ALPHA
STRUCTURAL ENGINEERS

STRUCTURAL REPORT

Project No	25727
Description	REAR EXTENSION & LOFT CONVERSION & NEW OUTBUILDING
Client Name	
Address	30 Old London Road Badgers Mount Sevenoaks TN14 7AE
Date	01.2025
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: 30 Old London Road Badgers Mount Sevenoaks TN14 7AE
Job: Rear Extension & Loft Conversion & New Outbuilding
Client:
Job number: 25727

ITEMS:

1: Beam: Flat Roof Joists
Span: 3.4 m.
Reactions (unfactored/factored): R1: 1.26/1.76 kN; R2: 1.26/1.76 kN
Use 50 x 175 C24

2: Beam: Double Trimmers for New Skylight
Span: 3.0 m.
Reactions (unfactored/factored): R1: 1.86/2.57 kN; R2: 1.86/2.57 kN
Use 2no 50 x 150 C24

3: Beam: B1 - Steel Beam12
Span: 5.2 m.
Reactions (unfactored/factored): R1: 14.35/20.12 kN; R2: 14.35/20.12 kN
Use 152 x 152 x 37 UKC S355
Bearing R1: 500 x 100 mm padstone

4: Column: SC1 - Steel Column
Net moments: M_y = 3.02 kNm M_z = 0.00kNm
Section: 100 x 100 x 5.0 SHS Grade S355
Sum of stress ratios = 0.125 OK

5: Column: SC2 - Steel Column
Net moments: M_y = 3.02 kNm M_z = 0.00kNm
Section: 100 x 100 x 5.0 SHS Grade S355
Sum of stress ratios = 0.168 OK

6: Beam: Floor Joists
Span: 3.4 m.
Reactions (unfactored/factored): R1: 1.60/2.31 kN; R2: 1.60/2.31 kN
Use 50 x 175 C24

7: Beam: B2 - Steel Beam
Span: 3.4 m.
Reactions (unfactored/factored): R1: 8.91/12.16 kN; R2: 18.19/25.18 kN
Use 152 x 152 x 30 UKC S355

8: Beam: B3 - Steel Beam
Span: 5.7 m.
Reactions (unfactored/factored): R1: 26.60/38.02 kN; R2: 34.58/48.90 kN
Use 203 x 203 x 46 UKC S355
Bearing R1: 900 x 100 mm padstone
Bearing R2: 1150 x 100 mm padstone

9: Beam: B4 - Steel Beam
Span: 5.7 m.
Reactions (unfactored/factored): R1: 20.94/30.28 kN; R2: 37.21/52.81 kN
Use 203 x 203 x 46 UKC S355
Bearing R1: 700 x 100 mm padstone
Bearing R2: 1250 x 100 mm padstone

10: Beam: Nominal Beam for B3
Span: 1.0 m.
Reactions (unfactored/factored): R1: 26.41/37.73 kN; R2: 0.34/0.48 kN
Use 127 x 76 x 13 UKB S355
Bearing R1: 900 x 100 mm padstone
Bearing R2: 76 x 100 mm padstone

11: Beam: Nominal Beam for B4
Span: 1.8 m.
Reactions (unfactored/factored): R1: 21.98/31.25 kN; R2: 13.58/19.52 kN
Use 127 x 76 x 13 UKB S355
Bearing R1: 950 x 100 mm padstone
Bearing R2: 450 x 100 mm padstone

12: Beam: Flat Roof Joists - Extension
Span: 3.0 m.
Reactions (unfactored/factored): R1: 1.11/1.55 kN; R2: 1.11/1.55 kN
Use 50 x 150 C24

13: Beam: B5 - Steel Beam
Span: 4.5 m.
Reactions (unfactored/factored): R1: 49.45/68.32 kN; R2: 49.45/68.32 kN
Use 203 x 203 x 60 UKC S355

14: Beam: B6 - Steel Beam
Span: 4.6 m.
Reactions (unfactored/factored): R1: 10.12/13.97 kN; R2: 10.12/13.97 kN
Use 203 x 133 x 30 UKB S355
Bearing R1: 206 x 100 mm padstone
Bearing R2: 200 x 100 mm padstone

15: Column: SC3 & SC4 - Steel Columns
Net moments: M_y = 12.2 kNm M_z = 0.00kNm
Section: 152 x 152 x 30 UC Grade S235
Sum of stress ratios = 0.342 OK

16: Beam: Outbuilding Flat Roof
Span: 4.1 m.
Reactions (unfactored/factored): R1: 1.52/2.12 kN; R2: 1.52/2.12 kN
Use 50 x 200 C24

17: Beam: B7 - Steel Beam
Span: 4.8 m.
Reactions (unfactored/factored): R1: 8.59/12.04 kN; R2: 8.59/12.04 kN
Use 203 x 133 x 30 UKB S355
Bearing R1: 175 x 100 mm padstone
Bearing R2: as R1

18: Beam: GB1 - Ground Beam
Span: 4.0 m.
Reactions (unfactored/factored): R1: 62.86/86.15 kN; R2: 62.86/86.15 kN
Section: 600 x 500 mm RC, C25/30 concrete
Reinforcement: 3 No H16
H8 links at 200 mm max c/s

19: Beam: GB2 - Ground Beam
Span: 4.1 m.
Reactions (unfactored/factored): R1: 64.43/88.31 kN; R2: 64.43/88.31 kN
Section: 600 x 500 mm RC, C25/30 concrete
Reinforcement: 3 No H16
H8 links at 200 mm max c/s

20: Beam: GB3 - Ground Beam
Span: 2.4 m.
Reactions (unfactored/factored): R1: 21.48/29.00 kN; R2: 21.48/29.00 kN
Section: 600 x 500 mm RC, C25/30 concrete
Reinforcement: 3 No H16
H8 links at 200 mm max c/s

21: Beam: GB4 - Ground Beam
Span: 2.5 m.
Reactions (unfactored/factored): R1: 22.38/30.21 kN; R2: 22.38/30.21 kN
Section: 600 x 500 mm RC, C25/30 concrete
Reinforcement: 3 No H16
H8 links at 200 mm max c/s

22: Beam: GB5 - Ground Beam
Span: 4.1 m.
Reactions (unfactored/factored): R1: 64.43/88.31 kN; R2: 64.43/88.31 kN
Section: 600 x 500 mm RC, C25/30 concrete
Reinforcement: 3 No H16
H8 links at 200 mm max c/s

23: Beam: GB6 - Ground Beam
Span: 4.0 m.
Reactions (unfactored/factored): R1: 62.86/86.15 kN; R2: 62.86/86.15 kN
Section: 600 x 500 mm RC, C25/30 concrete
Reinforcement: 3 No H16
H8 links at 200 mm max c/s

24: Beam: GB7 - Ground Beam
Span: 2.5 m.
Reactions (unfactored/factored): R1: 22.38/30.21 kN; R2: 22.38/30.21 kN
Section: 600 x 500 mm RC, C25/30 concrete
Reinforcement: 3 No H16
H8 links at 200 mm max c/s

25: Beam: GB8 - Ground Beam
Span: 2.4 m.
Reactions (unfactored/factored): R1: 21.48/29.00 kN; R2: 21.48/29.00 kN
Section: 600 x 500 mm RC, C25/30 concrete
Reinforcement: 3 No H16
H8 links at 200 mm max c/s

Beam: **Flat Roof Joists** **Span: 3.4 m.**

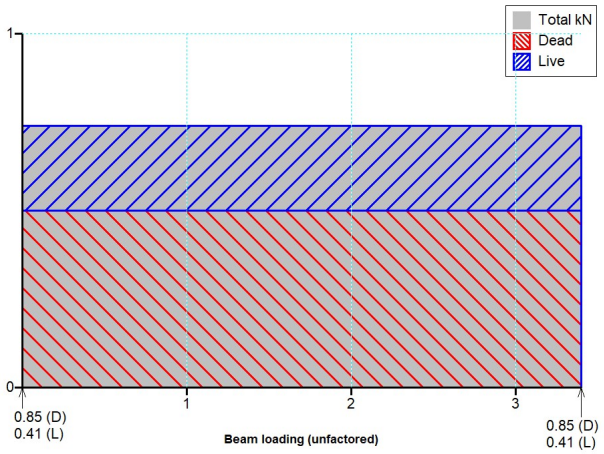
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O G	o.w.	0.1	0		L	0.17	0.17	0.17
U G	Roof Dead	1.0*0.4	0		L	0.68	0.68	0.70
U QA	Roof Live	0.6*0.4	0		L	0.41	0.41	0.42
				Total load (unfactored):	**2.52 kN**	**1.26**	**1.26**	1.29
				Dead/Permanent (unfactored):	1.70 kN	0.85	0.85	0.87
				Live/Variable (unfactored):	0.82 kN	0.41	0.41	0.42
				Factored (6.10):	3.52 kN	1.76	1.76	

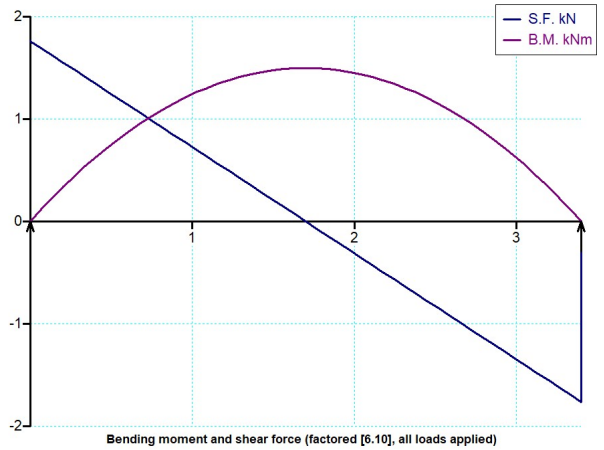
Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QA: Residential

Maximum B.M. = 1.50 kNm (6.10) at 1.70 m. from R1

Maximum S.F. = 1.76 kN (6.10) at R1

Mid-span deflections: Dead: 0.87 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 0.42 x 10^8/EI
Total: 1.29 x 10^8/EI





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 cm^3 I_y = 2,233 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, g_M = 1.3 [EC5 UK Table NA.3]

Loading modification factor, k_{mod} = 0.7 (Service class 1; Live load duration: Long 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, $f_{m,y,d} = f_{m,k}.k_{mod}.k_h.k_{sys}/g_m$ = 24.0 x 0.70 x 1.00 x 1.1/1.30 = 14.22 N/mm^2

Design bending stress, $s_{m,y,d}$ = 1.50 x 1000/255 = 5.86 N/mm^2 OK

Bending resistance = 14.22 x 255/1000 = 3.63 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, $f_{v,d} = f_{v,k}.k_{mod}.k_{sys}/g_m$ = 4.00 x 0.70 x 1.1/1.30 = 2.37 N/mm^2

Design shear stress, $s_{v,y,d}$ = 1.76 x 1000 x (3/2)/(33.5 x 175) = 0.45 N/mm^2 OK

Shear resistance = 2.37 x 33.5 x 175 x (2/3)/1000 = 9.26 kN

Deflection

Final deflection limit = 0.003L = 10.20 mm

Final deflection, $u_{fin} = Su_{inst}(1 + y_2.k_{def})$ [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 1.07 x 10^6/((11,000/16) x 50 x 175) = 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}	y_2	Final mm	
G:	0.87	3.54	0.60	1.00	5.67	
QA:	0.42	1.70	0.60	0.30	2.01	
Shear deflection:		0.21			0.31	
Total mm.		5.46			**7.99**	OK

Beam: **Double Trimmers for New Skylight** **Span: 3.0 m.**

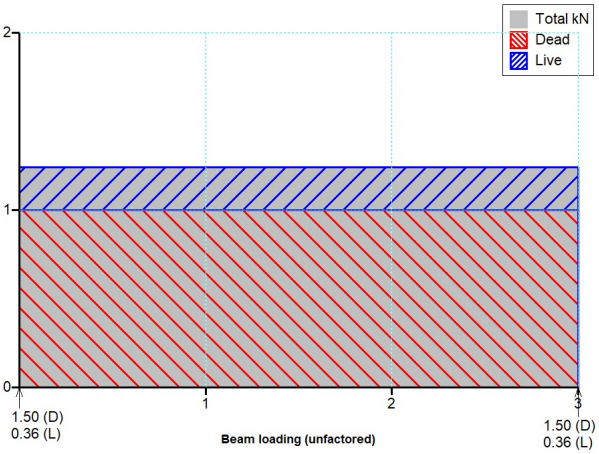
		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O	G	o.w.	0.1	0		L	0.15	0.15	0.11
U	G	Roof Dead	1.0*0.4	0		L	0.60	0.60	0.42
U	QA	Roof Live	0.6*0.4	0		L	0.36	0.36	0.25
U	G	Skylight	0.5	0		L	0.75	0.75	0.53
					Total load (unfactored):	**3.72 kN**	**1.86**	**1.86**	1.31
					Dead/Permanent (unfactored):	3.00 kN	1.50	1.50	1.05
					Live/Variable (unfactored):	0.72 kN	0.36	0.36	0.25
					Factored (6.10):	5.13 kN	2.57	2.57	

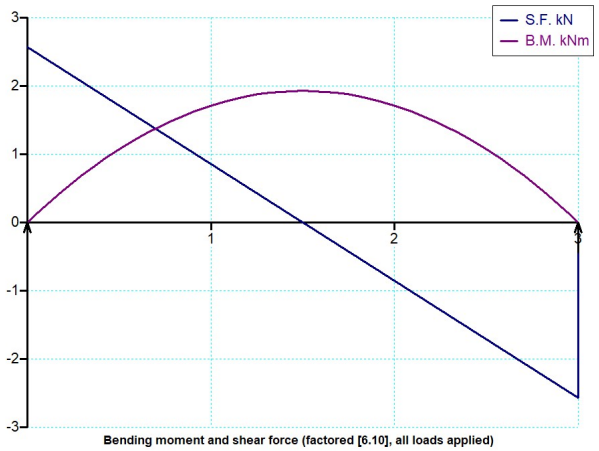
Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QA: Residential

Maximum B.M. = 1.92 kNm (6.10) at 1.50 m. from R1

Maximum S.F. = 2.57 kN (6.10) at R1

Mid-span deflections: Dead: 1.05 x 10^8/EI *(E in N/mm^2, I in cm^4)*
Live: 0.25 x 10^8/EI
Total: 1.31 x 10^8/EI





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

Use 2no 50 x 150 C24 *6.3 kg/m approx*

$W_{el,y}$ = 375.0 cm^3 I_y = 2,813 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, g_M = 1.3 [EC5 UK Table NA.3]

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

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

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

Bending

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

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

Design bending stress, $s_{m,y,d}$ = 1.92 x 1000/375 = 5.13 N/mm^2 OK

Bending resistance = 12.18 x 375/1000 = 4.57 kNm

Shear

Effective width for shear, $b_{ef} = k_{cr}.b$ = 0.67 x 100 = 67 mm. [A1:2008 (6.13a)]

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

Design shear stress, $s_{v,y,d}$ = 2.57 x 1000 x (3/2)/(67.0 x 150) = 0.38 N/mm^2 OK

Shear resistance = 2.03 x 67 x 150 x (2/3)/1000 = 13.61 kN

Deflection

Final deflection limit = 0.003L = 9.00 mm

Final deflection, $u_{fin} = Su_{inst}(1 + y_2.k_{def})$ [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 1.39 x 10^6/((11,000/16) x 100 x 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}	y_2	Final mm	
G:	1.05	3.41	0.60	1.00	5.45	
QA:	0.25	0.82	0.60	0.30	0.97	
Shear deflection:		0.16			0.25	
Total mm.		4.39			**6.67**	OK

Beam: **B1 - Steel Beam12** **Span: 5.2 m.**

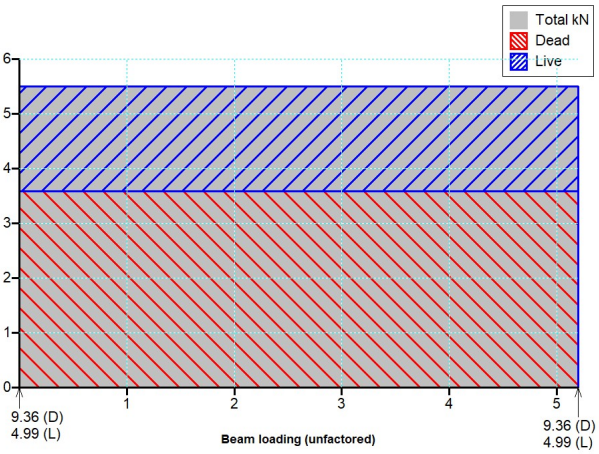
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.4	0		L	1.04	1.04	3.8
U G	Roof Dead	1.0*6.4*0.5	0		L	8.32	8.32	30.5
U QA	Roof Live	0.6*6.4*0.5	0		L	4.99	4.99	18.3
				Total load (unfactored):	**28.70 kN**	**14.35**	**14.35**	52.6
				Dead/Permanent (unfactored):	18.72 kN	9.36	9.36	34.3
				Live/Variable (unfactored):	9.98 kN	4.99	4.99	18.3
				Factored (6.10):	40.25 kN	20.12	20.12	

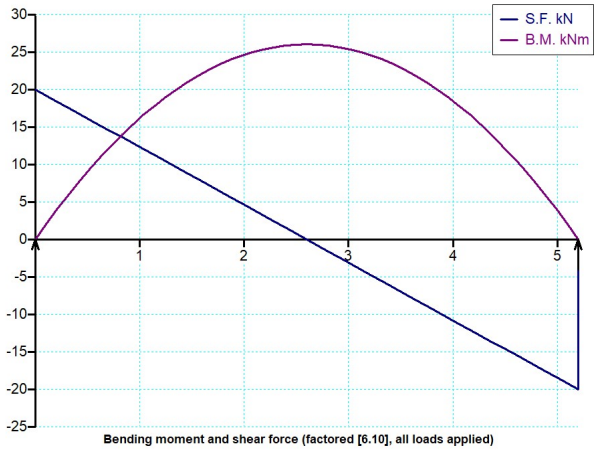
Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QA: Residential

Maximum B.M. = 26.16 kNm (6.10) at 2.60 m. from R1

Maximum S.F. = 20.12 kN (6.10) at R1

Mid-span deflections: Dead: 34.3 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 18.3 x 10^8/EI
Total: 52.6 x 10^8/EI





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 <= 9e (7.32): Class 1, plastic
EC3 Table 5.2 Web: c/t = 123.6/8.0 = 15.4 <= 72e (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 20.12 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/$\sqrt{3}$)/(1.0 x 1000) = 292 kN (>=20.12) OK [EC3 6.2.6]

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

Bending

Moment resistance

Design moment, M_{Ed} = 26.2 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, $M_{b,Rd} = c_{LT,mod}.M_{c,Rd}$

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

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

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

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

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

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

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

$M_{cr} = C_1(p^2EI_z/L_e^2)\sqrt{(I_w/I_z + L_e^2GI_T/p^2EI_z)}$ [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}	$\bar{l}_z$	$\bar{l}_{LT}$	f_{LT}	c_{LT}	$c_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-5.20	5.20	26.2	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 = 18.3 x 1e8/(210,000 x 2,210) = 3.9 mm (L/1320) OK
TL deflection = 52.6 x 1e8/(210,000 x 2,210) = 11.3 mm (L/459)

Bearings

152 x 152 x 37 UC stiff bearing length, $b_1 = t + 1.6r + 2T$ = 43.2 mm

R1 (20.12 kN) : 500 x 100 mm padstone

Masonry design strength = 0.44 N/mm^2 (User-entered value)

Stress under padstone = 20.12 x 1000/500 x 100 = 0.40 N/mm^2 OK

Padstone height is less than projection (228mm): reinforcement required - not checked

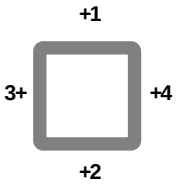
R2 (20.12 kN) : None

Column calculation to EN1993-1-1 using S355 steel

Location: SC1 - Steel Column

Length: 2.5 m.

