ALPHA
STRUCTURAL ENGINEERS

STRUCTURAL REPORT

Project No	24460
Description	NEW OUTBUILDING GARDEN SHED
Client Name	
Address	94 Temple Sheen Road London SW14 7RR
Date	12/2024
Prepared By	KE

1. Introduction

-THIS CALCULATION IS BASED UPON DRAWINGS PREPARED BY OTHER. DIMENSIONS USED HAVE BEEN SCALED FORM SUCH DRAWINGS.

-ALL DIMENSIONS TO BE RECONFIRMED ON SITE BY BUILDER AND ANY DISCREPANCIES SHALL BE NOTIFIED TO THE SURVEYOR/ENGINEER.

-THE CONTRACTOR IS ULTIMATELY RESPONSIBLE FOR ENSURING THE STABILITY AND SAFETY OF THE STRUCTURAL OPERATIONS, AND THERE FOR SHALL UNDERTAKE EVERY EFFORT NOT TO CAUSE DISTRESS TO THE STRUCTURAL FABRIC OF THE BUILDING.

-CONTRACTOR TO TAKE ALL NECESSARY PRECAUTIONS TO PROP AND BRACE DURING CONSTRUCTION. HEALTH & SAFETY CONSULTANT SHOULD BE EMPLOYED.

-CONTRACTOR TO ISSUE METHOD STATEMENT (PRIOR TO COMMENCEMENT OF ALL WORK) FOR APPROVAL.

-IT IS THE CLIENT/ BUILDING OWNER RESPONSIBILITY TO VERIFY AND AGREE ADJOINING OWNER OF BUILDING, LINE OF BOUNDARY, OR OBTAIN PARTY WALL AGREEMENT.

-ANY DOUBLED-UP JOISTS (AROUND SKYLIGHT, STAIRCASE AND UNDER PARTITION, OR DESIGNED AS DOUBLED JOISTS) TO BE BOLTED TOGETHER BY M12 BOLTS EVERY 500MM -50MM ABOVE AND BELOW LONGITUDINAL CENTER

-IT IS THE RESPONSIBILITY OF THE CONTRACT TO INFORM THE DISTRICT SURVEYOR OF EACH STAGE OF WORKS AND TO ENSURE INSPECTIONS ARE MADE.

-ANY WORKS THAT COMMENCE BEFORE THE BUILDING REGULATION HAVE BEEN APPROVED IS THE RESPONSIBILITY OF THE CLIENT

-ALL TEMPORARY PROPPING AND SHORING OF NEIGHBORING BUILDING TO CONTRACTOR DETAILS AND APPROVAL OF LOCAL AUTHORITY

- IT IS THE CLIENT RESPONSIBILTY TO SEEK AN ENGINNER/ OR SURVEYOR CONSOLTATIONS FOR THE POSIBILTY OF REMOVING ANY STRUCTRAL ELEMENTS BEFORE THE DESIGN IS CARRIED OUT.

-ANY DOUBLED BEAMS (IF EXISTED) TO BE BOLTED TOGETHER WITH TUBE SPACERS OR SUITABLE CONNECTION AT MAX 1.5MC/S. USE M16 BOLTS

-STEEL BEAMS AND COLUMNS TO BE PROTECTED FROM FIRE. MINIMUM 1HOURE FIR RESISTANCE IS REQUIRED. 2 COATS OF INTUMESCENT PAINT OR DOUBLE LAYER OF PLASTER BOARD TO BE PROVIDED

-DIMENSION OF STRUCTURAL OPENING TO BE CONFIRMED BEFORE ANY ORDERS ARE PLACE FOR

WINDOWS/DOORS

-ANY DEVIATION FROM WHAT IS SHOWN ON THIS CALCULATION AND RELEVENT DRAWINGS, CONTRACTOR TO CONSULT THE STRUCTURALENGINEER.

- THE ENGINEE IS NOT RESPONSIBLE FOR ANY EXISTING CRACK, DAMAGES OR STRESS THAT ARE NOT SHOWN IN THE DRAWINGS.

2. Loading

LOADING TABLE

DEAD LOADS

SUSPENDED TIMBER JOIST FLOOR	
TIMBER JOISTS	0.20 kN/m2
TIMBER BOARDS	0.15 kN/m2
PLASTERBOARD CEILING (2 LAYERS)	0.30 kN/m2
SERVICES	0.05 kN/m2
INSULATION	0.10 kN/m2
	0.80 kN/m2

SUSPENDED TIMBER JOIST FLAT ROOF	
TIMBER JOISTS	0.20 kN/m2
TIMBER BOARDS	0.15 kN/m2
PLASTERBOARD CEILING (2 LAYERS)	0.30 kN/m2
SERVICES	0.05 kN/m2
INSULATION	0.10 kN/m2
WATERPROOFING	0.50 kN/m2
	1.30 kN/m2

TIMBER PITCHED ROOF	
TIMBER RAFTERS	0.20 kN/m2
TIMBER BOARDS	0.15 kN/m2
PLASTERBOARD CEILING (2 LAYERS)	0.30 kN/m2
SERVICES	0.05 kN/m2
INSULATION	0.10 kN/m2
TILES, BATTENS AND FELT	0.70 kN/m2
	1.50 kN/m2

BRICK/BLOCK CAVITY MASONRY WALL	
100mm BLOCKWORK (1450kg/m3)	1.50 kN/m2
102mm BRICKWORK (2000kg/m)	2.20 kN/m2
PLASTERBOARD & SKIM (13mm ONE SIDE)	0.12 kN/m2
INSULATION & SERVICES	0.15 kN/m2
	3.97 kN/m2

TIMBER STUD WALL (INTERNAL)	
TIMBER STUDS	0.15 kN/m2
PLASTERBOARD (2 LAYERS EACH FACE)	0.55 kN/m2
SERVICES	0.05 kN/m2
INSULATION	0.10 kN/m2
	0.85 kN/m2

TIMBER STUD WALL (EXTERNAL)	
TIMBER STUDS	0.20 kN/m2
TIMBER BOARDS	0.15 kN/m2
PLASTERBOARD (2 LAYERS)	0.30 kN/m2
SERVICES	0.05 kN/m2
INSULATION	0.10 kN/m2
TILES, BATTENS AND FELT	0.70 kN/m2
	1.50 kN/m2

100mm BRICK MASONRY WALL	
100mm BRICKWORK (2000kg/m)	2.20 kN/m2
PLASTERBOARD & SKIM (13mm ONE SIDE)	0.12 kN/m2
INSULATION & SERVICES	0.15 kN/m2
	2.47 kN/m2

215mm BRICK MASONRY WALL	
215mm BRICKWORK (2000kg/m)	4.30 kN/m2
PLASTERBOARD & SKIM (13mm ONE SIDE)	0.12 kN/m2
INSULATION & SERVICES	0.15 kN/m2
	4.57 kN/m2

