ OK

Bending resistance = $12.18 \times 679 / 1000 = 8.28 \text{ kNm}$

Shear

Effective width for shear, $b_{\text{ef}} = k_{\text{cr}} \cdot b = 0.67 \times 141 = 94.5 \text{ mm}$. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{\text{mod}} \cdot k_{\text{sys}} / g_m = 4.00 \times 0.60 \times 1.1 / 1.30 = 2.03 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 9.37 \times 1000 \times (3/2) / (94.5 \times 170) = 0.88 \text{ N/mm}^2$ OK

Shear resistance = $2.03 \times 94.5 \times 170 \times (2/3) / 1000 = 21.74 \text{ kN}$

Deflection

Final deflection limit = $L/250 = 8.00 \text{ mm}$

Final deflection, $u_{\text{fin}} = S u_{\text{inst}} (1 + y_2 \cdot k_{\text{def}})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{\text{inst},v} = 1.2 M_{\text{unf}} / (G_{\text{mean}} A)$ where $(G = E/16)$

$u_{\text{inst},v} = 1.2 \times 5.33 \times 10^6 / ((11,000/16) \times 141 \times 170) = 0.39 \text{ mm}$

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 10^8 / EI$			Inst. mm	y_2	k_{def}	Fin. mm
Dead:	1.11	1.74	1.00	0.60	2.78		
Live QH:		0.78	1.23	0.00	0.60	1.23	
Shear deflection:				0.39		0.52	

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Total mm. 3.36 4.54 OK

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Beam: T4 Span: 2.6 m.

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp
		R2comp					
U	G	Floor dead	1.5x0.4	0	L	0.78	0.78
U	QH	Floor Live	1.5x0.4	0	L	0.78	0.78
P	G	Beam: Trimmer 3 : R1	3.84	1		2.36	1.48
P	QH	Beam: Trimmer 3 : R1	2.79	1		1.72	1.07

Total load (unfactored): 9.75 kN ~~5.64~~ ~~4.11~~

Dead/Permanent (unfactored): 5.40 kN 3.14 2.26

Live/Variable (unfactored): 4.35 kN 2.50 1.85

(6.10): 13.82 kN 7.99 5.83

Load types: U:UDL; P:Point load; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed; QH: Roof

Maximum B.M. = 7.13 kNm (6.10) at 1.00 m. from R1

Maximum S.F. = 7.99 kN (6.10) at R1

Mid-span deflections: Dead: $1.66 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $1.30 \times 10^8 / EI$

Total: $2.96 \times 10^8 / EI$

Timber beam calculation to BS EN1995-1-1 using C24 timber

Use 3no 47 x 175 C24 10.4 kg/m approx

$W_{el,y} = 719.7 \text{ cm}^3$ $I_y = 6,297 \text{ cm}^4$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.6$ (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]

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Load sharing factor, $k_{sys} = 1.1$ [EC5 6.6(2)]

Deflection modification factor, $k_{def} = 0.60$ (Service class 1, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = 1.0$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_m = 24.0 \times 0.60 \times 1.00 \times 1.1 / 1.30 = 12.18 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 7.13 \times 1000 / 720 = 9.91 \text{ N/mm}^2$ OK

Bending resistance = $12.18 \times 720 / 1000 = 8.77 \text{ kNm}$

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 141 = 94.5 \text{ mm}$. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / g_m = 4.00 \times 0.60 \times 1.1 / 1.30 = 2.03 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 7.99 \times 1000 \times (3/2) / (94.5 \times 175) = 0.72 \text{ N/mm}^2$ OK

Shear resistance = $2.03 \times 94.5 \times 175 \times (2/3) / 1000 = 22.38 \text{ kN}$

Deflection

Final deflection limit = $L / 250 = 10.40 \text{ mm}$

Final deflection, $u_{fin} = Su_{inst}(1 + \gamma_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2M_{unf} / (G_{mean}A)$ where $(G = E/16)$

$$u_{inst,v} = 1.2 \times 4.33 \times 10^6 / ((11,000/16) \times 141 \times 175) = 0.31 \text{ mm}$$

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 1e^8 / EI$			Inst. mm	γ_2	k_{def}	Fin. mm
Dead:	1.66	2.40	1.00	0.60	3.83		
Live QH:		1.30	1.88	0.00	0.60	1.88	
Shear deflection:				0.31		0.41	
Total mm.			<u>4.58</u>		<u>6.12</u>	OK	

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Beam: Trimmer 1 Span: 3.2 m.

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp
		R2comp					
U	G	o.w.	0.2 0	L	0.32 0.32		
U	G	Floor dead	1.5x0.4 0	L	0.96 0.96		
U	QH	Floor Live	1.5x0.4 0	L	0.96 0.96		
U	G	Wall	1x0.5 0	L	0.80 0.80		
U	G	Rafters	1.1x2.5 0	L	4.40 4.40		
U	QH	Rafters	0.8x2.5 0	L	3.20 3.20		
Total load (unfactored): 21.28 kN				10.64	10.64		
Dead/Permanent (unfactored): 12.96 kN				6.48	6.48		
Live/Variable (unfactored): 8.32 kN				4.16	4.16		
(6.10): 29.98 kN				14.99	14.99		

Load types: U:UDL; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed: QH: Roof

Maximum B.M. = 11.99 kNm (6.10) at 1.60 m. from R1

Maximum S.F. = 14.99 kN (6.10) at R1

Mid-span deflections: Dead: $5.53 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $3.55 \times 10^8 / EI$

Total: $9.08 \times 10^8 / EI$

Timber beam calculation to BS EN1995-1-1 using C24 timber

Use 5no 47 x 170 C24 16.8 kg/m approx

$W_{el,y} = 1,132 \text{ cm}^3$ $I_y = 9,621 \text{ cm}^4$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

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Loading modification factor, $k_{mod} = 0.6$ (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.1$ [EC5 6.6(2)]

Deflection modification factor, $k_{def} = 0.60$ (Service class 1, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = 1.0$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_m = 24.0 \times 0.60 \times 1.00 \times 1.1 / 1.30 = 12.18 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 11.99 \times 1000 / 1132 = 10.59 \text{ N/mm}^2$ OK

Bending resistance = $12.18 \times 1132 / 1000 = 13.79 \text{ kNm}$

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 235 = 157.4 \text{ mm}$. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / g_m = 4.00 \times 0.60 \times 1.1 / 1.30 = 2.03 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 14.99 \times 1000 \times (3/2) / (157.4 \times 170) = 0.84 \text{ N/mm}^2$ OK

Shear resistance = $2.03 \times 157.4 \times 170 \times (2/3) / 1000 = 36.24 \text{ kN}$

Deflection

Final deflection limit = $L / 250 = 12.80 \text{ mm}$

Final deflection, $u_{fin} = S u_{inst} (1 + \gamma_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where $(G = E / 16)$

$$u_{inst,v} = 1.2 \times 8.51 \times 10^6 / ((11,000 / 16) \times 235 \times 170) = 0.37 \text{ mm}$$

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 1e^8 / EI$				Inst. mm	γ_2	k_{def}	Fin. mm
Dead:	5.53	5.22	1.00	0.60	8.36			
Live QH:		3.55	3.35	0.00	0.60	3.35		
Shear deflection:				0.37		0.51		
Total mm.			<u>8.95</u>		<u>12.22</u>	OK		

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Calculations for timber post/stud to BS EN1995 using C16 timber

Location: Post

Pos	Dur	Load	kN	Factored	Offset	Moment y-y	Moment z-z
				6.10	6.10	6.10	
1	G	B/F Beam: Purlin 1 : R1	1.42	1.92	100	0.28	
1	QH	B/F Beam: Purlin 1 : R1	1.00	1.50	100	0.22	
2	G	B/F Beam: Purlin 2 : R1	1.14	1.54	100	-0.23	
2	QH	B/F Beam: Purlin 2 : R1	0.80	1.20	100	-0.18	
Total load			4.36	6.16	0.10	0.00	

Load offsets are measured in mm. from faces of member; moments in kNm

Load durations: G: Dead

Member length = 1.0 m

Use: 97 x 97 C16

Grade properties:

Loading duration factor, $k_{mod} = 0.6$ (Service class 1; live load duration: Permanent) [EC5 Table 3.1]

Load sharing factor, $k_{sys} = 1.0$

Material partial factor, $g_m = 1.3$ [EC3 NA Table NA.3]

Grade compression stress, $f_{c,0,k} = 17.0 \text{ N/mm}^2$

Design compressive strength, $f_{c,0,d} = k_{mod} \cdot k_{sys} \cdot f_{c,0,k} / g_m = 0.6 \times 1.0 \times 17.0 / 1.3 = 7.85 \text{ N/mm}^2$

Modulus of elasticity, min, $E_{0.05} = 5,400 \text{ N/mm}^2$

Grade bending stress, $f_{m,k} = 16.0 \text{ N/mm}^2$

Design bending strength, $f_{m,d} = k_{mod} \cdot k_{sys} \cdot f_{m,k} / g_m = 0.6 \times 1.0 \times 16.0 / 1.3 = 7.38 \text{ N/mm}^2$

Depth factors, $k_{h,y} = (150/97)^{0.2} = 1.091$; $k_{h,z} = (150/97)^{0.2} = 1.091$ [EC5 3.2]

Applying k_h depth factors $f_{m,y,d} = 1.091 \times 7.38 = 8.06 \text{ N/mm}^2$ $f_{m,z,d} = 1.091 \times 7.38 = 8.06 \text{ N/mm}^2$

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Compression y-y axis

Design compressive stress, $s_{c,0,d} = 6.16 \times 1000 / (97 \times 97) = 0.654 \text{ N/mm}^2$