Pos	Dur	Load	kN	Fact	Offset	M_{yy}	M_{zz}
				6.10		6.10	6.10
2	G	B/F Beam: B1 - Steel : R2	9.36	12.64	100	-1.26	0.00
2	QA	B/F Beam: B1 - Steel : R2	4.99	7.48	100	-0.75	0.00
Total load kN			14.35	**20.12**		**-2.01**	**0.00**



Load offsets are measured in mm. from faces of member; moments in kNm
Load durations: G: Dead; Qx: Imposed; QA: Residential

SECTION SIZE : **100 x 100 x 5.0 HF SHS S355** *(14.7 kg/m)*

Section properties: A_g = 18.7cm^2 i_z = 3.86 cm W_{pl} = 66.4 cm^3 W_{el} = 55.9 cm^3

Design strength, f_y = 355 N/mm^2 ε = 0.814

Classification: Flange: c/t = 85.0/5.0 = 17.0 <= 33ε (26.8): Class 1, plastic
[Table 5.2] Web: c/t = 85.0/5.0 = 17.0 <= 396ε/(13α - 1) = 54.3 (α = 0.533): Class 1, plastic
α = 0.5(1 + N_{Ed}/(f_y.c.t_w))) = 0.5 x (1 + 20.1 x 1000/(355 x 85.0 x 2 x 5.00)) = 0.533 [SCI P362 Table 5.1 note 1]

Major axis: L_{Ey} = 1.0L = 2.50 m. Slenderness, λ_y = 2.50 x 100/3.86 = 64.8
Minor axis: L_{Ez} = 1.0L = 2.50 m. Slenderness, λ_z = 2.50 x 100/3.86 = 64.8

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

Compression

Design axial load, N_{Ed} = 20.12 kN

Design compression resistance, $N_{c,Rd}$ = A.f_y/γ_{M0} = 18.70 x 100 x 355/(1.0 x 1000) = 664 kN OK

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

λ_1 = 93.9ε = 93.9 x 0.814 = 76.4

Buckling about y-y (major) axis

$\bar{\lambda}_y$ = λ_y/λ_1 = 64.8/76.4 = 0.848 [EC3 6.3.1.3]

Use curve a: α = 0.210 ϕ = 0.5(1 + α($\bar{\lambda}_y$ - 0.2) + $\bar{\lambda}_y^2$) = 0.93 [EC3 (6.49)]

Flexural buckling reduction factor, χ_y = 1/(ϕ + $\sqrt{(\phi^2 - \bar{\lambda}_y^2)}$) = 0.77 [EC3 (6.49)]

Design buckling resistance, $N_{b,y,Rd}$ = χ_yA.f_y/γ_{M1} = 0.77 x 18.70 x 100 x 355/(1.0 x 1000) = 509 kN OK [EC3 (6.47)]

Buckling about z-z (minor) axis

$\bar{\lambda}_z$ = λ_z/λ_1 = 64.8/76.4 = 0.848 [EC3 6.3.1.3]

Use curve a: α = 0.210 ϕ = 0.5(1 + α($\bar{\lambda}_z$ - 0.2) + $\bar{\lambda}_z^2$) = 0.93 [EC3 Table 6.2]

Flexural buckling reduction factor, χ_z = 1/(ϕ + $\sqrt{(\phi^2 - \bar{\lambda}_z^2)}$) = 0.77 [EC3 (6.49)]

Design buckling resistance, $N_{b,z,Rd}$ = χ_zA.f_y/γ_{M1} = 0.77 x 18.70 x 100 x 355/(1.0 x 1000) = 509 kN OK [EC3 (6.47)]

Bending about y-y (major) axis:

Design moment, $M_{y,Ed}$ = 2.01 kNm

Moment resistance, $M_{c,y,Rd}$ = f_y.$W_{pl,y}$ = 355 x 66.4/1000 = 23.57 kNm OK

Lateral-torsional buckling check not required for square hollow sections

Bending about z-z (minor) axis: N/A

Summary:

N_{Ed}/$N_{min,b,Rd}$ = 20.12/509 = 0.040 [1]
$M_{y,Ed}$/M_{bs} = 2.01/23.57 = 0.085 [2]
$M_{z,Ed}$/$M_{z,cb,Rd}$ = 0.000 [3] x 1.5 [NCCI SN048]

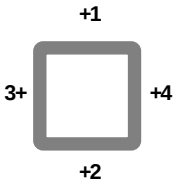
Sum of stress ratios [1] + [2] + 1.5 x [3] = **0.125** **OK** *subject to all SN048b criteria being complied with*

Column calculation to EN1993-1-1 using S355 steel

Location: SC2 - Steel Column

Length: 2.5 m.

Pos	Dur	Load	kN	Fact	Offset	M_{yy}	M_{zz}
				6.10		6.10	6.10
2	G	B/F Beam: B1 - Steel : R2	9.36	12.64	100	-1.90	0.00
2	QA	B/F Beam: B1 - Steel : R2	4.99	7.48	100	-1.12	0.00
Total load kN			14.35	**20.12**		**-3.02**	**0.00**



Load offsets are measured in mm. from faces of member; moments in kNm
Load durations: G: Dead; Qx: Imposed; QA: Residential

SECTION SIZE : **100 x 100 x 5.0 HF SHS S355** *(14.7 kg/m)*

Section properties: A_g = 18.7cm^2 i_z = 3.86 cm W_{pl} = 66.4 cm^3 W_{el} = 55.9 cm^3

Design strength, f_y = 355 N/mm^2 ε = 0.814

Classification: Flange: c/t = 85.0/5.0 = 17.0 <= 33ε (26.8): Class 1, plastic
[Table 5.2] Web: c/t = 85.0/5.0 = 17.0 <= 396ε/(13α - 1) = 54.3 (α = 0.533): Class 1, plastic
α = 0.5(1 + N_{Ed}/(f_y.c.t_w))) = 0.5 x (1 + 20.1 x 1000/(355 x 85.0 x 2 x 5.00)) = 0.533 [SCI P362 Table 5.1 note 1]

Major axis: L_{Ey} = 1.0L = 2.50 m. Slenderness, λ_y = 2.50 x 100/3.86 = 64.8
Minor axis: L_{Ez} = 1.0L = 2.50 m. Slenderness, λ_z = 2.50 x 100/3.86 = 64.8

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

Compression

Design axial load, N_{Ed} = 20.12 kN

Design compression resistance, $N_{c,Rd}$ = A.f_y/γ_{M0} = 18.70 x 100 x 355/(1.0 x 1000) = 664 kN OK

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

λ_1 = 93.9ε = 93.9 x 0.814 = 76.4

Buckling about y-y (major) axis

$\bar{\lambda}_y$ = λ_y/λ_1 = 64.8/76.4 = 0.848 [EC3 6.3.1.3]

Use curve a: α = 0.210 ϕ = 0.5(1 + α($\bar{\lambda}_y$ - 0.2) + $\bar{\lambda}_y^2$) = 0.93 [EC3 (6.49)]

Flexural buckling reduction factor, χ_y = 1/(ϕ + $\sqrt{(\phi^2 - \bar{\lambda}_y^2)}$) = 0.77 [EC3 (6.49)]

Design buckling resistance, $N_{b,y,Rd}$ = χ_yA.f_y/γ_{M1} = 0.77 x 18.70 x 100 x 355/(1.0 x 1000) = 509 kN OK [EC3 (6.47)]

Buckling about z-z (minor) axis

$\bar{\lambda}_z$ = λ_z/λ_1 = 64.8/76.4 = 0.848 [EC3 6.3.1.3]

Use curve a: α = 0.210 ϕ = 0.5(1 + α($\bar{\lambda}_z$ - 0.2) + $\bar{\lambda}_z^2$) = 0.93 [EC3 Table 6.2]

Flexural buckling reduction factor, χ_z = 1/(ϕ + $\sqrt{(\phi^2 - \bar{\lambda}_z^2)}$) = 0.77 [EC3 (6.49)]

Design buckling resistance, $N_{b,z,Rd}$ = χ_zA.f_y/γ_{M1} = 0.77 x 18.70 x 100 x 355/(1.0 x 1000) = 509 kN OK [EC3 (6.47)]

Bending about y-y (major) axis:

Design moment, $M_{y,Ed}$ = 3.02 kNm

Moment resistance, $M_{c,y,Rd}$ = f_y.$W_{pl,y}$ = 355 x 66.4/1000 = 23.57 kNm OK

Lateral-torsional buckling check not required for square hollow sections

Bending about z-z (minor) axis: N/A

Summary:

$N_{Ed}/N_{min,b,Rd}$ = 20.12/509 = 0.040 [1]
$M_{y,Ed}/M_{bs}$ = 3.02/23.57 = 0.128 [2]
$M_{z,Ed}/M_{z,cb,Rd}$ = 0.000 [3] x 1.5 [NCCI SN048]

Sum of stress ratios [1] + [2] + 1.5 x [3] = **0.168** **OK** *subject to all SN048b criteria being complied with*

Beam: **Floor Joists** **Span: 3.4 m.**

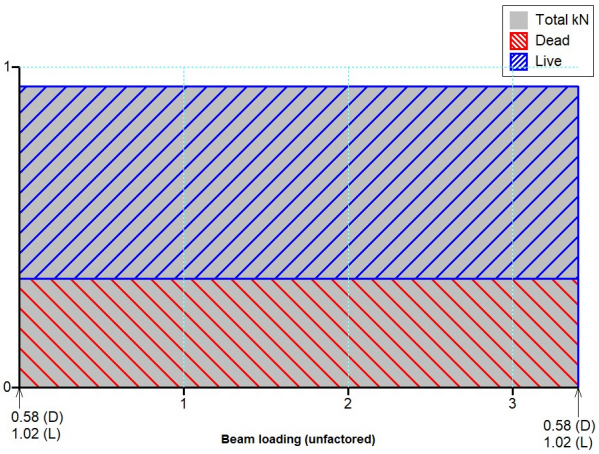
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.1	0		L	0.17	0.17	0.17
U G	Floor Dead	0.6*0.4	0		L	0.41	0.41	0.42
U QA	Floor Live	1.5*0.4	0		L	1.02	1.02	1.04
				Total load (unfactored):	**3.20 kN**	**1.60**	**1.60**	1.64
				Dead/Permanent (unfactored):	1.16 kN	0.58	0.58	0.59
				Live/Variable (unfactored):	2.04 kN	1.02	1.02	1.04
				Factored (6.10):	4.62 kN	2.31	2.31	

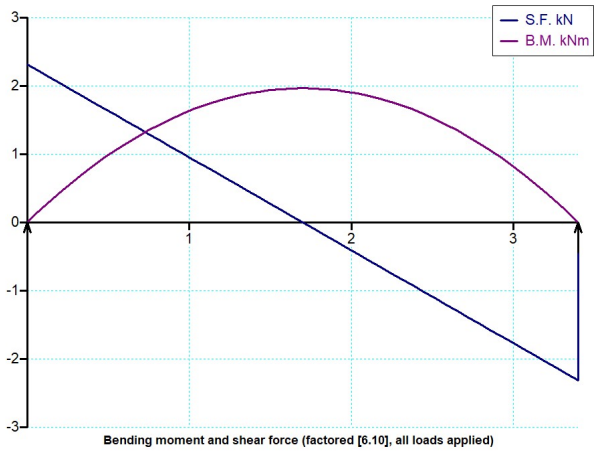
Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QA: Residential

Maximum B.M. = 1.96 kNm (6.10) at 1.70 m. from R1

Maximum S.F. = 2.31 kN (6.10) at R1

Mid-span deflections: Dead: 0.59 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 1.04 x 10^8/EI
Total: 1.64 x 10^8/EI





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 cm^3 I_y = 2,233 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} = 0.6 (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/mm^2

Design bending stress, $\sigma_{m,y,d}$ = 1.96 x 1000/255 = 7.69 N/mm^2 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/mm^2

Design shear stress, $\tau_{v,y,d}$ = 2.31 x 1000 x (3/2)/(33.5 x 175) = 0.59 N/mm^2 OK

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

Deflection

Final deflection limit = 0.003L = 10.20 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 1.36 x 10^6/((11,000/16) x 50 x 175) = 0.27 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:	0.59	2.41	0.60	1.00	3.85	
QA:	1.04	4.25	0.60	0.30	5.02	
Shear deflection:		0.27			0.36	
Total mm.		6.93			**9.23**	OK

Beam: **B2 - Steel Beam** **Span: 3.4 m.**

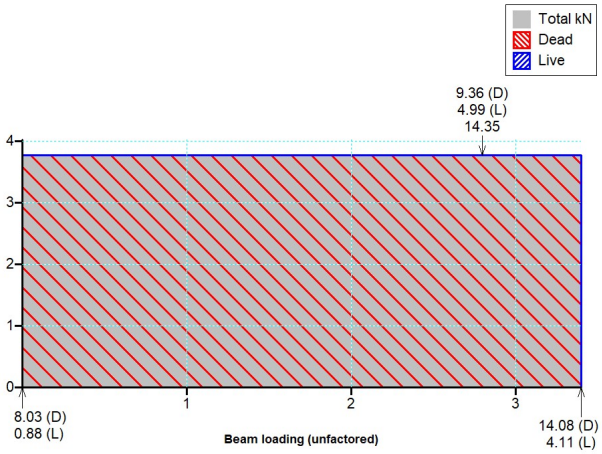
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.3	0		L	0.51	0.51	0.52
U G	Dormer Wall	1.5*2.3	0		L	5.87	5.87	6.00
P G	Col: SC1 - Steel Column	9.36	2.8			1.65	7.71	3.89
P QA	Col: SC1 - Steel Column	4.99	2.8			0.88	4.11	2.07
			Total load (unfactored):		**27.1 kN**	**8.91**	**18.19**	12.49
			Dead/Permanent (unfactored):		22.1 kN	8.03	14.08	10.42
			Live/Variable (unfactored):		5.0 kN	0.88	4.11	2.07
			Factored (6.10):		37.3 kN	12.16	25.18	

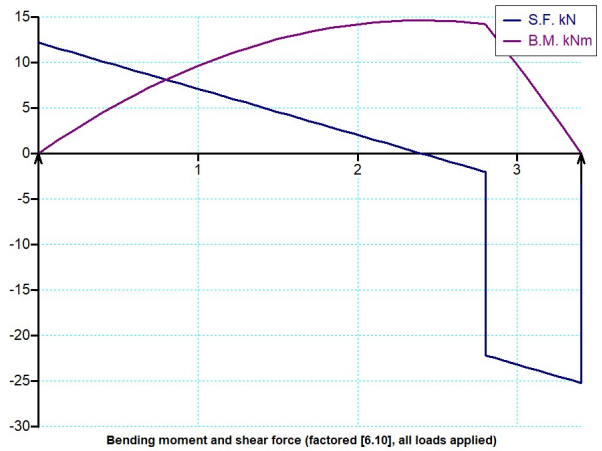
Load types: O:Beam o.w.; U:UDL; P:Point load; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed:

Maximum B.M. = 14.60 kNm (6.10) at 2.41 m. from R1

Maximum S.F. = -25.18 kN (6.10) at R2

Mid-span deflections: Dead: 10.42 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 2.07 x 10^8/EI
Total: 12.49 x 10^8/EI





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: <= 9e (7.32): Class 1, plastic
EC3 Table 5.2 Web: c/t = 123.6/6.5 = 19.0 <= 72e (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 25.2 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, $V_{pl,Rd} = A_v.(f_y/\sqrt{3})/\gamma_{M0}$ = 1,159 x (355/$\sqrt{3}$)/(1.0 x 1000) = 238 kN (>=25.2) OK [EC3 6.2.6]

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

Bending

Moment resistance

Design moment, M_{Ed} = 14.60 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 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]

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

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

$\bar{\lambda}_{LT,0}$ = 0.4 β = 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 = 248.0 cm^3 I_w = 0.031 dm^6 I_T = 10.5 cm^4 I_z = 560 cm^4 G = 81,000N/mm^2

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{\lambda}_z$	$\bar{\lambda}_{LT}$	f_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-3.40	3.40	14.6	1.00d	1.00	118.7	1.162	0.861	0.857	0.783	0.783	88.0	68.9	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 = 2.07 x 1e8/(210,000 x 1,750) = 0.6 mm (L/6036) OK
TL deflection = 12.49 x 1e8/(210,000 x 1,750) = 3.4 mm (L/1000)