LIVE LOADS

OCCUPANCY - DOMESTIC/RESIDENTIAL	1.50 kN/m2
SNOW LOAD ON FLAT ROOF	0.60 kN/m2
SNOW LOAD ON PITCHED ROOF	0.60 kN/m2

EuroBeam Project Summary

Site address: 94 Temple Sheen Road London SW14 7RR
Job: New Outbuilding
Client:
Job number: 24460

ITEMS:

1: Beam: Flat Roof Joists
Span: 3.5 m.
Reactions (unfactored/factored): R1: 1.29/1.81 kN; R2: 1.29/1.81 kN
Use 50 x 175 C24

2: Beam: RB1 - Steel Beam
Span: 5.2 m.
Reactions (unfactored/factored): R1: 16.02/22.46 kN; R2: 16.02/22.46 kN
Use 152 x 152 x 37 UKC S355

3: Beam: RB2 - Steel Beam
Span: 4.8 m.
Reactions (unfactored/factored): R1: 11.61/16.26 kN; R2: 5.61/7.83 kN
Use 203 x 133 x 25 UKB S355
Bearing R1: 250 x 100 mm padstone
Bearing R2: 134 x 100 mm padstone

4: Beam: FLOOR JOISTS
Span: 1.7 m.
Reactions (unfactored/factored): R1: 1.62/2.26 kN; R2: 1.62/2.26 kN
Use 50 x 150 C24

5: Beam: B1 - Steel Beam
Span: 2.9 m.
Reactions (unfactored/factored): R1: 20.05/27.58 kN; R2: 20.05/27.58 kN
Use 152 x 152 x 23 UKC S355
Bearing R1: 400 x 100 mm padstone
Bearing R2: as R1

6: Beam: B2 - Steel Beam
Span: 2.6 m.
Reactions (unfactored/factored): R1: 17.98/24.73 kN; R2: 17.98/24.73 kN
Use 152 x 152 x 23 UKC S355
Bearing R1: 400 x 100 mm padstone
Bearing R2: as R1

7: Beam: B3 - Steel Beam
Span: 2.6 m.
Reactions (unfactored/factored): R1: 6.05/8.66 kN; R2: 6.05/8.66 kN
Use 152 x 152 x 23 UKC S355
Bearing R1: 400 x 100 mm padstone
Bearing R2: as R1

8: Beam: B4 - Steel Beam
Span: 1.8 m.
Reactions (unfactored/factored): R1: 12.07/16.50 kN; R2: 12.07/16.50 kN
Use 152 x 152 x 23 UKC S355
Bearing R1: 400 x 100 mm padstone
Bearing R2: as R1

9: Beam: B5 - Steel Beam
Span: 1.8 m.
Reactions (unfactored/factored): R1: 0.98/1.41 kN; R2: 0.98/1.41 kN
Use 127 x 76 x 13 UKB S355
Bearing R1: 400 x 100 mm padstone
Bearing R2: as R1

Beam: Flat Roof Joists **Span: 3.5 m.**

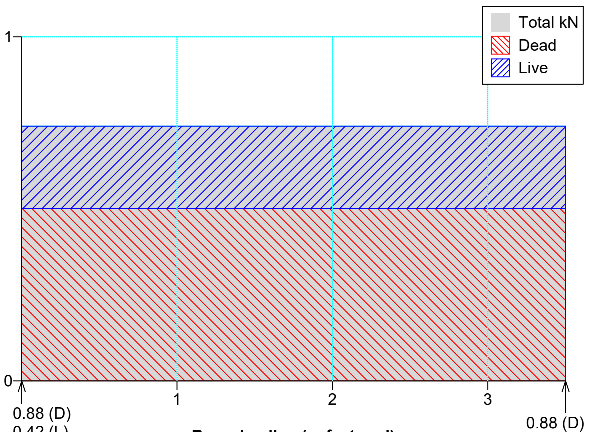
		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O	G	o.w.	0.1	0		L	0.17	0.17	0.20
U	G	Roof Dead	1.0*0.4	0		L	0.70	0.70	0.78
U	QA	Roof Live	0.6*0.4	0		L	0.42	0.42	0.47
				Total load (unfactored):		**2.59 kN**	**1.29**	**1.29**	1.45
			Dead/Permanent (unfactored):			1.75 kN	0.88	0.88	0.98
			Live/Variable (unfactored):			0.84 kN	0.42	0.42	0.47
			Factored (6.10):			3.62 kN	1.81	1.81	

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

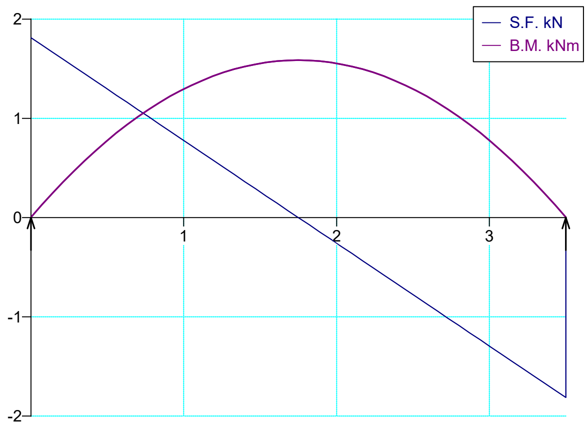
Maximum B.M. = 1.58 kNm (6.10) at 1.75 m. from R1

Maximum S.F. = 1.81 kN (6.10) at R1

Mid-span deflections: Dead: 0.98 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 0.47 x 10^8/EI
Total: 1.45 x 10^8/EI



Beam loading (unfactored)



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

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

Use 50 x 175 C24 *3.7 kg/m approx*

$W_{el,y}$ = 255.2 cm³ I_y = 2,233 cm⁴

Timber grade: C24

Grade bending strength, $f_{m,k}$ = 24.0 N/mm² [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k}$ = 4.0 N/mm² [BS EN 338: 2009 Table 1]

$E_{0.05}$ = 7,400 N/mm²; $E_{0,mean}$ = 11,000 N/mm² [BS EN 338: 2009 Table 1]

Material partial factor, γ_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]

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 0.60 x 1.00 x 1.1/1.30 = 12.18 N/mm²

Design bending stress, $\sigma_{m,y,d}$ = 1.58 x 1000/255 = 6.21 N/mm² OK

Bending resistance = 12.18 x 255/1000 = 3.11 kNm

Shear

Effective width for shear, $b_{ef} = k_{cr}.b$ = 0.67 x 50 = 33.5 mm. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k}.k_{mod}.k_{sys}/\gamma_m$ = 4.00 x 0.60 x 1.1/1.30 = 2.03 N/mm²

Design shear stress, $\sigma_{v,y,d}$ = 1.81 x 1000 x (3/2)/(33.5 x 175) = 0.46 N/mm² OK

Shear resistance = 2.03 x 33.5 x 175 x (2/3)/1000 = 7.94 kN

Deflection

Final deflection limit = 0.003L = 10.50 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)
$u_{inst,v} = 1.2 \times 1.13 \times 10^6/((11,000/16) \times 50 \times 175) = 0.23$ 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:	0.98	3.98	0.60	1.00	6.36	
QA:	0.47	1.91	0.60	0.30	2.25	
Shear deflection:		0.23			0.33	
Total mm.		6.11			**8.95**	OK

