CORE STRUCTURES
CHARTERED STRUCTURAL ENGINEERS

STRUCTURAL DESIGN CALCULATIONS

PROJECT REF:	**1131-01**
PROJECT ADDRESS:	**2 FURZEHOLME, HIGH SALVINGTON**
DESCRIPTION:	**PROPOSED LOFT CONVERSION & ALTERATIONS**
CLIENT:	**MARIE GALLAGHER**
STATUS:	**FOR BUILDING REGULATIONS APPROVAL**

Prepared by	Checked by	Date	Revision
ST	RJ	02.06.25	-

1. Introduction

The following calculations are the structural design for the proposed loft conversion and internal alterations at the above property.

The property is a detached bungalow (with a semi-detached side garage) understood to date from the mid-late 20th century. The construction is typical of the era. External walls are presumed to be of cavity masonry construction clad in facing brick. The pitched roof is constructed in loose cut timber and clad externally in plain clay tiles. The suspended ground floors are presumed to be formed in joisted timber. Internal walls are generally solid and for the purpose of design are assumed to be founded on dedicated strip footings.

A brief visual inspection of the property was carried out on Thursday 17th April 2025. This was a non-intrusive inspection with much of the existing structure hidden from view. Any assumptions about the existing structural arrangement made in the design – namely the presence of existing dedicated footings beneath internal walls utilised for proposed support, will need to be exposed and verified on site prior to commencement of the main structural works.

2. Design Codes

All design calculations have been carried out in accordance with Eurocodes unless noted otherwise.

LOADING DEAD AND IMPOSED LOAD	BS EN 1991-1-1 and UK National Annex (NA)
WIND LOADING	BS EN 1991-1-4 and NA
FOUNDATIONS	BS EN 1997-1 and BS EN 1997-2
HOT ROLLED STEEL	BS EN 1993-1-1 and NA
MASONRY	BS EN 1996-1-1 and BS EN 1996-1-1
TIMBER	BS EN 1995-1-1 and NA

3. Ground Conditions & Foundations

The British Geological Survey 1:50000 series, scale map shows expected bedrock of Tarrant Chalk Formation (Chalk), overlaid by superficial Head Deposits (Clay, Silt, Sand & Gravel).

4. Stability

To be achieved/maintained in the conventional way with horizontal loads acting on the structure transmitted to stiff elements and foundations via horizontal diaphragm action of the roof. Stiff elements comprise ply-sheathed buttress walls and/or residual piers with minimum sizing in accordance with the requirements outlined in Part A of the current Building Regulations. In the absence of which, stability framing or bracing will be required which will be clearly identified on drawings.

5. Disproportionate Collapse

In accordance with Part A3 of the current Building Regulations, the building is Consequence Class 1 and should therefore conform to basic requirements outlined in relevant Eurocodes BS EN 1990-1999.

6. Health & Safety

Specific construction hazards and residual risks will be highlighted on drawings to assist with the planning and construction phase.

7. Important notes to be read in conjunction with design calculations and drawings

i. No dimensions are to be scaled off any drawing or sketches produced here. Any missing dimensions are to be determined from site measurements or the Architect's drawings.

ii. This drawing has been prepared for structural purposes only and does not necessarily show architectural details, such as thermal insulation, ventilation, waterproofing, fire escape provisions, fire protection measures, services etc.

iii. In the case of alterations to existing buildings, this is based on what was clearly visible at the time of the inspection and the details may need amending during the works or further exposure works, to take account of features that were initially hidden from view.

iv. It is recommended that no materials are ordered, or work carried out until Planning Permission and Building Regulations approval is obtained, if appropriate, as they and the local fire officer may have additional requirements that affect the structure. It is also essential that we and Building Control are fully notified of any subsequent changes, either in the design office or on site, prior to being actioned.

v. Structural steelwork shall normally be boxed-in for fire protection to local authority requirements and protected from corrosion if in contact with potentially damp walls or humid atmosphere. If the works are close to or could affect neighbouring properties, Party Wall Notices may be required, and a party wall surveyor should be consulted. This is a legal requirement and to start the works without Party Wall Awards could seriously delay the works.

vi. It is the client and contractor's responsibility to provide adequate health and safety measures in accordance with the obligations set out in the CDM Regulations (2015). The contractor is also responsible for temporary works to support/stabilise all existing structures while the permanent works are being carried out.

vii. In the case of alterations to existing structures, the redistribution of stresses can cause slight cracking in the brittle finishes above and below and this should be allowed for in assessing or specifying the scope of works.

8. Gravity Loading

(kN/m^2)	Permanent (Gk)	Variable (Qk)	Total
Pitched Roof (40deg.)			
Tiles	0.60		
Battens & Felt	0.10		
Rafters	0.15		
Insulated Soffit	0.15		
(Slope)	**1.00**	**0.60**	**1.60**
(Plan)	**1.31**		**1.91**
Existing Ceiling			
Joists & Insulation	0.15		
Plasterboard Ceiling	0.15		
	0.30	**0.25**	**0.55**
Dormer Flat Roof			
3 layer Felt	0.35		
Deck, Joists & Firrings	0.30		
Soffit & Insulation	0.15		
	0.80	**0.75**	**1.55**
1st Floor			
Boards & Joists	0.35		
Plasterboard Ceiling	0.15		
	0.50	**1.50**	**2.00**
Partitions			
Studwork & Noggings	0.20		
Plasterboard	0.20		
	0.40		**0.40**
Dormer Wall			
Hanging Tiles	0.55		
Battens, Felt & Insulation	0.10		
Studs	0.15		
Plasterboard	0.10		
	0.90		**0.90**
External Wall			
Brickwork	2.00		
Blockwork	1.60		
Internal Finishes	0.25		
	3.85		**3.85**

EuroBeam Project Summary

Site address: 2 Furzeholme, High Salvington
Job: Loft Conversion & Alterations
Client: Marie Gallagher
Job number: 1131

ITEMS:

1: Beam: [B21]
Span: 3.8 m.
Reactions (unfactored/factored): R1: 9.09/12.68 kN; R2: 9.09/12.68 kN
Use 152 x 89 x 16 UKB S355
Bearing R2: 125 x 100 x 50h mm padstone

2: Beam: [B22]
Span: 5.0 m.
Reactions (unfactored/factored): R1: 15.07/21.02 kN; R2: 13.45/18.76 kN
Use 152 x 152 x 30 UKC S355

3: Beam: [B23]
Span: 3.2 m.
Reactions (unfactored/factored): R1: 7.65/10.68 kN; R2: 7.65/10.68 kN
Use 152 x 89 x 16 UKB S355

4: Beam: [R1]
Span: 3.2 m.
Reactions (unfactored/factored): R1: 1.38/1.92 kN; R2: 1.38/1.92 kN
Use 47 x 195 C24

5: Beam: [2R1]
Span: 3.2 m.
Reactions (unfactored/factored): R1: 3.98/5.55 kN; R2: 3.98/5.55 kN
Use 2no 47 x 195 C24

6: Beam: [R2]
Span: 2.4 m.
Reactions (unfactored/factored): R1: 1.04/1.44 kN; R2: 1.04/1.44 kN
Use 47 x 150 C24

7: Beam: [2R2]
Span: 2.4 m.
Reactions (unfactored/factored): R1: 1.95/2.72 kN; R2: 1.95/2.72 kN
Use 2no 47 x 150 C24

8: Beam: [B24] [B25] [B11]
Span: 1.4 m.
Reactions (unfactored/factored): R1: 14.63/20.40 kN; R2: 8.19/11.42 kN
Use 152 x 89 x 16 UKB S355

9: Beam: [L21]
Span: 1.2 m.
Reactions (unfactored/factored): R1: 1.38/1.92 kN; R2: 1.38/1.92 kN
Use 2no 47 x 120 C24

10: Beam: [B12]
Span: 3.8 m.
Reactions (unfactored/factored): R1: 12.38/17.49 kN; R2: 12.38/17.49 kN
Use 152 x 89 x 16 UKB S355
Bearing R1: 175 x 100 x 75h mm padstone

11: Beam: [B13]
Span: 2.8 m.
Reactions (unfactored/factored): R1: 10.87/15.48 kN; R2: 10.87/15.48 kN
Use 152 x 89 x 16 UKB S355

12: Beam: Front Stair Trimmer
Span: 2.4 m.
Reactions (unfactored/factored): R1: 5.30/7.37 kN; R2: 5.30/7.37 kN
Use 2no 47 x 195 C24

13: Beam: Rear Stair Trimmer
Span: 2.4 m.
Reactions (unfactored/factored): R1: 0.37/0.55 kN; R2: 1.87/2.73 kN

14: Beam: [B14]
Span: 4.1 m.
Reactions (unfactored/factored): R1: 12.05/16.96 kN; R2: 8.20/11.61 kN
Use 178 x 102 x 19 UKB S355

15: Beam: [B15]
Span: 3.2 m.
Reactions (unfactored/factored): R1: 11.38/16.13 kN; R2: 11.38/16.13 kN
Use 152 x 89 x 16 UKB S355

16: Beam: [B16]
Span: 3.2 m.
Reactions (unfactored/factored): R1: 10.01/14.47 kN; R2: 10.01/14.47 kN
Use 152 x 89 x 16 UKB S355

17: Beam: [B17]
Span: 3.2 m.
Reactions (unfactored/factored): R1: 12.77/18.00 kN; R2: 12.77/18.00 kN
Use 152 x 89 x 16 UKB S355

18: Beam: [B18]
Span: 5.0 m.
Reactions (unfactored/factored): R1: 19.59/27.78 kN; R2: 18.59/26.44 kN
Use 203 x 133 x 30 UKB S355
Bearing R1: 250 x 100 x 125h mm padstone

19: Beam: [B19]
Span: 3.8 m.
Reactions (unfactored/factored): R1: 14.71/20.76 kN; R2: 14.71/20.76 kN
Use 178 x 102 x 19 UKB S355

20: Beam: [FJ1]
Span: 3.8 m.
Reactions (unfactored/factored): R1: 1.60/2.33 kN; R2: 1.60/2.33 kN
Use 47 x 195 C24

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21 Timber post/stud: [TP2]
Use 100 x 100 C24

Beam: [B21] **Span: 3.8 m.**

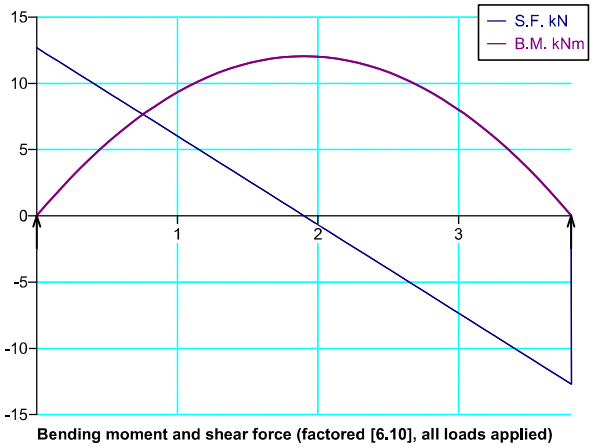
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.2	0		L	0.38	0.38	0.54
U G	Pitched Roof Dead	1.31*4.8/2	0		L	5.97	5.97	8.54
U QH	Pitched Roof Live	0.6*4.8/2	0		L	2.74	2.74	3.91
				Total load (unfactored):	**18.18 kN**	**9.09**	**9.09**	12.99
				Dead/Permanent (unfactored):	12.71 kN	6.35	6.35	9.08
				Live/Variable (unfactored):	5.47 kN	2.74	2.74	3.91
				Factored (6.10):	25.36 kN	12.68	12.68	

Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QH: Roof

Maximum B.M. = 12.05 kNm (6.10) at 1.90 m. from R1

Maximum S.F. = 12.68 kN (6.10) at R1

Mid-span deflections: Dead: 9.08 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 3.91 x 10^8/EI
Total: 12.99 x 10^8/EI



Bending moment and shear force (factored [6.10], all loads applied)

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **152 x 89 x 16 UKB** S355

D=152.4 mm B=88.7 mm t=4.5 mm T=7.7 mm I_y=834 cm^4 i_z=2.10 cm $W_{pl,y}$=123 cm^3 $W_{el,y}$=109 cm^3

Classification: Flange: c/t = 34.5/7.7 = 4.48 <= 9ε (7.32): Class 1, plastic
EC3 Table 5.2 Web: c/t = 121.8/4.5 = 27.1 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 12.68 kN

Shear area, A_v = A - 2bt_f + (t_w+2r)t_f = 20.3x100 - 2x88.7x7.70 + (4.50+2x7.60) x7.70 = 816 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd}$ = A_v.(f_y/√3)/γ_{M0} = 816 x (355/√3)/(1.0 x 1000) = 167 kN (>=12.68) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 137.0/4.5 = 30.44 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 12.05 kNm

Moment resistance, $M_{c,y,Rd}$ = f_y.$W_{pl,y}$ = 355 x 123/1000 = 43.7 kNm OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

Support conditions R1/R2: Compression flange laterally unrestrained; both flanges free to rotate on plan. Partial torsional restraint by dead bearing of bottom flange to support (1.2L+2D)

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

$\chi_{LT,mod}$ = χ_{LT}/f (but <=1/$\overline{\lambda}_{LT}^2$ and <=1.0) [Eq.6.58]