Beam: **B3 - Steel Beam** **Span: 5.7 m.**

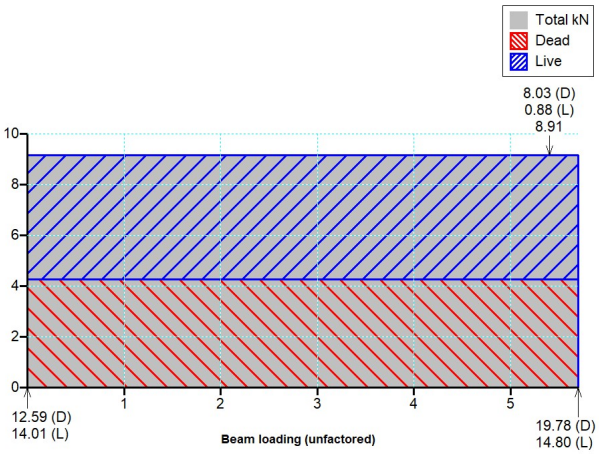
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.5	0		L	1.42	1.42	7
U G	Floor Dead	0.6*3.4*0.5	0		L	2.91	2.91	14
U QA	Floor Live	1.5*3.4*0.5	0		L	7.27	7.27	35
U G	Roof Dead	1.0*4.5*0.5	0		L	6.41	6.41	31
U QA	Roof Live	0.6*4.5*0.5	0		L	3.85	3.85	19
U G	Ceiling Dead	0.5*2.0*0.5	0		L	1.42	1.42	7
U QA	Ceiling Live	1.0*2.0*0.5	0		L	2.85	2.85	14
P G	Bm: B2 - Steel Beam : R1	8.03	5.4			0.42	7.61	5
P QA	Bm: B2 - Steel Beam : R1	0.88	5.4			0.05	0.83	1
				Total load (unfactored):	**61.2 kN**	**26.60**	**34.58**	131
				Dead/Permanent (unfactored):	32.4 kN	12.59	19.78	64
				Live/Variable (unfactored):	28.8 kN	14.01	14.80	68
				Factored (6.10):	86.9 kN	38.02	48.90	

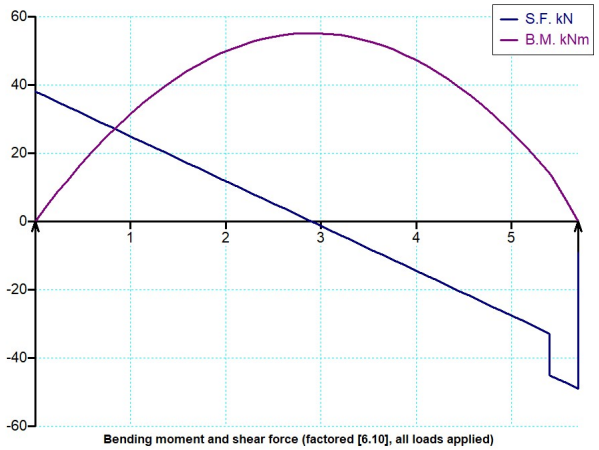
Load types: O:Beam o.w.; U:UDL; P:Point load; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QA: Residential

Maximum B.M. = 55.1 kNm (6.10) at 2.90 m. from R1

Maximum S.F. = -48.9 kN (6.10) at R2

Mid-span deflections: Dead: 64 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 68 x 10^8/EI
Total: 131 x 10^8/EI





Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **203 x 203 x 46 UKC** S355

D=203.2 mm B=203.6 mm t=7.2 mm T=11.0 mm I_y=4,570 cm^4 i_z=5.13 cm $W_{pl,y}$=497 cm^3 $W_{el,y}$=450 cm^3

Classification: Flange: c/t = 88.0/11.0 = 8.00 <= 10ε (8.14): Class 2, compact
EC3 Table 5.2 Web: c/t = 160.8/7.2 = 22.3 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 48.9 kN

Shear area, A_v = A - 2bt$_f$ + (t_w+2r)t_f = 58.7x100 - 2x204x11.0 + (7.20+2x10.2) x11.0 = 1,694 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd}$ = A_v.(f_y/√3)/γ_{M0} = 1,694 x (355/√3)/(1.0 x 1000) = 347 kN (>=48.9) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 181.2/7.2 = 25.17 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 55.1 kNm

Moment resistance, $M_{c,y,Rd}$ = f_y.$W_{pl,y}$ = 355 x 497/1000 = 176 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, $M_{b,Rd}$ = $\chi_{LT,mod}$.$M_{c,Rd}$

$\chi_{LT,mod}$ = χ_{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/√C_1 [NA2.18]

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

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

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

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

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

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

W_y = 497.0 cm^3 I_w = 0.143 dm^6 I_T = 22.2 cm^4 I_z = 1,550 cm^4 G = 81,000N/mm^2

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{\lambda}_z$	$\bar{\lambda}_{LT}$	f_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-5.70	5.70	55.1	1.00d	1.00	163.5	1.454	1.039	1.013	0.676	0.676	176.4	119.3	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 = 68 x 1e8/(210,000 x 4,570) = 7.1 mm (L/806) OK
TL deflection = 131 x 1e8/(210,000 x 4,570) = 13.7 mm (L/416)

Bearings

203 x 203 x 46 UC stiff bearing length, b_1 = t + 1.6r + 2T = 45.5 mm

Design masonry strength = 0.440 N/mm^2 (User-entered value)

R1 (38.0 kN) : 900 x 100 mm padstone

Stress under padstone = 38.0 x 1000/900 x 100 = 0.42 N/mm^2 OK

Padstone height is less than projection (427mm): reinforcement required - not checked

R2 (48.9 kN) : 1150 x 100 mm padstone

Stress under padstone = 48.9 x 1000/1150 x 100 = 0.43 N/mm^2 OK

Padstone height is less than projection (552mm): reinforcement required - not checked

Beam: **B4 - Steel Beam** **Span: 5.7 m.**

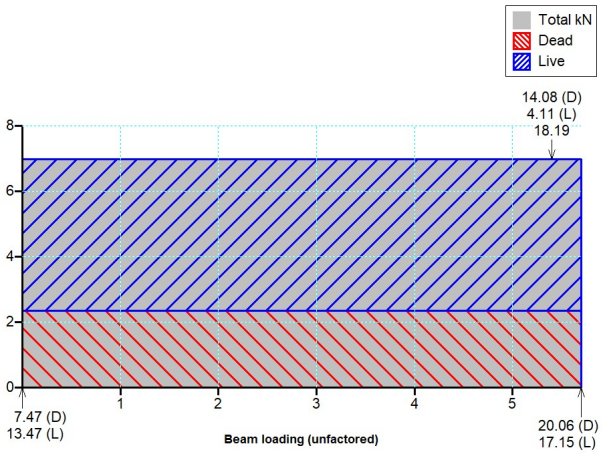
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.5	0		L	1.42	1.42	7
U G	Floor Dead	0.6*6.2*0.5	0		L	5.30	5.30	26
U QA	Floor Live	1.5*6.2*0.5	0		L	13.25	13.25	64
P G	Bm: B2 - Steel Beam : R2	14.08	5.4			0.74	13.34	9
P QA	Bm: B2 - Steel Beam : R2	4.11	5.4			0.22	3.89	2
				Total load (unfactored):	**58.1 kN**	**20.94**	**37.21**	107
				Dead/Permanent (unfactored):	27.5 kN	7.47	20.06	41
				Live/Variable (unfactored):	30.6 kN	13.47	17.15	66
				Factored (6.10):	83.1 kN	30.28	52.81	

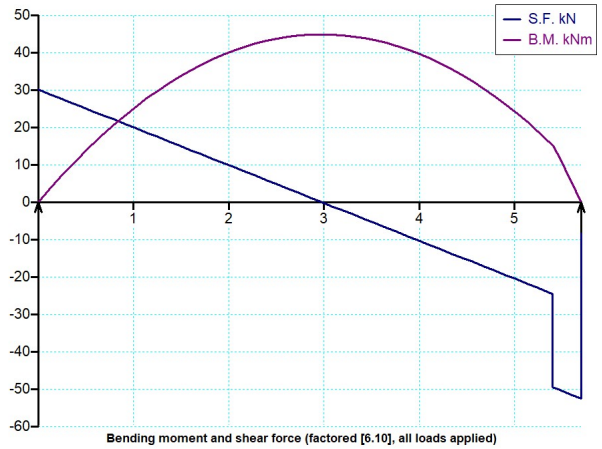
Load types: O:Beam o.w.; U:UDL; P:Point load; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QA: Residential

Maximum B.M. = 45.13 kNm (6.10) at 2.98 m. from R1

Maximum S.F. = -52.8 kN (6.10) at R2

Mid-span deflections: Dead: 41 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 66 x 10^8/EI
Total: 107 x 10^8/EI





Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **203 x 203 x 46 UKC** S355

D=203.2 mm B=203.6 mm t=7.2 mm T=11.0 mm I_y=4,570 cm^4 i_z=5.13 cm $W_{pl,y}$=497 cm^3 $W_{el,y}$=450 cm^3

Classification: Flange: c/t = 88.0/11.0 = 8.00 <= 10e (8.14): Class 2, compact
EC3 Table 5.2 Web: c/t = 160.8/7.2 = 22.3 <= 72e (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 52.8 kN

Shear area, $A_v = A - 2bt_f + (t_w+2r)t_f$ = 58.7x100 - 2x204x11.0 + (7.20+2x10.2) x11.0 = 1,694 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd} = A_v.(f_y/\sqrt{3})/\gamma_{M0}$ = 1,694 x (355/$\sqrt{3}$)/(1.0 x 1000) = 347 kN (>=52.8) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 181.2/7.2 = 25.17 <= 72e (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 45.1 kNm

Moment resistance, $M_{c,y,Rd} = f_y.W_{pl,y}$ = 355 x 497/1000 = 176 kNm OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

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

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

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

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

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

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

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

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

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

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

W_y = 497.0 cm^3 I_w = 0.143 dm^6 I_T = 22.2 cm^4 I_z = 1,550 cm^4 G = 81,000N/mm^2

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{l}_z$	$\bar{l}_{LT}$	f_{LT}	c_{LT}	$c_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-5.70	5.70	45.1	1.00d	1.00	163.5	1.454	1.039	1.013	0.676	0.676	176.4	119.3	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 = 66 x 1e8/(210,000 x 4,570) = 6.9 mm (L/824) OK
TL deflection = 107 x 1e8/(210,000 x 4,570) = 11.2 mm (L/509)

Bearings

203 x 203 x 46 UC stiff bearing length, $b_1 = t + 1.6r + 2T$ = 45.5 mm

Design masonry strength = 0.440 N/mm^2 (User-entered value)

R1 (30.3 kN) : 700 x 100 mm padstone

Stress under padstone = 30.3 x 1000/700 x 100 = 0.43 N/mm^2 OK

Padstone height is less than projection (327mm): reinforcement required - not checked

R2 (52.8 kN) : 1250 x 100 mm padstone

Stress under padstone = 52.8 x 1000/1250 x 100 = 0.42 N/mm^2 OK

Padstone height is less than projection (602mm): reinforcement required - not checked

Beam: **Nominal Beam for B3** **Span: 1.0 m.**

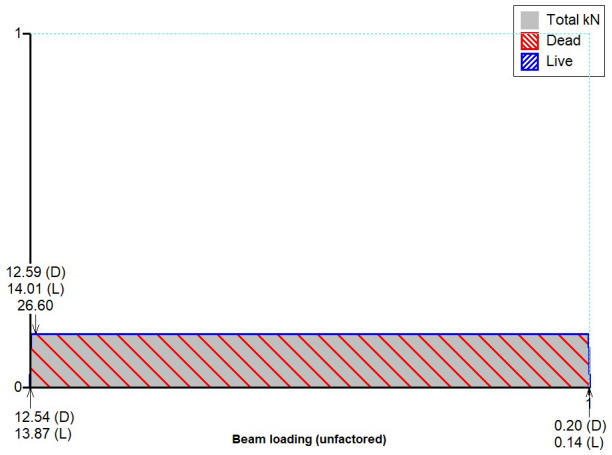
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.15	0		L	0.08	0.08	0.002
P G	Bm: B3 - Steel Beam : R1	12.59	0.01			12.46	0.13	0.008
P QA	Bm: B3 - Steel Beam : R1	14.01	0.01			13.87	0.14	0.009
				Total load (unfactored):	**26.8 kN**	**26.41**	**0.34**	0.019
				Dead/Permanent (unfactored):	12.7 kN	12.54	0.20	0.010
				Live/Variable (unfactored):	14.0 kN	13.87	0.14	0.009
				Factored (6.10):	38.2 kN	37.73	0.48	

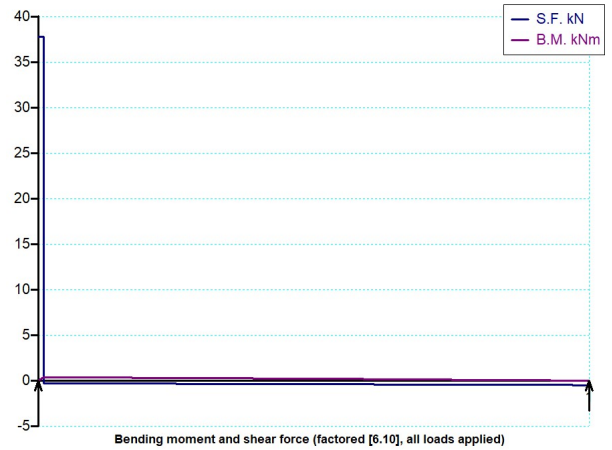
Load types: O:Beam o.w.; P:Point load; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed:

Maximum B.M. = 0.377 kNm (6.10) at 0.01 m. from R1

Maximum S.F. = 37.73 kN (6.10) at R1

Mid-span deflections: Dead: 0.010 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 0.009 x 10^8/EI
Total: 0.019 x 10^8/EI





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} = 37.7 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 (>=37.7) OK [EC3 6.2.6]

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

Bending

Moment resistance

Design moment, M_{Ed} = 0.377 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, $M_{b,Rd} = c_{LT,mod}.M_{c,Rd}$

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

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

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

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

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

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

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

$M_{cr} = C_1(p^2EI_z/L_e^2)\sqrt{(I_w/I_z + L_e^2GI_T/p^2EI_z)}$ [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 = 81,000N/mm^2

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{l}_z$	$\bar{l}_{LT}$	f_{LT}	c_{LT}	$c_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-1.00	1.00	0.4	1.00d	1.00	86.3	0.711	0.589	0.662	0.922	0.922	29.9	27.6	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 = 0.009 x 1e8/(210,000 x 473) = 0.0 mm OK
TL deflection = 0.019 x 1e8/(210,000 x 473) = 0.0 mm

Bearings

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

Design masonry strength = 0.440 N/mm^2 (User-entered value)

R1 (37.73 kN) : 900 x 100 mm padstone

Stress under padstone = 37.73 x 1000/900 x 100 = 0.42 N/mm^2 OK

Padstone height is less than projection (434mm): reinforcement required - not checked

R2 (0.48 kN) : 76 x 100 mm padstone

Stress under padstone = 0.48 x 1000/76 x 100 = 0.06 N/mm^2 OK

Padstone height is less than projection (22mm): reinforcement required - not checked

Beam: **Nominal Beam for B4** **Span: 1.8 m.**

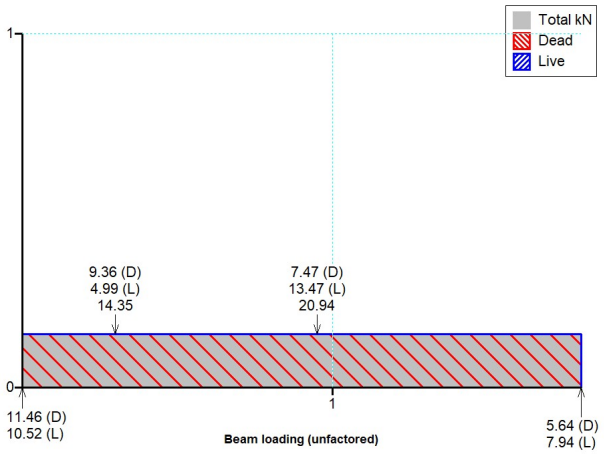
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.15	0		L	0.14	0.14	0.02
P G	Bm: B4 - Steel Beam : R1	7.47	0.95			3.53	3.94	0.90
P QA	Bm: B4 - Steel Beam : R1	13.47	0.95			6.36	7.11	1.63
P G	Col: SC2 - Steel Column	9.36	0.3			7.80	1.56	0.55
P QA	Col: SC2 - Steel Column	4.99	0.3			4.16	0.83	0.29
				Total load (unfactored):	**35.6 kN**	**21.98**	**13.58**	3.39
				Dead/Permanent (unfactored):	17.1 kN	11.46	5.64	1.47
				Live/Variable (unfactored):	18.5 kN	10.52	7.94	1.92
				Factored (6.10):	50.8 kN	31.25	19.52	

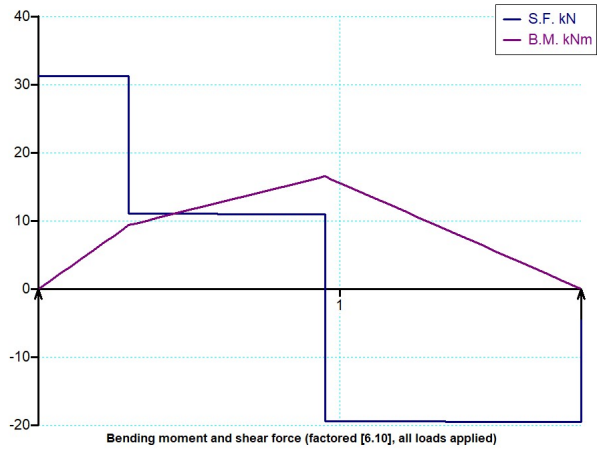
Load types: O:Beam o.w.; P:Point load; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed:

Maximum B.M. = 16.52 kNm (6.10) at 0.95 m. from R1

Maximum S.F. = 31.25 kN (6.10) at R1

Mid-span deflections: Dead: 1.47 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 1.92 x 10^8/EI
Total: 3.39 x 10^8/EI





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 <= 9e (7.32): Class 1, plastic
EC3 Table 5.2 Web: c/t = 96.6/4.0 = 24.1 <= 72e (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 31.3 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 (>=31.3) OK [EC3 6.2.6]

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

Bending

Moment resistance

Design moment, M_{Ed} = 16.52 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, $M_{b,Rd} = c_{LT,mod}.M_{c,Rd}$

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

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

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

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

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

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

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

$M_{cr} = C_1(p^2EI_z/L_e^2)\sqrt{(I_w/I_z + L_e^2GI_T/p^2EI_z)}$ [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 = 81,000N/mm^2

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{l}_z$	$\bar{l}_{LT}$	f_{LT}	c_{LT}	$c_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-1.80	1.80	16.5	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 = 1.92 x 1e8/(210,000 x 473) = 1.9 mm (L/931) OK
TL deflection = 3.39 x 1e8/(210,000 x 473) = 3.4 mm (L/527)

Bearings

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

Design masonry strength = 0.440 N/mm^2 (User-entered value)

R1 (31.3 kN) : 950 x 100 mm padstone

Stress under padstone = 31.3 x 1000/950 x 100 = 0.33 N/mm^2 OK

Padstone height is less than projection (459mm): reinforcement required - not checked