Beam: RB1 - Steel Beam **Span: 5.2 m.**

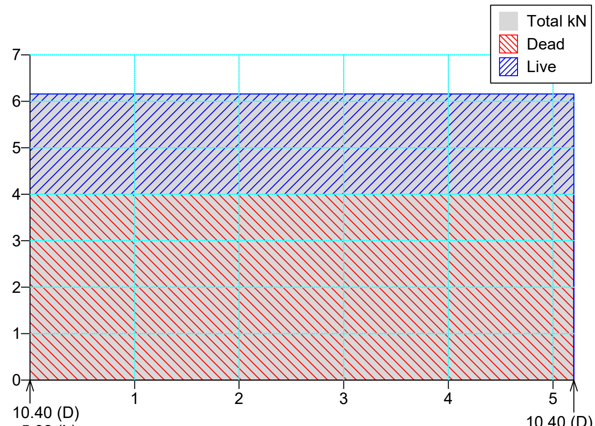
		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O	G	o.w.	0.4	0		L	1.04	1.04	3.8
U	G	Roof Dead	1.0*7.2*0.5	0		L	9.36	9.36	34.3
U	QA	Roof Live	0.6*7.2*0.5	0		L	5.62	5.62	20.6
				Total load (unfactored):		**32.03 kN**	**16.02**	**16.02**	58.6
			Dead/Permanent (unfactored):			20.80 kN	10.40	10.40	38.1
			Live/Variable (unfactored):			11.23 kN	5.62	5.62	20.6
			Factored (6.10):			44.93 kN	22.46	22.46	

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

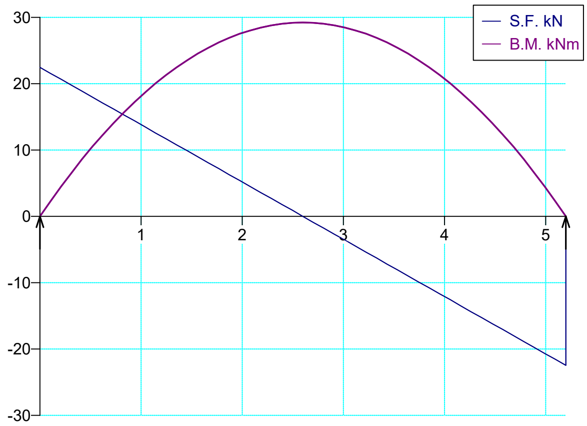
Maximum B.M. = 29.20 kNm (6.10) at 2.60 m. from R1

Maximum S.F. = 22.5 kN (6.10) at R1

Mid-span deflections: Dead: 38.1 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 20.6 x 10^8/EI
Total: 58.6 x 10^8/EI



Beam loading (unfactored)



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 37 UKC** S355

D=161.8 mm B=154.4 mm t=8.0 mm T=11.5 mm I_y=2,210 cm^4 i_z=3.87 cm $W_{pl,y}$=309 cm^3 $W_{el,y}$=273 cm^3

Classification: Flange: c/t = 65.6/11.5 = 5.70 <= 9ε (7.32): Class 1, plastic
EC3 Table 5.2 Web: c/t = 123.6/8.0 = 15.4 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 22.46 kN

Shear area, $A_v = A - 2bt_f + (t_w+2r)t_f$ = 47.1x100 - 2x154x11.5 + (8.00+2x7.60) x11.5 = 1,426 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd} = A_v.(f_y/\sqrt{3})/\gamma_{M0}$ = 1,426 x (355/√3)/(1.0 x 1000) = 292 kN (>=22.46) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 138.8/8.0 = 17.35 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 29.2 kNm

Moment resistance, $M_{c,y,Rd} = f_y.W_{pl,y}$ = 355 x 309/1000 = 110 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, $M_{b,Rd} = \chi_{LT,mod}.M_{c,Rd}$

$\chi_{LT,mod} = \chi_{LT}/f$ (but $<=1/\bar{\lambda}_{LT}^2$ 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) $k_c = 1/\sqrt{C_1}$ [NA2.18]

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

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

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

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

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

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

$W_y = 309.0\ cm^3$ $I_w = 0.040\ dm^6$ $I_T = 19.2\ cm^4$ $I_z = 706\ cm^4$ $G = 81{,}000N/mm^2$

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{\lambda}_z$	$\bar{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-5.20	5.20	29.2	1.00d	1.00	100.2	1.759	1.046	1.020	0.671	0.671	109.7	73.6	OK

C1 derivation: d: taken as 1.0 (conservative)

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 = 20.6 x 1e8/(210,000 x 2,210) = 4.4 mm (L/1174) OK
TL deflection = 58.6 x 1e8/(210,000 x 2,210) = 12.6 mm (L/412)

Beam: RB2 - Steel Beam **Span: 4.8 m.**

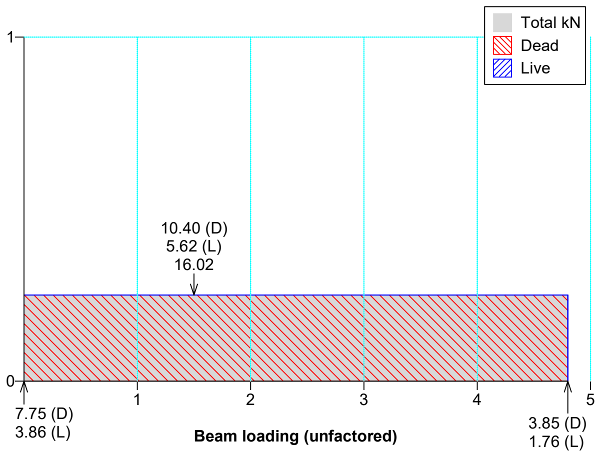
	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O G	o.w.	0.25	0		L	0.60	0.60	1.7
P G	Bm: B1 - Steel Beam : R1	10.40	1.5			7.15	3.25	19.5
P QA	Bm: B1 - Steel Beam : R1	5.62	1.5			3.86	1.76	10.6
			Total load (unfactored):	**17.22 kN**		**11.61**	**5.61**	31.8
			Dead/Permanent (unfactored):	11.60 kN		7.75	3.85	21.3
			Live/Variable (unfactored):	5.62 kN		3.86	1.76	10.6
			Factored (6.10):	24.09 kN		16.26	7.83	

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

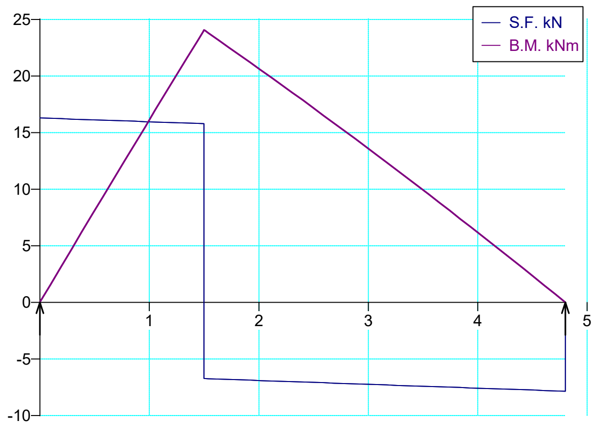
Maximum B.M. = 24.01 kNm (6.10) at 1.50 m. from R1