Effective length = $L_{E,y} = 1.0L = 1.0 \text{ m}$

Slenderness, $l_y = 1.0 \times 1000 / (\sqrt{12}) = 35.7$

$l_{rel,y} = (l_y / p) \cdot \ddot{O}(f_{c,0,k} / E_{0.05}) = (35.7 / p) \cdot \ddot{O}(17.0 / 5,400) = 0.638$ [EC5 6.3.2 Eq.(6.21)]

$k_y = 0.5(1 + B_c(l_{rel,y} - 0.3) + l_{rel,y}^2) = 0.5 \times (1 + 0.2 \times (0.638 - 0.3) + 0.638^2) = 0.737$ [EC5 6.3.2 Eq.(6.27)]

B_c is 0.2 for solid timber, 0.1 for glulam and LVL [EC5 6.3.2 Eq.(6.29)]

$k_{c,y} = 1 / (k_y + \ddot{O}(k_y^2 - l_{rel,y}^2)) = 1 / (0.737 + \ddot{O}(0.737^2 - 0.638^2)) = 0.903$ [EC5 6.3.2 Eq.(6.25)]

Design compressive strength = $k_{c,y} \cdot f_{c,0,d} = 0.903 \times 7.85 = 7.09 \text{ N/mm}^2$

Design axial load resistance = $7.09 \times 97 \times 97 / 1000 = 66.7 \text{ kN} \lll \text{ OK}$

Compression z-z axis

Design compressive stress, $s_{c,0,d} = 6.16 \times 1000 / (97 \times 97) = 0.654 \text{ N/mm}^2$

Effective length = $L_{E,z} = 1.0L = 1.0 \text{ m}$

Slenderness, $l_z = 1.0 \times 1000 / (\sqrt{12}) = 35.7$

$l_{rel,z} = (l_z / p) \cdot \ddot{O}(f_{c,0,k} / E_{0.05}) = (35.7 / p) \cdot \ddot{O}(17.0 / 5,400) = 0.638$ [EC5 6.3.2 Eq.(6.22)]

$k_z = 0.5(1 + B_c(l_{rel,z} - 0.3) + l_{rel,z}^2) = 0.5 \times (1 + 0.2 \times (0.638 - 0.3) + 0.638^2) = 0.737$ [EC5 6.3.2 Eq.(6.28)]

B_c is 0.2 for solid timber, 0.1 for glulam and LVL [EC5 6.3.2 Eq.(6.29)]

$k_{c,z} = 1 / (k_z + \ddot{O}(k_z^2 - l_{rel,z}^2)) = 1 / (0.737 + \ddot{O}(0.737^2 - 0.638^2)) = 0.903$ [EC5 6.3.2 Eq.(6.26)]

Design compressive strength = $k_{c,z} \cdot f_{c,0,d} = 0.903 \times 7.85 = 7.09 \text{ N/mm}^2$

Design axial load resistance = $7.09 \times 97 \times 97 / 1000 = 66.7 \text{ kN} \lll \text{ OK}$

Bending about y-y axis:

Design bending strength, $f_{m,y,d} = 8.06 \text{ N/mm}^2$

Elastic section modulus, $W_{yy} = 97 \times 97^2 / (6 \times 1000) = 152.1 \text{ cm}^4$

Design bending moment = 0.101 kNm

Design bending stress, $s_{m,y,d} = 0.101 \times 1000 / 152.1 = 0.66 \text{ N/mm}^2 \text{ OK}$

Moment resistance = $8.06 \times 152.1 / 1000 = 1.23 \text{ kNm OK}$

Bending about z-z axis:

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N/A

Combined compression and bending

Check $(s_{c,0,d}/k_{c,y} \cdot f_{c,0,d}) + (s_{m,y,d}/f_{m,y,d}) + k_m(s_{m,z,d}/f_{m,z,d}) \leq 1$ [EC5 6.2.4 Eq.(6.23)]

and $(s_{c,0,d}/k_{c,z} \cdot f_{c,0,d}) + k_m(s_{m,y,d}/f_{m,y,d}) + (s_{m,z,d}/k_{c,z} \cdot f_{m,z,d}) \leq 1$ [EC5 6.2.4 Eq.(6.24)]

For rectangular solid wood, glulam and LVL sections $k_m = 0.7$ [EC5 6.1.6]

Ratios: Buckling y-axis, $s_{c,0,d}/k_{c,y} \cdot f_{c,0,d}$: 0.65/7.09 = 0.092

Buckling z-axis, $s_{c,0,d}/k_{c,z} \cdot f_{c,0,d}$: 0.65/7.09 = 0.092

Bending y-axis, $s_{m,y,d}/f_{m,y,d}$: 0.66/8.06 = 0.082

Bending z-axis, $s_{m,z,d}/f_{m,z,d}$: 0.00/8.06 = 0.000

Check 1: $0.092 + 0.082 + 0.7 \times 0.000 = 0.174$ OK [EC5 Eq.(6.23)]

Check 2: $0.092 + 0.7 \times 0.082 + 0.000 = 0.150$ OK [EC5 Eq.(6.24)]

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Beam: Rafters - Roof Openings Span: 2.0 m.

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp
							R2comp

U	G	Roof	1.1x0.8 0	L	0.88	0.88	
---	---	------	-----------	---	------	------	--

U	QH	Roof	0.8x0.8 0	L	0.64	0.64	
---	----	------	-----------	---	------	------	--

Total load (unfactored): 3.04 kN ~~1.52~~ ~~1.52~~

Dead/Permanent (unfactored): 1.76 kN 0.88 0.88

Live/Variable (unfactored): 1.28 kN 0.64 0.64

(6.10): 4.30 kN 2.15 2.15

Load types: U:UDL; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed: QH: Roof

Maximum B.M. = 1.07 kNm (6.10) at 1.00 m. from R1

Maximum S.F. = 2.15 kN (6.10) at R1

Mid-span deflections: Dead: $0.18 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $0.13 \times 10^8 / EI$

Total: $0.32 \times 10^8 / EI$

Timber beam calculation to BS EN1995-1-1 using C16 timber

Use 5no50 x 72 C16 6.7 kg/m approx

$W_{el,y} = 216.0 \text{ cm}^3$ $I_y = 778 \text{ cm}^4$

Timber grade: C16

Grade bending strength, $f_{m,k} = 16.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 3.2 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 5,400 \text{ N/mm}^2$; $E_{0,mean} = 8,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.6$ (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.1$ [EC5 6.6(2)]

Deflection modification factor, $k_{def} = 0.60$ (Service class 1, solid timber/glulam/LVL) [EC5 Table 3.2]

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Bending

Height factor, $k_h = \min((150/h)^{0.2}, 1.3) = (150/72)^{0.2} = 1.16$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_m = 16.0 \times 0.60 \times 1.16 \times 1.1 / 1.30 = 9.41 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 1.07 \times 1000 / 216.0 = 4.97 \text{ N/mm}^2$ OK

Bending resistance = $9.41 \times 216.0 / 1000 = 2.03 \text{ kNm}$

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 250 = 167.5 \text{ mm}$. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / g_m = 3.20 \times 0.60 \times 1.1 / 1.30 = 1.62 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 2.15 \times 1000 \times (3/2) / (167.5 \times 72) = 0.27 \text{ N/mm}^2$ OK

Shear resistance = $1.62 \times 167.5 \times 72 \times (2/3) / 1000 = 13.06 \text{ kN}$

Deflection

Final deflection limit = $L / 250 = 8.00 \text{ mm}$

Final deflection, $u_{fin} = S u_{inst} (1 + \gamma_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where $(G = E / 16)$

$$u_{inst,v} = 1.2 \times 0.760 \times 10^6 / ((8,000 / 16) \times 250 \times 72) = 0.10 \text{ mm}$$

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 10^8 / EI$				Inst. mm	γ_2	k_{def}	Fin. mm
Dead:	0.18	2.95	1.00	0.60	4.72			
Live QH:		0.13	2.14	0.00	0.60	2.14		
Shear deflection:				0.10			0.14	
Total mm.			<u>5.19</u>		<u>7.00</u>			OK

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Beam: Roof Rear Extension Span: 2.8 m.

			Load name	Loading w1	Start x1	Loading w2	End x2	R1comp
								R2comp
U	G		o.w.	0.1	0	L	0.14	0.14
U	G		Roof	0.8x0.4	0	L	0.45	0.45
U	QA		Roof	2.5x0.4	0	L	1.40	1.40
Total load (unfactored): 3.98 kN 1.99 1.99								
Dead/Permanent (unfactored): 1.18 kN 0.59 0.59								
Live/Variable (unfactored): 2.80 kN 1.40 1.40								
(6.10): 5.79 kN 2.89 2.89								

Load types: U:UDL; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed; QA: Residential

Maximum B.M. = 2.03 kNm (6.10) at 1.40 m. from R1

Maximum S.F. = 2.89 kN (6.10) at R1

Mid-span deflections: Dead: $0.34 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $0.80 \times 10^8 / EI$

Total: $1.14 \times 10^8 / EI$

Timber beam calculation to BS EN1995-1-1 using C24 timber

Use 47 x 175 C24 3.5 kg/m approx

$W_{el,y} = 239.9 \text{ cm}^3$ $I_y = 2,099 \text{ cm}^4$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.6$ (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.1$ [EC5 6.6(2)]

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Deflection modification factor, $k_{def} = 0.60$ (Service class 1, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = 1.0$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_m = 24.0 \times 0.60 \times 1.00 \times 1.1 / 1.30 = 12.18 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 2.03 \times 1000 / 239.9 = 8.44 \text{ N/mm}^2$ OK

Bending resistance = $12.18 \times 239.9 / 1000 = 2.92 \text{ kNm}$

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 47 = 31.5 \text{ mm}$. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / g_m = 4.00 \times 0.60 \times 1.1 / 1.30 = 2.03 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 2.89 \times 1000 \times (3/2) / (31.5 \times 175) = 0.79 \text{ N/mm}^2$ OK

Shear resistance = $2.03 \times 31.5 \times 175 \times (2/3) / 1000 = 7.46 \text{ kN}$

Deflection

Final deflection limit = $L / 250 = 11.20 \text{ mm}$

Final deflection, $u_{fin} = S u_{inst} (1 + y_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where ($G = E / 16$)

$$u_{inst,v} = 1.2 \times 1.39 \times 10^6 / ((11,000 / 16) \times 47 \times 175) = 0.30 \text{ mm}$$

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 1e^8 / EI$				Inst. mm	y_2	k_{def}	Fin. mm
Dead:	0.34	1.46	1.00	0.60	2.33			
Live QA:		0.80	3.47	0.30	0.60	4.09		
Shear deflection:				0.30			0.39	
Total mm.			<u>5.22</u>		<u>6.80</u>	OK		

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Beam: Beam 2 Span: 5.2 m.

