ALPHA

STRUCTURAL ENGINEERS

STRUCTURAL REPORT

Project No	24685
Description	Ground Floor Extension & New Basement
Client Name	
Address	102 Green Lane Northwood Middlesex HA6 1AJ
Date	12.2024
Prepared By	Kemal Erturk

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

ALPHA Structural Engineers

Structural Calculation Report

Site address: 102

Job number:
Made by: KE
Page: 1

Beam: B1 **Span: 5.0 m.**

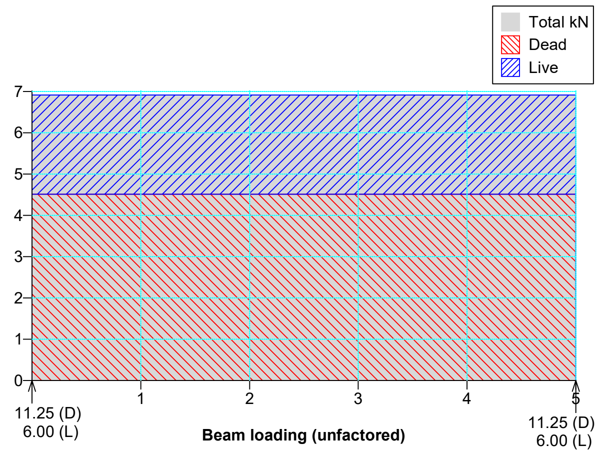
		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O	G	o.w.	0.5	0		L	1.25	1.25	4.1
U	G	ROOF DEAD	4*1	0		L	10.00	10.00	32.6
U	QA	ROOF LIVE	4*0.6	0		L	6.00	6.00	19.5
			Total load (unfactored):			**34.50 kN**	**17.25**	**17.25**	56.2
			Dead/Permanent (unfactored):			22.50 kN	11.25	11.25	36.6
			Live/Variable (unfactored):			12.00 kN	6.00	6.00	19.5
			Factored (6.10):			48.38 kN	24.19	24.19	

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

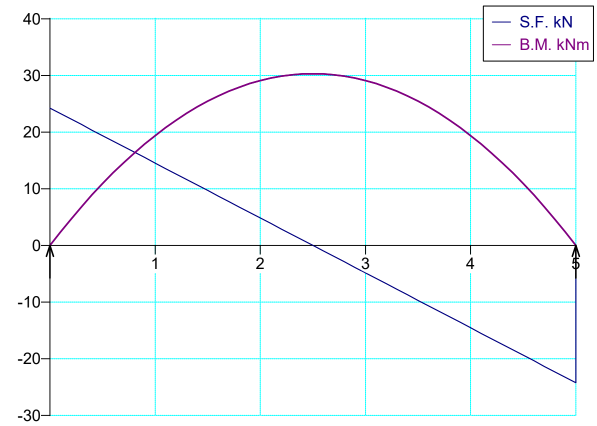
Maximum B.M. = 30.23 kNm (6.10) at 2.50 m. from R1

Maximum S.F. = 24.2 kN (6.10) at R1

Mid-span deflections: Dead: 36.6 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 19.5 x 10^8/EI
Total: 56.2 x 10^8/EI



Beam loading (unfactored)



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

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **203 x 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} = 24.19 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 (>=24.19) 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} = 30.2 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, $M_{b,Rd} = \chi_{LT,mod}.M_{c,Rd}$

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

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

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

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

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

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

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

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

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

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\overline{\lambda}_z$	$\overline{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT, mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-5.00	5.00	30.2	1.00d	1.00	195.6	1.276	0.950	0.932	0.730	0.730	176.4	128.8	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 = 19.5 x 1e8/(210,000 x 4,570) = 2.0 mm (L/2457) OK
TL deflection = 56.2 x 1e8/(210,000 x 4,570) = 5.9 mm (L/855)

Beam: **B2** **Span: 8.0 m.**

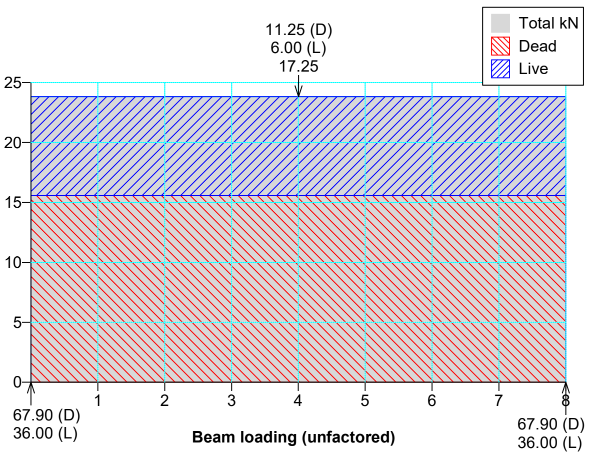
		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O	G	o.w.	1.3	0		L	5.20	5.20	69
U	G	FLOOR DEAD	2.75*0.5*2	0		L	11.00	11.00	147
U	QA	FLOOR LIVE	2.75*1.5*2	0		L	33.00	33.00	440
U	G	WALL	4.8*2.4	0		L	46.08	46.08	614
P	G	Beam: B1 [3.8 m.] : R1	11.25	4			5.63	5.63	120
P	QA	Beam: B1 [3.8 m.] : R1	6.00	4			3.00	3.00	64
				Total load (unfactored):		**208 kN**	**103.90**	**103.90**	1454
			Dead/Permanent (unfactored):			136 kN	67.90	67.90	950
			Live/Variable (unfactored):			72 kN	36.00	36.00	504
			Factored (6.10):			291 kN	145.67	145.67	

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

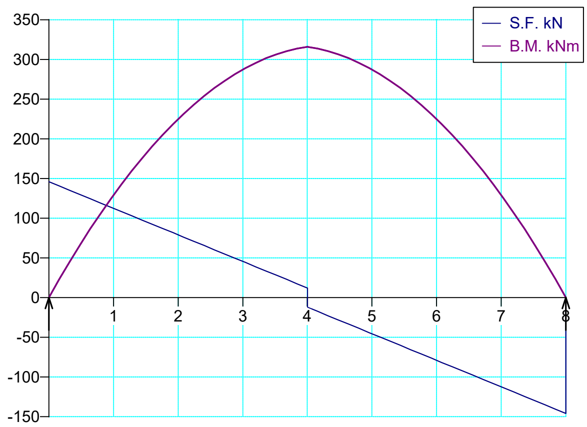
Maximum B.M. = 316 kNm (6.10) at 4.00 m. from R1

Maximum S.F. = 146 kN (6.10) at R1

Mid-span deflections: Dead: 950 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 504 x 10^8/EI
Total: 1,454 x 10^8/EI



Beam loading (unfactored)



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

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **356 x 368 x 129 UKC** S355

D=355.6 mm B=368.6 mm t=10.4 mm T=17.5 mm I_y=40,200 cm^4 i_z=9.43 cm $W_{pl,y}$=2,480 cm^3 $W_{el,y}$=2,260 cm^3

Classification: Flange: c/t = 163.9/17.5 = 9.37 <= 14ε (11.55): Class 3, semi-compact
EC3 Table 5.2 Web: c/t = 290.2/10.4 = 27.9 <= 72ε (59.4): Class 1, plastic

Shear

Design shear force, V_{Ed} = 146 kN

Shear area, $A_v = A - 2bt_f + (t_w+2r)t_f$ = 164x100 - 2x369x17.5 + (10.4+2x15.2) x17.5 = 4,213 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd} = A_v.(f_y/\sqrt{3})/\gamma_{M0}$ = 4,213 x (345/$\sqrt{3}$)/(1.0 x 1000) = 839 kN (>=146) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 320.6/10.4 = 30.83 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 316 kNm

Moment resistance, $M_{c,y,Rd} = f_y.W_{el,y}$ = 345 x 2,260/1000 = 780 kNm OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

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

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

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

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

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

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

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

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

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

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

$W_y = 2{,}260\ cm^3$ $I_w = 4.18\ dm^6$ $I_T = 153\ cm^4$ $I_z = 14{,}600\ cm^4$ $G = 81{,}000 N/mm^2$

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{\lambda}_z$	$\bar{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT, mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-8.00	8.00	315.5	1.00d	1.00	1106.3	1.095	0.840	0.839	0.795	0.795	779.7	619.9	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 = 504 x 1e8/(210,000 x 40,200) = 6.0 mm (L/1340) OK
TL deflection = 1454 x 1e8/(210,000 x 40,200) = 17.2 mm (L/464)