Maximum S.F. = 16.26 kN (6.10) at R1

Mid-span deflections: Dead: 21.3 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 10.6 x 10^8/EI
Total: 31.8 x 10^8/EI



Beam loading (unfactored)



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 25 UKB** S355

D=203.2 mm B=133.2 mm t=5.7 mm T=7.8 mm I_y=2,340 cm^4 i_z=3.10 cm $W_{pl,y}$=258 cm^3 $W_{el,y}$=230 cm^3

Classification: Flange: c/t = 56.1/7.8 = 7.20 <= 9ε (7.32): Class 1, plastic
EC3 Table 5.2 Web: c/t = 172.4/5.7 = 30.2 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 16.26 kN

Shear area, $A_v = A - 2bt_f + (t_w+2r)t_f$ = 32.0x100 - 2x133x7.80 + (5.70+2x7.60) x7.80 = 1,285 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd} = A_v.(f_y/\sqrt{3})/\gamma_{M0}$ = 1,285 x (355/√3)/(1.0 x 1000) = 263 kN (>=16.26) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 187.6/5.7 = 32.91 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 24.01 kNm

Moment resistance, $M_{c,y,Rd} = f_y.W_{pl,y}$ = 355 x 258/1000 = 91.6 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, $M_{b,Rd} = \chi_{LT,mod}.M_{c,Rd}$

$\chi_{LT,mod} = \chi_{LT}/f$ (but $<=1/\bar{\lambda}_{LT}^2$ 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) $k_c = 1/\sqrt{C_1}$ [NA2.18]

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

$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\Phi_{LT}^2-\beta\bar{\lambda}_{LT}^2)}]$ [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 = 258.0 cm^3 I_w = 0.029 dm^6 I_T = 5.96 cm^4 I_z = 308 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}	$\chi_{LT, mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-4.80	4.80	23.6	1.00d	1.00	45.4	2.026	1.420	1.429	0.463	0.463	91.6	42.4	OK

C1 derivation: d: taken as 1.0 (conservative)

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 = 10.6 x 1e8/(210,000 x 2,340) = 2.1 mm (L/2234) OK
TL deflection = 31.8 x 1e8/(210,000 x 2,340) = 6.5 mm (L/741)

Bearings

203 x 133 x 25 UB stiff bearing length, b_1 = t + 1.6r + 2T = 33.5 mm

Design masonry strength = 0.700 N/mm^2 (User-entered value)

R1 (16.26 kN) : 250 x 100 mm padstone

Stress under padstone = 16.26 x 1000/250 x 100 = 0.65 N/mm^2 OK

Padstone height is less than projection (108mm): reinforcement required - not checked

R2 (7.83 kN) : 134 x 100 mm padstone

Stress under padstone = 7.83 x 1000/134 x 100 = 0.58 N/mm^2 OK

Padstone height is less than projection (50mm): reinforcement required - not checked

Beam: FLOOR JOISTS **Span: 1.7 m.**

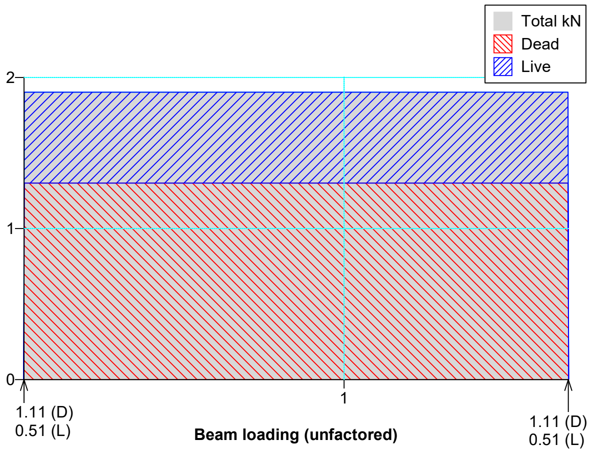
		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O	G	o.w.	0.1	0		L	0.09	0.09	0.01
U	G	FLOOR DEAD	0.4*0.5	0		L	0.17	0.17	0.02
U	G	PARTITION	1	0		L	0.85	0.85	0.11
U	QA	FLOOR LIVE	0.4*1.5	0		L	0.51	0.51	0.07
				Total load (unfactored):		**3.23 kN**	**1.62**	**1.62**	0.21
				Dead/Permanent (unfactored):		2.21 kN	1.11	1.11	0.14
				Live/Variable (unfactored):		1.02 kN	0.51	0.51	0.07
				Factored (6.10):		4.51 kN	2.26	2.26	

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

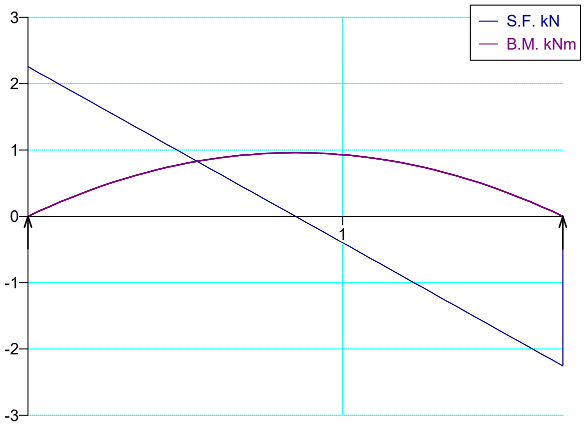
Maximum B.M. = 0.959 kNm (6.10) at 0.85 m. from R1

Maximum S.F. = 2.26 kN (6.10) at R1

Mid-span deflections: Dead: $0.14 \times 10^8/EI$ *(E in N/mm², I in cm⁴)*
Live: $0.07 \times 10^8/EI$
Total: $0.21 \times 10^8/EI$



Beam loading (unfactored)



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

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

Use 50 x 150 C24 *3.2 kg/m approx*

$W_{el,y}$ = 187.5 cm³ I_y = 1,406 cm⁴

Timber grade: C24

Grade bending strength, $f_{m,k}$ = 24.0 N/mm² [BS EN 338: 2009 Table 1]

Grade shear strength, $f_{v,k}$ = 4.0 N/mm² [BS EN 338: 2009 Table 1]

$E_{0.05}$ = 7,400 N/mm²; $E_{0,mean}$ = 11,000 N/mm² [BS EN 338: 2009 Table 1]

Material partial factor, γ_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]

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 0.60 x 1.00 x 1.1/1.30 = 12.18 N/mm²

Design bending stress, $\sigma_{m,y,d}$ = 0.96 x 1000/188 = 5.12 N/mm² OK

Bending resistance = 12.18 x 188/1000 = 2.28 kNm

Shear

Effective width for shear, $b_{ef} = k_{cr}.b$ = 0.67 x 50 = 33.5 mm. [A1:2008 (6.13a)]