			Load name	Loading w1	Start x1	Loading w2	End x2	R1comp
								R2comp
U	G		o.w.	0.3	0	L	0.78	0.78
U	G		Roof	0.8x2.5	0	L	5.20	5.20
U	QA		Roof	2.5x2.5	0	L	16.25	16.25
Total load (unfactored): 44.46 kN					22.23	22.23		
Dead/Permanent (unfactored): 11.96 kN					5.98	5.98		
Live/Variable (unfactored): 32.50 kN					16.25	16.25		
(6.10): 64.90 kN					32.45	32.45		

Load types: U:UDL; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed; QA: Residential

Maximum B.M. = 42.18 kNm (6.10) at 2.60 m. from R1

Maximum S.F. = 32.45 kN (6.10) at R1

Mid-span deflections: Dead: $21.9 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $59.5 \times 10^8 / EI$

Total: $81.4 \times 10^8 / EI$

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : 203 x 133 x 30 UB S355 (Class 1, plastic)

D=206.8 mm B=133.9 mm t=6.4 mm T=9.6 mm $I_y=2,900 \text{ cm}^4$ $i_z=3.17 \text{ cm}$ $W_{pl,y}=314 \text{ cm}^3$
 $W_{el,y}=280 \text{ cm}^3$

Classification: Flange: $c/t = 56.1/9.6 = 5.85 \leq 9e$ (7.32): Class 1, plastic

EC3 Table 5.2 Web: $c/t = 172.4/6.4 = 26.9 \leq 72e$ (58.6): Class 1, plastic

Shear

Design shear force, $V_{Ed} = 32.45 \text{ kN}$

Shear area, $A_v = A - 2bt_f + (t_w + 2r)t_f = 38.2 \times 100 - 2 \times 134 \times 9.60 + (6.40 + 2 \times 7.60) \times 9.60 = 1,456 \text{ mm}^2$ [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd} = A_v \cdot (f_y / \sqrt{3}) / g_{M0} = 1,456 \times (355 / \sqrt{3}) / (1.0 \times 1000) = 299 \text{ kN}$ (≥ 32.45) OK [EC3 6.2.6]

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Shear buckling: $h_w/t_w = 187.6/6.4 = 29.31 \leq 72e$ (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, $M_{Ed} = 42.18$ kNm

Moment resistance, $M_{c,y,Rd} = f_y \cdot W_{pl,y} = 355 \times 314/1000 = 111.5$ kNm OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

Support conditions R1/R2: Compression flange laterally restrained. Nominal torsional restraint. Both flanges free to rotate on plan (1.0L)

Design buckling resistance moment, $M_{b,Rd} = C_{LT,mod} \cdot M_{c,Rd}$

$C_{LT,mod} = C_{LT}/f$ (but $\leq 1/\bar{\lambda}_{LT}^2$ and ≤ 1.0) [Eq.6.58]

$f = 1 - 0.5(1-k_c)[1 - 2(\bar{\lambda}_{LT} - 0.8)^2]$ 6.3.2.3(2) $k_c = 1/\ddot{O}C_1$ [NA2.18]

Use buckling curve b: $a = 0.340$ [EC3 Tables 6.3/6.4 NA 2.17]

$C_{LT} = 1/[f_{LT} + \ddot{O}(F_{LT}^2 - b\bar{\lambda}_{LT}^2)]$ [EC3 (6.57)]

$F_{LT} = 0.5[1 + a_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + b\bar{\lambda}_{LT}^2]$

$\bar{\lambda}_{LT,0} = 0.4$ $b = 0.75$ [EC3 UK NA 2.17]

$\bar{\lambda}_{LT} = \ddot{O}(f_y \cdot W_{pl,y}/M_{cr})$

$M_{cr} = C_1(p^2 E I_z / L_e^2) \ddot{O}(I_w / I_z + L_e^2 G I_T / p^2 E I_z)$ [SN003: k and k_w taken as 1.0, conservative]

$W_y = 314.0$ cm³ $I_w = 0.037$ dm⁶ $I_T = 10.3$ cm⁴ $I_z = 385$ cm⁴ $G = 81,000$ N/mm²

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{\lambda}_z$	$\bar{\lambda}_{LT}$	f_{LT}	C_{LT}	$C_{LT,mod}$	M
c,Rd $M_{b,Rd}$											
0.00-5.20	5.20	42.2	1.13*	0.94	65.0	2.147	1.310	1.298	0.518	0.526	
	111.5	58.6	OK								

* $C_1 = 4M_{max}/\ddot{O}(M_{max}^2 + 4M_a^2 + 7M_b^2 + 4M_c^2) \leq 2.5$ [$M_{a,b,c}$ quarter pt moments] [Wong & Driver, AISC Eng. Journal, Q1 2010]

Combined bending and shear

$V_{Ed} \leq 0.5 V_{c,Rd}$: Check for bending/shear interaction not required [EC3 6.2.8(2)]

Web capacity at bearings

Resistance of web to transverse forces, $F_{Rd} = f_{yw} \cdot L_{eff} \cdot t_w / g_{M1}$

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$$f_{yw} = 355 \text{ N/mm}^2$$

$$L_{eff} = c_F l_y$$

$$c_F = 0.5/\bar{\lambda}_F \leq 1.0$$

$$\bar{\lambda}_F = \sqrt{I_y \cdot t_w \cdot f_{yw} / F_{cr}}$$

$$F_{cr} = 0.9 k_F \cdot E \cdot (t_w^3 / h_w)$$

$$k_F = 2 + 6((S_s + c)/h_w)^2 \leq 6$$

Type (c) load application assumed:

$$\text{Effective loaded length, } l_y = \min S_s + 2t_f(1 + \ddot{O}(m_1 + m_2)) \text{ [EC3-1-5 Eq.6.10]},$$

$$l_e + t_f \ddot{O}(m_1/2 + (l_e/t_f)^2 + m_2) \text{ [Eq.6.11]}$$

$$\text{or } l_e + t_f \ddot{O}(m_1 + m_2) \text{ [Eq.6.12]}$$

$$l_e = k_F \cdot E \cdot t_w^2 / (2 \cdot f_{yw} \cdot h_w) \leq s_s + c \text{ [Eq.6.13]}$$

$$m_1 = f_{yf} \cdot b_f / (f_{yw} \cdot t_w) \quad m_2 = 0.02(h_w/t_f)^2 \text{ if } \bar{\lambda}_F > 0.5 \text{ else } 0.0$$

Reactions R1 & 2: 32.45 kN

Required minimum stiff bearing length, $S_s = 0 \text{ mm}$

c (end of beam to stiff bearing) taken as 0.0

$$m_1 = 20.9 \quad m_2 = 0.00 \quad F_{cr} = 528 \text{ kN} \quad k_F = 2.00 \quad l_e = 0.00 \quad l_y = 31.05 \quad \bar{\lambda}_F = 0.365 \quad c_F = 1.00 \quad L_{eff} = 31.05$$

Resistance of web to transverse forces, $F_{R,d} = 355 \times 31.05 \times 6.4 / (1000 \times 1.0) = 70.5 \text{ kN OK}$

Deflection

$$\text{LL deflection} = 59.5 \times 1e8 / (210,000 \times 2,900) = 9.8 \text{ mm (L/532) OK}$$

$$\text{TL deflection} = 81.4 \times 1e8 / (210,000 \times 2,900) = 13.4 \text{ mm (L/389)}$$

Bearings

$$203 \times 133 \times 30 \text{ UB stiff bearing length, } b_1 = t + 1.6r + 2T = 37.8 \text{ mm}$$

R1 (32.45 kN) : 300 x 100 x 100h mm padstone

Masonry design strength = 1.50 N/mm^2 (User-entered value)

$$\text{Stress under padstone} = 32.45 \times 1000 / 300 \times 100 = 1.08 \text{ N/mm}^2 \text{ OK}$$

R2 (32.45 kN) : None

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Beam: Beam 3 Span: 2.5 m.