Beam: FLAT ROOF JOISTS **Span: 4.2 m.**

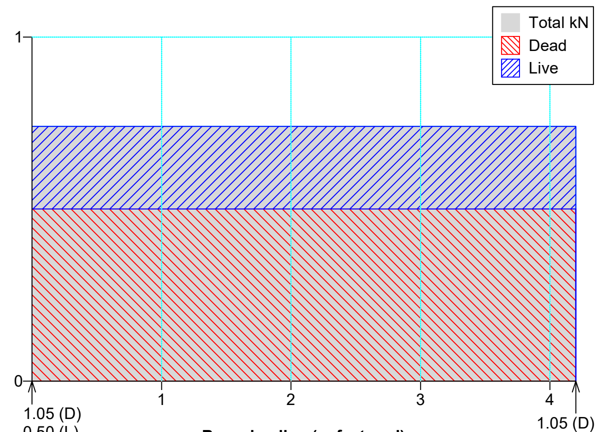
		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O	G	o.w.	0.1	0		L	0.21	0.21	0.41
U	G	ROOF DEAD	1*0.4	0		L	0.84	0.84	1.62
U	QA	ROOF LIVE	0.6*0.4	0		L	0.50	0.50	0.97
				Total load (unfactored):		**3.11 kN**	**1.55**	**1.55**	3.00
				Dead/Permanent (unfactored):		2.10 kN	1.05	1.05	2.03
				Live/Variable (unfactored):		1.01 kN	0.50	0.50	0.97
				Factored (6.10):		4.35 kN	2.17	2.17	

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

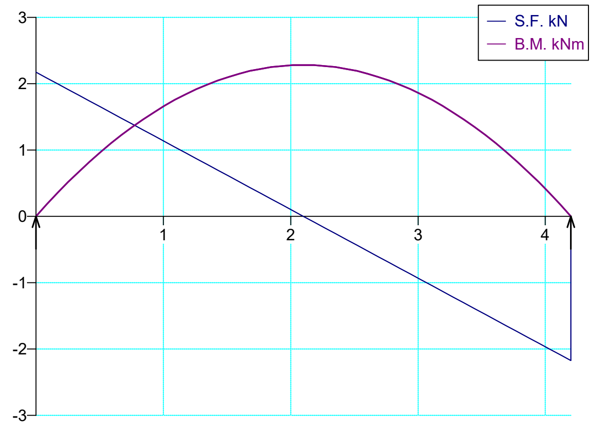
Maximum B.M. = 2.28 kNm (6.10) at 2.10 m. from R1

Maximum S.F. = 2.17 kN (6.10) at R1

Mid-span deflections: Dead: 2.03 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 0.97 x 10^8/EI
Total: 3.00 x 10^8/EI



Beam loading (unfactored)



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

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

Use 50 x 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, 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} = 1.0

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

Bending

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

Design bending strength, $f_{m,y,d} = f_{m,k}.k_{mod}.k_h.k_{sys}/\gamma_m$ = 24.0 x 0.60 x 1.00 x 1.0/1.30 = 11.08 N/mm^2

Design bending stress, $\sigma_{m,y,d}$ = 2.28 x 1000/333 = 6.85 N/mm^2 OK

Bending resistance = 11.08 x 333/1000 = 3.69 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 1.0/1.30 = 1.85 N/mm^2

Design shear stress, $\sigma_{v,y,d}$ = 2.17 x 1000 x (3/2)/(33.5 x 200) = 0.49 N/mm^2 OK

Shear resistance = 1.85 x 33.5 x 200 x (2/3)/1000 = 8.25 kN

Deflection

Final deflection limit = 0.003L = 12.60 mm
Final deflection, $u_{fin} = \Sigma u_{inst}(1 + \psi_2.k_{def})$ [EC2 Eq.2.29/2.30]

Instantaneous mid-span shear deflection, $u_{inst,v} = 1.2M_{unf}/(G_{mean}A)$ where (G = E/16)
$u_{inst,v} = 1.2 \times 1.63 \times 10^6/((11,000/16) \times 50 \times 200) = 0.28$ mm

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

Mid-span deflections:	x $1e^8$/EI	Inst. mm	k_{def}	ψ_2	Final mm	
G:	2.03	5.53	0.60	1.00	8.84	
QA:	0.97	2.65	0.60	0.30	3.13	
Shear deflection:		0.28			0.42	
Total mm.		8.46			**12.39**	OK

Beam: B3 **Span: 5.0 m.**

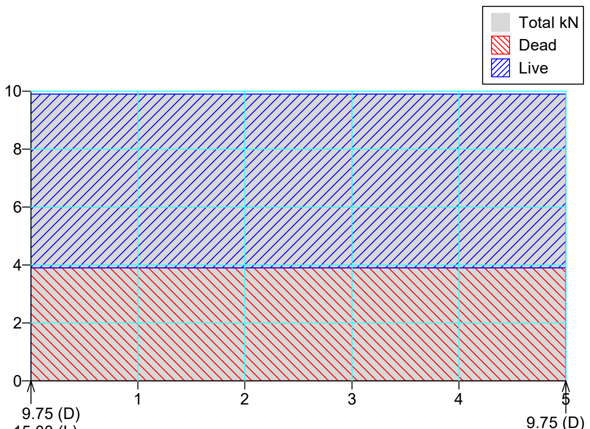
		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O	G	o.w.	0.5	0		L	1.25	1.25	4.1
U	G	FLOOR DEAD	4*0.6	0		L	6.00	6.00	19.5
U	QA	FLOOR LIVE	4*1.5	0		L	15.00	15.00	48.8
U	G	PARTITIONS	1	0		L	2.50	2.50	8.1
					Total load (unfactored):	**49.5 kN**	**24.75**	**24.75**	80.6
				Dead/Permanent (unfactored):		19.5 kN	9.75	9.75	31.7
				Live/Variable (unfactored):		30.0 kN	15.00	15.00	48.8
				Factored (6.10):		71.3 kN	35.66	35.66	

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

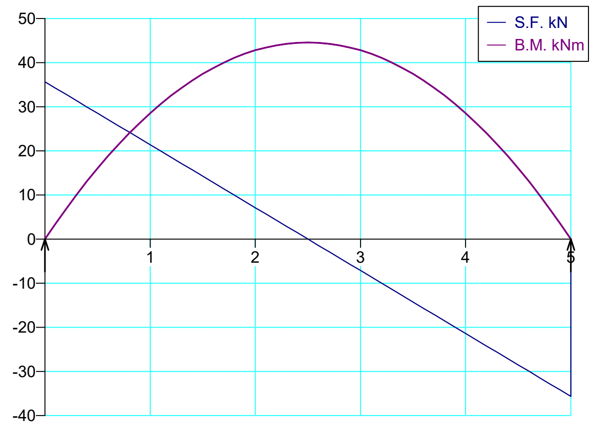
Maximum B.M. = 44.58 kNm (6.10) at 2.50 m. from R1

Maximum S.F. = 35.7 kN (6.10) at R1

Mid-span deflections: Dead: 31.7 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 48.8 x 10^8/EI
Total: 80.6 x 10^8/EI



Beam loading (unfactored)



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

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **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} = 35.7 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/√3)/(1.0 x 1000) = 347 kN (>=35.7) 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} = 44.6 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, $M_{b,Rd} = \chi_{LT,mod}.M_{c,Rd}$

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

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

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

$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta\bar{\lambda}_{LT}^2)}]$ [EC3 (6.57)]
$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta\bar{\lambda}_{LT}^2]$
$\bar{\lambda}_{LT,0} = 0.4$ $\beta = 0.75$ [EC3 UK NA 2.17]
$\bar{\lambda}_{LT} = \sqrt{(f_y.W_{pl,y}/M_{cr})}$
$M_{cr} = C_1(\pi^2 EI_z/L_e^2)\sqrt{(I_w/I_z + L_e^2 GI_T/\pi^2 EI_z)}$ [SN003: k and k_w taken as 1.0, conservative]
$W_y = 497.0$ 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}	$\bar{\lambda}_z$	$\bar{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-5.00	5.00	44.6	1.00d	1.00	195.6	1.276	0.950	0.932	0.730	0.730	176.4	128.8	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 = 48.8 x 1e8/(210,000 x 4,570) = 5.1 mm (L/983) OK
TL deflection = 80.6 x 1e8/(210,000 x 4,570) = 8.4 mm (L/596)

Beam: **B4** **Span: 5.0 m.**

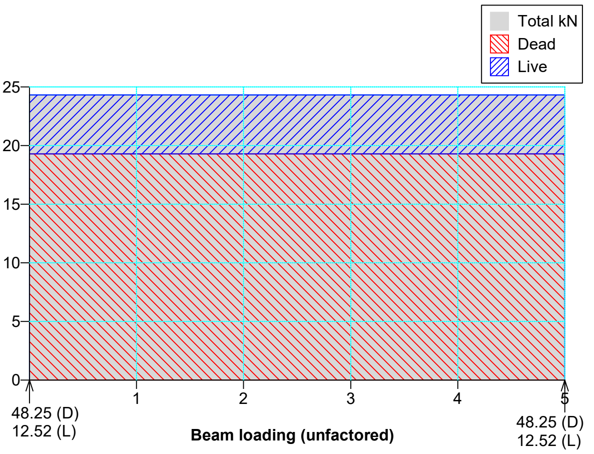
		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O	G	o.w.	0.75	0		L	1.88	1.88	6
U	G	ROOF DEAD	1*1.2	0		L	3.00	3.00	10
U	QA	ROOF LIVE	0.6*1.2	0		L	1.80	1.80	6
U	G	FLOOR DEAD	0.5*2.1	0		L	2.63	2.63	9
U	QA	FLOOR LIVE	1.5*2.1	0		L	7.88	7.88	26
U	G	WALL	3*4.8	0		L	36.00	36.00	117
U	G	EXTENSION ROOF	1*1.9	0		L	4.75	4.75	15
U	QA	EXTENSION ROOF LIVE	0.6*1.9	0		L	2.85	2.85	9
				Total load (unfactored):		**121.5 kN**	**60.77**	**60.77**	198
			Dead/Permanent (unfactored):			96.5 kN	48.25	48.25	157
			Live/Variable (unfactored):			25.0 kN	12.52	12.52	41
			Factored (6.10):			167.9 kN	83.93	83.93	