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

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

$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta\overline{\lambda}_{LT}^2)}]$ [EC3 (6.57)]

$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta\overline{\lambda}_{LT}^2]$

$\overline{\lambda}_{LT,0} = 0.4 \quad \beta = 0.75$ [EC3 UK NA 2.17]

$\overline{\lambda}_{LT} = \sqrt{(f_y.W_{pl,y}/M_{cr})}$

$M_{cr} = C_1(\pi^2 EI_z/L_e^2)\sqrt{(I_w/I_z + L_e^2 GI_T/\pi^2 EI_z)}$ [SN003: k and k_w taken as 1.0, conservative]

$W_y = 123.0\ cm^3 \quad I_w = 0.005\ dm^6 \quad I_T = 3.56\ cm^4 \quad I_z = 89.8\ cm^4 \quad G = 81{,}000 N/mm^2$

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\overline{\lambda}_z$	$\overline{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-3.80	4.86	12.0	1.13*	0.94	18.2	3.032	1.550	1.596	0.407	0.405	43.7	17.7	OK

* $C1 = 4M_{max}/\sqrt{(M_{max}^2 + 4M_a^2 + 7M_b^2 + 4M_c^2)} <= 2.5$ [$M_{a,b,c}$ quarter pt moments] [Wong & Driver, AISC Eng. Journal, Q1 2010]

Combined bending and shear

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

Web buckling and crushing have not been checked

Deflection

LL deflection = 3.91 x 1e8/(210,000 x 834) = 2.2 mm (L/1702) OK
TL deflection = 12.99 x 1e8/(210,000 x 834) = 7.4 mm (L/512)

Bearings

152 x 89 x 16 UB stiff bearing length, $b_1 = t + 1.6r + 2T = 32.1$ mm

R1 (12.68 kN) : None

R2 (12.68 kN) : 125 x 100 x 50h mm padstone

Masonry: 3.6N/mm² 100mm aerated concrete block, shape factor, $\delta = 1.38$ [Table A1 BS EN 772-1]
Normalised compressive strength, $f_b = 3.60 \times 1.38 = 4.97\ N/mm^2$
Mortar: M4: $f_m = 4.0\ N/mm^2$
K = 0.75, $\alpha = 0.7$, B = 0.3 [EC6-1-1 UK NA 2012 Table NA.4]
Characteristic compressive strength, $f_k = K.f_b^{\alpha}.f_m^{B} = 0.75 \times 4.97^{0.7} \times 4.00^{0.3} = 3.49\ N/mm^2$ [EC6-1 Eq(3.1)]
Category II mfr, class 2 execution: $\gamma_m = 3.0$ [EC6-3 UK NA Table NA.1]
Design compressive strength, $f_d = f_k/\gamma_m = 3.49/3.0 = 1.16\ N/mm^2$

Stress under padstone = 12.68 x 1000/125 x 100 = 1.01 N/mm² OK

Beam: [B22] **Span: 5.0 m.**

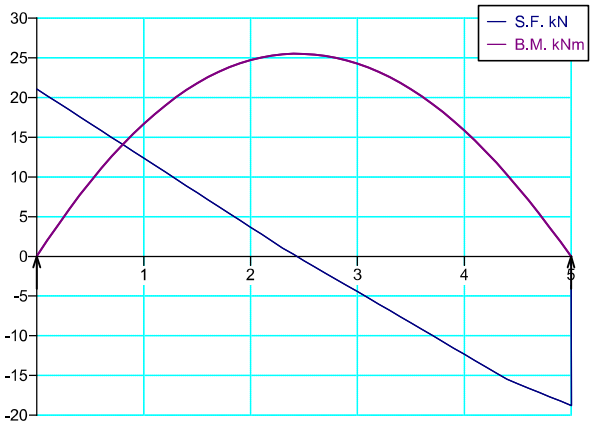
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.3	0		L	0.75	0.75	2.4
R G	Pitched Roof Dead	1.31*6.2/2	0		2.3	7.19	2.15	14.4
R QH	Pitched Roof Live	0.6*6.2/2	0		2.3	3.29	0.98	6.6
R G	Pitched Roof Dead	1.31*5.6/2	2.3		4.4	2.54	5.16	15.8
R QH	Pitched Roof Live	0.6*5.6/2	2.3		4.4	1.16	2.36	7.2
R G	Pitched Roof Dead	1.31*3.8/2	4.4		5.0	0.09	1.40	0.7
R QH	Pitched Roof Live	0.6*3.8/2	4.4		5.0	0.04	0.64	0.3
				Total load (unfactored):	**28.53 kN**	**15.07**	**13.45**	47.5
				Dead/Permanent (unfactored):	20.04 kN	10.57	9.46	33.4
				Live/Variable (unfactored):	8.49 kN	4.50	3.99	14.2
				Factored (6.10):	39.78 kN	21.02	18.76	

Load types: O:Beam o.w.; R:Part UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QH: Roof

Maximum B.M. = 25.47 kNm (6.10) at 2.43 m. from R1

Maximum S.F. = 21.02 kN (6.10) at R1

Mid-span deflections: Dead: 33.4 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 14.2 x 10^8/EI
Total: 47.5 x 10^8/EI



Bending moment and shear force (factored [6.10], all loads applied)

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **152 x 152 x 30 UKC** S355

D=157.6 mm B=152.9 mm t=6.5 mm T=9.4 mm I_y=1,750 cm^4 i_z=3.83 cm $W_{pl,y}$=248 cm^3 $W_{el,y}$=222 cm^3

Classification: Flange: c/t = 65.6/9.4 = 6.98 <= 9ε (7.32): Class 1, plastic
EC3 Table 5.2 Web: c/t = 123.6/6.5 = 19.0 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 21.02 kN

Shear area, A_v = A - 2bt_f + (t_w+2r)t_f = 38.3x100 - 2x153x9.40 + (6.50+2x7.60) x9.40 = 1,159 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd}$ = A_v.(f_y/√3)/γ_{M0} = 1,159 x (355/√3)/(1.0 x 1000) = 238 kN (>=21.02) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 138.8/6.5 = 21.35 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 25.5 kNm

Moment resistance, $M_{c,y,Rd}$ = f_y.$W_{pl,y}$ = 355 x 248/1000 = 88.0 kNm OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

Support conditions R1/R2: Compression flange laterally unrestrained; both flanges free to rotate on plan. Partial torsional restraint by dead bearing of bottom flange to support (1.2L+2D)

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

$\chi_{LT,mod} = \chi_{LT}/f$ (but <=$1/\overline{\lambda}_{LT}^2$ and <=1.0) [Eq.6.58]

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

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

$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta\overline{\lambda}_{LT}^2)}]$ [EC3 (6.57)]

$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta\overline{\lambda}_{LT}^2]$

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

$\overline{\lambda}_{LT} = \sqrt{(f_y.W_{pl,y}/M_{cr})}$

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

$W_y = 248.0\ cm^3$ $I_w = 0.031\ dm^6$ $I_T = 10.5\ cm^4$ $I_z = 560\ cm^4$ $G = 81{,}000 N/mm^2$

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\overline{\lambda}_z$	$\overline{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-5.00	6.32	25.5	1.13*	0.94	61.3	2.158	1.198	1.174	0.580	0.592	88.0	52.1	OK

* $C1 = 4M_{max}/\sqrt{(M_{max}^2+4M_a^2+7M_b^2+4M_c^2)}$ <= 2.5 [$M_{a,b,c}$ quarter pt moments] [Wong & Driver, AISC Eng. Journal, Q1 2010]

Combined bending and shear

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

Web buckling and crushing have not been checked

Deflection

LL deflection = 14.2 x 1e8/(210,000 x 1,750) = 3.9 mm (L/1298) OK
TL deflection = 47.5 x 1e8/(210,000 x 1,750) = 12.9 mm (L/387)

Beam: [B23] **Span: 3.2 m.**

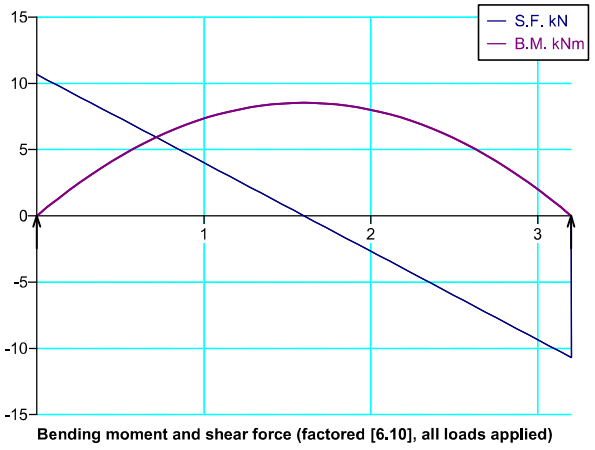
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.2	0		L	0.32	0.32	0.27
U G	Pitched Roof Dead	1.31*4.8/2	0		L	5.03	5.03	4.29
U QH	Pitched Roof Live	0.6*4.8/2	0		L	2.30	2.30	1.97
				Total load (unfactored):	**15.31 kN**	**7.65**	**7.65**	6.54
				Dead/Permanent (unfactored):	10.70 kN	5.35	5.35	4.57
				Live/Variable (unfactored):	4.61 kN	2.30	2.30	1.97
				Factored (6.10):	21.36 kN	10.68	10.68	

Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QH: Roof

Maximum B.M. = 8.54 kNm (6.10) at 1.60 m. from R1

Maximum S.F. = 10.68 kN (6.10) at R1

Mid-span deflections: Dead: 4.57 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 1.97 x 10^8/EI
Total: 6.54 x 10^8/EI



Bending moment and shear force (factored [6.10], all loads applied)

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **152 x 89 x 16 UKB** S355

D=152.4 mm B=88.7 mm t=4.5 mm T=7.7 mm I_y=834 cm^4 i_z=2.10 cm $W_{pl,y}$=123 cm^3 $W_{el,y}$=109 cm^3

Classification: Flange: c/t = 34.5/7.7 = 4.48 <= 9ε (7.32): Class 1, plastic
EC3 Table 5.2 Web: c/t = 121.8/4.5 = 27.1 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 10.68 kN

Shear area, A_v = A - 2bt_f + (t_w+2r)t_f = 20.3x100 - 2x88.7x7.70 + (4.50+2x7.60) x7.70 = 816 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd}$ = A_v.(f_y/√3)/γ_{M0} = 816 x (355/√3)/(1.0 x 1000) = 167 kN (>=10.68) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 137.0/4.5 = 30.44 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 8.54 kNm

Moment resistance, $M_{c,y,Rd}$ = f_y.$W_{pl,y}$ = 355 x 123/1000 = 43.7 kNm OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

Support conditions R1/R2: Compression flange laterally unrestrained; both flanges free to rotate on plan. Partial torsional restraint by dead bearing of bottom flange to support (1.2L+2D)

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

$\chi_{LT,mod}$ = χ_{LT}/f (but <=1/$\overline{\lambda}_{LT}^2$ and <=1.0) [Eq.6.58]

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

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

$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta\overline{\lambda}_{LT}^2)}]$ [EC3 (6.57)]

$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta\overline{\lambda}_{LT}^2]$

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

$\overline{\lambda}_{LT} = \sqrt{(f_y.W_{pl,y}/M_{cr})}$

$M_{cr} = C_1(\pi^2 EI_z/L_e^2)\sqrt{(I_w/I_z + L_e^2 GI_T/\pi^2 EI_z)}$ [SN003: k and k_w taken as 1.0, conservative]

$W_y = 123.0\ cm^3$ $I_w = 0.005\ dm^6$ $I_T = 3.56\ cm^4$ $I_z = 89.8\ cm^4$ $G = 81{,}000 N/mm^2$

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\overline{\lambda}_z$	$\overline{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-3.20	4.14	8.5	1.13*	0.94	21.8	2.583	1.414	1.422	0.466	0.470	43.7	20.5	OK

* $C1 = 4M_{max}/\sqrt{(M_{max}^2 + 4M_a^2 + 7M_b^2 + 4M_c^2)} <= 2.5$ [$M_{a,b,c}$ quarter pt moments] [Wong & Driver, AISC Eng. Journal, Q1 2010]

Combined bending and shear

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

Web buckling and crushing have not been checked

Deflection

LL deflection = 1.97 x 1e8/(210,000 x 834) = 1.1 mm (L/2845) OK
TL deflection = 6.54 x 1e8/(210,000 x 834) = 3.7 mm (L/857)

Beam: [R1] **Span: 3.2 m.**

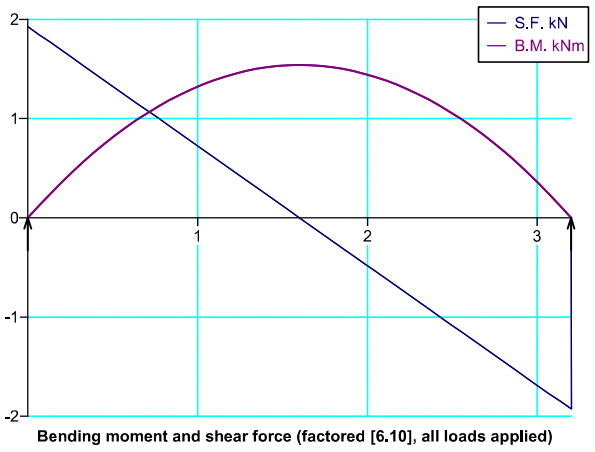
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.1	0		L	0.16	0.16	0.14
U G	Pitch Dead	1.31*0.4	0		L	0.84	0.84	0.72
U QH	Pitch Live	0.6*0.4	0		L	0.38	0.38	0.33
				Total load (unfactored):	**2.76 kN**	**1.38**	**1.38**	1.18
				Dead/Permanent (unfactored):	2.00 kN	1.00	1.00	0.85
				Live/Variable (unfactored):	0.77 kN	0.38	0.38	0.33
				Factored (6.10):	3.85 kN	1.92	1.92	

Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QH: Roof

Maximum B.M. = 1.54 kNm (6.10) at 1.60 m. from R1

Maximum S.F. = 1.92 kN (6.10) at R1

Mid-span deflections: Dead: 0.85 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 0.33 x 10^8/EI
Total: 1.18 x 10^8/EI



Bending moment and shear force (factored [6.10], all loads applied)

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

Use 47 x 195 C24 *3.8 kg/m approx*

$W_{el,y}$ = 297.9 cm^3 I_y = 2,904 cm^4

Timber grade: C24

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

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

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

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

Loading modification factor, k_{mod} = 0.8 (Service class 2; Live load duration: Medium term) [EC5 Tables 2.2/3.1]

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

Deflection modification factor, k_{def} = 0.80 (Service class 2, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

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

Design bending strength, $f_{m,y,d} = f_{m,k}.k_{mod}.k_h.k_{sys}/\gamma_m$ = 24.0 x 0.80 x 1.00 x 1.1/1.30 = 16.25 N/mm^2

Design bending stress, $\sigma_{m,y,d}$ = 1.54 x 1000/298 = 5.17 N/mm^2 OK

Bending resistance = 16.25 x 298/1000 = 4.84 kNm

Shear

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

Design shear strength, $f_{v,d} = f_{v,k}.k_{mod}.k_{sys}/\gamma_m$ = 4.00 x 0.80 x 1.1/1.30 = 2.71 N/mm^2

Design shear stress, $\sigma_{v,y,d}$ = 1.92 x 1000 x (3/2)/(31.5 x 195) = 0.47 N/mm^2 OK

Shear resistance = 2.71 x 31.5 x 195 x (2/3)/1000 = 11.08 kN

Deflection

Final deflection limit = 0.003L = 9.60 mm
Final deflection, $u_{fin} = \Sigma u_{inst}(1 + \psi_2.k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2M_{unf}/(G_{mean}A)$ where (G = E/16)
$u_{inst,v} = 1.2 \times 1.11 \times 10^6/((11{,}000/16) \times 47 \times 195) = 0.21$ mm

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

Mid-span deflections:	x $1e^8$/EI	Inst. mm	k_{def}	ψ_2	Final mm	
G:	0.85	2.67	0.80	1.00	4.80	
QH:	0.33	1.03	0.80	0.00	1.03	
Shear deflection:		0.21			0.33	
Total mm.		3.90			**6.16**	OK

Beam: [2R1] **Span: 3.2 m.**

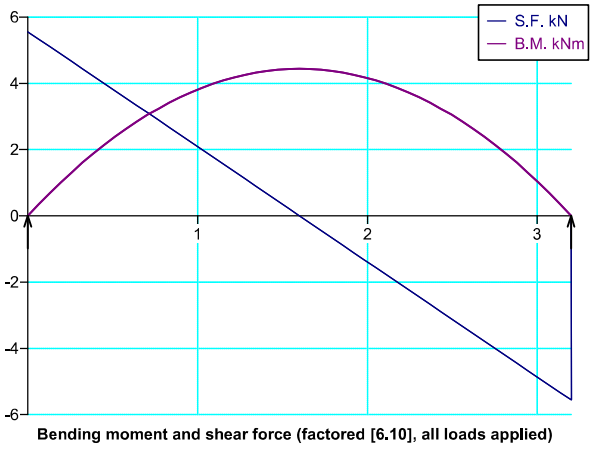
	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O G	o.w.	0.1	0		L	0.16	0.16	0.14
U G	Pitch Dead	1.31*2.5/2	0		L	2.62	2.62	2.24
U QH	Pitch Live	0.6*2.5/2	0		L	1.20	1.20	1.02
				Total load (unfactored):	**7.96 kN**	**3.98**	**3.98**	3.39
				Dead/Permanent (unfactored):	5.56 kN	2.78	2.78	2.37
				Live/Variable (unfactored):	2.40 kN	1.20	1.20	1.02
				Factored (6.10):	11.11 kN	5.55	5.55	

Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QH: Roof

Maximum B.M. = 4.44 kNm (6.10) at 1.60 m. from R1

Maximum S.F. = 5.55 kN (6.10) at R1

Mid-span deflections: Dead: 2.37 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 1.02 x 10^8/EI
Total: 3.39 x 10^8/EI



Bending moment and shear force (factored [6.10], all loads applied)

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

Use 2no 47 x 195 C24 *7.7 kg/m approx*

$W_{el,y}$ = 595.7 cm^3 I_y = 5,808 cm^4

Timber grade: C24

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

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

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

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

Loading modification factor, k_{mod} = 0.8 (Service class 2; Live load duration: Medium term) [EC5 Tables 2.2/3.1]

Load sharing factor, k_{sys} = 1.0

Deflection modification factor, k_{def} = 0.80 (Service class 2, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

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

Design bending strength, $f_{m,y,d} = f_{m,k}.k_{mod}.k_h.k_{sys}/\gamma_m$ = 24.0 x 0.80 x 1.00 x 1.0/1.30 = 14.77 N/mm^2

Design bending stress, $\sigma_{m,y,d}$ = 4.44 x 1000/596 = 7.46 N/mm^2 OK

Bending resistance = 14.77 x 596/1000 = 8.80 kNm

Shear

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

Design shear strength, $f_{v,d} = f_{v,k}.k_{mod}.k_{sys}/\gamma_m$ = 4.00 x 0.80 x 1.0/1.30 = 2.46 N/mm^2

Design shear stress, $\sigma_{v,y,d}$ = 5.55 x 1000 x (3/2)/(63.0 x 195) = 0.68 N/mm^2 OK

Shear resistance = 2.46 x 63 x 195 x (2/3)/1000 = 20.15 kN

Deflection

Final deflection limit = 0.003L = 9.60 mm
Final deflection, $u_{fin} = \Sigma u_{inst}(1 + \psi_2.k_{def})$ [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 x 3.18 x 10^6/((11,000/16) x 94 x 195) = 0.30 mm

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

Mid-span deflections:	x 1e^8/EI	Inst. mm	k_{def}	ψ_2	Final mm	
G:	2.37	3.71	0.80	1.00	6.68	
QH:	1.02	1.60	0.80	0.00	1.60	
Shear deflection:		0.30			0.47	
Total mm.		5.62			**8.76**	OK

Beam: [R2] **Span: 2.4 m.**

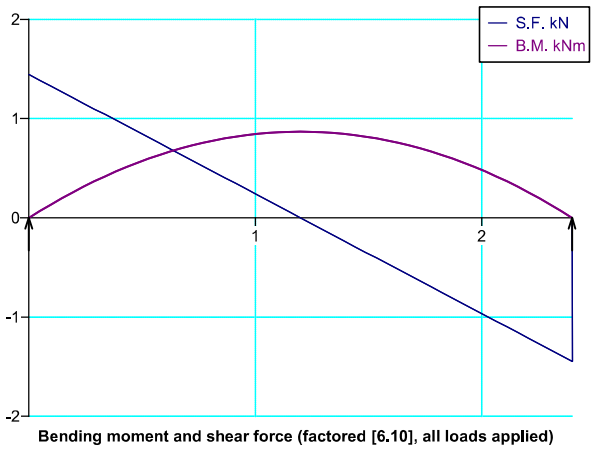
	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O G	o.w.	0.1	0		L	0.12	0.12	0.04
U G	Pitch Dead	1.31*0.4	0		L	0.63	0.63	0.23
U QH	Pitch Live	0.6*0.4	0		L	0.29	0.29	0.10
				Total load (unfactored):	**2.07 kN**	**1.04**	**1.04**	0.37
				Dead/Permanent (unfactored):	1.50 kN	0.75	0.75	0.27
				Live/Variable (unfactored):	0.58 kN	0.29	0.29	0.10
				Factored (6.10):	2.89 kN	1.44	1.44	

Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QH: Roof

Maximum B.M. = 0.866 kNm (6.10) at 1.20 m. from R1

Maximum S.F. = 1.44 kN (6.10) at R1

Mid-span deflections: Dead: 0.27 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 0.10 x 10^8/EI
Total: 0.37 x 10^8/EI



Bending moment and shear force (factored [6.10], all loads applied)

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

Use 47 x 150 C24 *3.0 kg/m approx*

$W_{el,y}$ = 176.3 cm^3 I_y = 1,322 cm^4

Timber grade: C24

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

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

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

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

Loading modification factor, k_{mod} = 0.8 (Service class 2; Live load duration: Medium term) [EC5 Tables 2.2/3.1]

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

Deflection modification factor, k_{def} = 0.80 (Service class 2, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

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

Design bending strength, $f_{m,y,d} = f_{m,k}.k_{mod}.k_h.k_{sys}/\gamma_m$ = 24.0 x 0.80 x 1.00 x 1.1/1.30 = 16.25 N/mm^2

Design bending stress, $\sigma_{m,y,d}$ = 0.87 x 1000/176 = 4.91 N/mm^2 OK

Bending resistance = 16.25 x 176/1000 = 2.86 kNm

Shear

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

Design shear strength, $f_{v,d} = f_{v,k}.k_{mod}.k_{sys}/\gamma_m$ = 4.00 x 0.80 x 1.1/1.30 = 2.71 N/mm^2

Design shear stress, $\sigma_{v,y,d}$ = 1.44 x 1000 x (3/2)/(31.5 x 150) = 0.46 N/mm^2 OK

Shear resistance = 2.71 x 31.5 x 150 x (2/3)/1000 = 8.53 kN

Deflection

Final deflection limit = 0.003L = 7.20 mm
Final deflection, $u_{fin} = \Sigma u_{inst}(1 + \psi_2.k_{def})$ [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 x 0.62 x 10^6/((11,000/16) x 47 x 150) = 0.15 mm

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

Mid-span deflections:	x 1e^8/EI	Inst. mm	k_{def}	ψ_2	Final mm	
G:	0.27	1.85	0.80	1.00	3.34	
QH:	0.10	0.71	0.80	0.00	0.71	
Shear deflection:		0.15			0.24	
Total mm.		2.72			**4.29**	OK

Beam: [2R2] **Span: 2.4 m.**

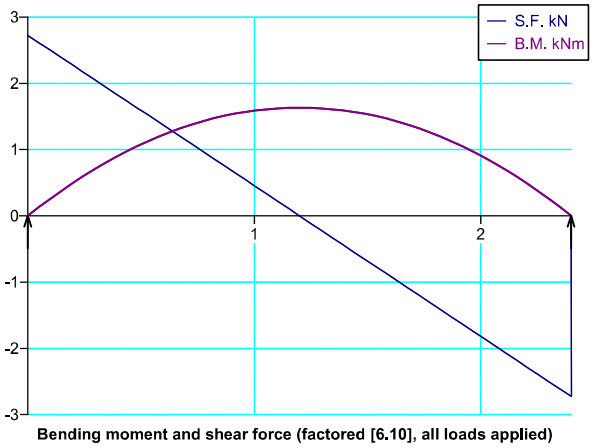
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.1	0		L	0.12	0.12	0.04
U G	Pitch Dead	1.31*1.6/2	0		L	1.26	1.26	0.45
U QH	Pitch Live	0.6*1.6/2	0		L	0.58	0.58	0.21
				Total load (unfactored):	**3.91 kN**	**1.95**	**1.95**	0.70
				Dead/Permanent (unfactored):	2.76 kN	1.38	1.38	0.50
				Live/Variable (unfactored):	1.15 kN	0.58	0.58	0.21
				Factored (6.10):	5.45 kN	2.72	2.72	

Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QH: Roof

Maximum B.M. = 1.63 kNm (6.10) at 1.20 m. from R1

Maximum S.F. = 2.72 kN (6.10) at R1

Mid-span deflections: Dead: 0.50 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 0.21 x 10^8/EI
Total: 0.70 x 10^8/EI



Bending moment and shear force (factored [6.10], all loads applied)

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

Use 2no 47 x 150 C24 *5.9 kg/m approx*

$W_{el,y}$ = 352.5 cm^3 I_y = 2,644 cm^4

Timber grade: C24

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

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

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

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

Loading modification factor, k_{mod} = 0.8 (Service class 2; Live load duration: Medium term) [EC5 Tables 2.2/3.1]

Load sharing factor, k_{sys} = 1.0

Deflection modification factor, k_{def} = 0.80 (Service class 2, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

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

Design bending strength, $f_{m,y,d} = f_{m,k}.k_{mod}.k_h.k_{sys}/\gamma_m$ = 24.0 x 0.80 x 1.00 x 1.0/1.30 = 14.77 N/mm^2