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U	G	o.w.	0.3 0	L	0.38 0.38			
U	G	Wall	3.35x0.5 0	L	2.09 2.09			
U	G	Roof	0.8x0.4 0	L	0.40 0.40			
U	QH	Roof	2.5x0.4 0	L	1.25 1.25			
P	G	Bm: Beam 2 [5.0 m.] : R1		5.98 0.2		5.50 0.48		
P	QA	Bm: Beam 2 [5.0 m.] : R1		16.25 0.2		14.95 1.30		
Total load (unfactored): 30.47 kN				24.57 5.90				
Dead/Permanent (unfactored): 11.72 kN				8.37 3.35				
Live/Variable (unfactored): 18.75 kN				16.20 2.55				
(6.10): 43.94 kN			35.60 8.34					

Load types: U:UDL; P:Point load; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed; QH: Roof

Maximum B.M. = 7.57 kNm (6.10) at 0.69 m. from R1

Maximum S.F. = 35.60 kN (6.10) at R1

Mid-span deflections: Dead: $1.63 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $1.77 \times 10^8 / EI$

Total: $3.40 \times 10^8 / EI$

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : 203 x 133 x 30 UB S355 (Class 1, plastic)

D=206.8 mm B=133.9 mm t=6.4 mm T=9.6 mm $I_y=2,900 \text{ cm}^4$ $i_z=3.17 \text{ cm}$ $W_{pl,y}=314 \text{ cm}^3$
 $W_{el,y}=280 \text{ cm}^3$

Classification: Flange: $c/t = 56.1/9.6 = 5.85 \leq 9e$ (7.32): Class 1, plastic

EC3 Table 5.2 Web: $c/t = 172.4/6.4 = 26.9 \leq 72e$ (58.6): Class 1, plastic

Shear

Design shear force, $V_{Ed} = 35.60 \text{ kN}$

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Shear area, $A_v = A - 2bt_f + (t_w + 2r)t_f = 38.2 \times 100 - 2 \times 134 \times 9.60 + (6.40 + 2 \times 7.60) \times 9.60 = 1,456 \text{ mm}^2$ [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd} = A_v \cdot (f_y / \sqrt{3}) / g_{M0} = 1,456 \times (355 / \sqrt{3}) / (1.0 \times 1000) = 299 \text{ kN}$ (≥ 35.60) OK [EC3 6.2.6]

Shear buckling: $h_w/t_w = 187.6/6.4 = 29.31 \leq 72e$ (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, $M_{Ed} = 7.57 \text{ kNm}$

Moment resistance, $M_{c,y,Rd} = f_y \cdot W_{pl,y} = 355 \times 314/1000 = 111.5 \text{ kNm}$ OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

Support conditions R1/R2: Compression flange laterally restrained. Nominal torsional restraint. Both flanges free to rotate on plan (1.0L)

Design buckling resistance moment, $M_{b,Rd} = C_{LT,mod} \cdot M_{c,Rd}$

$C_{LT,mod} = C_{LT}/f$ (but $\leq 1/\bar{\lambda}_{LT}^2$ and ≤ 1.0) [Eq.6.58]

$f = 1 - 0.5(1 - k_c)[1 - 2(\bar{\lambda}_{LT} - 0.8)^2]$ 6.3.2.3(2) $k_c = 1/\bar{\phi}_{C1}$ [NA2.18]

Use buckling curve b: $a = 0.340$ [EC3 Tables 6.3/6.4 NA 2.17]

$C_{LT} = 1/[f_{LT} + \bar{\phi}(F_{LT}^2 - b\bar{\lambda}_{LT}^2)]$ [EC3 (6.57)]

$F_{LT} = 0.5[1 + a_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + b\bar{\lambda}_{LT}^2]$

$\bar{\lambda}_{LT,0} = 0.4$ $b = 0.75$ [EC3 UK NA 2.17]

$\bar{\lambda}_{LT} = \sqrt{\phi(f_y \cdot W_{pl,y}/M_{cr})}$

$M_{cr} = C_1(p^2 E I_z / L_e^2) \bar{\phi}(I_w/I_z + L_e^2 G I_T / p^2 E I_z)$ [SN003: k and k_w taken as 1.0, conservative]

$W_y = 314.0 \text{ cm}^3$ $I_w = 0.037 \text{ dm}^6$ $I_T = 10.3 \text{ cm}^4$ $I_z = 385 \text{ cm}^4$ $G = 81,000 \text{ N/mm}^2$

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{\lambda}_z$	$\bar{\lambda}_{LT}$	f_{LT}	C_{LT}	$C_{LT,mod}$	M
c_{Rd}	$M_{b,Rd}$										
0.00-2.50	2.50	7.6	1.15*	0.93	187.8	1.032	0.770	0.786	0.833	0.863	
	111.5	96.2	OK								

* $C_1 = 4M_{max}/\bar{\phi}(M_{max}^2 + 4M_a^2 + 7M_b^2 + 4M_c^2) \leq 2.5$ [$M_{a,b,c}$ quarter pt moments] [Wong & Driver, AISC Eng. Journal, Q1 2010]

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Combined bending and shear

$V_{Ed} \leq 0.5 V_{c,Rd}$: Check for bending/shear interaction not required [EC3 6.2.8(2)]

Web capacity at bearings

Resistance of web to transverse forces, $F_{Rd} = f_{yw} \cdot L_{eff} \cdot t_w / g_{M1}$

$$f_{yw} = 355 \text{ N/mm}^2 \quad L_{eff} = c_F l_y$$

$$c_F = 0.5 / \bar{\lambda}_F \leq 1.0 \quad \bar{\lambda}_F = \sqrt{l_y \cdot t_w \cdot f_{yw} / F_{cr}}$$

$$F_{cr} = 0.9 k_F \cdot E \cdot (t_w^3 / h_w) \quad k_F = 2 + 6((S_s + c) / h_w)^2 \leq 6$$

Type (c) load application assumed:

Effective loaded length, $l_y = \min S_s + 2t_f(1 + \ddot{O}(m_1 + m_2))$ [EC3-1-5 Eq.6.10],

$$l_e + t_f \ddot{O}(m_1/2 + (l_e/t_f)^2 + m_2) \text{ [Eq.6.11]}$$

$$\text{or } l_e + t_f \ddot{O}(m_1 + m_2) \text{ [Eq.6.12]}$$

$$l_e = k_F \cdot E \cdot t_w^2 / (2 \cdot f_{yw} \cdot h_w) \leq S_s + c \text{ [Eq.6.13]}$$

$$m_1 = f_{yf} \cdot b_f / (f_{yw} \cdot t_w) \quad m_2 = 0.02(h_w/t_f)^2 \text{ if } \bar{\lambda}_F > 0.5 \text{ else } 0.0$$

Reaction R1: 35.60 kN

Required minimum stiff bearing length, $S_s = 0 \text{ mm}$

c (end of beam to stiff bearing) taken as 0.0

$$m_1 = 20.9 \quad m_2 = 0.00 \quad F_{cr} = 528 \text{ kN} \quad k_F = 2.00 \quad l_e = 0.00 \quad l_y = 31.05 \quad \bar{\lambda}_F = 0.365 \quad c_F = 1.00 \quad L_{eff} = 31.05$$

Resistance of web to transverse forces, $F_{R,d} = 355 \times 31.05 \times 6.4 / (1000 \times 1.0) = 70.5 \text{ kN OK}$

Reaction R2: 8.344 kN

Required minimum stiff bearing length, $S_s = 0 \text{ mm}$

c (end of beam to stiff bearing) taken as 0.0

$$m_1 = 20.9 \quad m_2 = 0.00 \quad F_{cr} = 528 \text{ kN} \quad k_F = 2.00 \quad l_e = 0.00 \quad l_y = 31.05 \quad \bar{\lambda}_F = 0.365 \quad c_F = 1.00 \quad L_{eff} = 31.05$$

Resistance of web to transverse forces, $F_{R,d} = 355 \times 31.05 \times 6.4 / (1000 \times 1.0) = 70.5 \text{ kN OK}$

Deflection

LL deflection = $1.77 \times 1e8 / (210,000 \times 2,900) = 0.3 \text{ mm}$ (L/8615) OK

TL deflection = $3.40 \times 1e8 / (210,000 \times 2,900) = 0.6 \text{ mm}$ (L/4481)

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Notched beam flexible end plate connection check (EC3 + UK NA)

Calculations are based on NCCI SN014a: Shear resistance of a simple end plate connection

Location: Beam 2 and Beam 3 Connection

Factored load at support = 32.45 kN

Beam: S355 203x133x30 UB connection to web of S355 203x133x30 UB

Section dims: 203 x 133 x 30 UB: D=206.8 mm B=133.9 mm t=6.4 mm T=9.6 mm $I_y=2,900 \text{ cm}^4$
 $i_z=3.17 \text{ cm}$ $W_{pl,y}=314 \text{ cm}^3$ $W_{el,y}=280 \text{ cm}^3$

203 x 133 x 30 UB: D=206.8 mm B=133.9 mm t=6.4 mm T=9.6 mm $I_y=2,900 \text{ cm}^4$ $i_z=3.17 \text{ cm}$
 $W_{pl,y}=314 \text{ cm}^3$ $W_{el,y}=280 \text{ cm}^3$

Notch dimensions: Top: 60 x 18mm Bottom: 60 x 19mm

Use 150 x 98 x 6mm end plate 2No M16 8.8 bolts at 90 mm centres (v), 50 mm centres (h), 6 mm fillet welds

Check detailing

Check plate/cleat length (150mm) $\geq 0.6 \times \text{beam depth (124mm)}$ OK

Check bolt pitch (90mm) $\geq 2.2d_0$ (39.6mm) OK

Check bolt end clearance (30mm) $\geq 1.2d_0$ (21.6mm) OK

Check bolt edge clearance (24mm) $\geq 1.2d_0$ (21.6mm) OK

Check plate thickness allows for adequate ductility [SN014a 12]

Limiting thickness = $(d/2.8) \cdot \sqrt{f_{ub}/f_{y,p}} = (16/2.8) \times \sqrt{800/275} = 9.7 \text{ mm}$

Check end plate thickness, t_p (6) $\leq 9.7 \text{ mm}$ OK

Weld design

Full strength double fillet welds required. Minimum throat size = $0.45 \times \text{beam web thickness}$ [NCCI SN014a 11]

Minimum weld throat size = 2.9mm; actual = $0.7 \times 6 = 4.2 \text{ mm}$ OK

Bolt resistance (M16 8.8 bolts)

Shear resistance per bolt = $F_{v,Rd} = a_v \cdot f_{ub} \cdot A_s / g_{M2} = 0.6 \times 800 \times 157 / (1.25 \times 1000) = 60.3 \text{ kN}$ [EC3-1-8 Table 3.4]