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

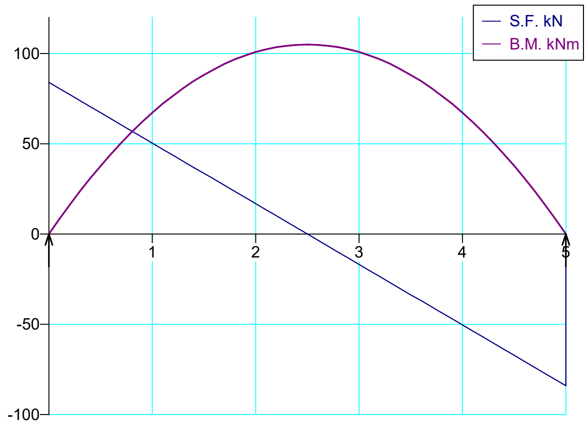
Maximum B.M. = 104.9 kNm (6.10) at 2.50 m. from R1

Maximum S.F. = 83.9 kN (6.10) at R1

Mid-span deflections: Dead: 157 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 41 x 10^8/EI
Total: 198 x 10^8/EI



Beam loading (unfactored)



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

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **203 x 203 x 71 UKC** S355

D=215.8 mm B=206.4 mm t=10.0 mm T=17.3 mm I_y=7,620 cm^4 i_z=5.30 cm $W_{pl,y}$=799 cm^3 $W_{el,y}$=706 cm^3

Classification: Flange: c/t = 88.0/17.3 = 5.09 <= 9ε (7.43): Class 1, plastic
EC3 Table 5.2 Web: c/t = 160.8/10.0 = 16.1 <= 72ε (59.4): Class 1, plastic

Shear

Design shear force, V_{Ed} = 83.9 kN

Shear area, $A_v = A - 2bt_f + (t_w+2r)t_f$ = 90.4x100 - 2x206x17.3 + (10.0+2x10.2) x17.3 = 2,424 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd} = A_v.(f_y/\sqrt{3})/\gamma_{M0}$ = 2,424 x (345/√3)/(1.0 x 1000) = 483 kN (>=83.9) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 181.2/10.0 = 18.12 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 105 kNm

Moment resistance, $M_{c,y,Rd} = f_y.W_{pl,y}$ = 345 x 799/1000 = 276 kNm OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

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

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

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

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

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

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

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

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

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

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

W_y = 799.0 cm^3 I_w = 0.250 dm^6 I_T = 80.2 cm^4 I_z = 2,540 cm^4 G = 81,000N/mm^2

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{\lambda}_z$	$\bar{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-5.00	5.00	104.9	1.00d	1.00	424.2	1.217	0.806	0.813	0.814	0.814	275.7	224.3	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 = 41 x 1e8/(210,000 x 7,620) = 2.5 mm (L/1962) OK
TL deflection = 198 x 1e8/(210,000 x 7,620) = 12.4 mm (L/404)

Beam: **B5** **Span: 6.7 m.**

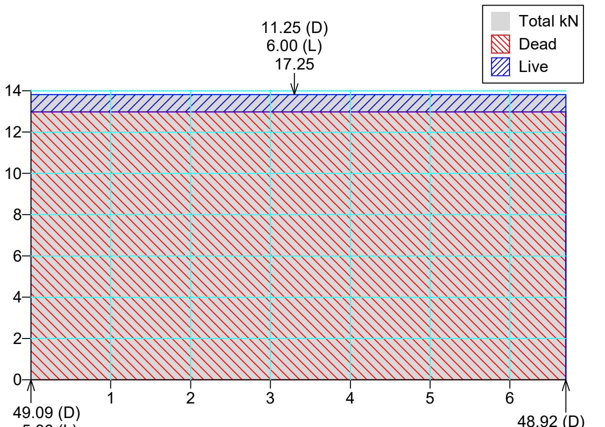
		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O	G	o.w.	1.05	0		L	3.52	3.52	28
U	G	ROOF DEAD	1*1.4	0		L	4.69	4.69	37
U	QA	ROOF LIVE	0.6*1.4	0		L	2.81	2.81	22
U	G	WALL	3*3.5	0		L	35.17	35.17	276
P	G	Beam: B1 [5.0 m.] : R1	11.25	3.3			5.71	5.54	70
P	QA	Beam: B1 [5.0 m.] : R1	6.00	3.3			3.04	2.96	38
				Total load (unfactored):		**109.6 kN**	**54.95**	**54.69**	470
				Dead/Permanent (unfactored):		98.0 kN	49.09	48.92	410
				Live/Variable (unfactored):		11.6 kN	5.86	5.77	60
				Factored (6.10):		149.8 kN	75.06	74.70	

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

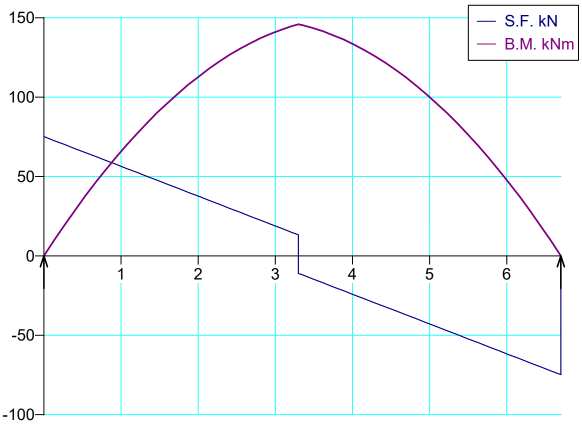
Maximum B.M. = 145.7 kNm (6.10) at 3.30 m. from R1

Maximum S.F. = 75 kN (6.10) at R1

Mid-span deflections: Dead: 410 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 60 x 10^8/EI
Total: 470 x 10^8/EI



Beam loading (unfactored)



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

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **254 x 254 x 107 UKC** S355

D=266.7 mm B=258.8 mm t=12.8 mm T=20.5 mm I_y=17,500 cm^4 i_z=6.59 cm $W_{pl,y}$=1,480 cm^3 $W_{el,y}$=1,310 cm^3

Classification: Flange: c/t = 110.3/20.5 = 5.38 <= 9ε (7.43): Class 1, plastic
EC3 Table 5.2 Web: c/t = 200.3/12.8 = 15.6 <= 72ε (59.4): Class 1, plastic

Shear

Design shear force, V_{Ed} = 75.1 kN

Shear area, $A_v = A - 2bt_f + (t_w+2r)t_f$ = 136x100 - 2x259x20.5 + (12.8+2x12.7) x20.5 = 3,772 mm^2 [EC3 6.2.6 (3)]

Shear resistance, $V_{pl,Rd} = A_v.(f_y/\sqrt{3})/\gamma_{M0}$ = 3,772 x (345/√3)/(1.0 x 1000) = 751 kN (>=75.1) OK [EC3 6.2.6]

Shear buckling: h_w/t_w = 225.7/12.8 = 17.63 <= 72ε (58.58): check not required [EC3 6.2.6(6)]

Bending

Moment resistance

Design moment, M_{Ed} = 146 kNm

Moment resistance, $M_{c,y,Rd} = f_y.W_{pl,y}$ = 345 x 1,480/1000 = 511 kNm OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

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

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

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

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

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

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

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

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

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

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

$W_y = 1{,}480\ cm^3$ $I_w = 0.898\ dm^6$ $I_T = 172\ cm^4$ $I_z = 5{,}930\ cm^4$ $G = 81{,}000 N/mm^2$

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{\lambda}_z$	$\bar{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-6.70	6.70	145.1	1.00d	1.00	702.5	1.312	0.853	0.849	0.788	0.788	510.6	402.2	OK

C1 derivation: d: taken as 1.0 (conservative)

Combined bending and shear

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

Web buckling and crushing have not been checked

Deflection

LL deflection = 60 x 1e8/(210,000 x 17,500) = 1.6 mm (L/4130) OK
TL deflection = 470 x 1e8/(210,000 x 17,500) = 12.8 mm (L/524)