Design bending stress, $\sigma_{m,y,d}$ = 1.63 x 1000/353 = 4.64 N/mm^2 OK

Bending resistance = 14.77 x 353/1000 = 5.21 kNm

Shear

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

Design shear strength, $f_{v,d} = f_{v,k}.k_{mod}.k_{sys}/\gamma_m$ = 4.00 x 0.80 x 1.0/1.30 = 2.46 N/mm^2

Design shear stress, $\sigma_{v,y,d}$ = 2.72 x 1000 x (3/2)/(63.0 x 150) = 0.43 N/mm^2 OK

Shear resistance = 2.46 x 63 x 150 x (2/3)/1000 = 15.50 kN

Deflection

Final deflection limit = 0.003L = 7.20 mm
Final deflection, $u_{fin} = \Sigma u_{inst}(1 + \psi_2.k_{def})$ [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 x 1.17 x 10^6/((11,000/16) x 94 x 150) = 0.15 mm

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

Mid-span deflections:	x 1e^8/EI	Inst. mm	k_{def}	ψ_2	Final mm	
G:	0.50	1.71	0.80	1.00	3.07	
QH:	0.21	0.71	0.80	0.00	0.71	
Shear deflection:		0.15			0.23	
Total mm.		2.56			**4.01**	OK

Beam: **[B24] [B25] [B11]** **Span: 1.4 m.**

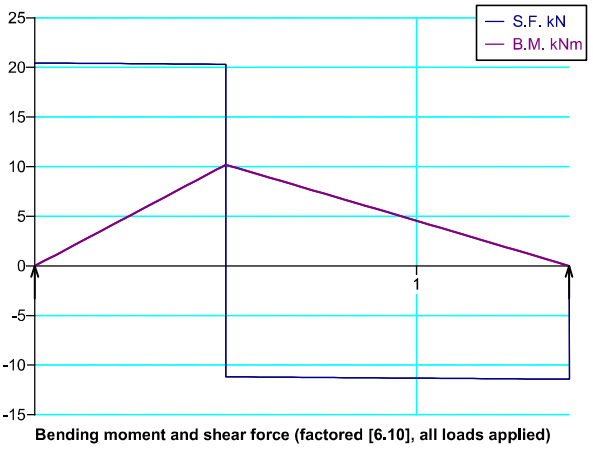
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.2	0		L	0.14	0.14	0.01
P G	Bm: [B21] [3.8 m.] : R1	6.35	0.5			4.08	2.27	0.32
P QH	Bm: [B21] [3.8 m.] : R1	2.74	0.5			1.76	0.98	0.14
P G	Bm: [B22] [5.0 m.] : R2	9.46	0.5			6.08	3.38	0.48
P QH	Bm: [B22] [5.0 m.] : R2	3.99	0.5			2.57	1.43	0.20
				Total load (unfactored):	**22.82 kN**	**14.63**	**8.19**	1.16
				Dead/Permanent (unfactored):	16.09 kN	10.30	5.79	0.81
				Live/Variable (unfactored):	6.73 kN	4.33	2.40	0.34
				Factored (6.10):	31.82 kN	20.40	11.42	

Load types: O:Beam o.w.; P:Point load; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed:

Maximum B.M. = 10.17 kNm (6.10) at 0.50 m. from R1

Maximum S.F. = 20.40 kN (6.10) at R1

Mid-span deflections: Dead: 0.81 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 0.34 x 10^8/EI
Total: 1.16 x 10^8/EI



Bending moment and shear force (factored [6.10], all loads applied)

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **152 x 89 x 16 UKB** S355

D=152.4 mm B=88.7 mm t=4.5 mm T=7.7 mm I_y=834 cm^4 i_z=2.10 cm $W_{pl,y}$=123 cm^3 $W_{el,y}$=109 cm^3

Classification: Flange: c/t = 34.5/7.7 = 4.48 <= 9ε (7.32): Class 1, plastic
EC3 Table 5.2 Web: c/t = 121.8/4.5 = 27.1 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 20.40 kN

Shear area, A_v = A - 2bt_f + (t_w+2r)t_f = 20.3x100 - 2x88.7x7.70 + (4.50+2x7.60) x7.70 = 816 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd}$ = A_v.(f_y/√3)/γ_{M0} = 816 x (355/√3)/(1.0 x 1000) = 167 kN (>=20.40) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 137.0/4.5 = 30.44 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 10.17 kNm

Moment resistance, $M_{c,y,Rd}$ = f_y.$W_{pl,y}$ = 355 x 123/1000 = 43.7 kNm OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

Support conditions R1/R2: Compression flange laterally unrestrained; both flanges free to rotate on plan. Partial torsional restraint by dead bearing of bottom flange to support (1.2L+2D)

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

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

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

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

$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta\overline{\lambda}_{LT}^2)}]$ [EC3 (6.57)]

$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta\overline{\lambda}_{LT}^2]$

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

$\overline{\lambda}_{LT} = \sqrt{(f_y.W_{pl,y}/M_{cr})}$

$M_{cr} = C_1(\pi^2 EI_z/L_e^2)\sqrt{(I_w/I_z + L_e^2 GI_T/\pi^2 EI_z)}$ [SN003: k and k_w taken as 1.0, conservative]

$W_y = 123.0\ cm^3$ $I_w = 0.005\ dm^6$ $I_T = 3.56\ cm^4$ $I_z = 89.8\ cm^4$ $G = 81{,}000 N/mm^2$

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\overline{\lambda}_z$	$\overline{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-1.40	1.98	10.0	1.40*	0.84	70.6	1.237	0.787	0.798	0.824	0.894	43.7	39.0	OK

* $C1 = 4M_{max}/\sqrt{(M_{max}^2+4M_a^2+7M_b^2+4M_c^2)} <= 2.5$ [$M_{a,b,c}$ quarter pt moments] [Wong & Driver, AISC Eng. Journal, Q1 2010]

Combined bending and shear

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

Web buckling and crushing have not been checked

Deflection

LL deflection = 0.34 x 1e8/(210,000 x 834) = 0.2 mm (L/7167) OK
TL deflection = 1.16 x 1e8/(210,000 x 834) = 0.7 mm (L/2121)

Beam: [L21] **Span: 1.2 m.**

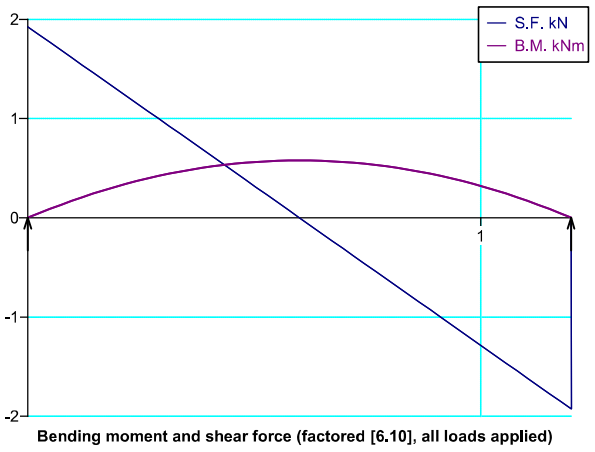
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.1	0		L	0.06	0.06	0.003
U G	Dormer Roof Dead	1.31*2.3/2	0		L	0.90	0.90	0.041
U QH	Dormer Roof Live	0.6*2.3/2	0		L	0.41	0.41	0.019
				Total load (unfactored):	**2.76 kN**	**1.38**	**1.38**	0.062
				Dead/Permanent (unfactored):	1.93 kN	0.96	0.96	0.043
				Live/Variable (unfactored):	0.83 kN	0.41	0.41	0.019
				Factored (6.10):	3.84 kN	1.92	1.92	

Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QH: Roof

Maximum B.M. = 0.577 kNm (6.10) at 0.60 m. from R1

Maximum S.F. = 1.92 kN (6.10) at R1

Mid-span deflections: Dead: 0.043 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 0.019 x 10^8/EI
Total: 0.062 x 10^8/EI



Bending moment and shear force (factored [6.10], all loads applied)

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

Use 2no 47 x 120 C24 *4.7 kg/m approx*

$W_{el,y}$ = 225.6 cm^3 I_y = 1,354 cm^4

Timber grade: C24

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

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

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

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

Loading modification factor, k_{mod} = 0.8 (Service class 2; Live load duration: Medium term) [EC5 Tables 2.2/3.1]

Load sharing factor, k_{sys} = 1.0

Deflection modification factor, k_{def} = 0.80 (Service class 2, solid timber/glulam/LVL) [EC5 Table 3.2]

Bending

Height factor, $k_h = \min((150/h)^{0.2}, 1.3) = (150/120)^{0.2} = 1.05$ [EC5 3.2(3)]

Design bending strength, $f_{m,y,d} = f_{m,k}.k_{mod}.k_h.k_{sys}/\gamma_m$ = 24.0 x 0.80 x 1.05 x 1.0/1.30 = 15.44 N/mm^2

Design bending stress, $\sigma_{m,y,d}$ = 0.58 x 1000/226 = 2.56 N/mm^2 OK

Bending resistance = 15.44 x 226/1000 = 3.48 kNm

Shear

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

Design shear strength, $f_{v,d} = f_{v,k}.k_{mod}.k_{sys}/\gamma_m$ = 4.00 x 0.80 x 1.0/1.30 = 2.46 N/mm^2

Design shear stress, $\sigma_{v,y,d}$ = 1.92 x 1000 x (3/2)/(63.0 x 120) = 0.38 N/mm^2 OK

Shear resistance = 2.46 x 63 x 120 x (2/3)/1000 = 12.40 kN

Deflection

Final deflection limit = 0.003L = 3.60 mm
Final deflection, $u_{fin} = \Sigma u_{inst}(1 + \psi_2.k_{def})$ [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 x 0.413 x 10^6/((11,000/16) x 94 x 120) = 0.06 mm

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

Mid-span deflections:	x 1e^8/EI	Inst. mm	k_{def}	ψ_2	Final mm	
G:	0.043	0.29	0.80	1.00	0.52	
QH:	0.019	0.13	0.80	0.00	0.13	
Shear deflection:		0.06			0.10	
Total mm.		0.48			**0.75**	OK

Beam: [B12] **Span: 3.8 m.**

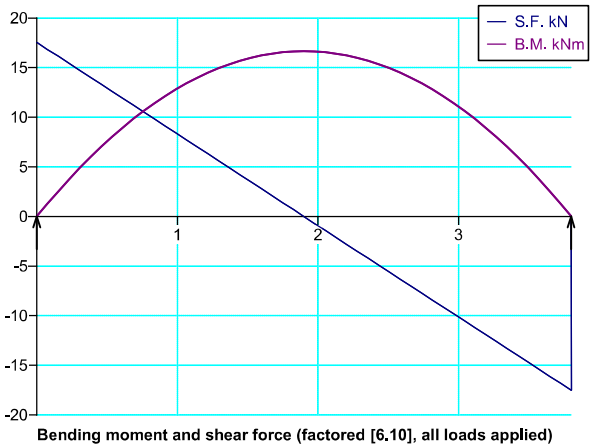
	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O G	o.w.	0.2	0		L	0.38	0.38	0.54
U G	Pitch Dead	1.31*4.1/2	0		L	5.10	5.10	7.29
U QH	Pitch Live	0.6*4.1/2	0		L	2.34	2.34	3.34
U G	Dwarf Stud	1*0.4	0		L	0.76	0.76	1.09
U G	Parallel Floor Dead	1*0.5	0		L	0.95	0.95	1.36
U QA	Parallel Floor Live	1*1.5	0		L	2.85	2.85	4.07
			Total load (unfactored):		**24.76 kN**	**12.38**	**12.38**	17.69
			Dead/Permanent (unfactored):		14.38 kN	7.19	7.19	10.28
			Live/Variable (unfactored):		10.37 kN	5.19	5.19	7.41
			Factored (6.10):		34.98 kN	17.49	17.49	

Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QA: Residential; QH: Roof

Maximum B.M. = 16.62 kNm (6.10) at 1.90 m. from R1

Maximum S.F. = 17.49 kN (6.10) at R1

Mid-span deflections: Dead: 10.28 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 7.41 x 10^8/EI
Total: 17.69 x 10^8/EI



Bending moment and shear force (factored [6.10], all loads applied)

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **152 x 89 x 16 UKB** S355

D=152.4 mm B=88.7 mm t=4.5 mm T=7.7 mm I_y=834 cm^4 i_z=2.10 cm $W_{pl,y}$=123 cm^3 $W_{el,y}$=109 cm^3

Classification: Flange: c/t = 34.5/7.7 = 4.48 <= 9ε (7.32): Class 1, plastic
EC3 Table 5.2 Web: c/t = 121.8/4.5 = 27.1 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 17.49 kN

Shear area, A_v = A - 2bt_f + (t_w+2r)t_f = 20.3x100 - 2x88.7x7.70 + (4.50+2x7.60) x7.70 = 816 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd}$ = A_v.(f_y/√3)/γ_{M0} = 816 x (355/√3)/(1.0 x 1000) = 167 kN (>=17.49) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 137.0/4.5 = 30.44 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 16.62 kNm

Moment resistance, $M_{c,y,Rd}$ = f_y.$W_{pl,y}$ = 355 x 123/1000 = 43.7 kNm OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

Support conditions R1/R2: Compression flange laterally unrestrained; both flanges free to rotate on plan. Partial torsional restraint by dead bearing of bottom flange to support (1.2L+2D)

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

$\chi_{LT,mod}$ = χ_{LT}/f (but <=1/$\overline{\lambda}_{LT}^2$ and <=1.0) [Eq.6.58]