Design shear strength, $f_{v,d} = f_{v,k}.k_{mod}.k_{sys}/\gamma_m$ = 4.00 x 0.60 x 1.1/1.30 = 2.03 N/mm²

Design shear stress, $\sigma_{v,y,d}$ = 2.26 x 1000 x (3/2)/(33.5 x 150) = 0.67 N/mm² OK

Shear resistance = 2.03 x 33.5 x 150 x (2/3)/1000 = 6.80 kN

Deflection

Final deflection limit = L/250 = 6.80 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)
$u_{inst,v} = 1.2 \times 0.69 \times 10^6/((11{,}000/16) \times 50 \times 150) = 0.16$ 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:	0.14	0.91	0.60	1.00	1.46	
QA:	0.07	0.42	0.60	0.30	0.50	
Shear deflection:		0.16			0.23	
Total mm.		1.50			**2.19**	OK

Beam: B1 - Steel Beam **Span: 2.9 m.**

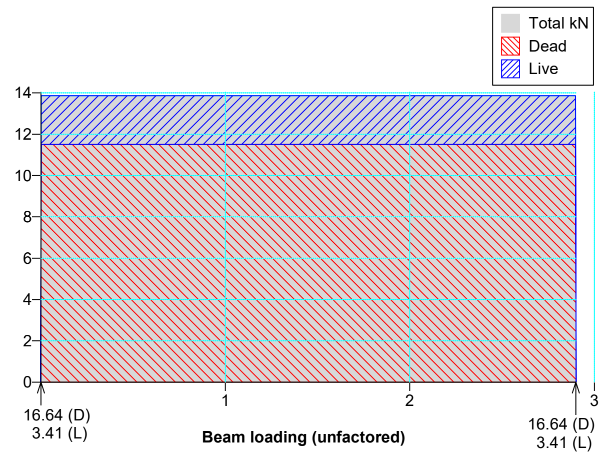
		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O	G	o.w.	0.25	0		L	0.36	0.36	0.23
U	G	ROOF DEAD	1.8*1	0		L	2.61	2.61	1.66
U	QA	ROOF LIVE	1.8*0.6	0		L	1.57	1.57	0.99
U	G	WALL	3.6*2.5	0		L	13.05	13.05	8.29
U	G	FLOOR DEAD	0.85*0.5	0		L	0.62	0.62	0.39
U	QA	FLOOR LIVE	0.85*1.5	0		L	1.85	1.85	1.17
				Total load (unfactored):	**40.1 kN**		**20.05**	**20.05**	12.74
			Dead/Permanent (unfactored):		33.3 kN		16.64	16.64	10.57
			Live/Variable (unfactored):		6.8 kN		3.41	3.41	2.17
			Factored (6.10):		55.2 kN		27.58	27.58	

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

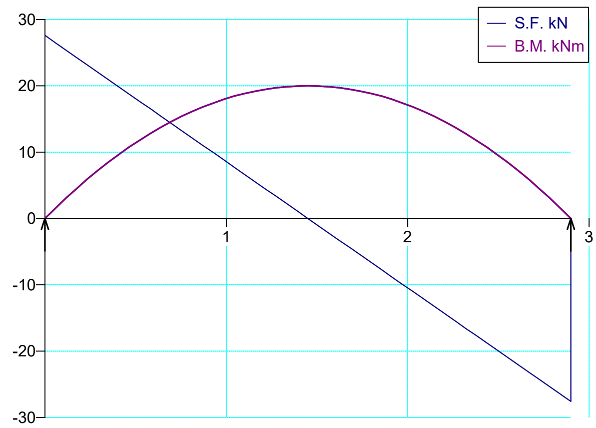
Maximum B.M. = 20.00 kNm (6.10) at 1.45 m. from R1

Maximum S.F. = 27.58 kN (6.10) at R1

Mid-span deflections: Dead: $10.57 \times 10^8/EI$ *(E in N/mm², I in cm⁴)*
Live: $2.17 \times 10^8/EI$
Total: $12.74 \times 10^8/EI$



Beam loading (unfactored)



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 23 UKC** S355

D=152.4 mm B=152.2 mm t=5.8 mm T=6.8 mm I_y=1,250 cm⁴ i_z=3.70 cm $W_{pl,y}$=182 cm³ $W_{el,y}$=164 cm³

Classification: Flange: c/t = 65.6/6.8 = 9.65 <= 14ε (11.39): Class 3, semi-compact
EC3 Table 5.2 Web: c/t = 123.6/5.8 = 21.3 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 27.6 kN

Shear area, $A_v = A - 2bt_f + (t_w+2r)t_f$ = 29.2x100 - 2x152x6.80 + (5.80+2x7.60) x6.80 = 993 mm² [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd} = A_v.(f_y/\sqrt{3})/\gamma_{M0}$ = 993 x (355/√3)/(1.0 x 1000) = 204 kN (>=27.6) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 138.8/5.8 = 23.93 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 20.00 kNm

Moment resistance, $M_{c,y,Rd} = f_y.W_{el,y}$ = 355 x 164/1000 = 58.2 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, $M_{b,Rd} = \chi_{LT,mod}.M_{c,Rd}$

$\chi_{LT,mod} = \chi_{LT}/f$ (but $<=1/\bar{\lambda}_{LT}^2$ 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) $k_c = 1/\sqrt{C_1}$ [NA2.18]

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

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

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

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

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

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

$W_y = 164.0\ cm^3$ $I_w = 0.021\ dm^6$ $I_T = 4.63\ cm^4$ $I_z = 400\ cm^4$ $G = 81{,}000 N/mm^2$

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{\lambda}_z$	$\bar{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-2.90	2.90	20.0	1.00d	1.00	93.7	1.026	0.788	0.799	0.824	0.824	58.2	48.0	OK

C1 derivation: d: taken as 1.0 (conservative)

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 = 2.17 x 1e8/(210,000 x 1,250) = 0.8 mm (L/3508) OK
TL deflection = 12.74 x 1e8/(210,000 x 1,250) = 4.9 mm (L/598)

Bearings

152 x 152 x 23 UC stiff bearing length, $b_1 = t + 1.6r + 2T = 31.6$ mm

Design masonry strength = 0.700 N/mm^2 (User-entered value)

R1 (27.6 kN) : 400 x 100 mm padstone

Stress under padstone = 27.6 x 1000/400 x 100 = 0.69 N/mm^2 OK

Padstone height is less than projection (184mm): reinforcement required - not checked