Beam: B6 **Span: 3.8 m.**

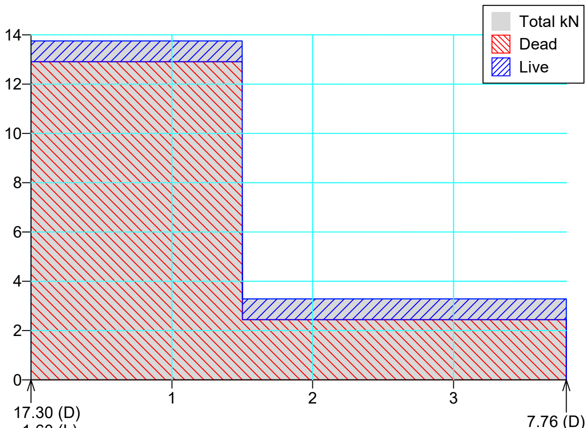
		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O	G	o.w.	1.05	0		L	1.99	1.99	2.85
U	G	ROOF DEAD	1*1.4	0		L	2.66	2.66	3.80
U	QA	ROOF LIVE	0.6*1.4	0		L	1.60	1.60	2.28
R	G	WALL	3*3.5	0		1.5	12.64	3.11	9.55
				Total load (unfactored):		**28.3 kN**	**18.89**	**9.36**	18.49
				Dead/Permanent (unfactored):		25.1 kN	17.30	7.76	16.21
				Live/Variable (unfactored):		3.2 kN	1.60	1.60	2.28
				Factored (6.10):		38.6 kN	25.74	12.87	

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

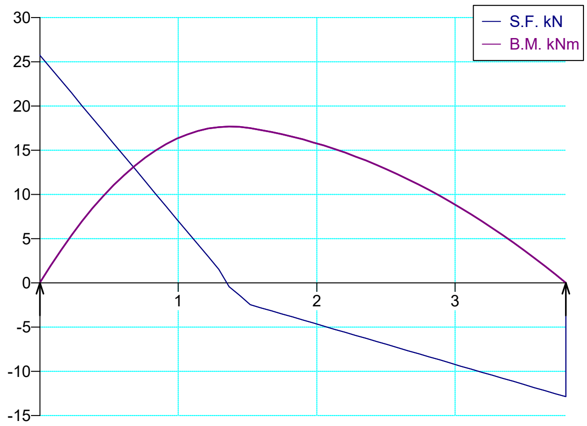
Maximum B.M. = 17.68 kNm (6.10) at 1.36 m. from R1

Maximum S.F. = 25.74 kN (6.10) at R1

Mid-span deflections: Dead: 16.21 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 2.28 x 10^8/EI
Total: 18.49 x 10^8/EI



Beam loading (unfactored)



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

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **203 x 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} = 25.7 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/√3)/(1.0 x 1000) = 347 kN (>=25.7) 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} = 17.68 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, $M_{b,Rd} = \chi_{LT,mod}.M_{c,Rd}$

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

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

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

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

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

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

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

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

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

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\overline{\lambda}_z$	$\overline{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT,mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-3.80	3.80	17.7	1.00d	1.00	292.4	0.969	0.777	0.790	0.830	0.830	176.4	146.4	OK

C1 derivation: d: taken as 1.0 (conservative)

Combined bending and shear

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

Web buckling and crushing have not been checked

Deflection

LL deflection = 2.28 x 1e8/(210,000 x 4,570) = 0.2 mm OK
TL deflection = 18.49 x 1e8/(210,000 x 4,570) = 1.9 mm (L/1972)

Beam: **B7** **Span: 5.1 m.**

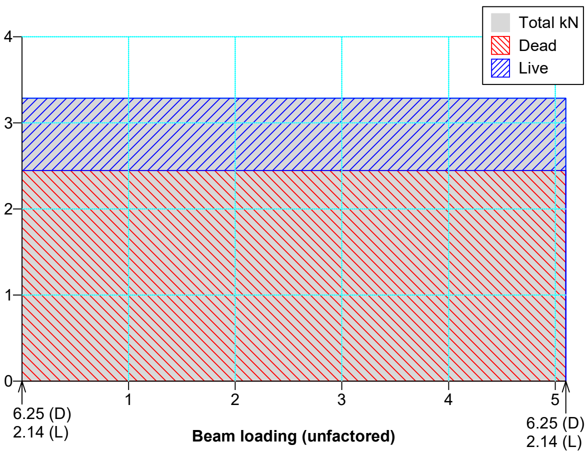
		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O	G	o.w.	1.05	0		L	2.68	2.68	9.2
U	G	ROOF DEAD	1*1.4	0		L	3.57	3.57	12.3
U	QA	ROOF LIVE	0.6*1.4	0		L	2.14	2.14	7.4
				Total load (unfactored):		**16.78 kN**	**8.39**	**8.39**	29.0
				Dead/Permanent (unfactored):		12.49 kN	6.25	6.25	21.6
				Live/Variable (unfactored):		4.28 kN	2.14	2.14	7.4
				Factored (6.10):		23.29 kN	11.65	11.65	

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

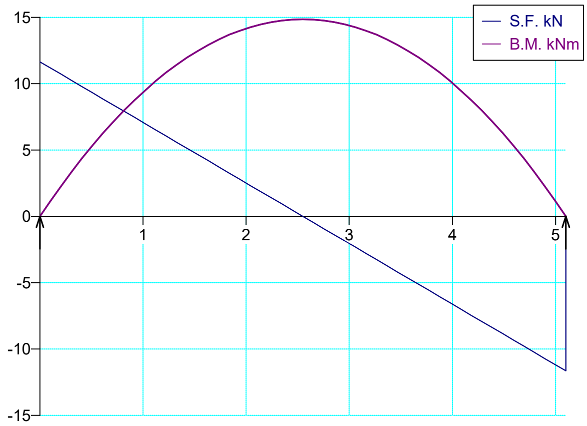
Maximum B.M. = 14.85 kNm (6.10) at 2.55 m. from R1

Maximum S.F. = 11.65 kN (6.10) at R1

Mid-span deflections: Dead: 21.6 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 7.4 x 10^8/EI
Total: 29.0 x 10^8/EI



Beam loading (unfactored)



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

Beam calculation to BS EN1993.1.1 using S355 steel

SECTION SIZE : **152 x 152 x 37 UKC** S355

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

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

Shear

Design shear force, V_{Ed} = 11.65 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, $V_{pl,Rd} = A_v.(f_y/\surd 3)/\gamma_{M0}$ = 1,426 x (355/√3)/(1.0 x 1000) = 292 kN (>=11.65) OK [EC3 6.2.6]

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

Bending

Moment resistance

Design moment, M_{Ed} = 14.85 kNm

Moment resistance, $M_{c,y,Rd} = f_y.W_{pl,y}$ = 355 x 309/1000 = 110 kNm OK

Lateral-torsional buckling check

Beam is laterally restrained at supports only

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

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

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

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

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

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

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

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

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

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

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

Segment	L_e	M_{Max}	C_1	k_c	M_{cr}	$\bar{\lambda}_z$	$\bar{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT, mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-5.10	5.10	14.9	1.00d	1.00	102.5	1.725	1.034	1.009	0.679	0.679	109.7	74.4	OK

C1 derivation: d: taken as 1.0 (conservative)

Combined bending and shear

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

Web buckling and crushing have not been checked

Deflection

LL deflection = 7.4 x 1e8/(210,000 x 2,210) = 1.6 mm (L/3199) OK
TL deflection = 29.0 x 1e8/(210,000 x 2,210) = 6.2 mm (L/817)

Beam: B8 **Span: 3.8 m.**

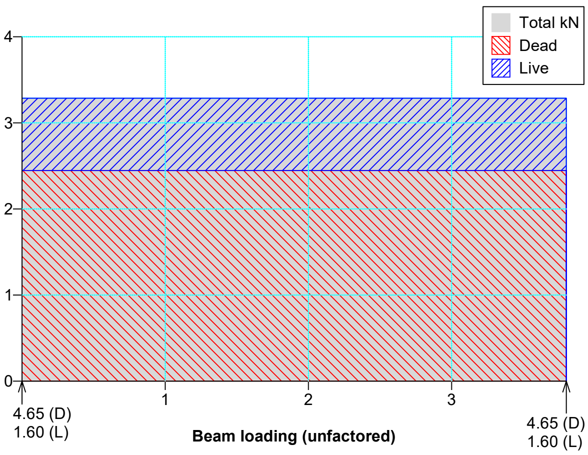
		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O	G	o.w.	1.05	0		L	1.99	1.99	2.85
U	G	ROOF DEAD	1*1.4	0		L	2.66	2.66	3.80
U	QA	ROOF LIVE	0.6*1.4	0		L	1.60	1.60	2.28
					Total load (unfactored):	**12.50 kN**	**6.25**	**6.25**	8.93
					Dead/Permanent (unfactored):	9.31 kN	4.65	4.65	6.65
					Live/Variable (unfactored):	3.19 kN	1.60	1.60	2.28
					Factored (6.10):	17.36 kN	8.68	8.68	

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

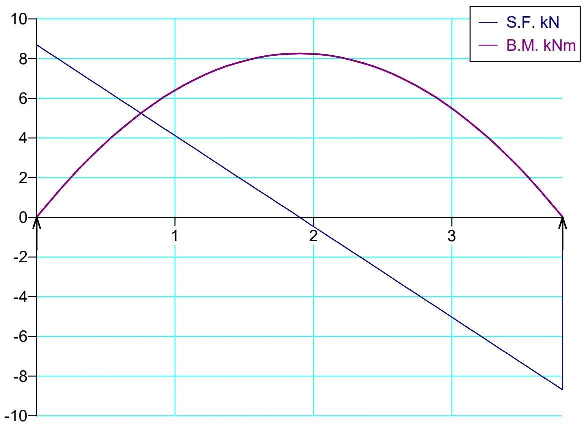
Maximum B.M. = 8.24 kNm (6.10) at 1.90 m. from R1