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

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

$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta\overline{\lambda}_{LT}^2)}]$ [EC3 (6.57)]

$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta\overline{\lambda}_{LT}^2]$

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

$\overline{\lambda}_{LT} = \sqrt{(f_y.W_{pl,y}/M_{cr})}$

$M_{cr} = C_1(\pi^2 EI_z/L_e^2)\sqrt{(I_w/I_z + L_e^2 GI_T/\pi^2 EI_z)}$ [SN003: k and k_w taken as 1.0, conservative]

$W_y = 123.0\ cm^3$ $I_w = 0.005\ dm^6$ $I_T = 3.56\ cm^4$ $I_z = 89.8\ cm^4$ $G = 81{,}000 N/mm^2$

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\overline{\lambda}_z$	$\overline{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-3.80	4.86	16.6	1.13*	0.94	18.2	3.032	1.550	1.596	0.407	0.405	43.7	17.7	OK

* $C1 = 4M_{max}/\sqrt{(M_{max}^2+4M_a^2+7M_b^2+4M_c^2)} <= 2.5$ [$M_{a,b,c}$ quarter pt moments] [Wong & Driver, AISC Eng. Journal, Q1 2010]

Combined bending and shear

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

Web buckling and crushing have not been checked

Deflection

LL deflection = 7.41 x 1e8/(210,000 x 834) = 4.2 mm (L/898) OK
TL deflection = 17.69 x 1e8/(210,000 x 834) = 10.1 mm (L/376)

Bearings

152 x 89 x 16 UB stiff bearing length, $b_1 = t + 1.6r + 2T = 32.1$ mm

R1 (17.49 kN) : 175 x 100 x 75h mm padstone

Masonry: $3.6 N/mm^2$ 100mm aerated concrete block, shape factor, $\delta = 1.38$ [Table A1 BS EN 772-1]
Normalised compressive strength, $f_b = 3.60 \times 1.38 = 4.97\ N/mm^2$
Mortar: M4: $f_m = 4.0\ N/mm^2$
K = 0.75, $\alpha = 0.7$, B = 0.3 [EC6-1-1 UK NA 2012 Table NA.4]
Characteristic compressive strength, $f_k = K.f_b^{\alpha}.f_m^{B} = 0.75 \times 4.97^{0.7} \times 4.00^{0.3} = 3.49\ N/mm^2$ [EC6-1 Eq(3.1)]
Category II mfr, class 2 execution: $\gamma_m = 3.0$ [EC6-3 UK NA Table NA.1]
Design compressive strength, $f_d = f_k/\gamma_m = 3.49/3.0 = 1.16\ N/mm^2$

Stress under padstone = $17.49 \times 1000/175 \times 100 = 1.00\ N/mm^2$ OK

R2 (17.49 kN) : None

Beam: **[B13]** **Span: 2.8 m.**

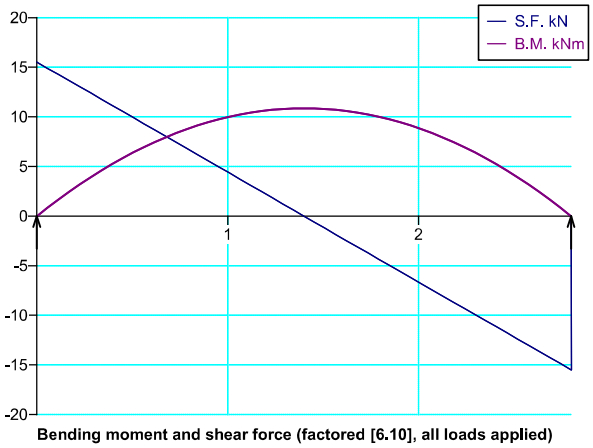
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.2	0		L	0.28	0.28	0.16
U G	Pitch Dead	1.31*4.1/2	0		L	3.76	3.76	2.15
U QH	Pitch Live	0.6*4.1/2	0		L	1.72	1.72	0.98
U G	Dwarf Stud	1*0.4	0		L	0.56	0.56	0.32
U G	Floor Dead	0.5*2.6/2	0		L	0.91	0.91	0.52
U QA	Floor Live + parts	2*2.6/2	0		L	3.64	3.64	2.08
				Total load (unfactored):	**21.74 kN**	**10.87**	**10.87**	6.22
				Dead/Permanent (unfactored):	11.02 kN	5.51	5.51	3.15
				Live/Variable (unfactored):	10.72 kN	5.36	5.36	3.07
				Factored (6.10):	30.96 kN	15.48	15.48	

Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QA: Residential; QH: Roof

Maximum B.M. = 10.84 kNm (6.10) at 1.40 m. from R1

Maximum S.F. = 15.48 kN (6.10) at R1

Mid-span deflections: Dead: 3.15 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 3.07 x 10^8/EI
Total: 6.22 x 10^8/EI



Bending moment and shear force (factored [6.10], all loads applied)

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **152 x 89 x 16 UKB** S355

D=152.4 mm B=88.7 mm t=4.5 mm T=7.7 mm I_y=834 cm^4 i_z=2.10 cm $W_{pl,y}$=123 cm^3 $W_{el,y}$=109 cm^3

Classification: Flange: c/t = 34.5/7.7 = 4.48 <= 9ε (7.32): Class 1, plastic
EC3 Table 5.2 Web: c/t = 121.8/4.5 = 27.1 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 15.48 kN

Shear area, A_v = A - 2bt_f + (t_w+2r)t_f = 20.3x100 - 2x88.7x7.70 + (4.50+2x7.60) x7.70 = 816 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd}$ = A_v.(f_y/√3)/γ_{M0} = 816 x (355/√3)/(1.0 x 1000) = 167 kN (>=15.48) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 137.0/4.5 = 30.44 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 10.84 kNm

Moment resistance, $M_{c,y,Rd}$ = f_y.$W_{pl,y}$ = 355 x 123/1000 = 43.7 kNm OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

Support conditions R1/R2: Compression flange laterally unrestrained; both flanges free to rotate on plan. Partial torsional restraint by dead bearing of bottom flange to support (1.2L+2D)

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

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

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

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

$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta\overline{\lambda}_{LT}^2)}]$ [EC3 (6.57)]

$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta\overline{\lambda}_{LT}^2]$

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

$\overline{\lambda}_{LT} = \sqrt{(f_y.W_{pl,y}/M_{cr})}$

$M_{cr} = C_1(\pi^2 EI_z/L_e^2)\sqrt{(I_w/I_z + L_e^2 GI_T/\pi^2 EI_z)}$ [SN003: k and k_w taken as 1.0, conservative]

W_y = 123.0 cm^3 I_w = 0.005 dm^6 I_T = 3.56 cm^4 I_z = 89.8 cm^4 G = 81,000N/mm^2

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\overline{\lambda}_z$	$\overline{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-2.80	3.66	10.8	1.13*	0.94	25.3	2.284	1.315	1.304	0.516	0.523	43.7	22.8	OK

* C1 = $4M_{max}/\sqrt{(M_{max}^2+4M_a^2+7M_b^2+4M_c^2)}$ <= 2.5 [$M_{a,b,c}$ quarter pt moments] [Wong & Driver, AISC Eng. Journal, Q1 2010]

Combined bending and shear

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

Web buckling and crushing have not been checked

Deflection

LL deflection = 3.07 x 1e8/(210,000 x 834) = 1.8 mm (L/1597) OK
TL deflection = 6.22 x 1e8/(210,000 x 834) = 3.6 mm (L/788)

Beam: **Front Stair Trimmer** **Span: 2.4 m.**

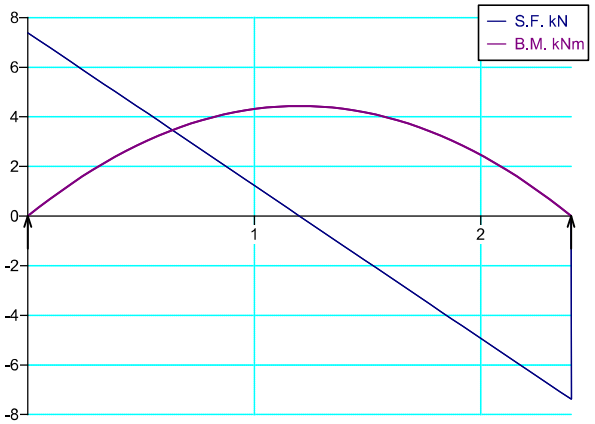
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.1	0		L	0.12	0.12	0.04
U G	Pitch Dead	1.31*4.1/2	0		L	3.22	3.22	1.16
U QH	Pitch Live	0.6*4.1/2	0		L	1.48	1.48	0.53
U G	Dwarf Stud	1*0.4	0		L	0.48	0.48	0.17
				Total load (unfactored):	**10.60 kN**	**5.30**	**5.30**	1.91
				Dead/Permanent (unfactored):	7.65 kN	3.82	3.82	1.38
				Live/Variable (unfactored):	2.95 kN	1.48	1.48	0.53
				Factored (6.10):	14.75 kN	7.37	7.37	

Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QH: Roof

Maximum B.M. = 4.42 kNm (6.10) at 1.20 m. from R1

Maximum S.F. = 7.37 kN (6.10) at R1

Mid-span deflections: Dead: 1.38 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 0.53 x 10^8/EI
Total: 1.91 x 10^8/EI



Bending moment and shear force (factored [6.10], all loads applied)

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

Use 2no 47 x 195 C24 *7.7 kg/m approx*

$W_{el,y}$ = 595.7 cm^3 I_y = 5,808 cm^4

Timber grade: C24

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

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

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

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

Loading modification factor, k_{mod} = 0.8 (Service class 1; Live load duration: Medium term) [EC5 Tables 2.2/3.1]

Load sharing factor, k_{sys} = 1.0

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, $f_{m,y,d} = f_{m,k}.k_{mod}.k_h.k_{sys}/\gamma_m$ = 24.0 x 0.80 x 1.00 x 1.0/1.30 = 14.77 N/mm^2

Design bending stress, $\sigma_{m,y,d}$ = 4.42 x 1000/596 = 7.43 N/mm^2 OK

Bending resistance = 14.77 x 596/1000 = 8.80 kNm

Shear

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

Design shear strength, $f_{v,d} = f_{v,k}.k_{mod}.k_{sys}/\gamma_m$ = 4.00 x 0.80 x 1.0/1.30 = 2.46 N/mm^2

Design shear stress, $\sigma_{v,y,d}$ = 7.37 x 1000 x (3/2)/(63.0 x 195) = 0.90 N/mm^2 OK

Shear resistance = 2.46 x 63 x 195 x (2/3)/1000 = 20.15 kN

Deflection

Final deflection limit = 0.003L = 7.20 mm
Final deflection, $u_{fin} = \Sigma u_{inst}(1 + \psi_2.k_{def})$ [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 x 3.18 x 10^6/((11,000/16) x 94 x 195) = 0.30 mm

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

Mid-span deflections:	x 1e^8/EI	Inst. mm	k_{def}	ψ_2	Final mm	
G:	1.38	2.15	0.60	1.00	3.45	
QH:	0.53	0.83	0.60	0.00	0.83	
Shear deflection:		0.30			0.43	
Total mm.		3.29			**4.71**	OK

Beam: **Rear Stair Trimmer** **Span: 2.4 m.**

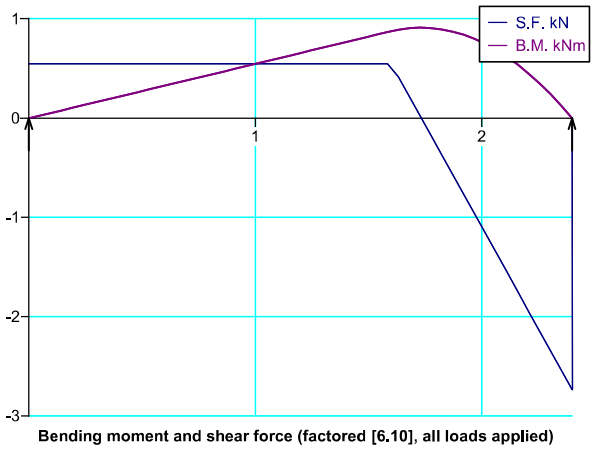
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
R G	Stair Dead	0.5*2.8/2	1.6		2.4	0.09	0.47	0.07
R QA	Stair Live	1.5*2.8/2	1.6		2.4	0.28	1.40	0.22
				Total load (unfactored):	**2.24 kN**	**0.37**	**1.87**	0.30
				Dead/Permanent (unfactored):	0.56 kN	0.09	0.47	0.07
				Live/Variable (unfactored):	1.68 kN	0.28	1.40	0.22
				Factored (6.10):	3.28 kN	0.55	2.73	

Load types: R:Part UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QA: Residential

Maximum B.M. = 0.910 kNm (6.10) at 1.73 m. from R1

Maximum S.F. = -2.73 kN (6.10) at R2

Mid-span deflections: Dead: 0.07 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 0.22 x 10^8/EI
Total: 0.30 x 10^8/EI



Bending moment and shear force (factored [6.10], all loads applied)