R2 (19.5 kN) : 450 x 100 mm padstone

Stress under padstone = 19.5 x 1000/450 x 100 = 0.43 N/mm^2 OK

Padstone height is less than projection (209mm): reinforcement required - not checked

Beam: **Flat Roof Joists - Extension** **Span: 3.0 m.**

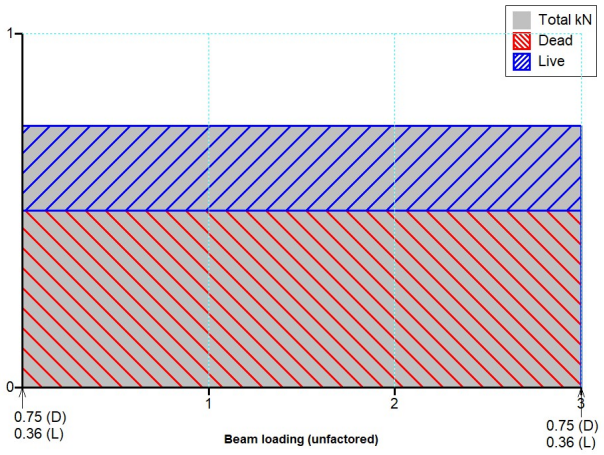
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.1	0		L	0.15	0.15	0.11
U G	Roof Dead	1.0*0.4	0		L	0.60	0.60	0.42
U QA	Roof Live	0.6*0.4	0		L	0.36	0.36	0.25
				Total load (unfactored):	**2.22 kN**	**1.11**	**1.11**	0.78
				Dead/Permanent (unfactored):	1.50 kN	0.75	0.75	0.53
				Live/Variable (unfactored):	0.72 kN	0.36	0.36	0.25
				Factored (6.10):	3.11 kN	1.55	1.55	

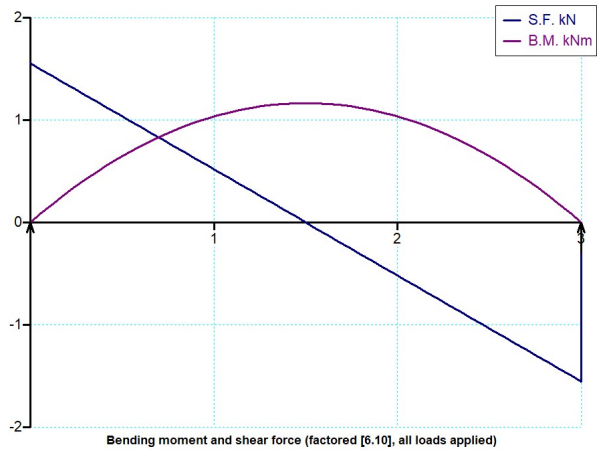
Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QA: Residential

Maximum B.M. = 1.16 kNm (6.10) at 1.50 m. from R1

Maximum S.F. = 1.55 kN (6.10) at R1

Mid-span deflections: Dead: 0.53 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 0.25 x 10^8/EI
Total: 0.78 x 10^8/EI





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 cm^3 I_y = 1,406 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, g_M = 1.3 [EC5 UK Table NA.3]

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

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

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

Bending

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

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

Design bending stress, $s_{m,y,d}$ = 1.16 x 1000/188 = 6.21 N/mm^2 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, $f_{v,d} = f_{v,k}.k_{mod}.k_{sys}/g_m$ = 4.00 x 0.60 x 1.1/1.30 = 2.03 N/mm^2

Design shear stress, $s_{v,y,d}$ = 1.55 x 1000 x (3/2)/(33.5 x 150) = 0.46 N/mm^2 OK

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

Deflection

Final deflection limit = 0.003L = 9.00 mm

Final deflection, $u_{fin} = Su_{inst}(1 + y_2.k_{def})$ [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 0.83 x 10^6/((11,000/16) x 50 x 150) = 0.19 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}	y_2	Final mm	
G:	0.53	3.41	0.60	1.00	5.45	
QA:	0.25	1.64	0.60	0.30	1.93	
Shear deflection:		0.19			0.28	
Total mm.		5.24			**7.67**	OK

Beam: **B5 - Steel Beam** **Span: 4.5 m.**

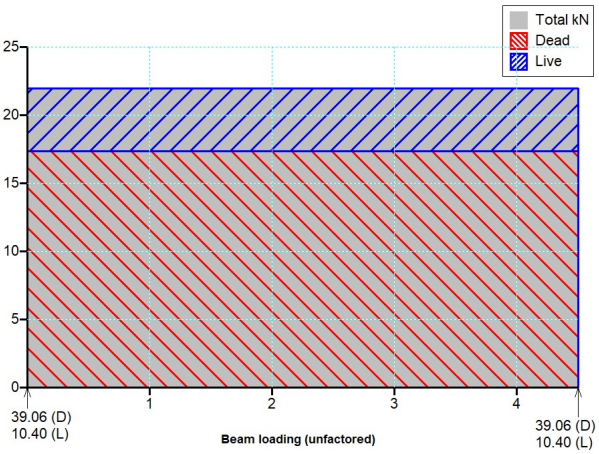
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.6	0		L	1.35	1.35	3
U G	Roof Dead	1.0*6.4*0.5	0		L	7.20	7.20	17
U QA	Roof Live	0.6*6.4*0.5	0		L	4.32	4.32	10
U G	Floor Dead	0.6*3.6*0.5	0		L	2.43	2.43	6
U QA	Floor Live	1.5*3.6*0.5	0		L	6.08	6.08	14
U G	External Wall	4.8*2.6	0		L	28.08	28.08	67
				Total load (unfactored):	**98.9 kN**	**49.45**	**49.45**	117
				Dead/Permanent (unfactored):	78.1 kN	39.06	39.06	93
				Live/Variable (unfactored):	20.8 kN	10.40	10.40	25
				Factored (6.10):	136.6 kN	68.32	68.32	

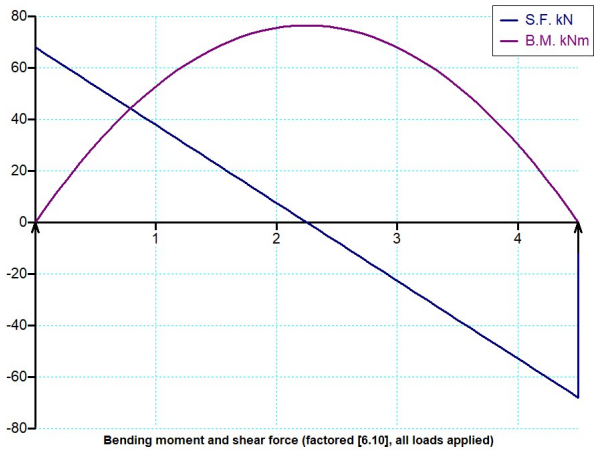
Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QA: Residential

Maximum B.M. = 76.9 kNm (6.10) at 2.25 m. from R1

Maximum S.F. = 68.3 kN (6.10) at R1

Mid-span deflections: Dead: 93 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 25 x 10^8/EI
Total: 117 x 10^8/EI





Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **203 x 203 x 60 UKC** S355

D=209.6 mm B=205.8 mm t=9.4 mm T=14.2 mm I_y=6,120 cm^4 i_z=5.20 cm $W_{pl,y}$=656 cm^3 $W_{el,y}$=584 cm^3

Classification: Flange: c/t = 88.0/14.2 = 6.20 <= 9ε (7.32): Class 1, plastic

EC3 Table 5.2 Web: c/t = 160.8/9.4 = 17.1 <= 72ε (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 68.3 kN

Shear area, $A_v = A - 2bt_f + (t_w+2r)t_f$ = 76.4x100 - 2x206x14.2 + (9.40+2x10.2) x14.2 = 2,218 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd} = A_v.(f_y/\sqrt{3})/\gamma_{M0}$ = 2,218 x (355/$\sqrt{3}$)/(1.0 x 1000) = 455 kN (>=68.3) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 181.2/9.4 = 19.28 <= 72e (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 76.9 kNm

Moment resistance, $M_{c,y,Rd} = f_y.W_{pl,y}$ = 355 x 656/1000 = 233 kNm OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

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

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

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

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

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

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

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

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

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

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

W_y = 656.0 cm^3 I_w = 0.197 dm^6 I_T = 47.2 cm^4 I_z = 2,060 cm^4 G = 81,000N/mm^2

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{l}_z$	$\bar{l}_{LT}$	f_{LT}	c_{LT}	$c_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-4.50	4.50	76.9	1.00d	1.00	350.5	1.133	0.815	0.820	0.809	0.809	232.9	188.3	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 = 25 x 1e8/(210,000 x 6,120) = 1.9 mm (L/2344) OK
TL deflection = 117 x 1e8/(210,000 x 6,120) = 9.1 mm (L/493)

Beam: **B6 - Steel Beam** **Span: 4.6 m.**

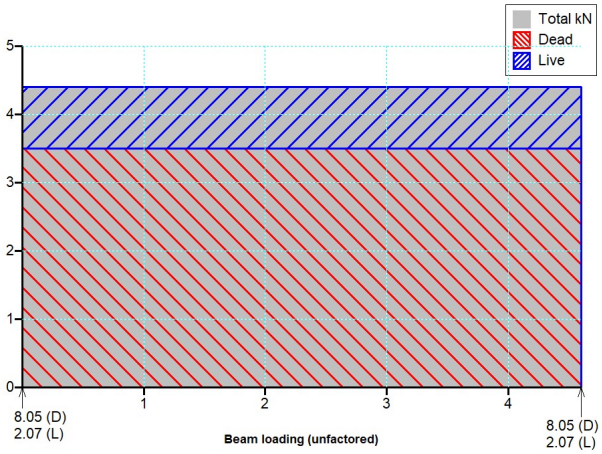
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.6	0		L	1.38	1.38	3.5
U G	Roof Dead	1.0*3.0*0.5	0		L	3.45	3.45	8.7
U QA	Roof Live	0.6*3.0*0.5	0		L	2.07	2.07	5.2
U G	Cavity Wall	3.5*0.4	0		L	3.22	3.22	8.2
				Total load (unfactored):	**20.24 kN**	**10.12**	**10.12**	25.7
				Dead/Permanent (unfactored):	16.10 kN	8.05	8.05	20.4
				Live/Variable (unfactored):	4.14 kN	2.07	2.07	5.3
				Factored (6.10):	27.94 kN	13.97	13.97	

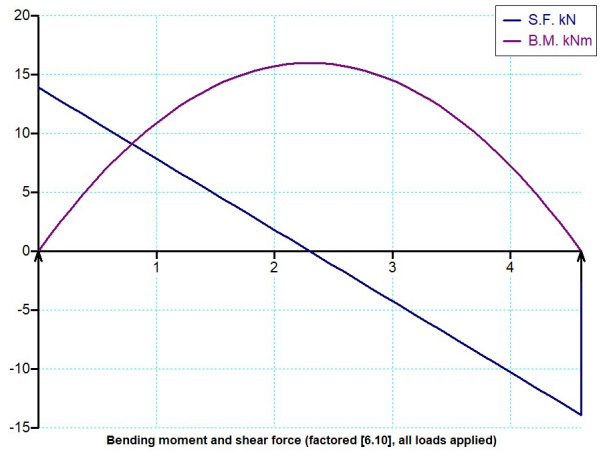
Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QA: Residential

Maximum B.M. = 16.07 kNm (6.10) at 2.30 m. from R1

Maximum S.F. = 13.97 kN (6.10) at R1

Mid-span deflections: Dead: 20.4 x 10^8/EI *(E in N/mm^2, I in cm^4)*
Live: 5.3 x 10^8/EI
Total: 25.7 x 10^8/EI





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 <= 9e (7.32): Class 1, plastic
EC3 Table 5.2 Web: c/t = 172.4/6.4 = 26.9 <= 72e (58.6): Class 1, plastic

Shear

Design shear force, V_{Ed} = 13.97 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, $V_{pl,Rd} = A_v.(f_y/\sqrt{3})/\gamma_{M0}$ = 1,456 x (355/$\sqrt{3}$)/(1.0 x 1000) = 299 kN (>=13.97) OK [EC3 6.2.6]

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

Bending

Moment resistance

Design moment, M_{Ed} = 16.07 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 restrained. Nominal torsional restraint. Both flanges free to rotate on plan (1.0L)

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

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

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

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

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

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

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

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

$M_{cr} = C_1(p^2EI_z/L_e^2)\sqrt{(I_w/I_z + L_e^2GI_T/p^2EI_z)}$ [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 = 81,000N/mm^2

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{l}_z$	$\bar{l}_{LT}$	f_{LT}	c_{LT}	$c_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-4.60	4.60	16.1	1.00d	1.00	67.2	1.899	1.288	1.273	0.530	0.530	111.5	59.1	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 = 5.3 x 1e8/(210,000 x 2,900) = 0.9 mm (L/5336) OK
TL deflection = 25.7 x 1e8/(210,000 x 2,900) = 4.2 mm (L/1092)

Bearings

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

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

R1 (13.97 kN) : 206 x 100 mm padstone

Stress under padstone = 13.97 x 1000/206 x 100 = 0.68 N/mm^2 OK

Padstone height is less than projection (84mm): reinforcement required - not checked

R2 (13.97 kN) : 200 x 100 mm padstone

Stress under padstone = 13.97 x 1000/200 x 100 = 0.70 N/mm^2 OK

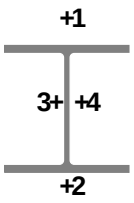
Padstone height is less than projection (81mm): reinforcement required - not checked

Column calculation to EN1993-1-1 using S235 steel

Location: SC3 & SC4 - Steel Columns

Length: 2.8 m.

Pos	Dur	Load	kN	Fact	Offset	M_{yy}	M_{zz}
				6.10		6.10	6.10
2	G	B/F Bm: B5 - Steel Beam : R1	39.1	52.7	100	-9.43	0.00
2	QA	B/F Bm: B5 - Steel Beam : R1	10.4	15.6	100	-2.79	0.00
Total load kN			49.5	**68.3**		**-12.22**	**0.00**



Load offsets are measured in mm. from faces of member; moments in kNm
Load durations: G: Dead; Qx: Imposed; QA: Residential

SECTION SIZE : **152 x 152 x 30 UKC** S235

Section properties: B = 152.9mm D = 157.6mm T = 9.4mm t = 6.5mm A_g = 38.3cm^2
i_y = 6.76cm i_z = 3.83cm $W_{pl,y}$ = 248cm^3 $W_{el,y}$ = 222cm^3

Design strength, f_y = 235 N/mm^2 ε = 1.00

Classification: Flange: c/t = 65.6/9.4 = 6.98 <= 9ε (9.00): Class 1, plastic
[Table 5.2] Web: c/t = 123.6/6.5 = 19.0 <= 396ε/(13α - 1) = 50.4 : Class 1, plastic
α = 0.5(1 + N_{Ed}/(f_y.c.t_w)) = 0.5 x (1 + 68.3 x 1000/(235 x 124 x 6.50)) = 0.681 [SCI P362 Table 5.1 note 1]

Major axis: L_{Ey} = 1.0L = 2.80 m. Slenderness, λ_y = 2.80 x 100/6.76 = 41.4
Minor axis: L_{Ez} = 1.0L = 2.80 m. Slenderness, λ_z = 2.80 x 100/3.83 = 73.1

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

Compression

Design axial load, N_{Ed} = 68.3 kN

Design compression resistance, $N_{c,Rd}$ = A.f_y/γ_{M0} = 38.3 x 100 x 235/(1.0 x 1000) = 900 kN OK

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

λ_1 = 93.9ε = 93.9 x 1.00 = 93.9

Buckling about y-y (major) axis

$\bar{\lambda}_y$ = λ_y/λ_1 = 41.4/93.9 = 0.441 [EC3 6.3.1.3]

Use curve b: α = 0.340 ϕ = 0.5(1 + α($\bar{\lambda}_y$ - 0.2) + $\bar{\lambda}_y^2$) = 0.64 [EC3 (6.49)]

Flexural buckling reduction factor, χ_y = 1/(ϕ + $\sqrt{(\phi^2 - \bar{\lambda}_y^2)}$) = 0.91 [EC3 (6.49)]

Design buckling resistance, $N_{b,y,Rd}$ = χ_yA.f_y/γ_{M1} = 0.91 x 38.3 x 100 x 235/(1.0 x 1000) = 819 kN OK [EC3 (6.47)]

Buckling about z-z (minor) axis

$\bar{\lambda}_z$ = λ_z/λ_1 = 73.1/93.9 = 0.779 [EC3 6.3.1.3]

Use curve c: α = 0.490 ϕ = 0.5(1 + α($\bar{\lambda}_z$ - 0.2) + $\bar{\lambda}_z^2$) = 0.94 [EC3 Table 6.2]

Flexural buckling reduction factor, χ_z = 1/(ϕ + $\sqrt{(\phi^2 - \bar{\lambda}_z^2)}$) = 0.68 [EC3 (6.49)]

Design buckling resistance, $N_{b,z,Rd}$ = χ_zA.f_y/γ_{M1} = 0.68 x 38.3 x 100 x 235/(1.0 x 1000) = 608 kN OK [EC3 (6.47)]

Bending about y-y (major) axis:

Design moment, $M_{y,Ed}$ = 12.22 kNm

Moment resistance, $M_{c,y,Rd}$ = f_y.$W_{pl,y}$ = 235 x 248/1000 = 58.3 kNm OK

Calculate buckling resistance moment

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

M_{cr} = C_1(π^2EI_z/L_{eff}^2)[$\sqrt{}$(I_w/I_z + L_{eff}^2GI_t/(π^2EI_z)] = 157 [NCCI SN003 2(1)]
C_1 = 1.0 (conservative) $\bar{\lambda}_{LT}$ = $\sqrt{}$($M_{c,y,Rd}$ /M_{cr}) = 0.61
$\bar{\lambda}_{LT,0}$ = 0.4 β = 0.75 [EC3 UK NA 2.17]
Use buckling curve b: α_{LT} = 0.340 [EC3 Tables 6.3/6.4 NA2.17]
ϕ_{LT} = 0.5[1 + α_{LT}($\bar{\lambda}_{LT}$ - $\bar{\lambda}_{LT,0}$) + $\beta\bar{\lambda}_{LT}^2$] = 0.67
χ_{LT} = 1/[ϕ_{LT} + $\sqrt{}$(ϕ_{LT}^2-$\beta\bar{\lambda}_{LT}^2$)] = 0.91 [EC3 (6.56)]

$M_{b,y,Rd}$ = χ_{LT}.$M_{c,y,Rd}$/γ_M = 0.91 x 58.3/1.0 = 53.2 kNm

Bending about z-z (minor) axis: N/A

Summary:

$N_{Ed}/N_{min,b,Rd}$ = 68.3/608 = 0.112 [1]

$M_{y,Ed}/M_{bs}$ = 12.22/53.2 = 0.230 [2]

$M_{z,Ed}/M_{z,cb,Rd}$ = 0.000 [3] x 1.5 [NCCI SN048]

Sum of stress ratios [1] + [2] + 1.5 x [3] = **0.342** **OK** *subject to all SN048b criteria being complied with*

Beam: **Outbuilding Flat Roof** **Span: 4.1 m.**

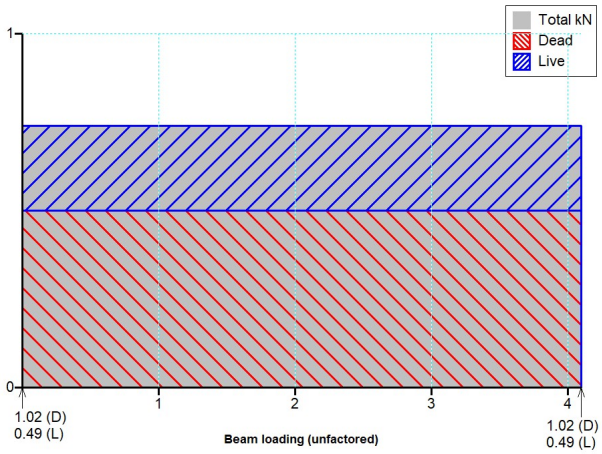
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.1	0		L	0.20	0.20	0.37
U G	Roof Dead	1.0*0.4	0		L	0.82	0.82	1.47
U QA	Roof Live	0.6*0.4	0		L	0.49	0.49	0.88
				Total load (unfactored):	**3.03 kN**	**1.52**	**1.52**	2.72
				Dead/Permanent (unfactored):	2.05 kN	1.02	1.02	1.84
				Live/Variable (unfactored):	0.98 kN	0.49	0.49	0.88
				Factored (6.10):	4.24 kN	2.12	2.12	

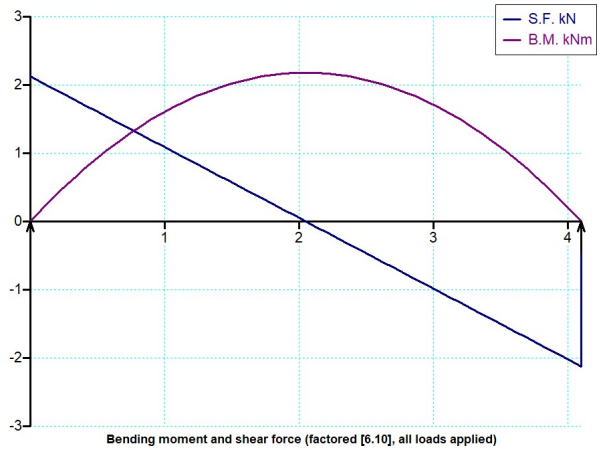
Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QA: Residential

Maximum B.M. = 2.17 kNm (6.10) at 2.05 m. from R1

Maximum S.F. = 2.12 kN (6.10) at R1

Mid-span deflections: Dead: 1.84 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 0.88 x 10^8/EI
Total: 2.72 x 10^8/EI





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

Use 50 x 200 C24 *4.2 kg/m approx*

$W_{el,y}$ = 333.3 cm^3 I_y = 3,333 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, g_M = 1.3 [EC5 UK Table NA.3]

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

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

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

Bending

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

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

Design bending stress, $s_{m,y,d}$ = 2.17 x 1000/333 = 6.52 N/mm^2 OK

Bending resistance = 12.18 x 333/1000 = 4.06 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, $f_{v,d} = f_{v,k}.k_{mod}.k_{sys}/g_m$ = 4.00 x 0.60 x 1.1/1.30 = 2.03 N/mm^2

Design shear stress, $s_{v,y,d}$ = 2.12 x 1000 x (3/2)/(33.5 x 200) = 0.48 N/mm^2 OK

Shear resistance = 2.03 x 33.5 x 200 x (2/3)/1000 = 9.07 kN

Deflection

Final deflection limit = 0.003L = 12.30 mm

Final deflection, $u_{fin} = Su_{inst}(1 + y_2.k_{def})$ [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 1.55 x 10^6/((11,000/16) x 50 x 200) = 0.27 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}	y_2	Final mm	
G:	1.84	5.02	0.60	1.00	8.03	
QA:	0.88	2.41	0.60	0.30	2.84	
Shear deflection:		0.27			0.40	
Total mm.		7.70			**11.27**	OK

Beam: **B7 - Steel Beam** **Span: 4.8 m.**

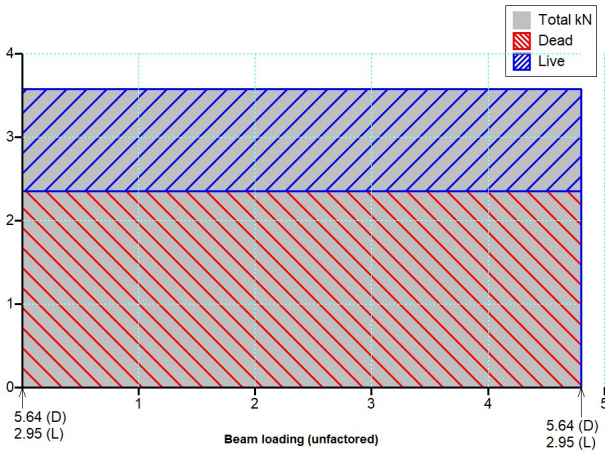
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	0.3	0		L	0.72	0.72	2.07
U G	Roof Dead	1.0*4.1*0.5	0		L	4.92	4.92	14.17
U QA	Roof Live	0.6*4.1*0.5	0		L	2.95	2.95	8.50
				Total load (unfactored):	**17.18 kN**	**8.59**	**8.59**	24.74
				Dead/Permanent (unfactored):	11.28 kN	5.64	5.64	16.24
				Live/Variable (unfactored):	5.90 kN	2.95	2.95	8.50
				Factored (6.10):	24.08 kN	12.04	12.04	

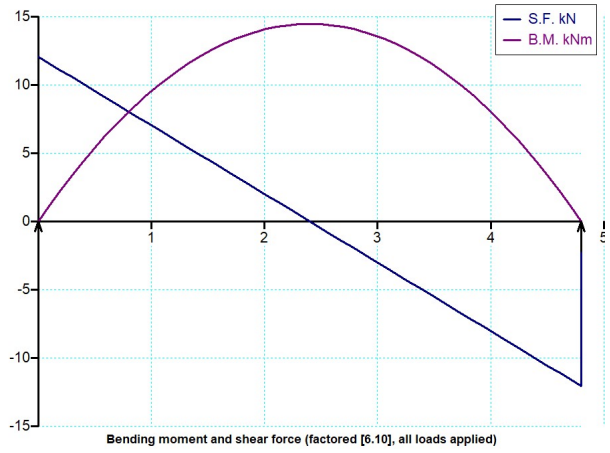
Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1
Load durations: G: Dead; Qx: Imposed: QA: Residential

Maximum B.M. = 14.45 kNm (6.10) at 2.40 m. from R1

Maximum S.F. = 12.04 kN (6.10) at R1

Mid-span deflections: Dead: 16.24 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 8.50 x 10^8/EI
Total: 24.74 x 10^8/EI





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} = 12.04 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, $V_{pl,Rd} = A_v.(f_y/\sqrt{3})/\gamma_{M0}$ = 1,456 x (355/$\sqrt{3}$)/(1.0 x 1000) = 299 kN (>=12.04) OK [EC3 6.2.6]

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

Bending

Moment resistance

Design moment, M_{Ed} = 14.45 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 restrained. Nominal torsional restraint. Both flanges free to rotate on plan (1.0L)

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

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

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

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

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

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

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

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

$M_{cr} = C_1(p^2EI_z/L_e^2)\sqrt{(I_w/I_z + L_e^2GI_T/p^2EI_z)}$ [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 = 81,000N/mm^2

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{l}_z$	$\bar{l}_{LT}$	f_{LT}	c_{LT}	$c_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-4.80	4.80	14.5	1.00d	1.00	63.6	1.982	1.324	1.314	0.511	0.511	111.5	57.0	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 = 8.50 x 1e8/(210,000 x 2,900) = 1.4 mm (L/3439) OK
TL deflection = 24.74 x 1e8/(210,000 x 2,900) = 4.1 mm (L/1182)

Bearings

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

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

R1 (12.04 kN) : 175 x 100 mm padstone

Stress under padstone = 12.04 x 1000/175 x 100 = 0.69 N/mm^2 OK

Padstone height is less than projection (69mm): reinforcement required - not checked

R2 as R1

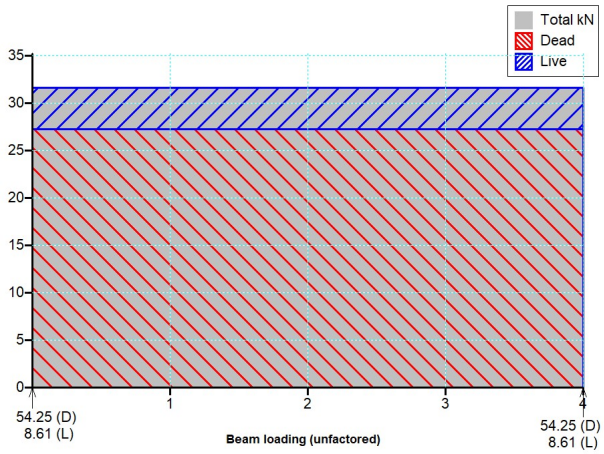
Beam: **GB1 - Ground Beam** **Span: 4.0 m.**

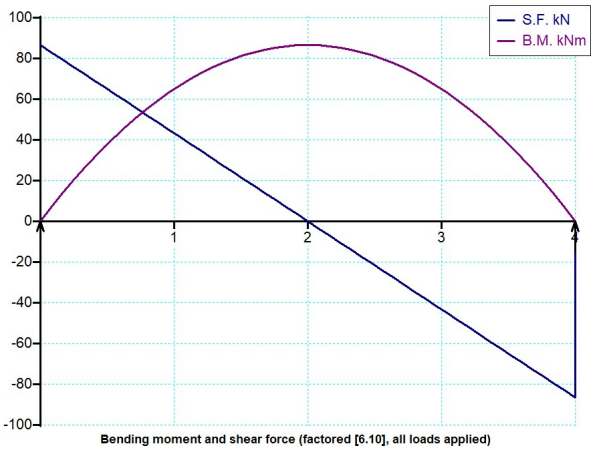
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	7.4	0		L	14.80	14.80	25
U G	Cavity Wall	3.5*3.0	0		L	21.00	21.00	35
U G	Floor Dead	3.5*4.1*0.5	0		L	14.35	14.35	24
U QA	Floor Live	1.5*4.1*0.5	0		L	6.15	6.15	10
U G	Roof Dead	1.0*4.1*0.5	0		L	4.10	4.10	7
U QA	Roof Live	0.6*4.1*0.5	0		L	2.46	2.46	4
				Total load (unfactored):	**125.7 kN**	**62.86**	**62.86**	105
				Dead/Permanent (unfactored):	108.5 kN	54.25	54.25	90
				Live/Variable (unfactored):	17.2 kN	8.61	8.61	14
				Factored (6.10):	172.3 kN	86.15	86.15	

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

Maximum B.M. = 86.2 kNm (6.10) at 2.00 m. from R1

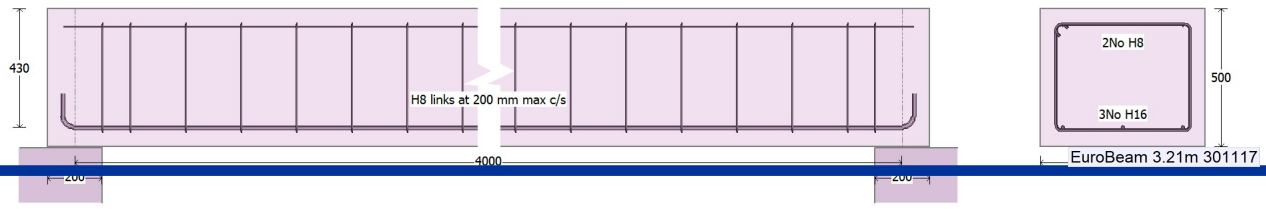
Maximum S.F. = 86.2 kN (6.10) at R1





RC beam calculation to BS EN1992.1.1

Member size: **600 x 500 C25/30**



Effective depth of beam, d = 430 mm

Bending

Design bending moment, M = 86.2 kNm

$K = M/(bd^2f_{ck}) = 86.2 \times 10^6/(600 \times 430^2 \times 25) = 0.031$

As K <= K', 0.168, no compression reinforcement required

$z = d(0.5 + \sqrt{}(0.25 - 0.881K)) = 430 \times (0.5 + \sqrt{}(0.25 - 0.881 \times 0.031)) = 418$ mm

Tension steel: area required, $A_s = M/(0.87.f_{yk}.z) = 86.2 \times 10^6/(0.87 \times 500 \times 418) = 474$ mm^2

Use **3 No H16**, area = 3 x 201 = 603 mm^2

Steel area limits: >= $0.0156f_{ck}^{2/3}$% = 0.13% = 400 mm^2 and <= 4% = 12,000 mm^2 OK

Shear

Check concrete shear capacity at face of support

Shear force, V_{Ed}, at face of supports = 81.8 kN (support width 200 mm)

Concrete shear capacity, $V_{Rd,c} = 0.12k(100.r_i.f_{ck})^{1/3}bd$ [EC2 6.2.2 Eq.(6.2a)]

$k = 1 + \sqrt{}(200/d) = 1 + \sqrt{}(200/430) = 1.68$

$r_i = A_{st}/(bd) = 603/(600 \times 430) = 0.002$ [6.2.2]

$V_{Rd,c} = 0.12 \times 1.68(100 \times 0.002 \times 25)^{1/3} \times 600 \times 430/1000 = 93.8$ kN

As $V_{Rd,c} >= V_{Ed}$ nominal shear reinforcement only required.

Use H8 links. Link area, A_{sw}, 2 H8 = 101 mm^2

$A_{sw}/s = 0.08.b.\sqrt{}f_{ck}/f_{yk} = 0.08 \times 600 \times \sqrt{}25/(1000 \times 500) = 0.480$

Maximum link spacing, s, = 101/0.480 = 209 mm

Provide H8 links at 200 mm max c/s

Deflection

Basic span/effective depth limits: r_1=0.35%: 30; r_1=0.50%: 20; r_1=1.5%: 14

Percent of main reinforcement, $r_1 = A_{st,reqd} \times 100/b.d = 474 \times 100/(600 \times 430) = 0.184\%$ [EC2 UK NA Table 5]

Raw span/depth limit = 30.0 Modification factor, $A_{st,prov}/A_{st,reqd}$ (but<=1.5) = 603/474 = 1.27

Adjusted span/depth limit = 30.0 x 1.27 = 38.2; Actual = 4.00 x 1000/430 = 9.30 OK

Check detailing

Minimum bar spacing needs to be greatest of f (16mm), 20mm or aggregate size + 5mm: not checked here

Minimum cover required, $c_{nom} = c_{min} + D_{c,dev}$ (taken as 10mm)

$c_{min} = \max(c_{min,b}; c_{min,dur}; 10)$

$c_{min,b}$: ensure adequate bond and compaction; $c_{min,dur}$: ensure adequate durability

$c_{min,b}$ = f = 16 mm

$c_{min,dur}$ = 35mm (exposure class: XC2, wet, rarely dry, many foundations, C25/30) [EC2 Table 4.1] [BS8500 Table A.4]

Required cover, c_{nom} = 35 + 10 = 45 mm

Cover to beam soffit = 500 - (430 + 16/2 + 8) = 54 mm. OK

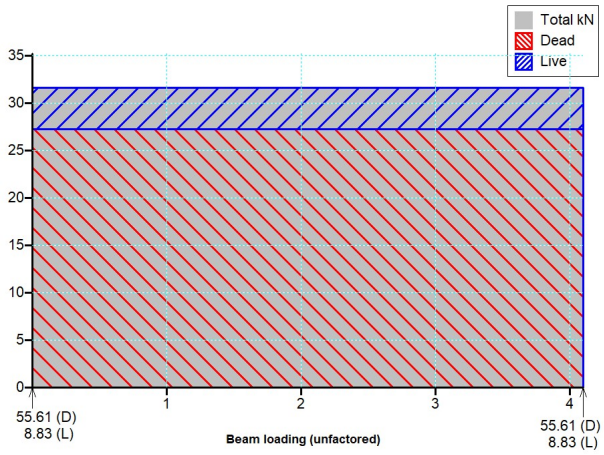
Beam: **GB2 - Ground Beam** **Span: 4.1 m.**

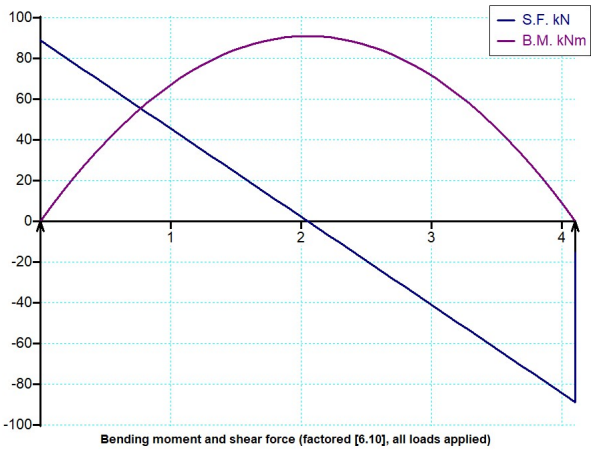
		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O	G	o.w.	7.4	0		L	15.17	15.17	27
U	G	Cavity Wall	3.5*3.0	0		L	21.52	21.52	39
U	G	Floor Dead	3.5*4.1*0.5	0		L	14.71	14.71	26
U	QA	Floor Live	1.5*4.1*0.5	0		L	6.30	6.30	11
U	G	Roof Dead	1.0*4.1*0.5	0		L	4.20	4.20	8
U	QA	Roof Live	0.6*4.1*0.5	0		L	2.52	2.52	5
					Total load (unfactored):	**128.9 kN**	**64.43**	**64.43**	116
					Dead/Permanent (unfactored):	111.2 kN	55.61	55.61	100
					Live/Variable (unfactored):	17.7 kN	8.83	8.83	16
					Factored (6.10):	176.6 kN	88.31	88.31	