Maximum S.F. = 8.68 kN (6.10) at R1

Mid-span deflections: Dead: 6.65 x 10^8/EI *(E in N/mm², I in cm⁴)*
Live: 2.28 x 10^8/EI
Total: 8.93 x 10^8/EI



Beam loading (unfactored)



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

Beam calculation to BS EN1993.1.1 using S355 steel

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

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

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

Shear

Design shear force, V_{Ed} = 8.68 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/√3)/(1.0 x 1000) = 238 kN (>=8.68) OK [EC3 6.2.6]

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

Bending

Moment resistance

Design moment, M_{Ed} = 8.24 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]

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

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

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

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

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

$W_y = 248.0$ 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}	$\overline{\lambda}_z$	$\overline{\lambda}_{LT}$	ϕ_{LT}	χ_{LT}	$\chi_{LT, mod}$	$M_{c,Rd}$	$M_{b,Rd}$	
0.00-3.80	3.80	8.2	1.00d	1.00	101.9	1.298	0.929	0.914	0.742	0.742	88.0	65.4	OK

C1 derivation: d: taken as 1.0 (conservative)

Combined bending and shear

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

Web buckling and crushing have not been checked

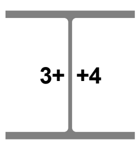
Deflection

LL deflection = 2.28 x 1e8/(210,000 x 1,750) = 0.6 mm (L/6125) OK
TL deflection = 8.93 x 1e8/(210,000 x 1,750) = 2.4 mm (L/1564)

Column calculation to EN1993-1-1 using S355 steel

Location: SC1-SC2

Length: 5.7 m.



Pos	Dur	Load	kN	Fact	Offset	M_{yy}	M_{zz}
				6.10		6.10	6.10
2	G	B/F Beam: B2 [8.0 m.] : R1	68	92	100	-20.81	0.00
2	QA	B/F Beam: B2 [8.0 m.] : R1	36	54	100	-12.26	0.00
Total load kN			104	**146**		**-33.07**	**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 : **254 x 254 x 73 UKC** S355

Section properties: B = 254.6mm D = 254.1mm T = 14.2mm t = 8.6mm A_g = 93.1cm^2
i_y = 11.1cm i_z = 6.48cm $W_{pl,y}$ = 992cm^3 $W_{el,y}$ = 898cm^3

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

Classification: Flange: c/t = 110.3/14.2 = 7.77 <= 10ε (8.14): Class 2, compact
[Table 5.2] Web: c/t = 200.3/8.6 = 23.3 <= 396ε/(13α - 1) = 45.7 : Class 1, plastic
$\alpha = 0.5(1 + N_{Ed}/(f_y.c.t_w))$ = 0.5 x (1 + 146 x 1000/(355 x 200 x 8.60)) = 0.619 [SCI P362 Table 5.1 note 1]

Major axis: L_{Ey} = 1.0L = 5.70 m. Slenderness, λ_y = 5.70 x 100/11.1 = 51.4
Minor axis: L_{Ez} = 1.0L = 5.70 m. Slenderness, λ_z = 5.70 x 100/6.48 = 88.0

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} = 146 kN

Design compression resistance, $N_{c,Rd} = A.f_y/\gamma_{M0}$ = 93.1 x 100 x 355/(1.0 x 1000) = 3,305 kN OK

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

$\lambda_1 = 93.9\varepsilon$ = 93.9 x 0.814 = 76.4

Buckling about y-y (major) axis

$\bar{\lambda}_y = \lambda_y/\lambda_1$ = 51.4/76.4 = 0.672 [EC3 6.3.1.3]

Use curve b: α = 0.340 $\phi = 0.5(1 + \alpha(\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = 0.81 [EC3 (6.49)]

Flexural buckling reduction factor, $\chi_y = 1/(\phi + \sqrt{(\phi^2 - \bar{\lambda}_y^2)})$ = 0.80 [EC3 (6.49)]

Design buckling resistance, $N_{b,y,Rd} = \chi_y A.f_y/\gamma_{M1}$ = 0.80 x 93.1 x 100 x 355/(1.0 x 1000) = 2,641 kN OK [EC3 (6.47)]

Buckling about z-z (minor) axis

$\bar{\lambda}_z = \lambda_z/\lambda_1$ = 88.0/76.4 = 1.15 [EC3 6.3.1.3]

Use curve c: α = 0.490 $\phi = 0.5(1 + \alpha(\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = 1.40 [EC3 Table 6.2]

Flexural buckling reduction factor, $\chi_z = 1/(\phi + \sqrt{(\phi^2 - \bar{\lambda}_z^2)})$ = 0.458 [EC3 (6.49)]

Design buckling resistance, $N_{b,z,Rd} = \chi_z A.f_y/\gamma_{M1}$ = 0.458 x 93.1 x 100 x 355/(1.0 x 1000) = 1,512 kN OK [EC3 (6.47)]

Bending about y-y (major) axis:

Design moment, $M_{y,Ed}$ = 33.1 kNm

Moment resistance, $M_{c,y,Rd} = f_y.W_{pl,y}$ = 355 x 992/1000 = 352 kNm OK

Calculate buckling resistance moment

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

$M_{cr} = C_1(\pi^2 EI_z/L_{eff}^2)[\sqrt{(I_w/I_z + L_{eff}^2 GI_t/(\pi^2 EI_z)}]$ = 453 [NCCI SN003 2(1)]
C_1 = 1.0 (conservative) $\bar{\lambda}_{LT} = \sqrt{(M_{c,y,Rd}/M_{cr})}$ = 0.88
$\bar{\lambda}_{LT,0}$ = 0.4 β = 0.75 [EC3 UK NA 2.17]
Use buckling curve b: α = 0.340 [EC3 Tables 6.3/6.4 NA2.17]
$\phi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta\bar{\lambda}_{LT}^2]$ = 0.87
$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta\bar{\lambda}_{LT}^2)}]$ = 0.77 [EC3 (6.56)]

$M_{b,y,Rd} = \chi_{LT}.M_{c,y,Rd}/\gamma_M$ = 0.77 x 352/1.0 = 271 kNm

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

Summary:

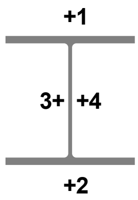
$N_{Ed}/N_{min,b,Rd}$ = 146/1512 = 0.096 [1]
$M_{y,Ed}/M_{bs}$ = 33.1/271 = 0.122 [2]
$M_{z,Ed}/M_{z,cb,Rd}$ = 0.000 [3] x 1.5 [NCCI SN048]
Sum of stress ratios [1] + [2] + 1.5 x [3] = **0.218** **OK** *subject to all SN048b criteria being complied with*

Column calculation to EN1993-1-1 using S355 steel

Location: SC3

Length: 3.0 m.

Pos	Dur	Load	kN	Fact	Offset	M_{yy}	M_{zz}
				6.10		6.10	6.10
2	G	B/F Beam: B3 [5.0 m.] : R1	9.8	13.2	100	-2.32	0.00
2	QA	B/F Beam: B3 [5.0 m.] : R1	15.0	22.5	100	-3.96	0.00
Total load kN			24.8	**35.7**		**-6.28**	**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 23 UKC** S355

Section properties: B = 152.2mm D = 152.4mm T = 6.8mm t = 5.8mm A_g = 29.2cm^2
i_y = 6.54cm i_z = 3.70cm $W_{pl,y}$ = 182cm^3 $W_{el,y}$ = 164cm^3

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

Classification: Flange: c/t = 65.6/6.8 = 9.65 <= 14ε (11.39): Class 3, semi-compact
[Table 5.2] Web: c/t = 123.6/5.8 = 21.3 <= 396ε/(13α - 1) = 50.3 : Class 1, plastic
$\alpha = 0.5(1 + N_{Ed}/(f_y.c.t_w))$ = 0.5 x (1 + 35.7 x 1000/(355 x 124 x 5.80)) = 0.570 [SCI P362 Table 5.1 note 1]

Major axis: L_{Ey} = 1.0L = 3.00 m. Slenderness, λ_y = 3.00 x 100/6.54 = 45.9
Minor axis: L_{Ez} = 1.0L = 3.00 m. Slenderness, λ_z = 3.00 x 100/3.70 = 81.1

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} = 35.7 kN

Design compression resistance, $N_{c,Rd} = A.f_y/\gamma_{M0}$ = 29.2 x 100 x 355/(1.0 x 1000) = 1,037 kN OK

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

$\lambda_1 = 93.9\varepsilon$ = 93.9 x 0.814 = 76.4

Buckling about y-y (major) axis

$\bar{\lambda}_y = \lambda_y/\lambda_1$ = 45.9/76.4 = 0.600 [EC3 6.3.1.3]

Use curve b: α = 0.340 $\phi = 0.5(1 + \alpha(\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = 0.75 [EC3 (6.49)]

Flexural buckling reduction factor, $\chi_y = 1/(\phi + \sqrt{(\phi^2 - \bar{\lambda}_y^2)})$ = 0.84 [EC3 (6.49)]

Design buckling resistance, $N_{b,y,Rd} = \chi_y A.f_y/\gamma_{M1}$ = 0.84 x 29.2 x 100 x 355/(1.0 x 1000) = 867 kN OK [EC3 (6.47)]