Beam: [B14] **Span: 4.1 m.**

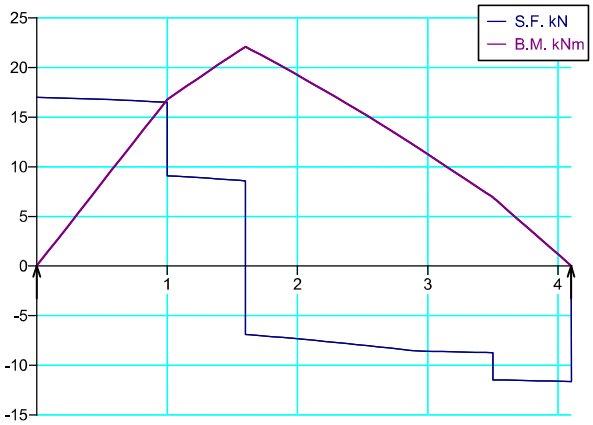
	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O G	o.w.	0.2	0		L	0.41	0.41	0.74
V G	Studwork	0	0	2.4*0.4	2.9	0.74	0.66	1.72
P G	Beam: Front Stair : R2	3.82	1			2.89	0.93	3.70
P QH	Beam: Front Stair : R2	1.48	1			1.12	0.36	1.43
P G	Beam: Rear Stair : R2	0.47	3.5			0.07	0.40	0.29
P QA	Beam: Rear Stair : R2	1.40	3.5			0.20	1.20	0.86
P G	Bm: [B13] [2.8 m.] : R1	5.51	1.6			3.36	2.15	7.38
P QA	Bm: [B13] [2.8 m.] : R1	3.64	1.6			2.22	1.42	4.88
P QH	Bm: [B13] [2.8 m.] : R1	1.72	1.6			1.05	0.67	2.30
				Total load (unfactored):	**20.25 kN**	**12.05**	**8.20**	23.29
				Dead/Permanent (unfactored):	12.01 kN	7.46	4.55	13.82
				Live/Variable (unfactored):	8.24 kN	4.59	3.65	9.47
				Factored (6.10):	28.58 kN	16.96	11.61	

Load types: O:Beam o.w.; V:Variable load; P:Point load; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed:

Maximum B.M. = 22.06 kNm (6.10) at 1.60 m. from R1

Maximum S.F. = 16.96 kN (6.10) at R1

Mid-span deflections: Dead: 13.82 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 9.47 x 10^8/EI
Total: 23.29 x 10^8/EI



Bending moment and shear force (factored [6.10], all loads applied)

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **178 x 102 x 19 UKB** S355

D=177.8 mm B=101.2 mm t=4.8 mm T=7.9 mm I_y=1,360 cm^4 i_z=2.37 cm $W_{pl,y}$=171 cm^3 $W_{el,y}$=153 cm^3

Classification: Flange: c/t = 40.6/7.9 = 5.14 <= 9ε (7.32): Class 1, plastic
EC3 Table 5.2 Web: c/t = 146.8/4.8 = 30.6 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 16.96 kN

Shear area, A_v = A - 2bt_f + (t_w+2r)t_f = 24.3x100 - 2x101x7.90 + (4.80+2x7.60) x7.90 = 989 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd}$ = A_v.(f_y/√3)/γ_{M0} = 989 x (355/√3)/(1.0 x 1000) = 203 kN (>=16.96) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 162.0/4.8 = 33.75 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 22.06 kNm

Moment resistance, $M_{c,y,Rd}$ = f_y.$W_{pl,y}$ = 355 x 171/1000 = 60.7 kNm OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

Support conditions R1/R2: Compression flange laterally unrestrained; both flanges free to rotate on plan. Partial torsional restraint by dead bearing of bottom flange to support (1.2L+2D)

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

$\chi_{LT,mod} = \chi_{LT}/f$ (but $<=1/\overline{\lambda}_{LT}^2$ and $<=1.0$) [Eq.6.58]

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

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

$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta\overline{\lambda}_{LT}^2)}]$ [EC3 (6.57)]

$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta\overline{\lambda}_{LT}^2]$

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

$\overline{\lambda}_{LT} = \sqrt{(f_y.W_{pl,y}/M_{cr})}$

$M_{cr} = C_1(\pi^2 EI_z/L_e^2)\sqrt{(I_w/I_z + L_e^2 GI_T/\pi^2 EI_z)}$ [SN003: k and k_w taken as 1.0, conservative]

$W_y = 171.0\ cm^3$ $I_w = 0.010\ dm^6$ $I_T = 4.41\ cm^4$ $I_z = 137\ cm^4$ $G = 81{,}000 N/mm^2$

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\overline{\lambda}_z$	$\overline{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-4.10	5.28	21.8	1.29*	0.88	27.0	2.913	1.499	1.530	0.428	0.428	60.7	26.0	OK

* $C1 = 4M_{max}/\sqrt{(M_{max}^2+4M_a^2+7M_b^2+4M_c^2)} <= 2.5$ [$M_{a,b,c}$ quarter pt moments] [Wong & Driver, AISC Eng. Journal, Q1 2010]

Combined bending and shear

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

Web buckling and crushing have not been checked

Deflection

LL deflection = 9.47 x 1e8/(210,000 x 1,360) = 3.3 mm (L/1236) OK
TL deflection = 23.29 x 1e8/(210,000 x 1,360) = 8.2 mm (L/503)

Beam: [B15] **Span: 3.2 m.**

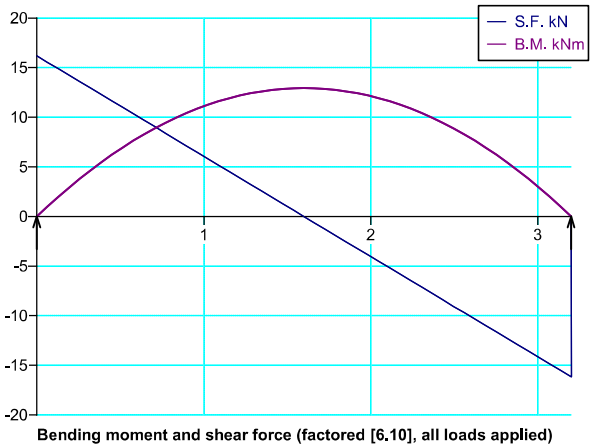
		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O	G	o.w.	0.2	0		L	0.32	0.32	0.27
U	G	Pitch Dead	1.31*4.1/2	0		L	4.30	4.30	3.67
U	QH	Pitch Live	0.6*4.1/2	0		L	1.97	1.97	1.68
U	G	Dwarf Stud	1*0.4	0		L	0.64	0.64	0.55
U	G	Floor Dead	0.5*2.6/2	0		L	1.04	1.04	0.89
U	QA	Floor Live	1.5*2.6/2	0		L	3.12	3.12	2.66
					Total load (unfactored):	**22.77 kN**	**11.38**	**11.38**	9.71
					Dead/Permanent (unfactored):	12.59 kN	6.30	6.30	5.37
					Live/Variable (unfactored):	10.18 kN	5.09	5.09	4.34
					Factored (6.10):	32.27 kN	16.13	16.13	

Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QA: Residential; QH: Roof

Maximum B.M. = 12.91 kNm (6.10) at 1.60 m. from R1

Maximum S.F. = 16.13 kN (6.10) at R1

Mid-span deflections: Dead: 5.37 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 4.34 x 10^8/EI
Total: 9.71 x 10^8/EI



Bending moment and shear force (factored [6.10], all loads applied)

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **152 x 89 x 16 UKB** S355

D=152.4 mm B=88.7 mm t=4.5 mm T=7.7 mm I_y=834 cm^4 i_z=2.10 cm $W_{pl,y}$=123 cm^3 $W_{el,y}$=109 cm^3

Classification: Flange: c/t = 34.5/7.7 = 4.48 <= 9ε (7.32): Class 1, plastic
EC3 Table 5.2 Web: c/t = 121.8/4.5 = 27.1 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 16.13 kN

Shear area, A_v = A - 2bt_f + (t_w+2r)t_f = 20.3x100 - 2x88.7x7.70 + (4.50+2x7.60) x7.70 = 816 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd}$ = A_v.(f_y/√3)/γ_{M0} = 816 x (355/√3)/(1.0 x 1000) = 167 kN (>=16.13) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 137.0/4.5 = 30.44 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 12.91 kNm

Moment resistance, $M_{c,y,Rd}$ = f_y.$W_{pl,y}$ = 355 x 123/1000 = 43.7 kNm OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

Support conditions R1/R2: Compression flange laterally unrestrained; both flanges free to rotate on plan. Partial torsional restraint by dead bearing of bottom flange to support (1.2L+2D)

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

$\chi_{LT,mod}$ = χ_{LT}/f (but <=1/$\overline{\lambda}_{LT}^2$ and <=1.0) [Eq.6.58]

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

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

$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta\overline{\lambda}_{LT}^2)}]$ [EC3 (6.57)]

$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta\overline{\lambda}_{LT}^2]$

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

$\overline{\lambda}_{LT} = \sqrt{(f_y.W_{pl,y}/M_{cr})}$

$M_{cr} = C_1(\pi^2 EI_z/L_e^2)\sqrt{(I_w/I_z + L_e^2 GI_T/\pi^2 EI_z)}$ [SN003: k and k_w taken as 1.0, conservative]

$W_y = 123.0\ cm^3$ $I_w = 0.005\ dm^6$ $I_T = 3.56\ cm^4$ $I_z = 89.8\ cm^4$ $G = 81{,}000 N/mm^2$

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\overline{\lambda}_z$	$\overline{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-3.20	4.14	12.9	1.13*	0.94	21.8	2.583	1.414	1.422	0.466	0.470	43.7	20.5	OK

* $C1 = 4M_{max}/\sqrt{(M_{max}^2 + 4M_a^2 + 7M_b^2 + 4M_c^2)} <= 2.5$ [$M_{a,b,c}$ quarter pt moments] [Wong & Driver, AISC Eng. Journal, Q1 2010]

Combined bending and shear

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

Web buckling and crushing have not been checked

Deflection

LL deflection = 4.34 x 1e8/(210,000 x 834) = 2.5 mm (L/1291) OK
TL deflection = 9.71 x 1e8/(210,000 x 834) = 5.5 mm (L/577)

Beam: [B16] **Span: 3.2 m.**

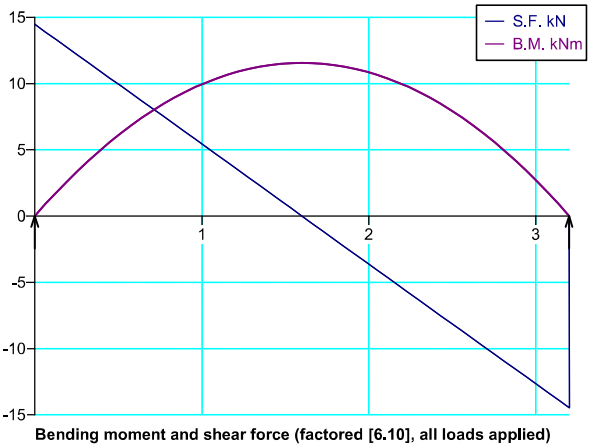
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.2	0		L	0.32	0.32	0.27
U G	Floor Dead	0.35*5.3/2	0		L	1.48	1.48	1.27
U QA	Floor Live	1.5*5.3/2	0		L	6.36	6.36	5.43
U G	Existing Ceiling	0.3*7.7/2	0		L	1.85	1.85	1.58
				Total load (unfactored):	**20.02 kN**	**10.01**	**10.01**	8.55
				Dead/Permanent (unfactored):	7.30 kN	3.65	3.65	3.12
				Live/Variable (unfactored):	12.72 kN	6.36	6.36	5.43
				Factored (6.10):	28.94 kN	14.47	14.47	

Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QA: Residential

Maximum B.M. = 11.58 kNm (6.10) at 1.60 m. from R1

Maximum S.F. = 14.47 kN (6.10) at R1

Mid-span deflections: Dead: 3.12 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 5.43 x 10^8/EI
Total: 8.55 x 10^8/EI



Bending moment and shear force (factored [6.10], all loads applied)

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **152 x 89 x 16 UKB** S355

D=152.4 mm B=88.7 mm t=4.5 mm T=7.7 mm I_y=834 cm^4 i_z=2.10 cm $W_{pl,y}$=123 cm^3 $W_{el,y}$=109 cm^3

Classification: Flange: c/t = 34.5/7.7 = 4.48 <= 9ε (7.32): Class 1, plastic
EC3 Table 5.2 Web: c/t = 121.8/4.5 = 27.1 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 14.47 kN

Shear area, A_v = A - 2bt_f + (t_w+2r)t_f = 20.3x100 - 2x88.7x7.70 + (4.50+2x7.60) x7.70 = 816 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd}$ = A_v.(f_y/√3)/γ_{M0} = 816 x (355/√3)/(1.0 x 1000) = 167 kN (>=14.47) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 137.0/4.5 = 30.44 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 11.58 kNm

Moment resistance, $M_{c,y,Rd}$ = f_y.$W_{pl,y}$ = 355 x 123/1000 = 43.7 kNm OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

Support conditions R1/R2: Compression flange laterally unrestrained; both flanges free to rotate on plan. Partial torsional restraint by dead bearing of bottom flange to support (1.2L+2D)

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

$\chi_{LT,mod}$ = χ_{LT}/f (but <=1/$\overline{\lambda}_{LT}^2$ and <=1.0) [Eq.6.58]