R2 as R1

Beam: B2 - Steel Beam **Span: 2.6 m.**

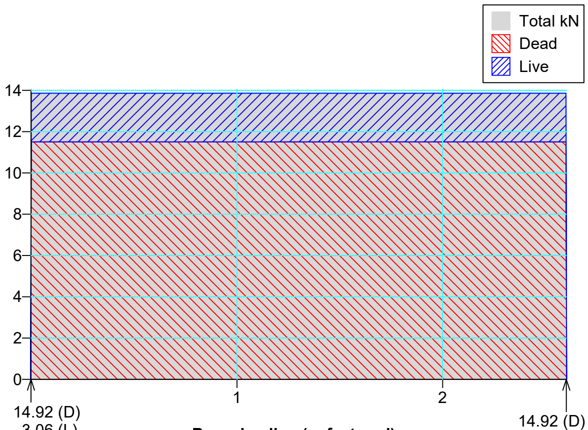
		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O	G	o.w.	0.25	0		L	0.32	0.32	0.15
U	G	ROOF DEAD	1.8*1	0		L	2.34	2.34	1.07
U	QA	ROOF LIVE	1.8*0.6	0		L	1.40	1.40	0.64
U	G	WALL	3.6*2.5	0		L	11.70	11.70	5.36
U	G	FLOOR DEAD	0.85*0.5	0		L	0.55	0.55	0.25
U	QA	FLOOR LIVE	0.85*1.5	0		L	1.66	1.66	0.76
				Total load (unfactored):		**35.96 kN**	**17.98**	**17.98**	8.23
				Dead/Permanent (unfactored):		29.83 kN	14.92	14.92	6.83
				Live/Variable (unfactored):		6.12 kN	3.06	3.06	1.40
				Factored (6.10):		49.46 kN	24.73	24.73	

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

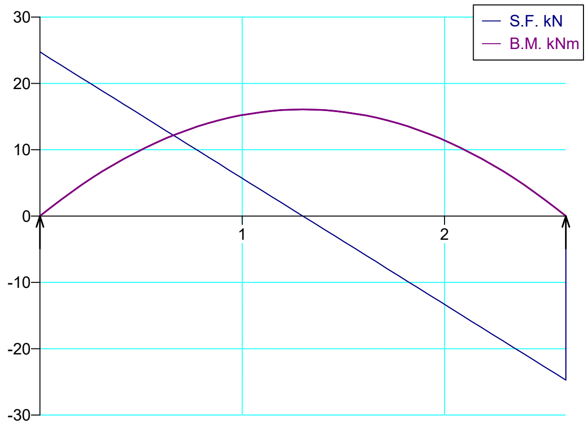
Maximum B.M. = 16.08 kNm (6.10) at 1.30 m. from R1

Maximum S.F. = 24.73 kN (6.10) at R1

Mid-span deflections: Dead: 6.83 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 1.40 x 10^8/EI
Total: 8.23 x 10^8/EI



Beam loading (unfactored)



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 23 UKC** S355

D=152.4 mm B=152.2 mm t=5.8 mm T=6.8 mm I_y=1,250 cm⁴ i_z=3.70 cm $W_{pl,y}$=182 cm³ $W_{el,y}$=164 cm³

Classification: Flange: c/t = 65.6/6.8 = 9.65 <= 14ε (11.39): Class 3, semi-compact
EC3 Table 5.2 Web: c/t = 123.6/5.8 = 21.3 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 24.73 kN

Shear area, $A_v = A - 2bt_f + (t_w+2r)t_f$ = 29.2x100 - 2x152x6.80 + (5.80+2x7.60) x6.80 = 993 mm² [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd} = A_v.(f_y/\sqrt{3})/\gamma_{M0}$ = 993 x (355/√3)/(1.0 x 1000) = 204 kN (>=24.73) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 138.8/5.8 = 23.93 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 16.08 kNm

Moment resistance, $M_{c,y,Rd} = f_y.W_{el,y}$ = 355 x 164/1000 = 58.2 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, $M_{b,Rd} = \chi_{LT,mod}.M_{c,Rd}$

$\chi_{LT,mod} = \chi_{LT}/f$ (but $<=1/\bar{\lambda}_{LT}^2$ 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) $k_c = 1/\sqrt{C_1}$ [NA2.18]

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

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

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

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

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

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

$W_y = 164.0$ cm^3 $I_w = 0.021$ dm^6 $I_T = 4.63$ cm^4 $I_z = 400$ cm^4 G = 81,000N/mm^2

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{\lambda}_z$	$\bar{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-2.60	2.60	16.1	1.00d	1.00	111.7	0.920	0.722	0.750	0.859	0.859	58.2	50.0	OK

C1 derivation: d: taken as 1.0 (conservative)

Combined bending and shear

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

Web buckling and crushing have not been checked

Deflection

LL deflection = 1.40 x 1e8/(210,000 x 1,250) = 0.5 mm (L/4875) OK
TL deflection = 8.23 x 1e8/(210,000 x 1,250) = 3.1 mm (L/829)

Bearings

152 x 152 x 23 UC stiff bearing length, $b_1 = t + 1.6r + 2T = 31.6$ mm

Design masonry strength = 0.700 N/mm^2 (User-entered value)

R1 (24.73 kN) : 400 x 100 mm padstone

Stress under padstone = 24.73 x 1000/400 x 100 = 0.62 N/mm^2 OK

Padstone height is less than projection (184mm): reinforcement required - not checked

R2 as R1

Beam: B3 - Steel Beam **Span: 2.6 m.**

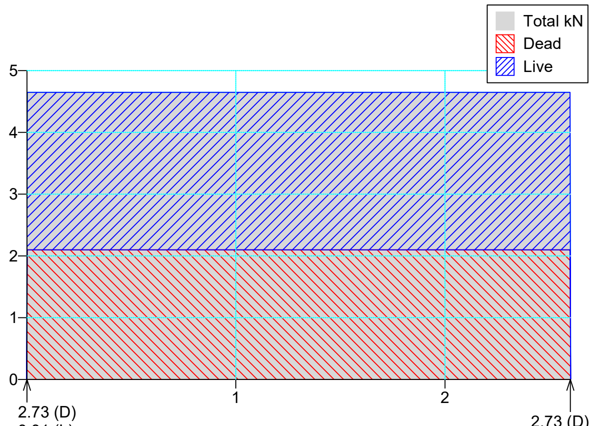
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.25	0		L	0.32	0.32	0.15
U G	PARTITION	1	0		L	1.30	1.30	0.60
U G	FLOOR DEAD	1.7*0.5	0		L	1.11	1.11	0.51
U QA	FLOOR LIVE	1.7*1.5	0		L	3.31	3.31	1.52
			Total load (unfactored):		**12.09 kN**	**6.05**	**6.05**	2.77
			Dead/Permanent (unfactored):		5.46 kN	2.73	2.73	1.25
			Live/Variable (unfactored):		6.63 kN	3.31	3.31	1.52
			Factored (6.10):		17.32 kN	8.66	8.66	

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

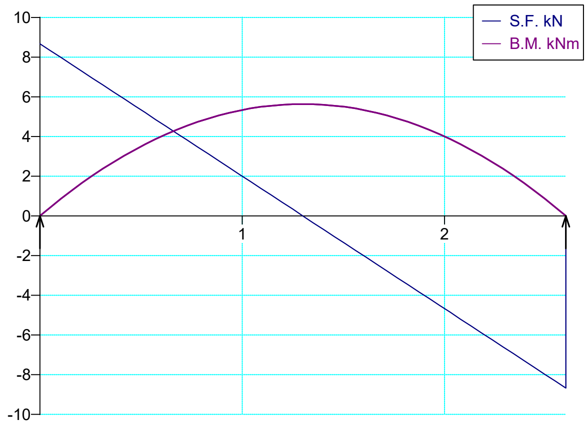
Maximum B.M. = 5.63 kNm (6.10) at 1.30 m. from R1