Buckling about z-z (minor) axis

$\bar{\lambda}_z = \lambda_z/\lambda_1$ = 81.1/76.4 = 1.06 [EC3 6.3.1.3]

Use curve c: α = 0.490 $\phi = 0.5(1 + \alpha(\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = 1.27 [EC3 Table 6.2]

Flexural buckling reduction factor, $\chi_z = 1/(\phi + \sqrt{(\phi^2 - \bar{\lambda}_z^2)})$ = 0.51 [EC3 (6.49)]

Design buckling resistance, $N_{b,z,Rd} = \chi_z A.f_y/\gamma_{M1}$ = 0.51 x 29.2 x 100 x 355/(1.0 x 1000) = 524 kN OK [EC3 (6.47)]

Bending about y-y (major) axis:

Design moment, $M_{y,Ed}$ = 6.28 kNm

Moment resistance, $M_{c,y,Rd} = f_y.W_{pl,y}$ = 355 x 182/1000 = 64.6 kNm OK [SN048b: use $W_{pl,y}$ although Class 3]

Calculate buckling resistance moment

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

$M_{cr} = C_1(\pi^2 EI_z/L_{eff}^2)[\sqrt{(I_w/I_z + L_{eff}^2 GI_t/(\pi^2 EI_z)})]$ = 88.9 [NCCI SN003 2(1)]
C_1 = 1.0 (conservative) $\bar{\lambda}_{LT} = \sqrt{(M_{c,y,Rd}/M_{cr})}$ = 0.81
$\bar{\lambda}_{LT,0}$ = 0.4 β = 0.75 [EC3 UK NA 2.17]
Use buckling curve b: α = 0.340 [EC3 Tables 6.3/6.4 NA2.17]
$\phi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta\bar{\lambda}_{LT}^2]$ = 0.82
$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta\bar{\lambda}_{LT}^2)}]$ = 0.81 [EC3 (6.56)]

$M_{b,y,Rd} = \chi_{LT}.M_{c,y,Rd}/\gamma_M$ = 0.81 x 64.6/1.0 = 52.5 kNm

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

Summary:

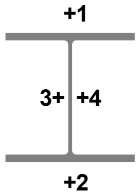
$N_{Ed}/N_{min,b,Rd}$ = 35.7/524 = 0.068 [1]
$M_{y,Ed}/M_{bs}$ = 6.28/52.5 = 0.120 [2]
$M_{z,Ed}/M_{z,cb,Rd}$ = 0.000 [3] x 1.5 [NCCI SN048]
Sum of stress ratios [1] + [2] + 1.5 x [3] = **0.188** **OK** *subject to all SN048b criteria being complied with*

Column calculation to EN1993-1-1 using S355 steel

Location: SC4

Length: 3.0 m.

Pos	Dur	Load	kN	Fact 6.10	Offset	M_{yy} 6.10	M_{zz} 6.10
2	G	B/F Beam: B5 [6.7 m.] : R1	49.1	66.3	100	-13.36	0.00
2	QA	B/F Beam: B5 [6.7 m.] : R1	5.9	8.8	100	-1.77	0.00
Total load kN			55.0	**75.1**		**-15.13**	**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 : **203 x 203 x 46 UKC** S355

Section properties: B = 203.6mm D = 203.2mm T = 11.0mm t = 7.2mm A_g = 58.7cm^2
i_y = 8.82cm i_z = 5.13cm $W_{pl,y}$ = 497cm^3 $W_{el,y}$ = 450cm^3

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

Classification: Flange: c/t = 88.0/11.0 = 8.00 <= 10ε (8.14): Class 2, compact
[Table 5.2] Web: c/t = 160.8/7.2 = 22.3 <= 396ε/(13α - 1) = 48.2 : Class 1, plastic
$\alpha = 0.5(1 + N_{Ed}/(f_y.c.t_w)) = 0.5 \times (1 + 75.1 \times 1000/(355 \times 161 \times 7.20)) = 0.591$ [SCI P362 Table 5.1 note 1]

Major axis: L_{Ey} = 1.0L = 3.00 m. Slenderness, λ_y = 3.00 x 100/8.82 = 34.0
Minor axis: L_{Ez} = 1.0L = 3.00 m. Slenderness, λ_z = 3.00 x 100/5.13 = 58.5

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} = 75.1 kN

Design compression resistance, $N_{c,Rd} = A.f_y/\gamma_{M0}$ = 58.7 x 100 x 355/(1.0 x 1000) = 2,084 kN OK

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

$\lambda_1 = 93.9\varepsilon = 93.9 \times 0.814 = 76.4$

Buckling about y-y (major) axis

$\bar{\lambda}_y = \lambda_y/\lambda_1 = 34.0/76.4 = 0.445$ [EC3 6.3.1.3]

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

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

Design buckling resistance, $N_{b,y,Rd} = \chi_y A.f_y/\gamma_{M1}$ = 0.91 x 58.7 x 100 x 355/(1.0 x 1000) = 1,892 kN OK [EC3 (6.47)]

Buckling about z-z (minor) axis

$\bar{\lambda}_z = \lambda_z/\lambda_1 = 58.5/76.4 = 0.765$ [EC3 6.3.1.3]

Use curve c: α = 0.490 $\phi = 0.5(1 + \alpha(\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) = 0.93$ [EC3 Table 6.2]

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

Design buckling resistance, $N_{b,z,Rd} = \chi_z A.f_y/\gamma_{M1}$ = 0.68 x 58.7 x 100 x 355/(1.0 x 1000) = 1,425 kN OK [EC3 (6.47)]

Bending about y-y (major) axis:

Design moment, $M_{y,Ed}$ = 15.13 kNm

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

Calculate buckling resistance moment

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

$M_{cr} = C_1(\pi^2 EI_z/L_{eff}^2)[\sqrt{(I_w/I_z + L_{eff}^2 GI_t/(\pi^2 EI_z))}] = 426$ [NCCI SN003 2(1)]
C_1 = 1.0 (conservative) $\bar{\lambda}_{LT} = \sqrt{(M_{c,y,Rd}/M_{cr})} = 0.64$
$\bar{\lambda}_{LT,0} = 0.4$ $\beta = 0.75$ [EC3 UK NA 2.17]
Use buckling curve b: α = 0.340 [EC3 Tables 6.3/6.4 NA2.17]
$\phi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta\bar{\lambda}_{LT}^2] = 0.70$
$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta\bar{\lambda}_{LT}^2)}] = 0.90$ [EC3 (6.56)]

$M_{b,y,Rd} = \chi_{LT}.M_{c,y,Rd}/\gamma_M$ = 0.90 x 176/1.0 = 158 kNm

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

Summary:

$N_{Ed}/N_{min,b,Rd}$ = 75.1/1425 = 0.053 [1]
$M_{y,Ed}/M_{bs}$ = 15.13/158 = 0.096 [2]
$M_{z,Ed}/M_{z,cb,Rd}$ = 0.000 [3] x 1.5 [NCCI SN048]

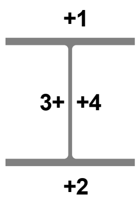
Sum of stress ratios [1] + [2] + 1.5 x [3] = **0.148** **OK** *subject to all SN048b criteria being complied with*

Column calculation to EN1993-1-1 using S355 steel

Location: SC5

Length: 3.0 m.

Pos	Dur	Load	kN	Fact	Offset	M_{yy}	M_{zz}
				6.10		6.10	6.10
1	G	B/F Beam: B6 [3.8 m.] : R1	17.3	23.4	100	4.71	0.00
1	QA	B/F Beam: B6 [3.8 m.] : R1	1.6	2.4	100	0.48	0.00
2	G	B/F Beam: B5 [6.7 m.] : R1	48.9	66.0	100	-13.31	0.00
2	QA	B/F Beam: B5 [6.7 m.] : R1	5.8	8.7	100	-1.74	0.00
Total load kN			73.6	**100.5**		**-9.87**	**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 : **203 x 203 x 46 UKC** S355

Section properties: B = 203.6mm D = 203.2mm T = 11.0mm t = 7.2mm A_g = 58.7cm^2
i_y = 8.82cm i_z = 5.13cm $W_{pl,y}$ = 497cm^3 $W_{el,y}$ = 450cm^3

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

Classification: Flange: c/t = 88.0/11.0 = 8.00 <= 10ε (8.14): Class 2, compact
[Table 5.2] Web: c/t = 160.8/7.2 = 22.3 <= 396ε/(13α - 1) = 45.5 : Class 1, plastic
$\alpha = 0.5(1 + N_{Ed}/(f_y.c.t_w))$ = 0.5 x (1 + 100 x 1000/(355 x 161 x 7.20)) = 0.622 [SCI P362 Table 5.1 note 1]

Major axis: L_{Ey} = 1.0L = 3.00 m. Slenderness, λ_y = 3.00 x 100/8.82 = 34.0
Minor axis: L_{Ez} = 1.0L = 3.00 m. Slenderness, λ_z = 3.00 x 100/5.13 = 58.5

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} = 100 kN

Design compression resistance, $N_{c,Rd} = A.f_y/\gamma_{M0}$ = 58.7 x 100 x 355/(1.0 x 1000) = 2,084 kN OK