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

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

$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta\overline{\lambda}_{LT}^2)}]$ [EC3 (6.57)]

$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta\overline{\lambda}_{LT}^2]$

$\overline{\lambda}_{LT,0} = 0.4 \quad \beta = 0.75$ [EC3 UK NA 2.17]

$\overline{\lambda}_{LT} = \sqrt{(f_y.W_{pl,y}/M_{cr})}$

$M_{cr} = C_1(\pi^2 EI_z/L_e^2)\sqrt{(I_w/I_z + L_e^2 GI_T/\pi^2 EI_z)}$ [SN003: k and k_w taken as 1.0, conservative]

W_y = 123.0 cm^3 I_w = 0.005 dm^6 I_T = 3.56 cm^4 I_z = 89.8 cm^4 G = 81,000N/mm^2

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\overline{\lambda}_z$	$\overline{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-3.20	4.14	11.6	1.13*	0.94	21.8	2.583	1.414	1.422	0.466	0.470	43.7	20.5	OK

* C1 = $4M_{max}/\sqrt{(M_{max}^2+4M_a^2+7M_b^2+4M_c^2)}$ <= 2.5 [$M_{a,b,c}$ quarter pt moments] [Wong & Driver, AISC Eng. Journal, Q1 2010]

Combined bending and shear

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

Web buckling and crushing have not been checked

Deflection

LL deflection = 5.43 x 1e8/(210,000 x 834) = 3.1 mm (L/1032) OK
TL deflection = 8.55 x 1e8/(210,000 x 834) = 4.9 mm (L/655)

Beam: [B17] **Span: 3.2 m.**

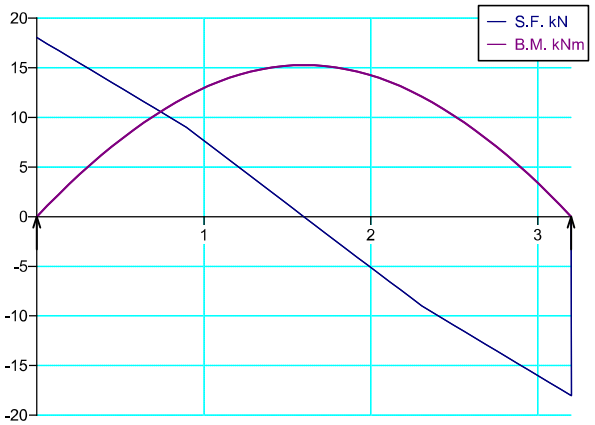
		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O	G	o.w.	0.2	0		L	0.32	0.32	0.27
U	G	Pitch Dead	1.31*4.1/2	0		L	4.30	4.30	3.67
U	QH	Pitch Live	0.6*4.1/2	0		L	1.97	1.97	1.68
U	G	Dwarf Stud	1*0.4	0		L	0.64	0.64	0.55
U	G	Floor Dead	0.5*2.6/2	0		L	1.04	1.04	0.89
U	QA	Floor Live	1.5*2.6/2	0		L	3.12	3.12	2.66
R	G	Dormer Wall	2.2*0.9	0.9		2.3	1.39	1.39	1.73
					Total load (unfactored):	**25.54 kN**	**12.77**	**12.77**	11.44
					Dead/Permanent (unfactored):	15.37 kN	7.68	7.68	7.10
					Live/Variable (unfactored):	10.18 kN	5.09	5.09	4.34
					Factored (6.10):	36.01 kN	18.00	18.00	

Load types: O:Beam o.w.; U:UDL; R:Part UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QA: Residential; QH: Roof

Maximum B.M. = 15.25 kNm (6.10) at 1.60 m. from R1

Maximum S.F. = 18.00 kN (6.10) at R1

Mid-span deflections: Dead: 7.10 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 4.34 x 10^8/EI
Total: 11.44 x 10^8/EI



Bending moment and shear force (factored [6.10], all loads applied)

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **152 x 89 x 16 UKB** S355

D=152.4 mm B=88.7 mm t=4.5 mm T=7.7 mm I_y=834 cm^4 i_z=2.10 cm $W_{pl,y}$=123 cm^3 $W_{el,y}$=109 cm^3

Classification: Flange: c/t = 34.5/7.7 = 4.48 <= 9ε (7.32): Class 1, plastic
EC3 Table 5.2 Web: c/t = 121.8/4.5 = 27.1 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 18.00 kN

Shear area, A_v = A - 2bt_f + (t_w+2r)t_f = 20.3x100 - 2x88.7x7.70 + (4.50+2x7.60) x7.70 = 816 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd}$ = A_v.(f_y/√3)/γ_{M0} = 816 x (355/√3)/(1.0 x 1000) = 167 kN (>=18.00) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 137.0/4.5 = 30.44 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 15.25 kNm

Moment resistance, $M_{c,y,Rd}$ = f_y.$W_{pl,y}$ = 355 x 123/1000 = 43.7 kNm OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

Support conditions R1/R2: Compression flange laterally unrestrained; both flanges free to rotate on plan. Partial torsional restraint by dead bearing of bottom flange to support (1.2L+2D)

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

$\chi_{LT,mod} = \chi_{LT}/f$ (but <=$1/\overline{\lambda}_{LT}^2$ and <=1.0) [Eq.6.58]

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

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

$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta\overline{\lambda}_{LT}^2)}]$ [EC3 (6.57)]

$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta\overline{\lambda}_{LT}^2]$

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

$\overline{\lambda}_{LT} = \sqrt{(f_y.W_{pl,y}/M_{cr})}$

$M_{cr} = C_1(\pi^2 EI_z/L_e^2)\sqrt{(I_w/I_z + L_e^2 GI_T/\pi^2 EI_z)}$ [SN003: k and k_w taken as 1.0, conservative]

$W_y = 123.0\ cm^3$ $I_w = 0.005\ dm^6$ $I_T = 3.56\ cm^4$ $I_z = 89.8\ cm^4$ $G = 81{,}000 N/mm^2$

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\overline{\lambda}_z$	$\overline{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-3.20	4.14	15.2	1.14*	0.94	22.0	2.583	1.408	1.415	0.469	0.473	43.7	20.6	OK

* C1 = $4M_{max}/\sqrt{(M_{max}^2+4M_a^2+7M_b^2+4M_c^2)}$ <= 2.5 [$M_{a,b,c}$ quarter pt moments] [Wong & Driver, AISC Eng. Journal, Q1 2010]

Combined bending and shear

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

Web buckling and crushing have not been checked

Deflection

LL deflection = 4.34 x 1e8/(210,000 x 834) = 2.5 mm (L/1291) OK
TL deflection = 11.44 x 1e8/(210,000 x 834) = 6.5 mm (L/490)

Beam: [B18] **Span: 5.0 m.**

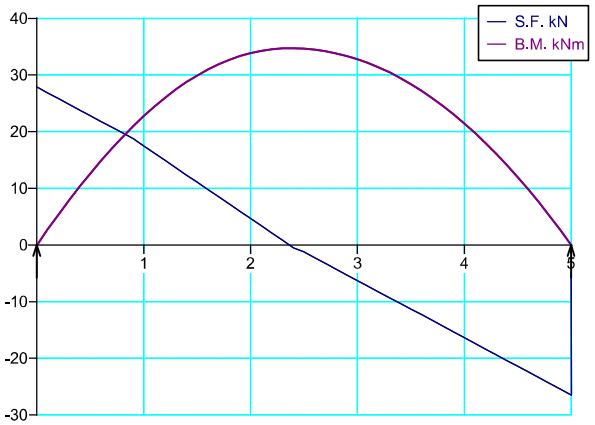
		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O	G	o.w.	0.3	0		L	0.75	0.75	2.4
U	G	Pitch Dead	1.31*4.1/2	0		L	6.71	6.71	21.9
U	QH	Pitch Live	0.6*4.1/2	0		L	3.08	3.08	10.0
U	G	Floor Dead	0.35*3.1/2	0		L	1.36	1.36	4.4
U	QA	Floor Live	1.5*3.1/2	0		L	5.81	5.81	18.9
R	G	Dormer Wall	2.2*0.9	0.9		2.3	1.88	0.89	5.8
					Total load (unfactored):	**38.2 kN**	**19.59**	**18.59**	63.4
					Dead/Permanent (unfactored):	20.4 kN	10.70	9.71	34.5
					Live/Variable (unfactored):	17.8 kN	8.89	8.89	28.9
					Factored (6.10):	54.2 kN	27.78	26.44	

Load types: O:Beam o.w.; U:UDL; R:Part UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QA: Residential; QH: Roof

Maximum B.M. = 34.61 kNm (6.10) at 2.38 m. from R1

Maximum S.F. = 27.8 kN (6.10) at R1

Mid-span deflections: Dead: 34.5 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 28.9 x 10^8/EI
Total: 63.4 x 10^8/EI



Bending moment and shear force (factored [6.10], all loads applied)

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **203 x 133 x 30 UKB** S355

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

Classification: Flange: c/t = 56.1/9.6 = 5.85 <= 9ε (7.32): Class 1, plastic
EC3 Table 5.2 Web: c/t = 172.4/6.4 = 26.9 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 27.8 kN

Shear area, A_v = A - 2bt_f + (t_w+2r)t_f = 38.2x100 - 2x134x9.60 + (6.40+2x7.60) x9.60 = 1,456 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd}$ = A_v.(f_y/√3)/γ_{M0} = 1,456 x (355/√3)/(1.0 x 1000) = 299 kN (>=27.8) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 187.6/6.4 = 29.31 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 34.6 kNm

Moment resistance, $M_{c,y,Rd}$ = f_y.$W_{pl,y}$ = 355 x 314/1000 = 111 kNm OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

Support conditions R1/R2: Compression flange laterally unrestrained; both flanges free to rotate on plan. Partial torsional restraint by dead bearing of bottom flange to support (1.2L+2D)

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

$\chi_{LT,mod}$ = χ_{LT}/f (but <=1/$\overline{\lambda}_{LT}^2$ and <=1.0) [Eq.6.58]

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

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

$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta\overline{\lambda}_{LT}^2)}]$ [EC3 (6.57)]

$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta\overline{\lambda}_{LT}^2]$

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

$\overline{\lambda}_{LT} = \sqrt{(f_y.W_{pl,y}/M_{cr})}$

$M_{cr} = C_1(\pi^2 EI_z/L_e^2)\sqrt{(I_w/I_z + L_e^2 GI_T/\pi^2 EI_z)}$ [SN003: k and k_w taken as 1.0, conservative]

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

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\overline{\lambda}_z$	$\overline{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-5.00	6.41	34.6	1.13*	0.94	50.4	2.648	1.487	1.514	0.433	0.434	111.5	48.3	OK

* $C1 = 4M_{max}/\sqrt{(M_{max}^2 + 4M_a^2 + 7M_b^2 + 4M_c^2)} <= 2.5$ [$M_{a,b,c}$ quarter pt moments] [Wong & Driver, AISC Eng. Journal, Q1 2010]

Combined bending and shear

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

Web buckling and crushing have not been checked

Deflection

LL deflection = 28.9 x 1e8/(210,000 x 2,900) = 4.8 mm (L/1053) OK
TL deflection = 63.4 x 1e8/(210,000 x 2,900) = 10.4 mm (L/480)

Bearings

203 x 133 x 30 UB stiff bearing length, $b_1 = t + 1.6r + 2T = 37.8$ mm

R1 (27.8 kN) : 250 x 100 x 125h mm padstone

Masonry: $3.6 N/mm^2$ 100mm aerated concrete block, shape factor, $\delta = 1.38$ [Table A1 BS EN 772-1]
Normalised compressive strength, $f_b = 3.60 \times 1.38 = 4.97\ N/mm^2$
Mortar: M4: $f_m = 4.0\ N/mm^2$
K = 0.75, $\alpha = 0.7$, B = 0.3 [EC6-1-1 UK NA 2012 Table NA.4]
Characteristic compressive strength, $f_k = K.f_b^{\alpha}.f_m^{B} = 0.75 \times 4.97^{0.7} \times 4.00^{0.3} = 3.49\ N/mm^2$ [EC6-1 Eq(3.1)]
Category II mfr, class 2 execution: $\gamma_m = 3.0$ [EC6-3 UK NA Table NA.1]
Design compressive strength, $f_d = f_k/\gamma_m = 3.49/3.0 = 1.16\ N/mm^2$