Maximum S.F. = 8.66 kN (6.10) at R1

Mid-span deflections: Dead: $1.25 \times 10^8/EI$ *(E in N/mm², I in cm⁴)*
Live: $1.52 \times 10^8/EI$
Total: $2.77 \times 10^8/EI$



Beam loading (unfactored)



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 23 UKC** S355

D=152.4 mm B=152.2 mm t=5.8 mm T=6.8 mm I_y=1,250 cm⁴ i_z=3.70 cm $W_{pl,y}$=182 cm³ $W_{el,y}$=164 cm³

Classification: Flange: c/t = 65.6/6.8 = 9.65 <= 14ε (11.39): Class 3, semi-compact
EC3 Table 5.2 Web: c/t = 123.6/5.8 = 21.3 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 8.66 kN

Shear area, $A_v = A - 2bt_f + (t_w+2r)t_f$ = 29.2x100 - 2x152x6.80 + (5.80+2x7.60) x6.80 = 993 mm² [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd} = A_v.(f_y/\sqrt{3})/\gamma_{M0}$ = 993 x (355/√3)/(1.0 x 1000) = 204 kN (>=8.66) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 138.8/5.8 = 23.93 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 5.63 kNm

Moment resistance, $M_{c,y,Rd} = f_y.W_{el,y}$ = 355 x 164/1000 = 58.2 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, $M_{b,Rd} = \chi_{LT,mod}.M_{c,Rd}$

$\chi_{LT,mod} = \chi_{LT}/f$ (but $<=1/\bar{\lambda}_{LT}^2$ 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) $k_c = 1/\sqrt{C_1}$ [NA2.18]

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

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

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

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

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

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

$W_y = 164.0$ cm^3 $I_w = 0.021$ dm^6 $I_T = 4.63$ cm^4 $I_z = 400$ cm^4 G = 81,000N/mm^2

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{\lambda}_z$	$\bar{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT, mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-2.60	2.60	5.6	1.00d	1.00	111.7	0.920	0.722	0.750	0.859	0.859	58.2	50.0	OK

C1 derivation: d: taken as 1.0 (conservative)

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 = 1.52 x 1e8/(210,000 x 1,250) = 0.6 mm (L/4490) OK
TL deflection = 2.77 x 1e8/(210,000 x 1,250) = 1.1 mm (L/2464)

Bearings

152 x 152 x 23 UC stiff bearing length, b_1 = t + 1.6r + 2T = 31.6 mm

Design masonry strength = 0.700 N/mm^2 (User-entered value)

R1 (8.66 kN) : 400 x 100 mm padstone

Stress under padstone = 8.66 x 1000/400 x 100 = 0.22 N/mm^2 OK

Padstone height is less than projection (184mm): reinforcement required - not checked

R2 as R1

Beam: B4 - Steel Beam **Span: 1.8 m.**

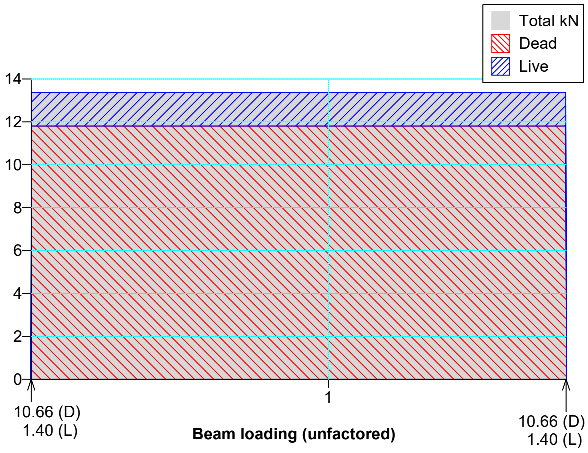
		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O	G	o.w.	0.25	0		L	0.22	0.22	0.03
U	G	ROOF DEAD	2.6*1	0		L	2.34	2.34	0.36
U	QA	ROOF LIVE	2.6*0.6	0		L	1.40	1.40	0.21
U	G	WALL	3.6*2.5	0		L	8.10	8.10	1.23
				Total load (unfactored):		**24.14 kN**	**12.07**	**12.07**	1.83
				Dead/Permanent (unfactored):		21.33 kN	10.66	10.66	1.62
				Live/Variable (unfactored):		2.81 kN	1.40	1.40	0.21
				Factored (6.10):		33.01 kN	16.50	16.50	

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

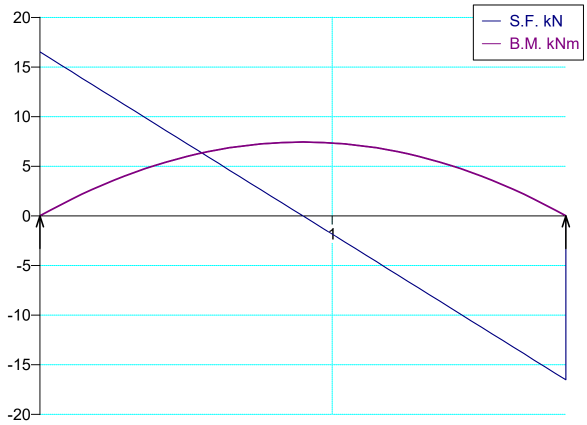
Maximum B.M. = 7.43 kNm (6.10) at 0.90 m. from R1

Maximum S.F. = 16.50 kN (6.10) at R1

Mid-span deflections: Dead: 1.62 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 0.21 x 10^8/EI
Total: 1.83 x 10^8/EI



Beam loading (unfactored)



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 23 UKC** S355

D=152.4 mm B=152.2 mm t=5.8 mm T=6.8 mm I_y=1,250 cm^4 i_z=3.70 cm $W_{pl,y}$=182 cm^3 $W_{el,y}$=164 cm^3

Classification: Flange: c/t = 65.6/6.8 = 9.65 <= 14ε (11.39): Class 3, semi-compact
EC3 Table 5.2 Web: c/t = 123.6/5.8 = 21.3 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 16.50 kN

Shear area, $A_v = A - 2bt_f + (t_w+2r)t_f$ = 29.2x100 - 2x152x6.80 + (5.80+2x7.60) x6.80 = 993 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd} = A_v.(f_y/\sqrt{3})/\gamma_{M0}$ = 993 x (355/√3)/(1.0 x 1000) = 204 kN (>=16.50) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 138.8/5.8 = 23.93 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 7.43 kNm

Moment resistance, $M_{c,y,Rd} = f_y.W_{el,y}$ = 355 x 164/1000 = 58.2 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, $M_{b,Rd} = \chi_{LT,mod}.M_{c,Rd}$