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

$\lambda_1 = 93.9\varepsilon$ = 93.9 x 0.814 = 76.4

Buckling about y-y (major) axis

$\bar{\lambda}_y = \lambda_y/\lambda_1$ = 34.0/76.4 = 0.445 [EC3 6.3.1.3]

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

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

Design buckling resistance, $N_{b,y,Rd} = \chi_y A.f_y/\gamma_{M1}$ = 0.91 x 58.7 x 100 x 355/(1.0 x 1000) = 1,892 kN OK [EC3 (6.47)]

Buckling about z-z (minor) axis

$\bar{\lambda}_z = \lambda_z/\lambda_1$ = 58.5/76.4 = 0.765 [EC3 6.3.1.3]

Use curve c: α = 0.490 $\phi = 0.5(1 + \alpha(\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = 0.93 [EC3 Table 6.2]

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

Design buckling resistance, $N_{b,z,Rd} = \chi_z A.f_y/\gamma_{M1}$ = 0.68 x 58.7 x 100 x 355/(1.0 x 1000) = 1,425 kN OK [EC3 (6.47)]

Bending about y-y (major) axis:

Design moment, $M_{y,Ed}$ = 9.87 kNm

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

Calculate buckling resistance moment

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

$M_{cr} = C_1(\pi^2 EI_z/L_{eff}^2)[\sqrt{(I_w/I_z + L_{eff}^2 GI_t/(\pi^2 EI_z)}]$ = 426 [NCCI SN003 2(1)]
C_1 = 1.0 (conservative) $\bar{\lambda}_{LT} = \sqrt{(M_{c,y,Rd}/M_{cr})}$ = 0.64
$\bar{\lambda}_{LT,0}$ = 0.4 β = 0.75 [EC3 UK NA 2.17]
Use buckling curve b: α = 0.340 [EC3 Tables 6.3/6.4 NA2.17]
$\phi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta\bar{\lambda}_{LT}^2]$ = 0.70
$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta\bar{\lambda}_{LT}^2)}]$ = 0.90 [EC3 (6.56)]

$M_{b,y,Rd} = \chi_{LT}.M_{c,y,Rd}/\gamma_M$ = 0.90 x 176/1.0 = 158 kNm

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

Summary:

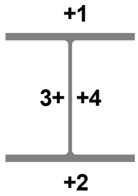
	$N_{Ed}/N_{min,b,Rd}$ = 100/1425 =	0.070	[1]
	$M_{y,Ed}/M_{bs}$ = 9.87/158 =	0.062	[2]
	$M_{z,Ed}/M_{z,cb,Rd}$ =	0.000	[3] x 1.5 [NCCI SN048]
Sum of stress ratios	[1] + [2] + 1.5 x [3] =	**0.133**	**OK** *subject to all SN048b criteria being complied with*

Column calculation to EN1993-1-1 using S355 steel

Location: SC6

Length: 3.0 m.

Pos	Dur	Load	kN	Fact	Offset	M_{yy}	M_{zz}
				6.10		6.10	6.10
2	G	B/F Beam: B6 [3.8 m.] : R2	7.76	10.48	100	-1.85	0.00
2	QA	B/F Beam: B6 [3.8 m.] : R2	1.60	2.40	100	-0.42	0.00
Total load kN			9.36	**12.88**		**-2.27**	**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 23 UKC** S355

Section properties: B = 152.2mm D = 152.4mm T = 6.8mm t = 5.8mm A_g = 29.2cm^2
i_y = 6.54cm i_z = 3.70cm $W_{pl,y}$ = 182cm^3 $W_{el,y}$ = 164cm^3

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

Classification: Flange: c/t = 65.6/6.8 = 9.65 <= 14ε (11.39): Class 3, semi-compact
[Table 5.2] Web: c/t = 123.6/5.8 = 21.3 <= 396ε/(13α - 1) = 55.3 : Class 1, plastic
$\alpha = 0.5(1 + N_{Ed}/(f_y.c.t_w)) = 0.5 \times (1 + 12.9 \times 1000/(355 \times 124 \times 5.80)) = 0.525$ [SCI P362 Table 5.1 note 1]

Major axis: L_{Ey} = 1.0L = 3.00 m. Slenderness, $\lambda_y = 3.00 \times 100/6.54 = 45.9$
Minor axis: L_{Ez} = 1.0L = 3.00 m. Slenderness, $\lambda_z = 3.00 \times 100/3.70 = 81.1$

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} = 12.88 kN

Design compression resistance, $N_{c,Rd} = A.f_y/\gamma_{M0} = 29.2 \times 100 \times 355/(1.0 \times 1000) = 1{,}037$ kN OK

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

$\lambda_1 = 93.9\varepsilon = 93.9 \times 0.814 = 76.4$

Buckling about y-y (major) axis

$\bar{\lambda}_y = \lambda_y/\lambda_1 = 45.9/76.4 = 0.600$ [EC3 6.3.1.3]

Use curve b: $\alpha = 0.340$ $\phi = 0.5(1 + \alpha(\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) = 0.75$ [EC3 (6.49)]

Flexural buckling reduction factor, $\chi_y = 1/(\phi + \sqrt{(\phi^2 - \bar{\lambda}_y^2)}) = 0.84$ [EC3 (6.49)]

Design buckling resistance, $N_{b,y,Rd} = \chi_y A.f_y/\gamma_{M1} = 0.84 \times 29.2 \times 100 \times 355/(1.0 \times 1000) = 867$ kN OK [EC3 (6.47)]

Buckling about z-z (minor) axis

$\bar{\lambda}_z = \lambda_z/\lambda_1 = 81.1/76.4 = 1.06$ [EC3 6.3.1.3]

Use curve c: $\alpha = 0.490$ $\phi = 0.5(1 + \alpha(\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) = 1.27$ [EC3 Table 6.2]

Flexural buckling reduction factor, $\chi_z = 1/(\phi + \sqrt{(\phi^2 - \bar{\lambda}_z^2)}) = 0.51$ [EC3 (6.49)]

Design buckling resistance, $N_{b,z,Rd} = \chi_z A.f_y/\gamma_{M1} = 0.51 \times 29.2 \times 100 \times 355/(1.0 \times 1000) = 524$ kN OK [EC3 (6.47)]

Bending about y-y (major) axis:

Design moment, $M_{y,Ed}$ = 2.27 kNm

Moment resistance, $M_{c,y,Rd} = f_y.W_{pl,y} = 355 \times 182/1000 = 64.6$ kNm OK [SN048b: use $W_{pl,y}$ although Class 3]

Calculate buckling resistance moment

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

$M_{cr} = C_1(\pi^2 EI_z/L_{eff}^2)[\sqrt{(I_w/I_z + L_{eff}^2 GI_t/(\pi^2 EI_z)}] = 88.9$ [NCCI SN003 2(1)]
C_1 = 1.0 (conservative) $\bar{\lambda}_{LT} = \sqrt{(M_{c,y,Rd}/M_{cr})} = 0.81$
$\bar{\lambda}_{LT,0} = 0.4$ $\beta = 0.75$ [EC3 UK NA 2.17]
Use buckling curve b: $\alpha = 0.340$ [EC3 Tables 6.3/6.4 NA2.17]
$\phi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta\bar{\lambda}_{LT}^2] = 0.82$
$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta\bar{\lambda}_{LT}^2)}] = 0.81$ [EC3 (6.56)]

$M_{b,y,Rd} = \chi_{LT}.M_{c,y,Rd}/\gamma_M = 0.81 \times 64.6/1.0 = 52.5$ kNm

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

Summary:

$N_{Ed}/N_{min,b,Rd}$ = 12.88/524 = 0.025 [1]
$M_{y,Ed}/M_{bs}$ = 2.27/52.5 = 0.043 [2]
$M_{z,Ed}/M_{z,cb,Rd}$ = 0.000 [3] x 1.5 [NCCI SN048]

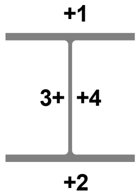
Sum of stress ratios [1] + [2] + 1.5 x [3] = **0.068** **OK** *subject to all SN048b criteria being complied with*

Column calculation to EN1993-1-1 using S355 steel

Location: SC7

Length: 3.0 m.