Bolt bearing resistance, $F_{b,Rd} = k_1 \cdot a_b \cdot f_u \cdot d \cdot t_p / g_{M2}$ [EC3-1-8 Table 3.4]

where $a_v = 0.6$; $f_{ub} = 800 \text{ N/mm}^2$; $A_s = 157.0 \text{ mm}^2$; $g_{M2} = 1.25$

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a_b is the smallest of a_d , f_{ub}/f_u or 1.0

$a_d = e_1/3d_0$ for end bolts, $(p_1/3d_0) - 0.25$ for inner bolts (*bolts in direction of load transfer*)

$k_1 = 2.8e_2/d_0 - 1.7$ but ≤ 2.5 for edge bolts (*bolts perpendicular to load transfer*)

and $1.4p_2/d_0 - 1.7$ but ≤ 2.5 for edge bolts (*do.*)

d_0 : bolt hole diameter (18 mm); e_1 : end distance; e_2 : edge distance; p_1 : pitch

f_u for $t = 3-100\text{mm}$: S275: 410N/mm²; S355: 470N/mm² [EC3-1-1 UK NA 2.4: use product values not those in EC3]

V_{Rd,1}, Bolts in shear

$F_{v,rd,red} = 0.8 F_{v,rd} = 0.8 \times 60.3 = 48.23 \text{ kN}$ (*0.8 reduction factor allows for tension in bolt*)

$V_{Rd,1} = n \cdot F_{v,rd,red} = 2 \times 2 \times 48.2 = 192.9 \text{ kN OK}$ [NCCI SN014a 3]

V_{Rd,2}, End plate in bearing

Bolt bearing resistance, $F_{b,Rd} = k_1 \cdot a_b \cdot f_u \cdot d \cdot t_p / g_{M2}$

Top bolts: $F_{b,Rd} = 2.03 \times 0.556 \times 410 \times 16 \times 6 / (1.25 \times 1000) = 35.6 \text{ kN}$

Other bolts: $F_{b,Rd} = 2.03 \times 1.000 \times 410 \times 16 \times 6 / (1.25 \times 1000) = 64.0 \text{ kN}$

As $F_{v,rd,red} (48.23 \text{ kN}) < F_{b,Rd,max} (64.0 \text{ kN})$, $V_{Rd,2} = n \cdot F_{b,rd,min}$

$V_{Rd,2} = 2 \times 2 \times 35.6 = 142.3 \text{ kN OK}$ [NCCI SN014a 4]

V_{Rd,3}, End plate in shear (gross section)

$V_{Rd,3} = A_v \cdot f_{y,p} / (1.27 \cdot \sqrt{3} g_{M0}) = 150 \times 6 \times 2 \times 275 / (1.27 \cdot \sqrt{3} \times 1.0 \times 1000) = 225.0 \text{ kN OK}$ [NCCI SN014a 6]

V_{Rd,4}, End plate in shear (net section)

$V_{Rd,4} = A_{v,net} \cdot f_{u,p} / (\sqrt{3} g_{M2}) = 114 \times 6 \times 2 \times 410 / (\sqrt{3} \times 1.10 \times 1000) = 294 \text{ kN OK}$ [NCCI SN014a 7]

V_{Rd,5}, End plate in shear (block shear)

$V_{Rd,5} = f_{u,p} A_{nt} / g_{M2} + f_{y,p} A_{nv} / (\sqrt{3} g_{M0})$ [EC3-1-8 3.10.2(2)]

A_{nt} = net area in tension = $2 \times 15 \times 6 = 180\text{mm}^2$

A_{nv} = net area in shear = $2 \times 93 \times 6 = 1,116\text{mm}^2$

$V_{Rd,5} = ((410 \times 180 / 1.10) + (275 \times 1,116 / (\sqrt{3} \times 1.0))) / 1000 = 244.3 \text{ kN OK}$ [NCCI SN014a 8]

V_{Rd,6}, End plate in bending

As $h_p (150\text{mm}) \geq 1.36p_3 (68.0\text{mm})$ check not required [NCCI SN014 9]

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V_{Rd,7}, Beam web in shear

$V_{Rd,7} = 0.9 \cdot A_v (f_{y,b} / \sqrt{3}) / g_{M0} = 0.9 \times 150 \times 6.4 \times 355 / (\sqrt{3} \times 1.0 \times 1000) = 177.1 \text{ kN OK}$ [0.9 factor see SCI P387]

V_{Rd,8}, Supporting member in bearing

$$V_{Rd,8} = n F_{b,Rd}$$

$$F_{b,Rd} = k_1 \cdot a_b \cdot f_u \cdot d \cdot t / g_{M2} = 2.19 \times 1.00 \times 470 \times 16 \times 6.4 / (1.25 \times 1000) = 84.3 \text{ kN}$$

$$V_{Rd,8} = 2 \times 2 \times 84.3 = 337.1 \text{ kN OK [NCCI SN014a 5]}$$

Ductility requirement

To ensure adequate ductility check $t_p (6) \leq (d/2.8) \sqrt{f_{ub}/f_{y,p}}$ (9.7mm) OK [SN014a 12]

Rotational capacity has not been checked

Summary:

V _{Rd,1}	Bolts in shear	193	kN
V _{Rd,2}	End plate in bearing	142	kN <<<
V _{Rd,3}	End plate in shear (gross section)	225	kN
V _{Rd,4}	End plate in shear (net section)	294	kN
V _{Rd,5}	End plate in shear (block shear)	244	kN
V _{Rd,6}	End plate in bending	N/A	
V _{Rd,7}	Beam web in shear	177	kN
V _{Rd,8}	Supporting member in bearing	337	kN

Critical design check: 2 - End plate in bearing: Connection resistance = 142kN OK

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Beam: Roof Rear Extension Span: 2.8 m.

			Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U	G		o.w.	0.15	0	L	0.21	0.21	
U	G		Roof	0.8x1.8	0	L	2.02	2.02	
U	QA		Roof	2.5x1.8	0	L	6.30	6.30	
Total load (unfactored): 17.05 kN						8.53	8.53		
Dead/Permanent (unfactored): 4.45 kN						2.23	2.23		
Live/Variable (unfactored): 12.60 kN						6.30	6.30		
(6.10): 24.91 kN						12.46	12.46		

Load types: U:UDL; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed; QA: Residential

Maximum B.M. = 8.72 kNm (6.10) at 1.40 m. from R1

Maximum S.F. = 12.46 kN (6.10) at R1

Mid-span deflections: Dead: $1.27 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $3.60 \times 10^8 / EI$

Total: $4.87 \times 10^8 / EI$

Timber beam calculation to BS EN1995-1-1 using C24 timber

Use 3no 47 x 175 C24 10.4 kg/m approx

$W_{el,y} = 719.7 \text{ cm}^3$ $I_y = 6,297 \text{ cm}^4$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.6$ (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.1$ [EC5 6.6(2)]

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Deflection modification factor, $k_{def} = 0.60$ (Service class 1, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = 1.0$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_m = 24.0 \times 0.60 \times 1.00 \times 1.1 / 1.30 = 12.18 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 8.72 \times 1000 / 720 = 12.11 \text{ N/mm}^2$ OK

Bending resistance = $12.18 \times 720 / 1000 = 8.77 \text{ kNm}$

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 141 = 94.5 \text{ mm}$. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / g_m = 4.00 \times 0.60 \times 1.1 / 1.30 = 2.03 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 12.46 \times 1000 \times (3/2) / (94.5 \times 175) = 1.13 \text{ N/mm}^2$ OK

Shear resistance = $2.03 \times 94.5 \times 175 \times (2/3) / 1000 = 22.38 \text{ kN}$

Deflection

Final deflection limit = $L / 250 = 11.20 \text{ mm}$

Final deflection, $u_{fin} = S u_{inst} (1 + y_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where ($G = E / 16$)

$$u_{inst,v} = 1.2 \times 5.97 \times 10^6 / ((11,000 / 16) \times 141 \times 175) = 0.42 \text{ mm}$$

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 1e^8 / EI$				Inst. mm	y_2	k_{def}	Fin. mm
Dead:	1.27	1.84	1.00	0.60	2.94			
Live QA:		3.60	5.20	0.30	0.60	6.14		
Shear deflection:				0.42			0.54	
Total mm.			<u>7.46</u>		<u>9.62</u>	OK		

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Column calculation to EN1993-1-1 using S355 steel

Location: Column 1

Length: 3.0 m.