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

Maximum B.M. = 90.5 kNm (6.10) at 2.05 m. from R1

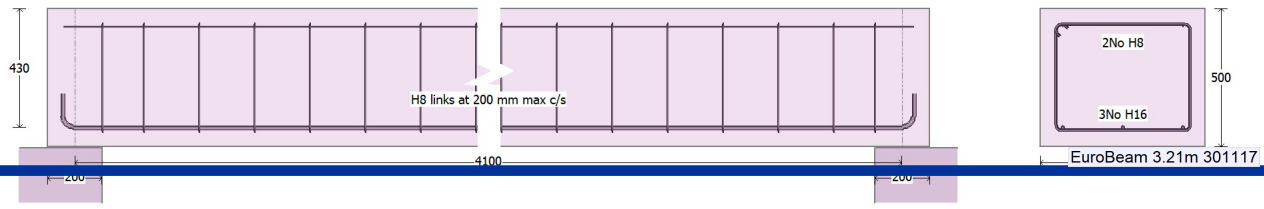
Maximum S.F. = 88.3 kN (6.10) at R1





RC beam calculation to BS EN1992.1.1

Member size: **600 x 500 C25/30**



Effective depth of beam, d = 430 mm

Bending

Design bending moment, M = 90.5 kNm

$K = M/(bd^2f_{ck}) = 90.5 \times 10^6/(600 \times 430^2 \times 25) = 0.033$

As K <= K', 0.168, no compression reinforcement required

$z = d(0.5 + \sqrt{}(0.25 - 0.881K)) = 430 \times (0.5 + \sqrt{}(0.25 - 0.881 \times 0.033)) = 417$ mm

Tension steel: area required, $A_s = M/(0.87.f_{yk}.z) = 90.5 \times 10^6/(0.87 \times 500 \times 417) = 499$ mm^2

Use **3 No H16**, area = 3 x 201 = 603 mm^2

Steel area limits: >= $0.0156f_{ck}^{2/3}$% = 0.13% = 400 mm^2 and <= 4% = 12,000 mm^2 OK

Shear

Check concrete shear capacity at face of support

Shear force, V_{Ed}, at face of supports = 84.0 kN (support width 200 mm)

Concrete shear capacity, $V_{Rd,c} = 0.12k(100.r_i.f_{ck})^{1/3}bd$ [EC2 6.2.2 Eq.(6.2a)]

$k = 1 + \sqrt{}(200/d) = 1 + \sqrt{}(200/430) = 1.68$

$r_i = A_{st}/(bd) = 603/(600 \times 430) = 0.002$ [6.2.2]

$V_{Rd,c} = 0.12 \times 1.68(100 \times 0.002 \times 25)^{1/3} \times 600 \times 430/1000 = 93.8$ kN

As $V_{Rd,c} >= V_{Ed}$ nominal shear reinforcement only required.

Use H8 links. Link area, A_{sw}, 2 H8 = 101 mm^2

$A_{sw}/s = 0.08.b.\sqrt{}f_{ck}/f_{yk} = 0.08 \times 600 \times \sqrt{}25/(1000 \times 500) = 0.480$

Maximum link spacing, s, = 101/0.480 = 209 mm

Provide H8 links at 200 mm max c/s

Deflection

Basic span/effective depth limits: r_1=0.35%: 30; r_1=0.50%: 20; r_1=1.5%: 14

Percent of main reinforcement, $r_1 = A_{st,reqd} \times 100/b.d = 499 \times 100/(600 \times 430) = 0.193\%$ [EC2 UK NA Table 5]

Raw span/depth limit = 30.0 Modification factor, $A_{st,prov}/A_{st,reqd}$ (but<=1.5) = 603/499 = 1.21

Adjusted span/depth limit = 30.0 x 1.21 = 36.3; Actual = 4.10 x 1000/430 = 9.53 OK

Check detailing

Minimum bar spacing needs to be greatest of f (16mm), 20mm or aggregate size + 5mm: not checked here

Minimum cover required, $c_{nom} = c_{min} + D_{c,dev}$ (taken as 10mm)

$c_{min} = \max(c_{min,b}; c_{min,dur}; 10)$

$c_{min,b}$: ensure adequate bond and compaction; $c_{min,dur}$: ensure adequate durability

$c_{min,b}$ = f = 16 mm

$c_{min,dur}$ = 35mm (exposure class: XC2, wet, rarely dry, many foundations, C25/30) [EC2 Table 4.1] [BS8500 Table A.4]

Required cover, c_{nom} = 35 + 10 = 45 mm

Cover to beam soffit = 500 - (430 + 16/2 + 8) = 54 mm. OK

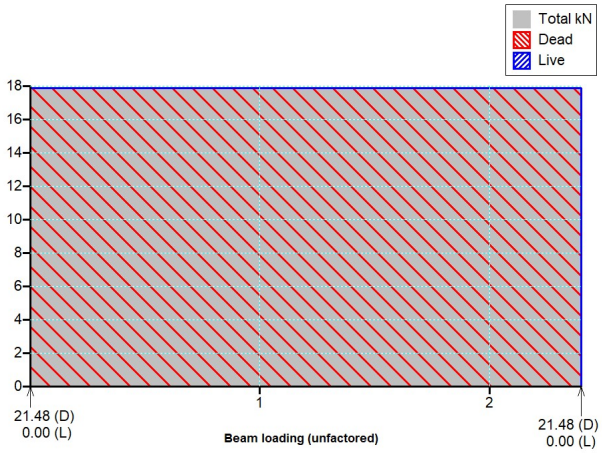
Beam: **GB3 - Ground Beam** **Span: 2.4 m.**

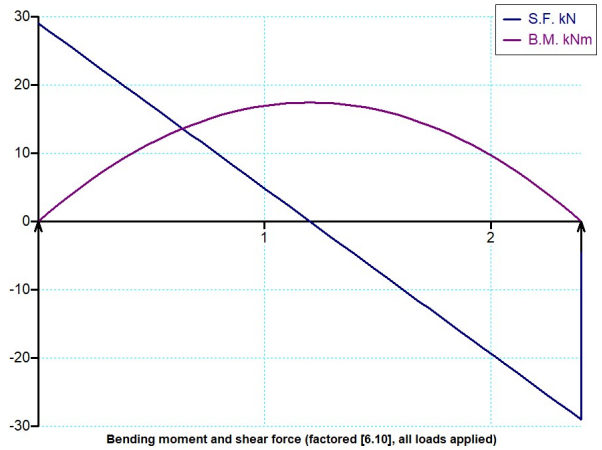
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	7.4	0		L	8.88	8.88	3.20
U G	Cavity Wall	3.5*3.0	0		L	12.60	12.60	4.54
				Total load (unfactored):	**43.0 kN**	**21.48**	**21.48**	7.73
				Dead/Permanent (unfactored):	43.0 kN	21.48	21.48	7.73
				Live/Variable (unfactored):	0.0 kN	0.00	0.00	0.00
				Factored (6.10):	58.0 kN	29.00	29.00	

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

Maximum B.M. = 17.40 kNm (6.10) at 1.20 m. from R1

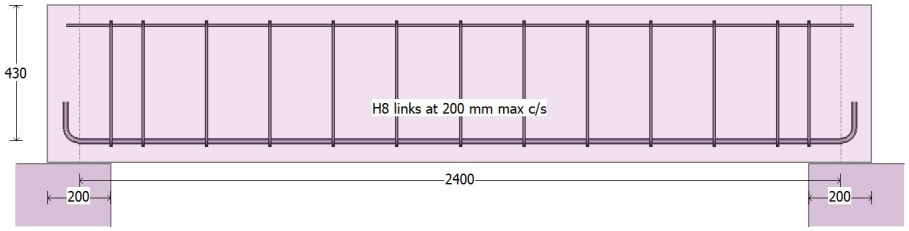
Maximum S.F. = 29.00 kN (6.10) at R1

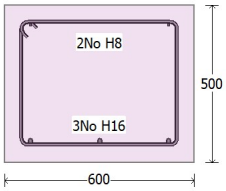




RC beam calculation to BS EN1992.1.1

Member size: **600 x 500 C25/30**





Effective depth of beam, d = 430 mm

Bending

Design bending moment, M = 17.40 kNm

$K = M/(bd^2f_{ck}) = 17.40 \times 10^6/(600 \times 430^2 \times 25) = 0.006$

As K <= K', 0.168, no compression reinforcement required

$z = d(0.5 + \sqrt{}(0.25 - 0.881K)) = 430 \times (0.5 + \sqrt{}(0.25 - 0.881 \times 0.006)) = 428$ mm

Tension steel: area required, $A_s = M/(0.87.f_{yk}.z) = 17.40 \times 10^6/(0.87 \times 500 \times 428) = 93.5$ mm^2

Use **3 No H16**, area = 3 x 201 = 603 mm^2

Steel area limits: $>= 0.0156f_{ck}^{2/3}\% = 0.13\% = 400$ mm^2 and <= 4% = 12,000 mm^2 OK

Shear

Check concrete shear capacity at face of support

Shear force, V_{Ed}, at face of supports = 26.6 kN (support width 200 mm)

Concrete shear capacity, $V_{Rd,c} = 0.12k(100.r_i.f_{ck})^{1/3}bd$ [EC2 6.2.2 Eq.(6.2a)]

$k = 1 + \sqrt{}(200/d) = 1 + \sqrt{}(200/430) = 1.68$

$r_i = A_{sl}/(bd) = 603/(600 \times 430) = 0.002$ [6.2.2]

$V_{Rd,c} = 0.12 \times 1.68(100 \times 0.002 \times 25)^{1/3} \times 600 \times 430/1000 = 93.8$ kN

As $V_{Rd,c} >= V_{Ed}$ nominal shear reinforcement only required.

Use H8 links. Link area, A_{sw}, 2 H8 = 101 mm^2

$A_{sw}/s = 0.08.b.\sqrt{}f_{ck}/f_{yk} = 0.08 \times 600 \times \sqrt{}25/(1000 \times 500) = 0.480$

Maximum link spacing, s, = 101/0.480 = 209 mm

Provide H8 links at 200 mm max c/s

Deflection

Basic span/effective depth limits: r_1=0.35%: 30; r_1=0.50%: 20; r_1=1.5%: 14

Percent of main reinforcement, $r_1 = A_{st,reqd} \times 100/b.d = 400 \times 100/(600 \times 430) = 0.155\%$ [EC2 UK NA Table 5]

Raw span/depth limit = 30.0 Modification factor, $A_{st,prov}/A_{st,reqd}$ (but<=1.5) = 603/400 = 1.51 so 1.5

Adjusted span/depth limit = 30.0 x 1.50 = 45.0; Actual = 2.40 x 1000/430 = 5.58 OK

Check detailing

Minimum bar spacing needs to be greatest of f (16mm), 20mm or aggregate size + 5mm: not checked here

Minimum cover required, $c_{nom} = c_{min} + D_{c,dev}$ (taken as 10mm)

$c_{min} = \max(c_{min,b}; c_{min,dur}; 10)$

$c_{min,b}$: ensure adequate bond and compaction; $c_{min,dur}$: ensure adequate durability

$c_{min,b}$ = f = 16 mm

$c_{min,dur}$ = 35mm (exposure class: XC2, wet, rarely dry, many foundations, C25/30) [EC2 Table 4.1] [BS8500 Table A.4]

Required cover, c_{nom} = 35 + 10 = 45 mm

Cover to beam soffit = 500 - (430 + 16/2 + 8) = 54 mm. OK

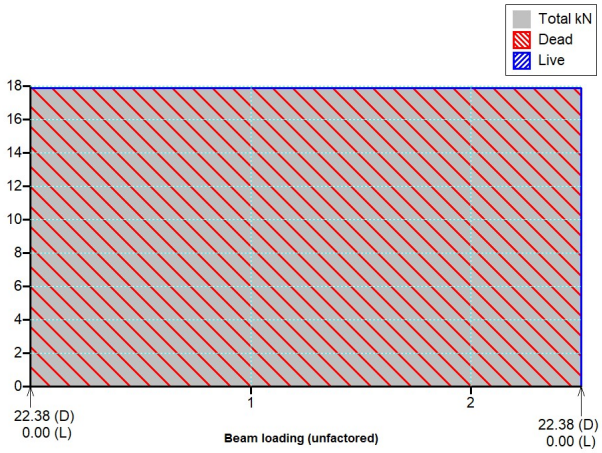
Beam: **GB4 - Ground Beam** **Span: 2.5 m.**

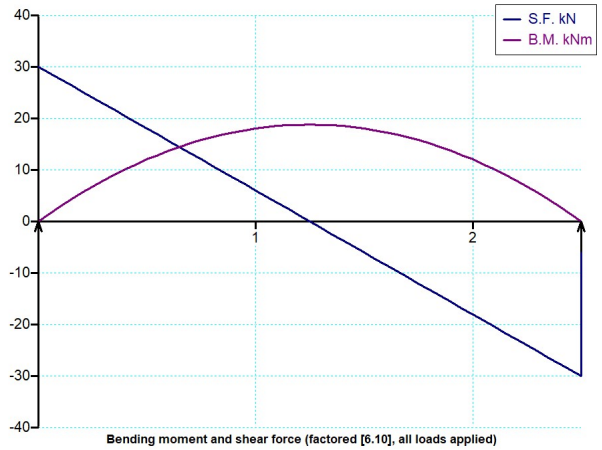
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	7.4	0		L	9.25	9.25	3.76
U G	Cavity Wall	3.5*3.0	0		L	13.13	13.13	5.34
				Total load (unfactored):	**44.8 kN**	**22.38**	**22.38**	9.10
				Dead/Permanent (unfactored):	44.8 kN	22.38	22.38	9.10
				Live/Variable (unfactored):	0.0 kN	0.00	0.00	0.00
				Factored (6.10):	60.4 kN	30.21	30.21	

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

Maximum B.M. = 18.88 kNm (6.10) at 1.25 m. from R1

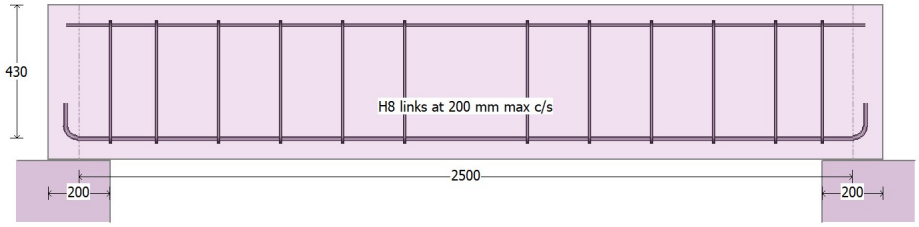
Maximum S.F. = 30.21 kN (6.10) at R1

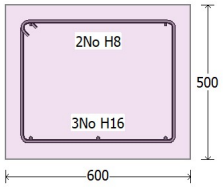




RC beam calculation to BS EN1992.1.1

Member size: **600 x 500 C25/30**





Effective depth of beam, d = 430 mm

Bending

Design bending moment, M = 18.88 kNm

$K = M/(bd^2f_{ck}) = 18.88 \times 10^6/(600 \times 430^2 \times 25) = 0.007$

As K <= K', 0.168, no compression reinforcement required

$z = d(0.5 + \sqrt{}(0.25 - 0.881K)) = 430 \times (0.5 + \sqrt{}(0.25 - 0.881 \times 0.007)) = 427$ mm

Tension steel: area required, $A_s = M/(0.87.f_{yk}.z) = 18.88 \times 10^6/(0.87 \times 500 \times 427) = 102$ mm^2

Use **3 No H16**, area = 3 x 201 = 603 mm^2

Steel area limits: >= $0.0156f_{ck}^{2/3}$% = 0.13% = 400 mm^2 and <= 4% = 12,000 mm^2 OK

Shear

Check concrete shear capacity at face of support

Shear force, V_{Ed}, at face of supports = 27.8 kN (support width 200 mm)

Concrete shear capacity, $V_{Rd,c} = 0.12k(100.r_i.f_{ck})^{1/3}bd$ [EC2 6.2.2 Eq.(6.2a)]

$k = 1 + \sqrt{}(200/d) = 1 + \sqrt{}(200/430) = 1.68$

$r_i = A_{sl}/(bd) = 603/(600 \times 430) = 0.002$ [6.2.2]

$V_{Rd,c} = 0.12 \times 1.68(100 \times 0.002 \times 25)^{1/3} \times 600 \times 430/1000 = 93.8$ kN

As $V_{Rd,c}$ >= V_{Ed} nominal shear reinforcement only required.