$\chi_{LT,mod} = \chi_{LT}/f$ (but $<=1/\bar{\lambda}_{LT}^2$ 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) $k_c = 1/\sqrt{C_1}$ [NA2.18]

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

$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta\bar{\lambda}_{LT}^2)}]$ [EC3 (6.57)]
$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta\bar{\lambda}_{LT}^2]$
$\bar{\lambda}_{LT,0} = 0.4$ $\beta = 0.75$ [EC3 UK NA 2.17]
$\bar{\lambda}_{LT} = \sqrt{(f_y.W_{pl,y}/M_{cr})}$
$M_{cr} = C_1(\pi^2 EI_z/L_e^2)\sqrt{(I_w/I_z + L_e^2 GI_T/\pi^2 EI_z)}$ [SN003: k and k_w taken as 1.0, conservative]
$W_y = 164.0\ cm^3$ $I_w = 0.021\ dm^6$ $I_T = 4.63\ cm^4$ $I_z = 400\ cm^4$ $G = 81{,}000 N/mm^2$

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{\lambda}_z$	$\bar{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT,\,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-1.80	1.80	7.4	1.00d	1.00	209.6	0.637	0.527	0.626	0.949	0.949	58.2	55.2	OK

C1 derivation: d: taken as 1.0 (conservative)

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 = 0.21 x 1e8/(210,000 x 1,250) = 0.1 mm OK
TL deflection = 1.83 x 1e8/(210,000 x 1,250) = 0.7 mm (L/2578)

Bearings

152 x 152 x 23 UC stiff bearing length, $b_1 = t + 1.6r + 2T = 31.6$ mm
Design masonry strength = 0.700 N/mm^2 (User-entered value)

R1 (16.50 kN) : 400 x 100 mm padstone

Stress under padstone = 16.50 x 1000/400 x 100 = 0.41 N/mm^2 OK

Padstone height is less than projection (184mm): reinforcement required - not checked

R2 as R1

Beam: B5 - Steel Beam **Span: 1.8 m.**

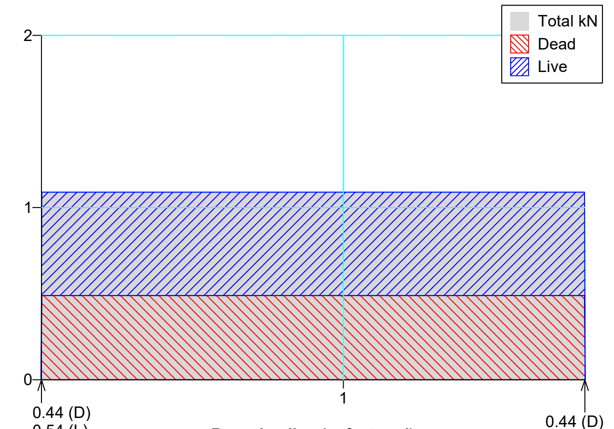
	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O G	o.w.	0.25	0		L	0.22	0.22	0.03
U G	FLOOR DEAD	0.4*0.6	0		L	0.22	0.22	0.03
U QA	FLOOR LIVE	1.5*0.4	0		L	0.54	0.54	0.08
				Total load (unfactored):	**1.96 kN**	**0.98**	**0.98**	0.15
				Dead/Permanent (unfactored):	0.88 kN	0.44	0.44	0.07
				Live/Variable (unfactored):	1.08 kN	0.54	0.54	0.08
				Factored (6.10):	2.81 kN	1.41	1.41	

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

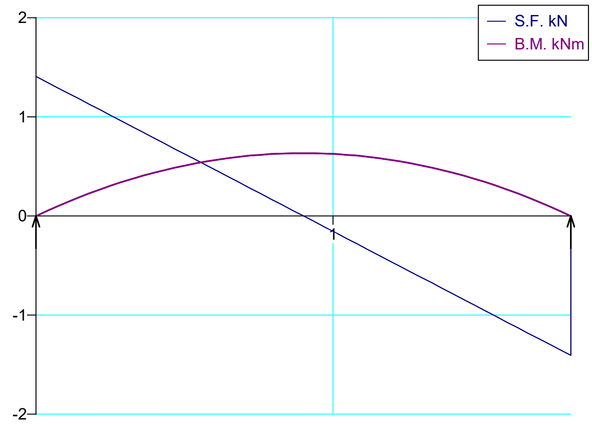
Maximum B.M. = 0.632 kNm (6.10) at 0.90 m. from R1

Maximum S.F. = 1.41 kN (6.10) at R1

Mid-span deflections: Dead: 0.07 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 0.08 x 10^8/EI
Total: 0.15 x 10^8/EI



Beam loading (unfactored)



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

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **127 x 76 x 13 UKB** S355

D=127.0 mm B=76.0 mm t=4.0 mm T=7.6 mm I_y=473 cm^4 i_z=1.84 cm $W_{pl,y}$=84.2 cm^3 $W_{el,y}$=74.6 cm^3

Classification: Flange: c/t = 28.4/7.6 = 3.74 <= 9ε (7.32): Class 1, plastic
EC3 Table 5.2 Web: c/t = 96.6/4.0 = 24.1 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 1.41 kN

Shear area, $A_v = A - 2bt_f + (t_w+2r)t_f$ = 16.5x100 - 2x76.0x7.60 + (4.00+2x7.60) x7.60 = 641 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd} = A_v.(f_y/\sqrt{3})/\gamma_{M0}$ = 641 x (355/$\sqrt{}$3)/(1.0 x 1000) = 131 kN (>=1.41) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 111.8/4.0 = 27.95 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 0.63 kNm

Moment resistance, $M_{c,y,Rd} = f_y.W_{pl,y}$ = 355 x 84.2/1000 = 29.9 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, $M_{b,Rd} = \chi_{LT,mod}.M_{c,Rd}$

$\chi_{LT,mod} = \chi_{LT}/f$ (but <=$1/\bar{\lambda}_{LT}^2$ 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) $k_c = 1/\sqrt{C_1}$ [NA2.18]

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

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

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

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

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

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

$W_y = 84.20\ cm^3$ $I_w = 0.002\ dm^6$ $I_T = 2.85\ cm^4$ $I_z = 55.7\ cm^4$ $G = 81{,}000 N/mm^2$

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{\lambda}_z$	$\bar{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT, mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-1.80	1.80	0.6	1.00d	1.00	35.7	1.280	0.915	0.901	0.751	0.751	29.9	22.5	OK

C1 derivation: d: taken as 1.0 (conservative)

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 = 0.08 x 1e8/(210,000 x 473) = 0.1 mm OK
TL deflection = 0.15 x 1e8/(210,000 x 473) = 0.1 mm

Bearings

127 x 76 x 13 UB stiff bearing length, b_1 = t + 1.6r + 2T = 31.4 mm

Design masonry strength = 0.700 N/mm^2 (User-entered value)

R1 (1.41 kN) : 400 x 100 mm padstone

Stress under padstone = 1.41 x 1000/400 x 100 = 0.04 N/mm^2 OK

Padstone height is less than projection (184mm): reinforcement required - not checked

R2 as R1