Pos	Dur	Load	kN	Factored	Offset	Moment y-y	
				6.10	6.10	6.10	
1	G	B/F Bm: Beam 2 [5.2 m.] : R1	5.98	8.07	100	1.63	
1	QA	B/F Bm: Beam 2 [5.2 m.] : R1	16.25	24.38	100	4.91	
4	G	B/F Bm: Beam 3 [2.5 m.] : R1	8.37	11.30	100	-	
1.16							
4	QA	B/F Bm: Beam 3 [2.5 m.] : R1	14.95	22.42	100	-	
2.30							
4	QH	B/F Bm: Beam 3 [2.5 m.] : R1	1.25	1.88	100	-	
0.19							
Total load			46.80	68.05	6.54	-3.66	

Load offsets are measured in mm. from faces of member; moments in kNm

Load durations: G: Dead

SECTION SIZE : 203 x 102 x 23 UB S355

Section properties: B = 101.8mm D = 203.2mm T = 9.3mm t = 5.4mm $A_g = 29.4\text{cm}^2$

$i_y = 8.46\text{cm}$ $i_z = 2.36\text{cm}$ $W_{pl,y} = 234\text{cm}^3$ $W_{el,y} = 207\text{cm}^3$ $W_{pl,z} = 49.7\text{cm}^3$ $W_{el,z} = 32.2\text{cm}^3$

Design strength, $f_y = 355\text{ N/mm}^2$ $e = 0.814$

Classification: Flange: $c/t = 40.6/9.3 = 4.37 \leq 9e$ (7.32): Class 1, plastic

[Table 5.2] Web: $c/t = 169.4/5.4 = 31.4 \leq 396e/(13a - 1) = 47.0$: Class 1, plastic

$a = 0.5(1 + N_{Ed}/(f_y \cdot c \cdot t_w)) = 0.5 \times (1 + 68.0 \times 1000/(355 \times 169 \times 5.40)) = 0.605$ [SCI P362

Table 5.1 note 1]

Major axis: $L_{Ex} = 1.0L = 3.00\text{ m}$. Slenderness, $I_y = 3.00 \times 100/8.46 = 35.5$

Minor axis: $L_{Ey} = 1.0L = 3.00\text{ m}$. Slenderness, $I_z = 3.00 \times 100/2.36 = 127$

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This calculation is based on NCCI SN048b - only valid if bending moment diagrams about each axis are linear and the member is restrained at each floor level but unrestrained between floors. Only Class 1, 2 or 3 hot rolled I or H sections or rectangular hollow sections can be considered

Compression

Design axial load, $N_{Ed} = 68.0$ kN

Design compression resistance, $N_{c,Rd} = A \cdot f_y / g_{M0} = 29.40 \times 100 \times 355 / (1.0 \times 1000) = 1,044$ kN OK

Calculate flexural buckling resistances, $N_{b,Rd}$

$$I_1 = 93.9e = 93.9 \times 0.814 = 76.4$$

Buckling about y-y (major) axis

$$\bar{\lambda}_y = I_y / I_1 = 35.5 / 76.4 = 0.464 \quad [\text{EC3 6.3.1.3}]$$

$$\text{Use curve a: } a = 0.210 \quad f = 0.5(1 + a(\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) = 0.635 \quad [\text{EC3 (6.49)}]$$

$$\text{Flexural buckling reduction factor, } c_y = 1 / (f + \ddot{O}(f^2 - \bar{\lambda}_y^2) <'>) = 0.935 \quad [\text{EC3 (6.49)}]$$

$$\text{Design buckling resistance, } N_{b,y,Rd} = c_y A \cdot f_y / g_{M1} = 0.935 \times 29.40 \times 100 \times 355 / (1.0 \times 1000) = 976 \text{ kN OK} \quad [\text{EC3 (6.47)}]$$

Buckling about z-z (minor) axis

$$\bar{\lambda}_z = I_z / I_1 = 127 / 76.4 = 1.66 \quad [\text{EC3 6.3.1.3}]$$

$$\text{Use curve b: } a = 0.340 \quad f = 0.5(1 + a(\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) = 2.13 \quad [\text{EC3 Table 6.2}]$$

$$\text{Flexural buckling reduction factor, } c_z = 1 / (f + \ddot{O}(f^2 - \bar{\lambda}_z^2) <'>) = 0.288 \quad [\text{EC3 (6.49)}]$$

$$\text{Design buckling resistance, } N_{b,z,Rd} = c_z A \cdot f_y / g_{M1} = 0.288 \times 29.40 \times 100 \times 355 / (1.0 \times 1000) = 301 \text{ kN OK} \quad [\text{EC3 (6.47)}]$$

Bending about y-y (major) axis:

Design moment, $M_{y,Ed} = 6.54$ kNm

$$\text{Moment resistance, } M_{c,y,Rd} = f_y \cdot W_{pl,y} = 355 \times 234 / 1000 = 83.1 \text{ kNm OK}$$

Calculate buckling resistance moment

$$\text{Design buckling resistance moment, } M_{b,y,Rd} = c_{LT,mod} \cdot M_{c,y,Rd}$$

$$M_{cr} = C_1(p^2 E I_z / L_{eff}^2) [\ddot{O}(I_w / I_z + L_{eff}^2 G I_t / (p^2 E I_z))] = 59.0 \quad [\text{NCCI SN003 2(1)}]$$

$$C_1 = 1.0 \text{ (conservative)} \quad \bar{\lambda}_{LT} = \ddot{O}(M_{c,y,Rd} / M_{cr}) = 1.19$$

$$\bar{\lambda}_{LT,0} = 0.4 \quad b = 0.75 \quad [\text{EC3 UK NA 2.17}]$$

$$\text{Use buckling curve b: } a = 0.340 \quad [\text{EC3 Tables 6.3/6.4 NA2.17}]$$

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$$\bar{\tau}_{LT} = 0.5[1 + a_{LT}(\bar{\tau}_{LT} - \bar{\tau}_{LT,0}) + b\bar{\tau}_{LT}^2] = 1.16$$

$$c_{LT} = 1/[f_{LT} + \ddot{O}(f_{LT}^2 - b\bar{\tau}_{LT}^2)] <'> = 0.587 \quad [\text{EC3 (6.56)}]$$

$$M_{b,y,Rd} = c_{LT} \cdot M_{c,y,Rd} / g_M = 0.587 \times 83.1 / 1.0 = 48.76 \text{ kNm}$$

Bending about z-z (minor) axis:

Design moment, $M_{z,Ed} = 3.66 \text{ kNm}$

Moment resistance, $M_{z,cb,Rd} = f_y \cdot W_{pl,z} = 355 \times 49.70 / 1000 = 17.64 \text{ kNm}$

Summary:

$$N_{Ed} / N_{min,b,Rd} = 68.0 / 301 = 0.226 \quad [1]$$

$$M_{y,Ed} / M_{bs} = 6.54 / 48.76 = 0.134 \quad [2]$$

$$M_{z,Ed} / M_{z,cb,Rd} = 3.66 / 17.64 = 0.207 \quad [3]$$

Sum of stress ratios $[1] + [2] + 1.5 \times [3] = 0.671$ **OK subject to all SN048b criteria being complied with**

Baseplate calculation (considering axial load only)

Design compression force on baseplate, $N_{Ed} = 68.0 \text{ kN}$

Concrete grade C20/25: cylinder strength, $f_{ck} = 20 \text{ N/mm}^2$

Concrete strength, $f_{cd} = a_{cc} \cdot f_{ck} / g_M = 0.85 \times 20 / 1.5 = 11.3 \text{ N/mm}^2$

Concrete design strength, $f_{jd} = b_j \cdot a \cdot f_{cd} = 11.3 \text{ N/mm}^2$ ($b = 2/3$; a taken as 1.5) [SN037 A2]

Minimum area required $= F_c / f_{jd} = 68.0 \times 1000 / 11.3 = 6,004 \text{ mm}^2$

Base is sized as a large projection base plate (equal projection from all faces of member) [EC3-1-8 6.2.5]

Min required projection, $c = 3.81 \text{ mm}$ Minimum base plate size = $211 \times 110 \text{ mm}$

Minimum thickness $= c \ddot{O}(3 \times f_{jd} / f_y) <'> = 3.81 \ddot{O}(3 \times 11.3 / 275) <'> = 1.34 \text{ mm}$

Use **225 x 175 x 5mm S275** base plate

Pressure on underside of plate $= 68.0 / 6,004 = 11.3 \text{ N/mm}^2$

Bending stress at root of plate projection $= 11.3 \times 3.81 \times (3.81/2) / (5 \times 5/6) = 19.7 \text{ N/mm}^2$ OK

As a taken as 1.5, min. base size = $1.5 \times$ min plate size = $165 \times 317 \text{ mm}$. - actual size to be determined by soil conditions

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Flexible end plate connection check (EC3 + UK NA)

Calculations are based on NCCI SN014a: Shear resistance of a simple end plate connection

Location: Column 1 and Beam 2 Connection

Factored load at support = 32.45 kN

Beam: S355 203x133x30 UB connection to web of S355 203x102x23 UB column

Section dims: 203 x 133 x 30 UB: D = 206.8 mm B = 133.9 mm t = 6.4 mm T = 9.6 mm r = 7.60 mm

203 x 102 x 23 UB: D=203.2 mm B=101.8 mm t=5.4 mm T=9.3 mm $I_y=2,100 \text{ cm}^4$ $i_z=2.36 \text{ cm}$ $W_{pl,y}=234 \text{ cm}^3$ $W_{el,y}=207 \text{ cm}^3$

Use 150 x 98 x 8mm end plate 2No M16 8.8 bolts at 90 mm centres (v), 50 mm centres (h), 6 mm fillet welds

Check detailing

Check plate/cleat length (150mm) $\geq 0.6 \times$ beam depth (124mm) OK

Check bolt pitch (90mm) $\geq 2.2d_0$ (39.6mm) OK

Check bolt end clearance (30mm) $\geq 1.2d_0$ (21.6mm) OK

Check bolt edge clearance (24mm) $\geq 1.2d_0$ (21.6mm) OK

Check plate thickness allows for adequate ductility [SN014a 12]

Limiting thickness = $(d/2.8) \sqrt{f_{ub}/f_{y,p}} = (16/2.8) \times \sqrt{800/275} = 9.7 \text{ mm}$

Check end plate thickness, t_p (8) $\leq 9.7 \text{ mm}$ OK

Weld design

Full strength double fillet welds required. Minimum throat size = $0.45 \times$ beam web thickness [NCCI SN014a 11]

Minimum weld throat size = 2.9mm; actual = $0.7 \times 6 = 4.2 \text{ mm}$ OK

Bolt resistance (M16 8.8 bolts)

Shear resistance per bolt = $F_{v,Rd} = a_v \cdot f_{ub} \cdot A_s / g_{M2} = 0.6 \times 800 \times 157 / (1.25 \times 1000) = 60.3 \text{ kN}$ [EC3-1-8 Table 3.4]