Use H8 links. Link area, A_{sw}, 2 H8 = 101 mm^2

$A_{sw}/s = 0.08.b.\sqrt{}f_{ck}/f_{yk} = 0.08 \times 600 \times \sqrt{}25/(1000 \times 500) = 0.480$

Maximum link spacing, s, = 101/0.480 = 209 mm

Provide H8 links at 200 mm max c/s

Deflection

Basic span/effective depth limits: r_1=0.35%: 30; r_1=0.50%: 20; r_1=1.5%: 14

Percent of main reinforcement, $r_1 = A_{st,reqd} \times 100/b.d = 400 \times 100/(600 \times 430) = 0.155\%$ [EC2 UK NA Table 5]

Raw span/depth limit = 30.0 Modification factor, $A_{st,prov}/A_{st,reqd}$ (but<=1.5) = 603/400 = 1.51 so 1.5

Adjusted span/depth limit = 30.0 x 1.50 = 45.0; Actual = 2.50 x 1000/430 = 5.81 OK

Check detailing

Minimum bar spacing needs to be greatest of f (16mm), 20mm or aggregate size + 5mm: not checked here

Minimum cover required, $c_{nom} = c_{min} + D_{c,dev}$ (taken as 10mm)

$c_{min} = \max(c_{min,b}; c_{min,dur}; 10)$

$c_{min,b}$: ensure adequate bond and compaction; $c_{min,dur}$: ensure adequate durability

$c_{min,b}$ = f = 16 mm

$c_{min,dur}$ = 35mm (exposure class: XC2, wet, rarely dry, many foundations, C25/30) [EC2 Table 4.1] [BS8500 Table A.4]

Required cover, c_{nom} = 35 + 10 = 45 mm

Cover to beam soffit = 500 - (430 + 16/2 + 8) = 54 mm. OK

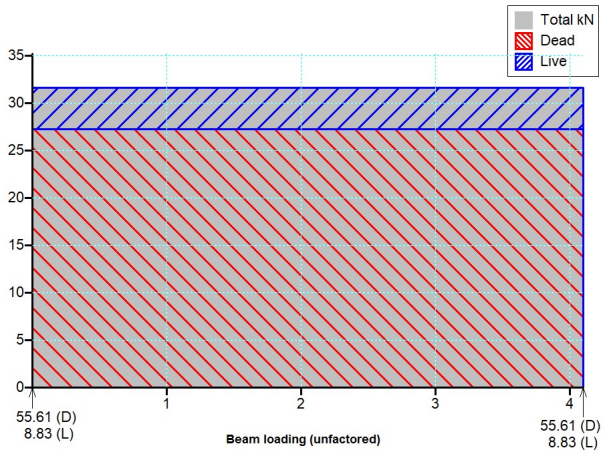
Beam: **GB5 - Ground Beam** **Span: 4.1 m.**

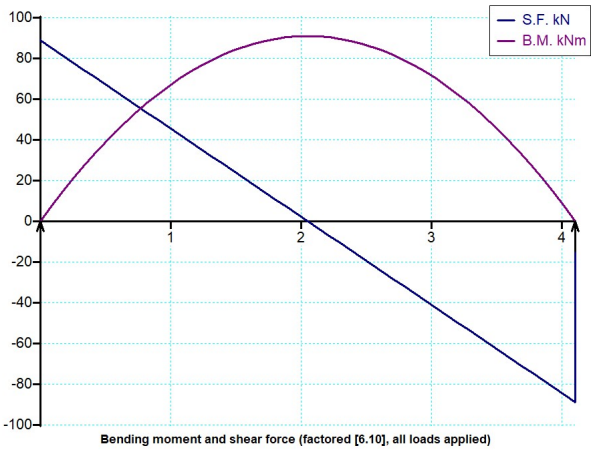
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	7.4	0		L	15.17	15.17	27
U G	Cavity Wall	3.5*3.0	0		L	21.52	21.52	39
U G	Floor Dead	3.5*4.1*0.5	0		L	14.71	14.71	26
U QA	Floor Live	1.5*4.1*0.5	0		L	6.30	6.30	11
U G	Roof Dead	1.0*4.1*0.5	0		L	4.20	4.20	8
U QA	Roof Live	0.6*4.1*0.5	0		L	2.52	2.52	5
				Total load (unfactored):	**128.9 kN**	**64.43**	**64.43**	116
				Dead/Permanent (unfactored):	111.2 kN	55.61	55.61	100
				Live/Variable (unfactored):	17.7 kN	8.83	8.83	16
				Factored (6.10):	176.6 kN	88.31	88.31	

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

Maximum B.M. = 90.5 kNm (6.10) at 2.05 m. from R1

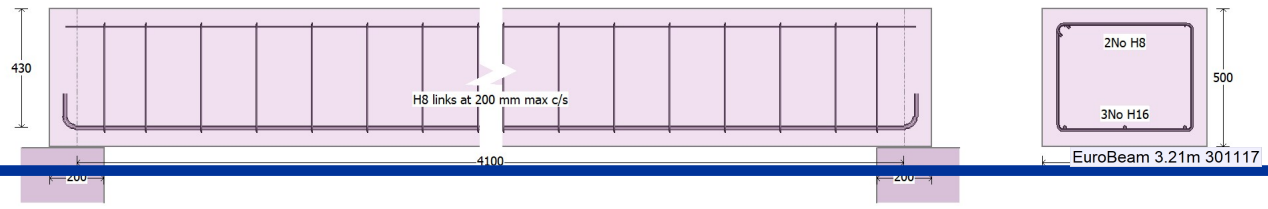
Maximum S.F. = 88.3 kN (6.10) at R1





RC beam calculation to BS EN1992.1.1

Member size: **600 x 500 C25/30**



Effective depth of beam, d = 430 mm

Bending

Design bending moment, M = 90.5 kNm

$K = M/(bd^2f_{ck}) = 90.5 \times 10^6/(600 \times 430^2 \times 25) = 0.033$

As K <= K', 0.168, no compression reinforcement required

$z = d(0.5 + \sqrt{}(0.25 - 0.881K)) = 430 \times (0.5 + \sqrt{}(0.25 - 0.881 \times 0.033)) = 417$ mm

Tension steel: area required, $A_s = M/(0.87.f_{yk}.z) = 90.5 \times 10^6/(0.87 \times 500 \times 417) = 499$ mm^2

Use **3 No H16**, area = 3 x 201 = 603 mm^2

Steel area limits: >= $0.0156f_{ck}^{2/3}$% = 0.13% = 400 mm^2 and <= 4% = 12,000 mm^2 OK

Shear

Check concrete shear capacity at face of support

Shear force, V_{Ed}, at face of supports = 84.0 kN (support width 200 mm)

Concrete shear capacity, $V_{Rd,c} = 0.12k(100.r_i.f_{ck})^{1/3}bd$ [EC2 6.2.2 Eq.(6.2a)]

$k = 1 + \sqrt{}(200/d) = 1 + \sqrt{}(200/430) = 1.68$

$r_i = A_{st}/(bd) = 603/(600 \times 430) = 0.002$ [6.2.2]

$V_{Rd,c} = 0.12 \times 1.68(100 \times 0.002 \times 25)^{1/3} \times 600 \times 430/1000 = 93.8$ kN

As $V_{Rd,c} >= V_{Ed}$ nominal shear reinforcement only required.

Use H8 links. Link area, A_{sw}, 2 H8 = 101 mm^2

$A_{sw}/s = 0.08.b.\sqrt{}f_{ck}/f_{yk} = 0.08 \times 600 \times \sqrt{}25/(1000 \times 500) = 0.480$

Maximum link spacing, s, = 101/0.480 = 209 mm

Provide H8 links at 200 mm max c/s

Deflection

Basic span/effective depth limits: r_1=0.35%: 30; r_1=0.50%: 20; r_1=1.5%: 14

Percent of main reinforcement, $r_1 = A_{st,reqd} \times 100/b.d = 499 \times 100/(600 \times 430) = 0.193\%$ [EC2 UK NA Table 5]

Raw span/depth limit = 30.0 Modification factor, $A_{st,prov}/A_{st,reqd}$ (but<=1.5) = 603/499 = 1.21

Adjusted span/depth limit = 30.0 x 1.21 = 36.3; Actual = 4.10 x 1000/430 = 9.53 OK

Check detailing

Minimum bar spacing needs to be greatest of f (16mm), 20mm or aggregate size + 5mm: not checked here

Minimum cover required, $c_{nom} = c_{min} + D_{c,dev}$ (taken as 10mm)

$c_{min} = \max(c_{min,b}; c_{min,dur}; 10)$

$c_{min,b}$: ensure adequate bond and compaction; $c_{min,dur}$: ensure adequate durability

$c_{min,b}$ = f = 16 mm

$c_{min,dur}$ = 35mm (exposure class: XC2, wet, rarely dry, many foundations, C25/30) [EC2 Table 4.1] [BS8500 Table A.4]

Required cover, c_{nom} = 35 + 10 = 45 mm

Cover to beam soffit = 500 - (430 + 16/2 + 8) = 54 mm. OK

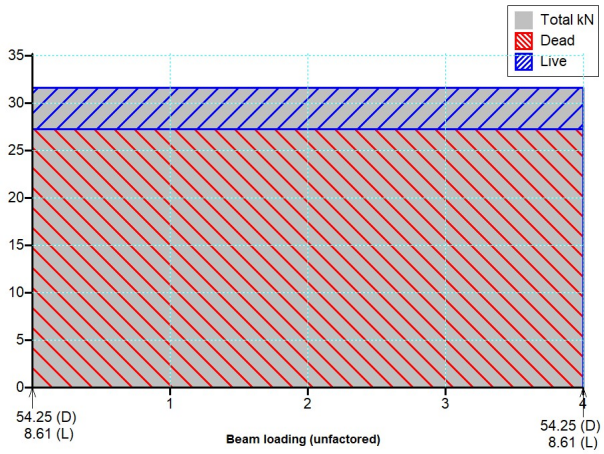
Beam: **GB6 - Ground Beam** **Span: 4.0 m.**

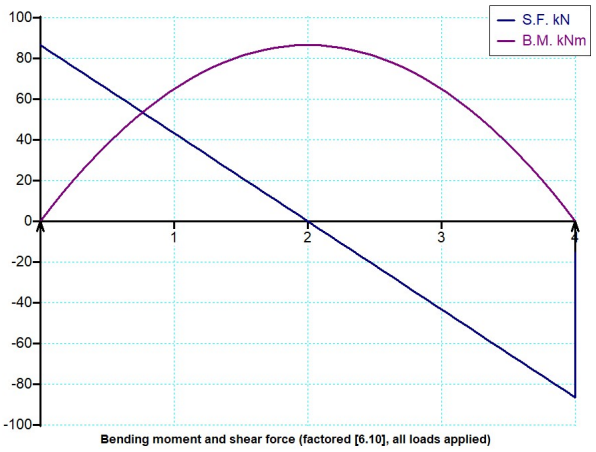
		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O	G	o.w.	7.4	0		L	14.80	14.80	25
U	G	Cavity Wall	3.5*3.0	0		L	21.00	21.00	35
U	G	Floor Dead	3.5*4.1*0.5	0		L	14.35	14.35	24
U	QA	Floor Live	1.5*4.1*0.5	0		L	6.15	6.15	10
U	G	Roof Dead	1.0*4.1*0.5	0		L	4.10	4.10	7
U	QA	Roof Live	0.6*4.1*0.5	0		L	2.46	2.46	4
					Total load (unfactored):	**125.7 kN**	**62.86**	**62.86**	105
					Dead/Permanent (unfactored):	108.5 kN	54.25	54.25	90
					Live/Variable (unfactored):	17.2 kN	8.61	8.61	14
					Factored (6.10):	172.3 kN	86.15	86.15	

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

Maximum B.M. = 86.2 kNm (6.10) at 2.00 m. from R1

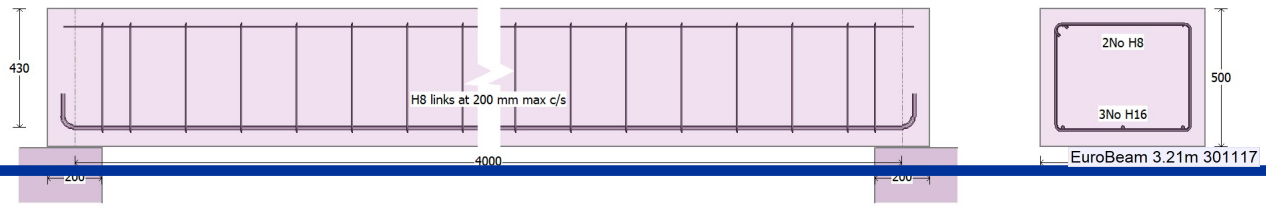
Maximum S.F. = 86.2 kN (6.10) at R1





RC beam calculation to BS EN1992.1.1

Member size: **600 x 500 C25/30**



Effective depth of beam, d = 430 mm

Bending

Design bending moment, M = 86.2 kNm

$K = M/(bd^2f_{ck}) = 86.2 \times 10^6/(600 \times 430^2 \times 25) = 0.031$

As K <= K', 0.168, no compression reinforcement required

$z = d(0.5 + \sqrt{}(0.25 - 0.881K)) = 430 \times (0.5 + \sqrt{}(0.25 - 0.881 \times 0.031)) = 418$ mm

Tension steel: area required, $A_s = M/(0.87.f_{yk}.z) = 86.2 \times 10^6/(0.87 \times 500 \times 418) = 474$ mm^2

Use **3 No H16**, area = 3 x 201 = 603 mm^2

Steel area limits: >= $0.0156f_{ck}^{2/3}$% = 0.13% = 400 mm^2 and <= 4% = 12,000 mm^2 OK

Shear

Check concrete shear capacity at face of support

Shear force, V_{Ed}, at face of supports = 81.8 kN (support width 200 mm)

Concrete shear capacity, $V_{Rd,c} = 0.12k(100.r_i.f_{ck})^{1/3}bd$ [EC2 6.2.2 Eq.(6.2a)]

$k = 1 + \sqrt{}(200/d) = 1 + \sqrt{}(200/430) = 1.68$

$r_i = A_{st}/(bd) = 603/(600 \times 430) = 0.002$ [6.2.2]

$V_{Rd,c} = 0.12 \times 1.68(100 \times 0.002 \times 25)^{1/3} \times 600 \times 430/1000 = 93.8$ kN

As $V_{Rd,c} >= V_{Ed}$ nominal shear reinforcement only required.

Use H8 links. Link area, A_{sw}, 2 H8 = 101 mm^2

$A_{sw}/s = 0.08.b.\sqrt{}f_{ck}/f_{yk} = 0.08 \times 600 \times \sqrt{}25/(1000 \times 500) = 0.480$

Maximum link spacing, s, = 101/0.480 = 209 mm

Provide H8 links at 200 mm max c/s

Deflection

Basic span/effective depth limits: r_1=0.35%: 30; r_1=0.50%: 20; r_1=1.5%: 14

Percent of main reinforcement, $r_1 = A_{st,reqd} \times 100/b.d = 474 \times 100/(600 \times 430) = 0.184\%$ [EC2 UK NA Table 5]

Raw span/depth limit = 30.0 Modification factor, $A_{st,prov}/A_{st,reqd}$ (but<=1.5) = 603/474 = 1.27

Adjusted span/depth limit = 30.0 x 1.27 = 38.2; Actual = 4.00 x 1000/430 = 9.30 OK

Check detailing

Minimum bar spacing needs to be greatest of f (16mm), 20mm or aggregate size + 5mm: not checked here

Minimum cover required, $c_{nom} = c_{min} + D_{c,dev}$ (taken as 10mm)

$c_{min} = \max(c_{min,b}; c_{min,dur}; 10)$

$c_{min,b}$: ensure adequate bond and compaction; $c_{min,dur}$: ensure adequate durability

$c_{min,b}$ = f = 16 mm

$c_{min,dur}$ = 35mm (exposure class: XC2, wet, rarely dry, many foundations, C25/30) [EC2 Table 4.1] [BS8500 Table A.4]

Required cover, c_{nom} = 35 + 10 = 45 mm

Cover to beam soffit = 500 - (430 + 16/2 + 8) = 54 mm. OK

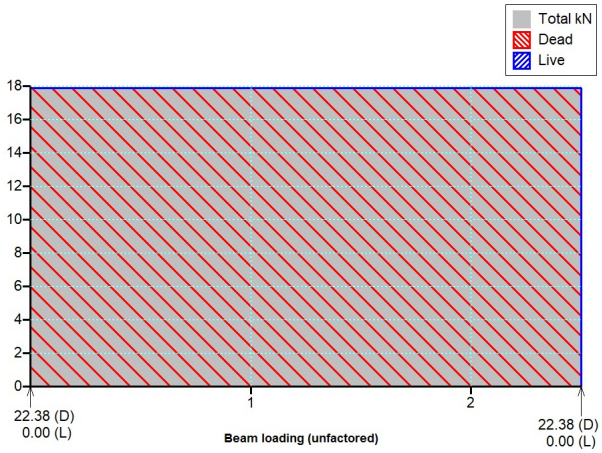
Beam: **GB7 - Ground Beam** **Span: 2.5 m.**

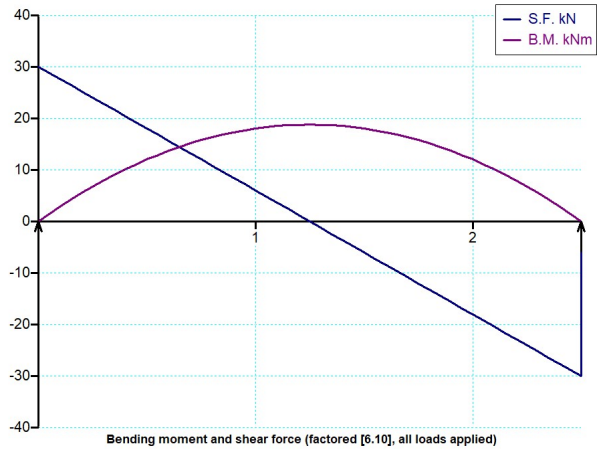
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	7.4	0		L	9.25	9.25	3.76
U G	Cavity Wall	3.5*3.0	0		L	13.13	13.13	5.34
				Total load (unfactored):	**44.8 kN**	**22.38**	**22.38**	9.10
				Dead/Permanent (unfactored):	44.8 kN	22.38	22.38	9.10
				Live/Variable (unfactored):	0.0 kN	0.00	0.00	0.00
				Factored (6.10):	60.4 kN	30.21	30.21	

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

Maximum B.M. = 18.88 kNm (6.10) at 1.25 m. from R1

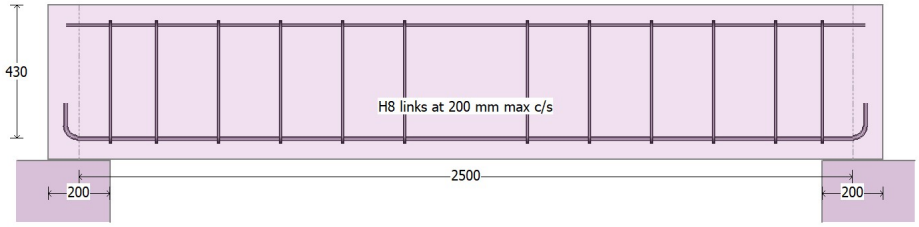
Maximum S.F. = 30.21 kN (6.10) at R1

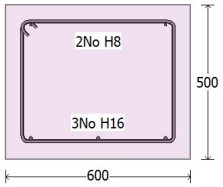




RC beam calculation to BS EN1992.1.1

Member size: **600 x 500 C25/30**





Effective depth of beam, d = 430 mm

Bending

Design bending moment, M = 18.88 kNm

$K = M/(bd^2f_{ck}) = 18.88 \times 10^6/(600 \times 430^2 \times 25) = 0.007$

As K <= K', 0.168, no compression reinforcement required

$z = d(0.5 + \sqrt{}(0.25 - 0.881K)) = 430 \times (0.5 + \sqrt{}(0.25 - 0.881 \times 0.007)) = 427$ mm

Tension steel: area required, $A_s = M/(0.87.f_{yk}.z) = 18.88 \times 10^6/(0.87 \times 500 \times 427) = 102$ mm^2

Use **3 No H16**, area = 3 x 201 = 603 mm^2

Steel area limits: >= $0.0156f_{ck}^{2/3}$% = 0.13% = 400 mm^2 and <= 4% = 12,000 mm^2 OK

Shear

Check concrete shear capacity at face of support

Shear force, V_{Ed}, at face of supports = 27.8 kN (support width 200 mm)

Concrete shear capacity, $V_{Rd,c} = 0.12k(100.r_i.f_{ck})^{1/3}bd$ [EC2 6.2.2 Eq.(6.2a)]

$k = 1 + \sqrt{}(200/d) = 1 + \sqrt{}(200/430) = 1.68$

$r_i = A_{sl}/(bd) = 603/(600 \times 430) = 0.002$ [6.2.2]

$V_{Rd,c} = 0.12 \times 1.68(100 \times 0.002 \times 25)^{1/3} \times 600 \times 430/1000 = 93.8$ kN

As $V_{Rd,c}$ >= V_{Ed} nominal shear reinforcement only required.