Stress under padstone = $27.8 \times 1000/250 \times 100 = 1.11\ N/mm^2$ OK

R2 (26.4 kN) : None

Beam: [B19] **Span: 3.8 m.**

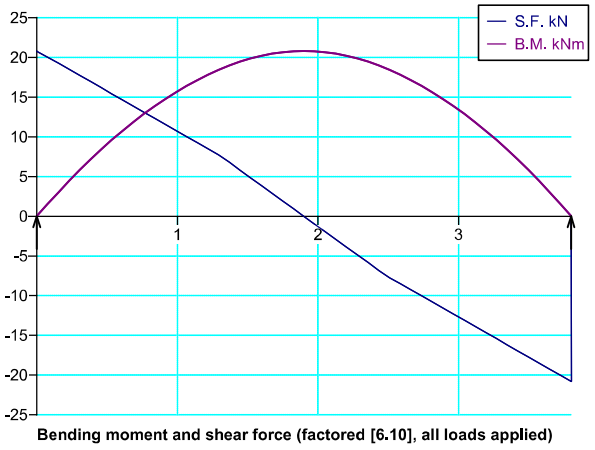
		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O	G	o.w.	0.2	0		L	0.38	0.38	0.54
U	G	Pitch Dead	1.31*4.1/2	0		L	5.10	5.10	7.29
U	QH	Pitch Live	0.6*4.1/2	0		L	2.34	2.34	3.34
U	G	Dwarf Stud	1*0.4	0		L	0.76	0.76	1.09
U	G	Floor Dead	0.5*2.6/2	0		L	1.23	1.23	1.76
U	QA	Floor Live	1.5*2.6/2	0		L	3.71	3.71	5.29
R	G	Dormer Wall	2.2*0.9	1.3		2.5	1.19	1.19	2.59
					Total load (unfactored):	**29.41 kN**	**14.71**	**14.71**	21.91
					Dead/Permanent (unfactored):	17.33 kN	8.67	8.67	13.28
					Live/Variable (unfactored):	12.08 kN	6.04	6.04	8.63
					Factored (6.10):	41.52 kN	20.76	20.76	

Load types: O:Beam o.w.; U:UDL; R:Part UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QA: Residential; QH: Roof

Maximum B.M. = 20.77 kNm (6.10) at 1.90 m. from R1

Maximum S.F. = 20.76 kN (6.10) at R1

Mid-span deflections: Dead: 13.28 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 8.63 x 10^8/EI
Total: 21.91 x 10^8/EI



Bending moment and shear force (factored [6.10], all loads applied)

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **178 x 102 x 19 UKB** S355

D=177.8 mm B=101.2 mm t=4.8 mm T=7.9 mm I_y=1,360 cm^4 i_z=2.37 cm $W_{pl,y}$=171 cm^3 $W_{el,y}$=153 cm^3

Classification: Flange: c/t = 40.6/7.9 = 5.14 <= 9ε (7.32): Class 1, plastic
EC3 Table 5.2 Web: c/t = 146.8/4.8 = 30.6 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 20.76 kN

Shear area, A_v = A - 2bt_f + (t_w+2r)t_f = 24.3x100 - 2x101x7.90 + (4.80+2x7.60) x7.90 = 989 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd}$ = A_v.(f_y/√3)/γ_{M0} = 989 x (355/√3)/(1.0 x 1000) = 203 kN (>=20.76) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 162.0/4.8 = 33.75 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 20.77 kNm

Moment resistance, $M_{c,y,Rd}$ = f_y.$W_{pl,y}$ = 355 x 171/1000 = 60.7 kNm OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

Support conditions R1/R2: Compression flange laterally unrestrained; both flanges free to rotate on plan. Partial torsional restraint by dead bearing of bottom flange to support (1.2L+2D)

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

$\chi_{LT,mod} = \chi_{LT}/f$ (but <=$1/\overline{\lambda}_{LT}^2$ and <=1.0) [Eq.6.58]

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

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

$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta\overline{\lambda}_{LT}^2)}]$ [EC3 (6.57)]

$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta\overline{\lambda}_{LT}^2]$

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

$\overline{\lambda}_{LT} = \sqrt{(f_y.W_{pl,y}/M_{cr})}$

$M_{cr} = C_1(\pi^2 EI_z/L_e^2)\sqrt{(I_w/I_z + L_e^2 GI_T/\pi^2 EI_z)}$ [SN003: k and k_w taken as 1.0, conservative]

$W_y = 171.0\ cm^3$ $I_w = 0.010\ dm^6$ $I_T = 4.41\ cm^4$ $I_z = 137\ cm^4$ $G = 81{,}000 N/mm^2$

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\overline{\lambda}_z$	$\overline{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-3.80	4.92	20.8	1.14*	0.94	26.0	2.714	1.528	1.568	0.415	0.414	60.7	25.2	OK

* $C1 = 4M_{max}/\sqrt{(M_{max}^2+4M_a^2+7M_b^2+4M_c^2)}$ <= 2.5 [$M_{a,b,c}$ quarter pt moments] [Wong & Driver, AISC Eng. Journal, Q1 2010]

Combined bending and shear

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

Web buckling and crushing have not been checked

Deflection

LL deflection = 8.63 x 1e8/(210,000 x 1,360) = 3.0 mm (L/1258) OK
TL deflection = 21.91 x 1e8/(210,000 x 1,360) = 7.7 mm (L/495)

Beam: **[FJ1]** **Span: 3.8 m.**

	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.1	0		L	0.19	0.19	0.27
U G	Floor Dead	0.35*0.4	0		L	0.27	0.27	0.38
U QA	Floor Live	1.50*0.4	0		L	1.14	1.14	1.63
				Total load (unfactored):	**3.19 kN**	**1.60**	**1.60**	2.28
				Dead/Permanent (unfactored):	0.91 kN	0.46	0.46	0.65
				Live/Variable (unfactored):	2.28 kN	1.14	1.14	1.63
				Factored (6.10):	4.65 kN	2.33	2.33	

Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QA: Residential

Maximum B.M. = 2.21 kNm (6.10) at 1.90 m. from R1

Maximum S.F. = 2.33 kN (6.10) at R1

Mid-span deflections: Dead: 0.65 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 1.63 x 10^8/EI
Total: 2.28 x 10^8/EI

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

Use 47 x 195 C24 *3.8 kg/m approx*

$W_{el,y}$ = 297.9 cm^3 I_y = 2,904 cm^4

Timber grade: C24

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

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

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

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

Loading modification factor, k_{mod} = 0.8 (Service class 1; Live load duration: Medium term) [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, $f_{m,y,d} = f_{m,k}.k_{mod}.k_h.k_{sys}/\gamma_m$ = 24.0 x 0.80 x 1.00 x 1.1/1.30 = 16.25 N/mm^2

Design bending stress, $\sigma_{m,y,d}$ = 2.21 x 1000/298 = 7.42 N/mm^2 OK

Bending resistance = 16.25 x 298/1000 = 4.84 kNm

Shear

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

Design shear strength, $f_{v,d} = f_{v,k}.k_{mod}.k_{sys}/\gamma_m$ = 4.00 x 0.80 x 1.1/1.30 = 2.71 N/mm^2

Design shear stress, $\sigma_{v,y,d}$ = 2.33 x 1000 x (3/2)/(31.5 x 195) = 0.57 N/mm^2 OK

Shear resistance = 2.71 x 31.5 x 195 x (2/3)/1000 = 11.08 kN

Deflection

Final deflection limit = 0.003L = 11.40 mm
Final deflection, $u_{fin} = \Sigma u_{inst}(1 + \psi_2.k_{def})$ [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 x 1.52 x 10^6/((11,000/16) x 47 x 195) = 0.29 mm

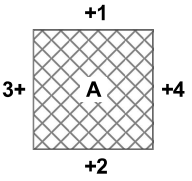
Final shear deflection is assumed to increase in proportion to total bending deflection

Mid-span deflections:	x 1e^8/EI	Inst. mm	k_{def}	ψ_2	Final mm	
G:	0.65	2.04	0.60	1.00	3.26	
QA:	1.63	5.10	0.60	0.30	6.02	
Shear deflection:		0.29			0.38	
Total mm.		7.43			**9.66**	OK

Calculations for timber post/stud to BS EN1995 using C24 timber

Location: [TP2]

Pos	Dur	Load	kN	Fact	Offset	M_{yy}	M_{zz}
				6.10		6.10	6.10
2	G	B/F Beam: [B24] [B25] : R1	10.30	13.91	0	-0.70	0.00
2	QH	B/F Beam: [B24] [B25] : R1	4.33	6.49	0	-0.32	0.00
Total load kN			14.63	**20.40**		**-1.02**	**0.00**



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

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

Member length = 2.4 m

Use: **100 x 100 C24**

Grade properties:

Loading duration factor, k_{mod} = 0.7 (Service class 1; live load duration: Long term) [EC5 Table 3.1]
Load sharing factor, k_{sys} = 1.0
Material partial factor, γ_m = 1.3 [EC3 NA Table NA.3]

Grade compression stress, $f_{c,0,k}$ = 21.0 N/mm^2
Design compressive strength, $f_{c,0,d} = k_{mod}.k_{sys}.f_{c,0,k}/\gamma_m$ = 0.7 x 1.0 x 21.0/1.3 = 11.31 N/mm^2
Modulus of elasticity, min, $E_{0,05}$ = 7,400 N/mm^2

Grade bending stress, $f_{m,k}$ = 24.0 N/mm^2
Design bending strength, $f_{m,d} = k_{mod}.k_{sys}.f_{m,k}/\gamma_m$ = 0.7 x 1.0 x 24.0/1.3 = 12.92 N/mm^2
Depth factors, $k_{h,y} = (150/100)^{0.2}$ = 1.084; $k_{h,z} = (150/100)^{0.2}$ = 1.084 [EC5 3.2]
Applying k_h depth factors $f_{m,y,d}$ = 1.084 x 12.92 = 14.01 N/mm^2 $f_{m,z,d}$ = 1.084 x 12.92 = 14.01 N/mm^2

Compression y-y axis

Design compressive stress, $\sigma_{c,0,d}$ = 20.40 x 1000/(100 x 100) = 2.04 N/mm^2
Effective length = L_{Ey} = 1.0L = 2.4 m
Slenderness, λ_y = 2.4 x 1000/(100/$\sqrt{}$12) = 83.1
$\lambda_{rel,y} = (\lambda_y/\pi)\sqrt{}(f_{c,O,k}/E_{0,05}))$ = (83.1/π)$\sqrt{}$(21.0/7,400) = 1.41 [EC5 6.3.2 Eq.(6.21)]
$k_v = 0.5(1+ B_c(\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2)$ = 0.5 x (1+ 0.2 x (1.41 - 0.3) + 1.41^2) = 1.60 [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)]
$k_{c,y} = 1/(k_y + \sqrt{}(k_y^2 - \lambda_{rel,y}^2))$ = 1/(1.60 + $\sqrt{}$(1.60^2 - 1.41^2)) = 0.422 [EC5 6.3.2 Eq.(6.25)]
Design compressive strength = $k_{c,y}.f_{c,0,d}$ = 0.422 x 11.31 = 4.77 N/mm^2
Design axial load resistance = 4.77 x 100 x 100/1000 = 47.7 kN ← OK

Compression z-z axis

Design compressive stress, $\sigma_{c,0,d}$ = 20.40 x 1000/(100 x 100) = 2.04 N/mm^2
Effective length = $L_{e,z}$ = 1.0L = 2.4 m
Slenderness, λ_z = 2.4 x 1000/(100/$\sqrt{}$12) = 83.1
$\lambda_{rel,z} = (\lambda_z/\pi)\sqrt{}(f_{c,O,k}/E_{0,05}))$ = (83.1/π)$\sqrt{}$(21.0/7,400) = 1.41 [EC5 6.3.2 Eq.(6.22)]
$k_z = 0.5(1+ B_c(\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2)$ = 0.5 x (1+ 0.2 x (1.41 - 0.3) + 1.41^2) = 1.60 [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)]
$k_{c,z} = 1/(k_z + \sqrt{}(k_z^2 - \lambda_{rel,z}^2))$ = 1/(1.60 + $\sqrt{}$(1.60^2 - 1.41^2)) = 0.422 [EC5 6.3.2 Eq.(6.26)]
Design compressive strength = $k_{c,z}.f_{c,0,d}$ = 0.422 x 11.31 = 4.77 N/mm^2
Design axial load resistance = 4.77 x 100 x 100/1000 = 47.7 kN ← OK

Bending about y-y axis:

Design bending strength, $f_{m,y,d}$ = 14.01 N/mm^2
Elastic section modulus, W_{yy} = 100 x 100^2/(6 x 1000) = 167 cm^4

Design bending moment = 1.02 kNm
Design bending stress, $\sigma_{m,y,d}$ = 1.02 x 1000/167 = 6.12 N/mm^2 OK
Moment resistance = 14.01 x 167/1000 = 2.34 kNm OK

Bending about z-z axis:

N/A

Combined compression and bending

Check $(\sigma_{c,0,d}/k_{c,y}.f_{c,0,d}) + (\sigma_{m.v.d}/f_{m,y,d}) + k_m(\sigma_{m,z,d}/f_{m,z,d}) <= 1$ [EC5 6.2.4 Eq.(6.23)]

and $(\sigma_{c,0,d}/k_{c,z}.f_{c,0,d}) + k_m(\sigma_{m,y,d}/f_{m,y,d}) + (\sigma_{m,z,d}/f_{m,z,d}) <= 1$ [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, $\sigma_{c,0,d}/k_{c,y}.f_{c,0,d}$:	2.04/4.77 = 0.428
	Buckling z-axis, $\sigma_{c,0,d}/k_{c,z}.f_{c,0,d}$:	2.04/4.77 = 0.428
	Bending y-axis, $\sigma_{m,y,d}/f_{m,y,d}$:	6.12/14.01 = 0.437
	Bending z-axis,$\sigma_{m,z,d}/f_{m,z,d}$:	0.00/14.01 = 0.000

Check 1: 0.428 + 0.437 + 0.7 x 0.000 = 0.864 OK [EC5 Eq.(6.23)]

Check 2: 0.428 + 0.7 x 0.437 + 0.000 = 0.733 OK [EC5 Eq.(6.24)]