Bolt bearing resistance, $F_{b,Rd} = k_1 \cdot a_b \cdot f_u \cdot d \cdot t_p / g_{M2}$ [EC3-18 Table 3.4]

where $a_v = 0.6$; $f_{ub} = 800 \text{ N/mm}^2$; $A_s = 157.0 \text{ mm}^2$; $g_{M2} = 1.25$

a_b is the smallest of a_d , f_{ub}/f_u or 1.0

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$a_d = e_1/3d_0$ for end bolts, $(p_1/3d_0) - 0.25$ for inner bolts (*bolts in direction of load transfer*)

$k_1 = 2.8e_2/d_0 - 1.7$ but ≤ 2.5 for edge bolts (*bolts perpendicular to load transfer*)

and $1.4p_2/d_0 - 1.7$ but ≤ 2.5 for edge bolts (*do.*)

d_0 : bolt hole diameter (18 mm); e_1 : end distance; e_2 : edge distance; p_1 : pitch

f_u for $t = 3-100\text{mm}$: S275: 410N/mm^2 ; S355: 470N/mm^2 [EC3-1-1 UK NA 2.4: use product values not those in EC3]

V_{Rd,1}, Bolts in shear

$F_{v,rd,red} = 0.8 F_{v,rd} = 0.8 \times 60.3 = 48.23 \text{ kN}$ (*0.8 reduction factor allows for tension in bolt*)

$V_{Rd,1} = n.F_{v,rd,red} = 2 \times 2 \times 48.2 = 192.9 \text{ kN OK}$ [NCCI SN014a 3]

V_{Rd,2}, End plate in bearing

Bolt bearing resistance, $F_{b,Rd} = k_1 \cdot a_b \cdot f_u \cdot d \cdot t_p / g_{M2}$

Top bolts: $F_{b,Rd} = 2.03 \times 0.556 \times 410 \times 16 \times 8 / (1.25 \times 1000) = 47.4 \text{ kN}$

Other bolts: $F_{b,Rd} = 2.03 \times 1.000 \times 410 \times 16 \times 8 / (1.25 \times 1000) = 85.4 \text{ kN}$

As $F_{v,rd,red} (48.23 \text{ kN}) < F_{b,Rd,max} (85.4 \text{ kN})$, $V_{Rd,2} = n.F_{b,rd,min}$

$V_{Rd,2} = 2 \times 2 \times 47.4 = 189.7 \text{ kN OK}$ [NCCI SN014a 4]

V_{Rd,3}, End plate in shear (gross section)

$V_{Rd,3} = A_v \cdot f_{y,p} / (1.27 \sqrt{3} g_{M0}) = 150 \times 8 \times 2 \times 275 / (1.27 \sqrt{3} \times 1.0 \times 1000) = 300 \text{ kN OK}$ [NCCI SN014a 6]

V_{Rd,4}, End plate in shear (net section)

$V_{Rd,4} = A_{v,net} \cdot f_{u,p} / (\sqrt{3} g_{M2}) = 114 \times 8 \times 2 \times 410 / (\sqrt{3} \times 1.10 \times 1000) = 393 \text{ kN OK}$ [NCCI SN014a 7]

V_{Rd,5}, End plate in shear (block shear)

$V_{Rd,5} = f_{u,p} A_{nt} / g_{M2} + f_{y,p} A_{nv} / (\sqrt{3} g_{M0})$ [EC3-1-8 3.10.2(2)]

A_{nt} = net area in tension = $2 \times 15 \times 8 = 240\text{mm}^2$

A_{nv} = net area in shear = $2 \times 93 \times 8 = 1,488\text{mm}^2$

$V_{Rd,5} = ((410 \times 240 / 1.10) + (275 \times 1,488 / (\sqrt{3} \times 1.0))) / 1000 = 326 \text{ kN OK}$ [NCCI SN014a 8]

V_{Rd,6}, End plate in bending

As $h_p (150\text{mm}) \geq 1.36p_3 (68.0\text{mm})$ check not required [NCCI SN014 9]

V_{Rd,7}, Beam web in shear

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$V_{Rd,7} = 0.9 \cdot A_v(f_{y,b}/\sqrt{3})/g_{M0} = 0.9 \times 150 \times 6.4 \times 355/(\sqrt{3} \times 1.0 \times 1000) = 177.1 \text{ kN OK}$ [0.9 factor see SCI P387]

$V_{Rd,8}$, Supporting member in bearing

$$V_{Rd,8} = nF_{b,Rd}$$

$$F_{b,Rd} = k_1 \cdot a_b \cdot f_u \cdot d \cdot t / g_{M2} = 2.19 \times 1.00 \times 470 \times 16 \times 5.4 / (1.25 \times 1000) = 71.1 \text{ kN}$$

$$V_{Rd,8} = 2 \times 2 \times 71.1 = 284.4 \text{ kN OK}$$
 [NCCI SN014a 5]

Ductility requirement

To ensure adequate ductility check $t_p(8) \leq (d/2.8)\sqrt{(f_{ub}/f_{y,p})}$ (9.7mm) OK [SN014a 12]

Rotational capacity has not been checked

Summary:

$V_{Rd,1}$	Bolts in shear	193	kN
$V_{Rd,2}$	End plate in bearing	190	kN
$V_{Rd,3}$	End plate in shear (gross section)	300	kN
$V_{Rd,4}$	End plate in shear (net section)	393	kN
$V_{Rd,5}$	End plate in shear (block shear)	326	kN
$V_{Rd,6}$	End plate in bending	N/A	
$V_{Rd,7}$	Beam web in shear	177	kN <<<
$V_{Rd,8}$	Supporting member in bearing	284	kN

Critical design check: 7 - Beam web in shear: Connection resistance = 177kN OK

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Flexible end plate connection check (EC3 + UK NA)

Calculations are based on NCCI SN014a: Shear resistance of a simple end plate connection

Location: Column 1 and Beam 3 - Connection

Factored load at support = 70.00 kN

Beam: S355 203x133x30 UB connection to flange of S355 203x102x23 UB column

Section dims: 203 x 133 x 30 UB: D = 206.8 mm B = 133.9 mm t = 6.4 mm T = 9.6 mm r = 7.60 mm

203 x 102 x 23 UB: D=203.2 mm B=101.8 mm t=5.4 mm T=9.3 mm $I_y=2,100 \text{ cm}^4$ $i_z=2.36 \text{ cm}$ $W_{pl,y}=234 \text{ cm}^3$ $W_{el,y}=207 \text{ cm}^3$

Use 150 x 98 x 10mm end plate 2No M16 8.8 bolts at 90 mm centres (v), 50 mm centres (h), 6 mm fillet welds

Check detailing

Check plate/cleat length (150mm) $\geq 0.6 \times$ beam depth (124mm) OK

Check bolt pitch (90mm) $\geq 2.2d_0$ (39.6mm) OK

Check bolt end clearance (30mm) $\geq 1.2d_0$ (21.6mm) OK

Check bolt edge clearance (24mm) $\geq 1.2d_0$ (21.6mm) OK

Check plate thickness allows for adequate ductility [SN014a 12]

Limiting thickness = $(d/2.8) \sqrt{f_{ub}/f_{y,p}} = (16/2.8) \times \sqrt{800/275} = 9.7 \text{ mm}$

Check column flange, $t_{f,c}$ (9.3mm) or plate thickness, t_p (10mm) $\leq 9.7 \text{ mm}$ OK

Weld design

Full strength double fillet welds required. Minimum throat size = $0.45 \times$ beam web thickness [NCCI SN014a 11]

Minimum weld throat size = 2.9mm; actual = $0.7 \times 6 = 4.2 \text{ mm}$ OK

Bolt resistance (M16 8.8 bolts)

Shear resistance per bolt = $F_{v,Rd} = a_v \cdot f_{ub} \cdot A_s / g_{M2} = 0.6 \times 800 \times 157 / (1.25 \times 1000) = 60.3 \text{ kN}$ [EC3-1-8 Table 3.4]

Bolt bearing resistance, $F_{b,Rd} = k_1 \cdot a_b \cdot f_u \cdot d \cdot t_p / g_{M2}$ [EC3-18 Table 3.4]

where $a_v = 0.6$; $f_{ub} = 800 \text{ N/mm}^2$; $A_s = 157.0 \text{ mm}^2$; $g_{M2} = 1.25$

a_b is the smallest of a_d , f_{ub}/f_u or 1.0

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$a_d = e_1/3d_0$ for end bolts, $(p_1/3d_0) - 0.25$ for inner bolts (*bolts in direction of load transfer*)

$k_1 = 2.8e_2/d_0 - 1.7$ but ≤ 2.5 for edge bolts (*bolts perpendicular to load transfer*)

and $1.4p_2/d_0 - 1.7$ but ≤ 2.5 for edge bolts (*do.*)

d_0 : bolt hole diameter (18 mm); e_1 : end distance; e_2 : edge distance; p_1 : pitch

f_u for $t = 3-100\text{mm}$: S275: 410N/mm^2 ; S355: 470N/mm^2 [EC3-1-1 UK NA 2.4: use product values not those in EC3]

V_{Rd,1}, Bolts in shear

$F_{v,rd,red} = 0.8 F_{v,rd} = 0.8 \times 60.3 = 48.23 \text{ kN}$ (*0.8 reduction factor allows for tension in bolt*)

$V_{Rd,1} = n \cdot F_{v,rd,red} = 2 \times 2 \times 48.2 = 192.9 \text{ kN OK}$ [NCCI SN014a 3]

V_{Rd,2}, End plate in bearing

Bolt bearing resistance, $F_{b,Rd} = k_1 \cdot a_b \cdot f_u \cdot d \cdot t_p / g_{M2}$

Top bolts: $F_{b,Rd} = 2.03 \times 0.556 \times 410 \times 16 \times 10 / (1.25 \times 1000) = 59.3 \text{ kN}$

Other bolts: $F_{b,Rd} = 2.03 \times 1.000 \times 410 \times 16 \times 10 / (1.25 \times 1000) = 106.7 \text{ kN}$