Use H8 links. Link area, A_{sw}, 2 H8 = 101 mm^2

$A_{sw}/s = 0.08.b.\sqrt{}f_{ck}/f_{yk} = 0.08 \times 600 \times \sqrt{}25/(1000 \times 500) = 0.480$

Maximum link spacing, s, = 101/0.480 = 209 mm

Provide H8 links at 200 mm max c/s

Deflection

Basic span/effective depth limits: r_1=0.35%: 30; r_1=0.50%: 20; r_1=1.5%: 14

Percent of main reinforcement, $r_1 = A_{st,reqd} \times 100/b.d = 400 \times 100/(600 \times 430) = 0.155\%$ [EC2 UK NA Table 5]

Raw span/depth limit = 30.0 Modification factor, $A_{st,prov}/A_{st,reqd}$ (but<=1.5) = 603/400 = 1.51 so 1.5

Adjusted span/depth limit = 30.0 x 1.50 = 45.0; Actual = 2.50 x 1000/430 = 5.81 OK

Check detailing

Minimum bar spacing needs to be greatest of f (16mm), 20mm or aggregate size + 5mm: not checked here

Minimum cover required, $c_{nom} = c_{min} + D_{c,dev}$ (taken as 10mm)

$c_{min} = \max(c_{min,b}; c_{min,dur}; 10)$

$c_{min,b}$: ensure adequate bond and compaction; $c_{min,dur}$: ensure adequate durability

$c_{min,b}$ = f = 16 mm

$c_{min,dur}$ = 35mm (exposure class: XC2, wet, rarely dry, many foundations, C25/30) [EC2 Table 4.1] [BS8500 Table A.4]

Required cover, c_{nom} = 35 + 10 = 45 mm

Cover to beam soffit = 500 - (430 + 16/2 + 8) = 54 mm. OK

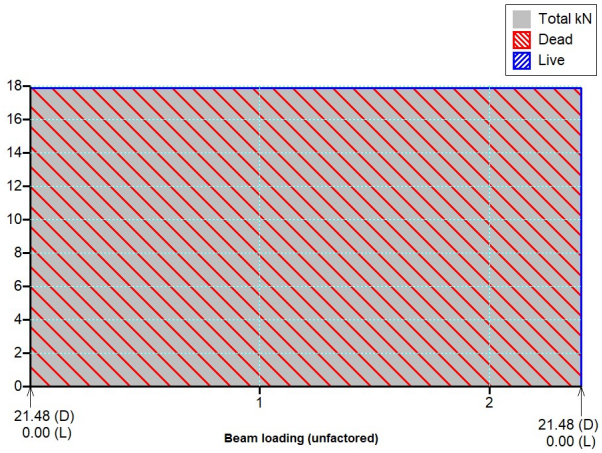
Beam: **GB8 - Ground Beam** **Span: 2.4 m.**

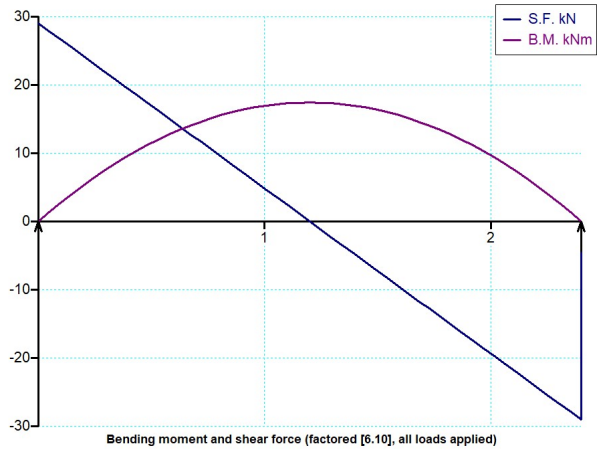
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O G	o.w.	7.4	0		L	8.88	8.88	3.20
U G	Cavity Wall	3.5*3.0	0		L	12.60	12.60	4.54
				Total load (unfactored):	**43.0 kN**	**21.48**	**21.48**	7.73
				Dead/Permanent (unfactored):	43.0 kN	21.48	21.48	7.73
				Live/Variable (unfactored):	0.0 kN	0.00	0.00	0.00
				Factored (6.10):	58.0 kN	29.00	29.00	

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

Maximum B.M. = 17.40 kNm (6.10) at 1.20 m. from R1

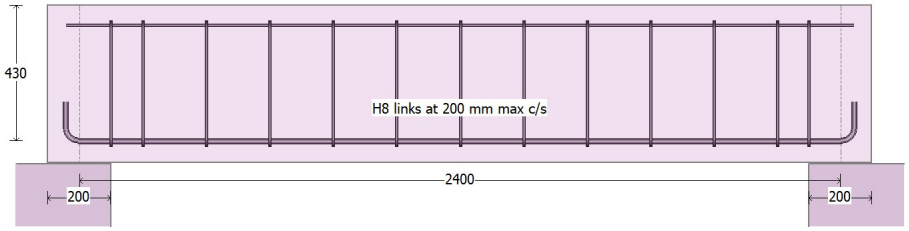
Maximum S.F. = 29.00 kN (6.10) at R1

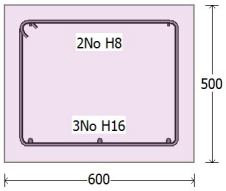




RC beam calculation to BS EN1992.1.1

Member size: **600 x 500 C25/30**





Effective depth of beam, d = 430 mm

Bending

Design bending moment, M = 17.40 kNm

$K = M/(bd^2f_{ck}) = 17.40 \times 10^6/(600 \times 430^2 \times 25) = 0.006$

As K <= K', 0.168, no compression reinforcement required

$z = d(0.5 + \sqrt{}(0.25 - 0.881K)) = 430 \times (0.5 + \sqrt{}(0.25 - 0.881 \times 0.006)) = 428$ mm

$A_s = M/(0.87.f_{yk}.z) = 17.40 \times 10^6/(0.87 \times 500 \times 428) = 93.5$ mm^2

Use **3 No H16**, area = 3 x 201 = 603 mm^2

Steel area limits: $>= 0.0156f_{ck}^{2/3}\% = 0.13\% = 400$ mm^2 and <= 4% = 12,000 mm^2 OK

Shear

Check concrete shear capacity at face of support

Shear force, V_{Ed}, at face of supports = 26.6 kN (support width 200 mm)

Concrete shear capacity, $V_{Rd,c} = 0.12k(100.r_i.f_{ck})^{1/3}bd$ [EC2 6.2.2 Eq.(6.2a)]

$k = 1 + \sqrt{}(200/d) = 1 + \sqrt{}(200/430) = 1.68$

$r_i = A_{sl}/(bd) = 603/(600 \times 430) = 0.002$ [6.2.2]

$V_{Rd,c} = 0.12 \times 1.68(100 \times 0.002 \times 25)^{1/3} \times 600 \times 430/1000 = 93.8$ kN

As $V_{Rd,c} >= V_{Ed}$ nominal shear reinforcement only required.

Use H8 links. Link area, A_{sw}, 2 H8 = 101 mm^2

$A_{sw}/s = 0.08.b.\sqrt{}f_{ck}/f_{yk} = 0.08 \times 600 \times \sqrt{}25/(1000 \times 500) = 0.480$

Maximum link spacing, s, = 101/0.480 = 209 mm

Provide H8 links at 200 mm max c/s

Deflection

Basic span/effective depth limits: r_1=0.35%: 30; r_1=0.50%: 20; r_1=1.5%: 14

Percent of main reinforcement, $r_1 = A_{st,reqd} \times 100/b.d = 400 \times 100/(600 \times 430) = 0.155\%$ [EC2 UK NA Table 5]

Raw span/depth limit = 30.0 Modification factor, $A_{st,prov}/A_{st,reqd}$ (but<=1.5) = 603/400 = 1.51 so 1.5

Adjusted span/depth limit = 30.0 x 1.50 = 45.0; Actual = 2.40 x 1000/430 = 5.58 OK

Check detailing

Minimum bar spacing needs to be greatest of f (16mm), 20mm or aggregate size + 5mm: not checked here

Minimum cover required, $c_{nom} = c_{min} + D_{c,dev}$ (taken as 10mm)

$c_{min} = \max(c_{min,b}; c_{min,dur}; 10)$

$c_{min,b}$: ensure adequate bond and compaction; $c_{min,dur}$: ensure adequate durability

$c_{min,b}$ = f = 16 mm

$c_{min,dur}$ = 35mm (exposure class: XC2, wet, rarely dry, many foundations, C25/30) [EC2 Table 4.1] [BS8500 Table A.4]

Required cover, c_{nom} = 35 + 10 = 45 mm

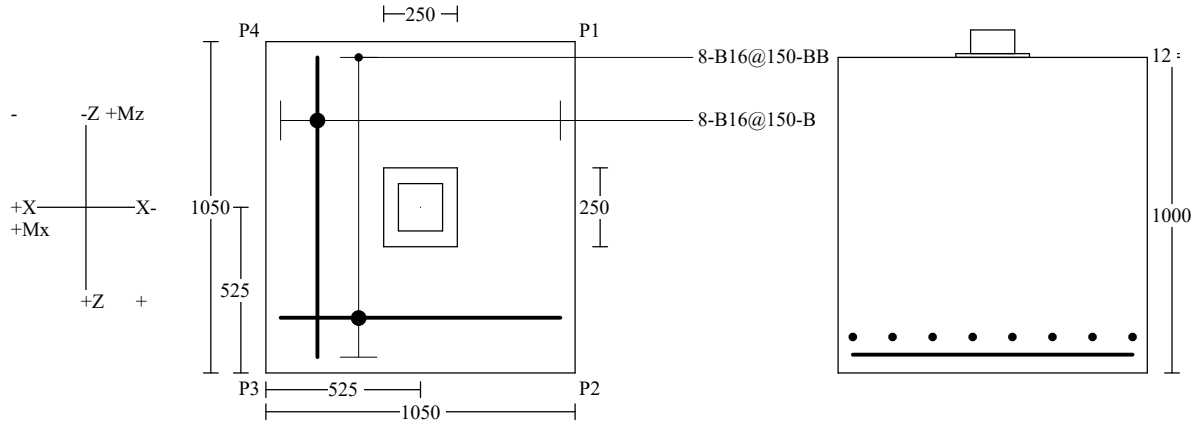
Cover to beam soffit = 500 - (430 + 16/2 + 8) = 54 mm. OK

Alpha Structural Engineers
79, Venice House,
Hatton Road,
Harrow HA0 1QL

Cloud dd298

Job Ref :
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Checked :
Approved :

Pad @ SC1 & SC2



Basic Properties

Design to	BS 8110: 1997
Fy, Fcu, Covers T, B, S	460 N/mm², 30 N/mm², 50 mm, 50 mm, 50 mm
Gross: Area, Area1, Z zz, Z xx	1.103, 0.063, 0.193, 0.193
Conc Den, Surcharge, LFsrv , LFult, SWP	23.4, 0.0, 1.0, 1.0, 100

Wall Loading

Loading F, os xx, os zz	30.0 kN/m, XX = 0 mm, ZZ = 0 mm
Note	All wall loads are resolved back to the column as a point load and induced moments.

Loading

Right (L, Xecc, Zecc, F).r	400, 0.000, 0.325	12.0 kN

Mxx Moments

Mzz Moments

Mzz.r = F.r.EccZ.r	12.00 x 0.325	3.9 kN.m

Resultant Forces from Wall at Column

F.w = F.t + F.b + F.l + F.r	0.0 + 0.0 + 0.0 + 12.0	12.0 kN
Mx.w = Mxx.t+Mxx.b+Mxx.l+Mxx.r	0.0 + 0.0 + 0.0 + 0.0	0.0 kN.m
Mz.w = Mzz.t+Mzz.b+Mzz.l+Mzz.r	0.0 + 0.0 + 0.0 + 3.9	3.9 kN.m

Z-Z Axis Section Capacities

As Bottom bars	8-B16@150	1608 mm²
X/d = Fn(AS, Fy, K1, Beff, Fcu, γ_m)	1608, 460, 0.4, 1050, 30, 0.95	0.06
Mu conc = Fn(Z, Beff, X, K1, Fcu)	895, 1050, 55, 0.4, 30	629 kN.m
vc = Fn(AS, bv, d, Fcu)	1608, 1050, 942, 30	0.3 N/mm²

X-X Axis Section Capacities

As Bottom bars	8-B16@150	1608 mm²
X/d = Fn(AS, Fy, K1, Beff, Fcu, γ_m)	1608, 460, 0.4, 1050, 30, 0.95	0.06
Mu conc = Fn(Z, Beff, X, K1, Fcu)	880, 1050, 55, 0.4, 30	618 kN.m
vc = Fn(AS, bv, d, Fcu)	1608, 1050, 926, 30	0.3 N/mm²

Critical Serviceability : 2 : Dead plus Live

Fpad = Den•d•Area•LF	23.4 x 1.0 x 1.103 x 1.00	25.8 kN
Fstub = Den•H•Area1•LF	23.4 x 0.012 x 0.063 x 1.00	0.0 kN
Fwall = Fwall•LF	12.0x1.00	12.0 kN
Fcol = F + Fstub + Fwall	49.5 + + 0.0 + 12.0	61.5 kN
Fres = F + Fpad + Fstub + Fwall	49.5 + 25.8 + 0.0 + 12.0	87.3 kN
Mzz = Mzz + Vx•D + Fcol•ezz	0.0 + (0.0 x 1.012) + (61.5 x 0.0)	0.0 kN.m
Mzz = Mzz + MzW•LF	0.0 + 3.9 x 1.0	3.9 kN.m
Effective L (Le) = Fn(Mzz,Fres,L)	3.9, 87.3, 1050	1050 mm

Pressure

Pmax = Fn(Pa, Pzz, Pxx, p1-4)	79.2, ±20.2, ±0.0, 99.4, 99.4, 59.0, 59.0	99.4 kN/m²	OK

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Alpha Structural Engineers
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FOS Overturning

Mzz Rest = (F+stub)•e+(pad)•L/2+wall•e1			
	(50 + 0) x 0.525 + (26) x 0.525 + 12 x 0.525	46 kN.m	
FOS OT zz = Mzz Rest / Mzz ot	46 / 0	45840.92 > 1.5	OK

Critical Ultimate : 1 : Dead plus Live

Fpad = Den•d•Area•LF	23.4 x 1.0 x 1.103 x 1.00	25.8 kN	
Fstub = Den•H•Area1•LF	23.4 x 0.012 x 0.063 x 1.00	0.0 kN	
Fwall = Fwall•LF	12.0x1.00	12.0 kN	
Fcol = F + Fstub + Fwall	71.4 + + 0.0 + 12.0	83.4 kN	
Fres = F + Fpad + Fstub + Fwall	71.4 + 25.8 + 0.0 + 12.0	109.2 kN	
Mzz = Mzz + Vx•D + Fcol•ezz	0.0 + (0.0 x 1.012) + (83.4 x 0.0)	0.0 kN.m	
Mzz = Mzz + MzW•LF	0.0 + 3.9 x 1.0	3.9 kN.m	
Effective L (Le) = Fn(Mzz,Fres,L)	3.9, 109.2, 1050	1050 mm	

Pressure

Pmax = Fn(Pa, Pzz, Pxx, p1-4)	99.0, ±20.2, ±0.0, 119.3, 119.3, 78.8, 78.8	119.3 kN/m²	(ULS)

FOS Overturning

Mzz Rest = (F+stub)•e+(pad)•L/2+wall•e1			
	(71 + 0) x 0.525 + (26) x 0.525 + 12 x 0.525	57 kN.m	
FOS OT zz = Mzz Rest / Mzz ot	57 / 0	57327.92 > 1.0	OK

Moments and Shears

Static load reduction w=(Sur + Den•D)•L_F

(0.0 + 23.4 x 1.0) x 1.00 — 23.4kN/m²

Moments at Column Face

X-X Moment Lower M - w•la²•B/2	8.3 - 23.4 • 1050 • 400² / 2	6.4 kN.m	OK
X-X Moment Upper M - w•La²•B/2	8.3 - 23.4 • 1050 • 400² / 2	6.4 kN.m	OK
Z-Z Moment Left M - w•La²•B/2	7.1 - 23.4 • 1050 • 400² / 2	5.1 kN.m	OK
Z-Z Moment Right M - w•la²•B/2	9.6 - 23.4 • 1050 • 400² / 2	7.6 kN.m	OK

Shear at d and 2d from Column Face

Checking <= 2vc at d & <= vc at 2d from column face

X-X lower V_d, V_{2d}, la_d, la_{2d}, d, b, w	0, 0, -526, -1452, 926, 1050, 23.4	0.01, 0.04 N/mm²	OK
X-X upper V_d, V_{2d}, la_d, la_{2d}, d, b, w	0, 0, -526, -1452, 926, 1050, 23.4	0.01, 0.04 N/mm²	OK
Z-Z left V_d, V_{2d}, la_d, la_{2d}, d, b, w	0, 0, -542, -1484, 942, 1050, 23.4	0.01, 0.04 N/mm²	OK
Z-Z right V_d, V_{2d}, la_d, la_{2d}, d, b, w	0, 0, -542, -1484, 942, 1050, 23.4	0.01, 0.04 N/mm²	OK

Punching Shear

Sub Zone Loads	F_0=109.2 + F_1=0 + F_2=0 + F_3=0 + F_4=0		
	+ F_{12}=0 + F_{23}=0 + F_{34}=0 + F_{41}=0	109.2 kN	
Sub Zone Error = Fres/∑(F_0-F_{41})	(1 - 109.196 / 109.196)•100	0.000%	OK
Zone Locations	P_4---------- P_1		
	| F_4 | F_{41} | F_1 |		
	|----------|		
	| F_{34}| F_0 | F_{12} | F_0 is bound by Punching Perimiter		
	|----------| @ 1.5 d from column face.		
	| F_3 | F_{23} | F_2 |		
	P_3---------- P_2		
Projections from Column face	2 opposite Projections are less than 1.5 d =>	Not Applicable	