As $F_{v,rd,red} (48.23 \text{ kN}) < F_{b,Rd,max} (106.7 \text{ kN})$, $V_{Rd,2} = n \cdot F_{b,rd,min}$

$V_{Rd,2} = 2 \times 2 \times 59.3 = 237.1 \text{ kN OK}$ [NCCI SN014a 4]

V_{Rd,3}, End plate in shear (gross section)

$V_{Rd,3} = A_v \cdot f_{y,p} / (1.27 \cdot \sqrt{3} g_{M0}) = 150 \times 10 \times 2 \times 275 / (1.27 \cdot \sqrt{3} \times 1.0 \times 1000) = 375 \text{ kN OK}$ [NCCI SN014a 6]

V_{Rd,4}, End plate in shear (net section)

$V_{Rd,4} = A_{v,net} \cdot f_{u,p} / (\sqrt{3} g_{M2}) = 114 \times 10 \times 2 \times 410 / (\sqrt{3} \times 1.10 \times 1000) = 491 \text{ kN OK}$ [NCCI SN014a 7]

V_{Rd,5}, End plate in shear (block shear)

$V_{Rd,5} = f_{u,p} A_{nt} / g_{M2} + f_{y,p} A_{nv} / (\sqrt{3} g_{M0})$ [EC3-1-8 3.10.2(2)]

A_{nt} = net area in tension = $2 \times 15 \times 10 = 300\text{mm}^2$

A_{nv} = net area in shear = $2 \times 93 \times 10 = 1,860\text{mm}^2$

$V_{Rd,5} = ((410 \times 300 / 1.10) + (275 \times 1,860 / (\sqrt{3} \times 1.0))) / 1000 = 407 \text{ kN OK}$ [NCCI SN014a 8]

V_{Rd,6}, End plate in bending

As $h_p (150\text{mm}) \geq 1.36p_3 (68.0\text{mm})$ check not required [NCCI SN014 9]

V_{Rd,7}, Beam web in shear

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$V_{Rd,7} = 0.9 \cdot A_v(f_{y,b}/\sqrt{3})/g_{M0} = 0.9 \times 150 \times 6.4 \times 355/(\sqrt{3} \times 1.0 \times 1000) = 177.1 \text{ kN OK}$ [0.9 factor see SCI P387]

$V_{Rd,8}$, Supporting member in bearing

$$V_{Rd,8} = nF_{b,Rd}$$

$$F_{b,Rd} = k_1 \cdot a_b \cdot f_u \cdot d \cdot t / g_{M2} = 2.33 \times 1.00 \times 470 \times 16 \times 9.3 / (1.25 \times 1000) = 130.3 \text{ kN}$$

$$V_{Rd,8} = 2 \times 2 \times 130.3 = 521.2 \text{ kN OK [NCCI SN014a 5]}$$

Ductility requirement

To ensure adequate ductility check $t_{f,c}$ (9.3) or t_p (10mm) $\leq (d/2.8)\sqrt{f_{ub}/f_{y,p}}$ (9.7mm) OK [SN014a 12]

Rotational capacity has not been checked

Summary:

$V_{Rd,1}$	Bolts in shear	193	kN
$V_{Rd,2}$	End plate in bearing	237	kN
$V_{Rd,3}$	End plate in shear (gross section)	375	kN
$V_{Rd,4}$	End plate in shear (net section)	491	kN
$V_{Rd,5}$	End plate in shear (block shear)	407	kN
$V_{Rd,6}$	End plate in bending	N/A	
$V_{Rd,7}$	Beam web in shear	177	kN <<<
$V_{Rd,8}$	Supporting member in bearing	521	kN

Critical design check: 7 - Beam web in shear: Connection resistance = 177kN OK

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Beam: Roof Rear Extension -Trimmers Span: 2.8 m.

			Load name	Loading w1	Start x1	Loading w2	End x2	R1comp
								R2comp
U	G		o.w.	0.1 0	L	0.14 0.14		
U	G		Roof	0.8x0.6 0	L	0.67 0.67		
U	QA		Roof	2.5x0.6 0	L	2.10 2.10		

Total load (unfactored): 5.82 kN ~~2.91~~ ~~2.91~~

Dead/Permanent (unfactored): 1.62 kN 0.81 0.81

Live/Variable (unfactored): 4.20 kN 2.10 2.10

(6.10): 8.49 kN 4.25 4.25

Load types: U:UDL; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed; QA: Residential

Maximum B.M. = 2.97 kNm (6.10) at 1.40 m. from R1

Maximum S.F. = -4.25 kN (6.10) at R2

Mid-span deflections: Dead: $0.46 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $1.20 \times 10^8 / EI$

Total: $1.66 \times 10^8 / EI$

Timber beam calculation to BS EN1995-1-1 using C24 timber

Use 2no 47 x 175 C24 6.9 kg/m approx

$W_{el,y} = 479.8 \text{ cm}^3$ $I_y = 4,198 \text{ cm}^4$

Timber grade: C24

Grade bending strength, $f_{m,k} = 24.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k} = 4.0 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

$E_{0.05} = 7,400 \text{ N/mm}^2$; $E_{0,mean} = 11,000 \text{ N/mm}^2$ [BS EN 338: 2009 Table 1]

Material partial factor, $g_M = 1.3$ [EC5 UK Table NA.3]

Loading modification factor, $k_{mod} = 0.6$ (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.1$ [EC5 6.6(2)]

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Deflection modification factor, $k_{def} = 0.60$ (Service class 1, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = 1.0$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_m = 24.0 \times 0.60 \times 1.00 \times 1.1 / 1.30 = 12.18 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 2.97 \times 1000 / 480 = 6.20 \text{ N/mm}^2$ OK

Bending resistance = $12.18 \times 480 / 1000 = 5.85 \text{ kNm}$

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 94 = 63 \text{ mm}$. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / g_m = 4.00 \times 0.60 \times 1.1 / 1.30 = 2.03 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 4.25 \times 1000 \times (3/2) / (63.0 \times 175) = 0.58 \text{ N/mm}^2$ OK

Shear resistance = $2.03 \times 63 \times 175 \times (2/3) / 1000 = 14.92 \text{ kN}$

Deflection

Final deflection limit = $L / 250 = 11.20 \text{ mm}$

Final deflection, $u_{fin} = S u_{inst} (1 + y_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2 M_{unf} / (G_{mean} A)$ where ($G = E / 16$)

$$u_{inst,v} = 1.2 \times 2.04 \times 10^6 / ((11,000 / 16) \times 94 \times 175) = 0.22 \text{ mm}$$

Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	$\times 1e^8 / EI$				Inst. mm	y_2	k_{def}	Fin. mm
Dead:	0.46	1.01	1.00	0.60	1.61			
Live QA:		1.20	2.60	0.30	0.60	3.07		
Shear deflection:				0.22			0.28	
Total mm.			<u>3.82</u>		<u>4.96</u>	OK		

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	Designed By BG	Checked By BG	Date 02/02/25

Beam: Roof Rear Extension -Trimmers Span: 2.8 m.

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp
							R2comp
U	G	o.w.	0.1 0	L	0.14 0.14		
U	G	Roof	0.8x0.6 0	L	0.67 0.67		
U	QA	Roof	2.5x0.6 0	L	2.10 2.10		
Total load (unfactored): 5.82 kN 2.91 2.91							
Dead/Permanent (unfactored): 1.62 kN 0.81 0.81							
Live/Variable (unfactored): 4.20 kN 2.10 2.10							
(6.10): 8.49 kN 4.25 4.25							

Load types: U:UDL; Load positions are measured in m. from R1

Load durations: G: Dead; Qx: Imposed; QA: Residential

Maximum B.M. = 2.97 kNm (6.10) at 1.40 m. from R1

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Live: $1.20 \times 10^8 / EI$

Total: $1.66 \times 10^8 / EI$

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Loading modification factor, $k_{mod} = 0.6$ (Service class 1; Live load duration: Permanent) [EC5 Tables 2.2/3.1]

Load sharing factor, $k_{sys} = 1.1$ [EC5 6.6(2)]

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Deflection modification factor, $k_{def} = 0.60$ (Service class 1, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = 1.0$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k} \cdot k_{mod} \cdot k_h \cdot k_{sys} / g_m = 24.0 \times 0.60 \times 1.00 \times 1.1 / 1.30 = 12.18 \text{ N/mm}^2$

Design bending stress, $s_{m,y,d} = 2.97 \times 1000 / 480 = 6.20 \text{ N/mm}^2$ OK

Bending resistance = $12.18 \times 480 / 1000 = 5.85 \text{ kNm}$

Shear

Effective width for shear, $b_{ef} = k_{cr} \cdot b = 0.67 \times 94 = 63 \text{ mm}$. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k} \cdot k_{mod} \cdot k_{sys} / g_m = 4.00 \times 0.60 \times 1.1 / 1.30 = 2.03 \text{ N/mm}^2$

Design shear stress, $s_{v,y,d} = 4.25 \times 1000 \times (3/2) / (63.0 \times 175) = 0.58 \text{ N/mm}^2$ OK

Shear resistance = $2.03 \times 63 \times 175 \times (2/3) / 1000 = 14.92 \text{ kN}$

Deflection

Final deflection limit = $L / 250 = 11.20 \text{ mm}$

Final deflection, $u_{fin} = S u_{inst} (1 + y_2 \cdot k_{def})$ [EC2 Eq.2.29/2.30]

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Mid-span deflections:	$\times 1e^8 / EI$				Inst. mm	y_2	k_{def}	Fin. mm
Dead:	0.46	1.01	1.00	0.60	1.61			
Live QA:		1.20	2.60	0.30	0.60	3.07		
Shear deflection:				0.22			0.28	
Total mm.			<u>3.82</u>		<u>4.96</u>	OK		

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