Pos	Dur	Load	kN	Fact	Offset	M_{yy}	M_{zz}
				6.10		6.10	6.10
2	G	B/F Beam: B7 [5.1 m.] : R1	6.25	8.44	100	-1.49	0.00
2	QA	B/F Beam: B7 [5.1 m.] : R1	2.14	3.21	100	-0.57	0.00
Total load kN			8.39	**11.65**		**-2.05**	**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 23 UKC** S355

Section properties: B = 152.2mm D = 152.4mm T = 6.8mm t = 5.8mm A_g = 29.2cm^2
i_y = 6.54cm i_z = 3.70cm $W_{pl,y}$ = 182cm^3 $W_{el,y}$ = 164cm^3

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

Classification: Flange: c/t = 65.6/6.8 = 9.65 <= 14ε (11.39): Class 3, semi-compact
[Table 5.2] Web: c/t = 123.6/5.8 = 21.3 <= 396ε/(13α - 1) = 55.6 : Class 1, plastic
$\alpha = 0.5(1 + N_{Ed}/(f_y.c.t_w))$ = 0.5 x (1 + 11.6 x 1000/(355 x 124 x 5.80)) = 0.523 [SCI P362 Table 5.1 note 1]

Major axis: L_{Ey} = 1.0L = 3.00 m. Slenderness, λ_y = 3.00 x 100/6.54 = 45.9
Minor axis: L_{Ez} = 1.0L = 3.00 m. Slenderness, λ_z = 3.00 x 100/3.70 = 81.1

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} = 11.65 kN

Design compression resistance, $N_{c,Rd} = A.f_y/\gamma_{M0}$ = 29.2 x 100 x 355/(1.0 x 1000) = 1,037 kN OK

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

$\lambda_1 = 93.9\varepsilon$ = 93.9 x 0.814 = 76.4

Buckling about y-y (major) axis

$\bar{\lambda}_y = \lambda_y/\lambda_1$ = 45.9/76.4 = 0.600 [EC3 6.3.1.3]

Use curve b: α = 0.340 $\phi = 0.5(1 + \alpha(\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = 0.75 [EC3 (6.49)]

Flexural buckling reduction factor, $\chi_y = 1/(\phi + \sqrt{(\phi^2 - \bar{\lambda}_y^2)})$ = 0.84 [EC3 (6.49)]

Design buckling resistance, $N_{b,y,Rd} = \chi_y A.f_y/\gamma_{M1}$ = 0.84 x 29.2 x 100 x 355/(1.0 x 1000) = 867 kN OK [EC3 (6.47)]

Buckling about z-z (minor) axis

$\bar{\lambda}_z = \lambda_z/\lambda_1$ = 81.1/76.4 = 1.06 [EC3 6.3.1.3]

Use curve c: α = 0.490 $\phi = 0.5(1 + \alpha(\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = 1.27 [EC3 Table 6.2]

Flexural buckling reduction factor, $\chi_z = 1/(\phi + \sqrt{(\phi^2 - \bar{\lambda}_z^2)})$ = 0.51 [EC3 (6.49)]

Design buckling resistance, $N_{b,z,Rd} = \chi_z A.f_y/\gamma_{M1}$ = 0.51 x 29.2 x 100 x 355/(1.0 x 1000) = 524 kN OK [EC3 (6.47)]

Bending about y-y (major) axis:

Design moment, $M_{y,Ed}$ = 2.05 kNm

Moment resistance, $M_{c,y,Rd} = f_y.W_{pl,y}$ = 355 x 182/1000 = 64.6 kNm OK [SN048b: use $W_{pl,y}$ although Class 3]

Calculate buckling resistance moment

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

$M_{cr} = C_1(\pi^2 EI_z/L_{eff}^2)[\sqrt{(I_w/I_z + L_{eff}^2 GI_t/(\pi^2 EI_z)}]$ = 88.9 [NCCI SN003 2(1)]
C_1 = 1.0 (conservative) $\bar{\lambda}_{LT} = \sqrt{(M_{c,y,Rd}/M_{cr})}$ = 0.81
$\bar{\lambda}_{LT,0}$ = 0.4 β = 0.75 [EC3 UK NA 2.17]
Use buckling curve b: α = 0.340 [EC3 Tables 6.3/6.4 NA2.17]
$\phi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta\bar{\lambda}_{LT}^2]$ = 0.82
$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta\bar{\lambda}_{LT}^2)}]$ = 0.81 [EC3 (6.56)]

$M_{b,y,Rd} = \chi_{LT}.M_{c,y,Rd}/\gamma_M$ = 0.81 x 64.6/1.0 = 52.5 kNm

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

Summary:

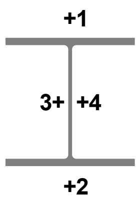
$N_{Ed}/N_{min,b,Rd}$ = 11.65/524 = 0.022 [1]
$M_{y,Ed}/M_{bs}$ = 2.05/52.5 = 0.039 [2]
$M_{z,Ed}/M_{z,cb,Rd}$ = 0.000 [3] x 1.5 [NCCI SN048]
Sum of stress ratios [1] + [2] + 1.5 x [3] = **0.061** **OK** *subject to all SN048b criteria being complied with*

Column calculation to EN1993-1-1 using S355 steel

Location: SC8

Length: 3.0 m.

Pos	Dur	Load	kN	Fact	Offset	M_{yy}	M_{zz}
				6.10		6.10	6.10
2	G	B/F Beam: B8 [3.8 m.] : R1	4.65	6.28	100	-1.11	0.00
2	QA	B/F Beam: B8 [3.8 m.] : R1	1.60	2.40	100	-0.42	0.00
Total load kN			6.25	**8.68**		**-1.53**	**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 23 UKC** S355

Section properties: B = 152.2mm D = 152.4mm T = 6.8mm t = 5.8mm A_g = 29.2cm^2
i_y = 6.54cm i_z = 3.70cm $W_{pl,y}$ = 182cm^3 $W_{el,y}$ = 164cm^3

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

Classification: Flange: c/t = 65.6/6.8 = 9.65 <= 14ε (11.39): Class 3, semi-compact
[Table 5.2] Web: c/t = 123.6/5.8 = 21.3 <= 396ε/(13α - 1) = 56.3 : Class 1, plastic
$\alpha = 0.5(1 + N_{Ed}/(f_y.c.t_w))$ = 0.5 x (1 + 8.68 x 1000/(355 x 124 x 5.80)) = 0.517 [SCI P362 Table 5.1 note 1]

Major axis: L_{Ey} = 1.0L = 3.00 m. Slenderness, λ_y = 3.00 x 100/6.54 = 45.9
Minor axis: L_{Ez} = 1.0L = 3.00 m. Slenderness, λ_z = 3.00 x 100/3.70 = 81.1

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} = 8.68 kN

Design compression resistance, $N_{c,Rd} = A.f_y/\gamma_{M0}$ = 29.2 x 100 x 355/(1.0 x 1000) = 1,037 kN OK

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

$\lambda_1 = 93.9\varepsilon$ = 93.9 x 0.814 = 76.4

Buckling about y-y (major) axis

$\bar{\lambda}_y = \lambda_y/\lambda_1$ = 45.9/76.4 = 0.600 [EC3 6.3.1.3]

Use curve b: α = 0.340 $\phi = 0.5(1 + \alpha(\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = 0.75 [EC3 (6.49)]

Flexural buckling reduction factor, $\chi_y = 1/(\phi + \sqrt{(\phi^2 - \bar{\lambda}_y^2)})$ = 0.84 [EC3 (6.49)]

Design buckling resistance, $N_{b,y,Rd} = \chi_y A.f_y/\gamma_{M1}$ = 0.84 x 29.2 x 100 x 355/(1.0 x 1000) = 867 kN OK [EC3 (6.47)]

Buckling about z-z (minor) axis

$\bar{\lambda}_z = \lambda_z/\lambda_1$ = 81.1/76.4 = 1.06 [EC3 6.3.1.3]

Use curve c: α = 0.490 $\phi = 0.5(1 + \alpha(\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = 1.27 [EC3 Table 6.2]

Flexural buckling reduction factor, $\chi_z = 1/(\phi + \sqrt{(\phi^2 - \bar{\lambda}_z^2)})$ = 0.51 [EC3 (6.49)]

Design buckling resistance, $N_{b,z,Rd} = \chi_z A.f_y/\gamma_{M1}$ = 0.51 x 29.2 x 100 x 355/(1.0 x 1000) = 524 kN OK [EC3 (6.47)]

Bending about y-y (major) axis:

Design moment, $M_{y,Ed}$ = 1.53 kNm

Moment resistance, $M_{c,y,Rd} = f_y.W_{pl,y}$ = 355 x 182/1000 = 64.6 kNm OK [SN048b: use $W_{pl,y}$ although Class 3]

Calculate buckling resistance moment

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

$M_{cr} = C_1(\pi^2 EI_z/L_{eff}^2)[\sqrt{(I_w/I_z + L_{eff}^2 GI_t/(\pi^2 EI_z)})]$ = 88.9 [NCCI SN003 2(1)]
C_1 = 1.0 (conservative) $\bar{\lambda}_{LT} = \sqrt{(M_{c,y,Rd}/M_{cr})}$ = 0.81
$\bar{\lambda}_{LT,0}$ = 0.4 β = 0.75 [EC3 UK NA 2.17]
Use buckling curve b: α = 0.340 [EC3 Tables 6.3/6.4 NA2.17]
$\phi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta\bar{\lambda}_{LT}^2]$ = 0.82
$\chi_{LT} = 1/[\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta\bar{\lambda}_{LT}^2)}]$ = 0.81 [EC3 (6.56)]

$M_{b,y,Rd} = \chi_{LT}.M_{c,y,Rd}/\gamma_M$ = 0.81 x 64.6/1.0 = 52.5 kNm

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

Summary:

$N_{Ed}/N_{min,b,Rd}$ = 8.68/524 = 0.017 [1]
$M_{y,Ed}/M_{bs}$ = 1.53/52.5 = 0.029 [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] = **0.046** **OK** *subject to all SN048b criteria being complied with*

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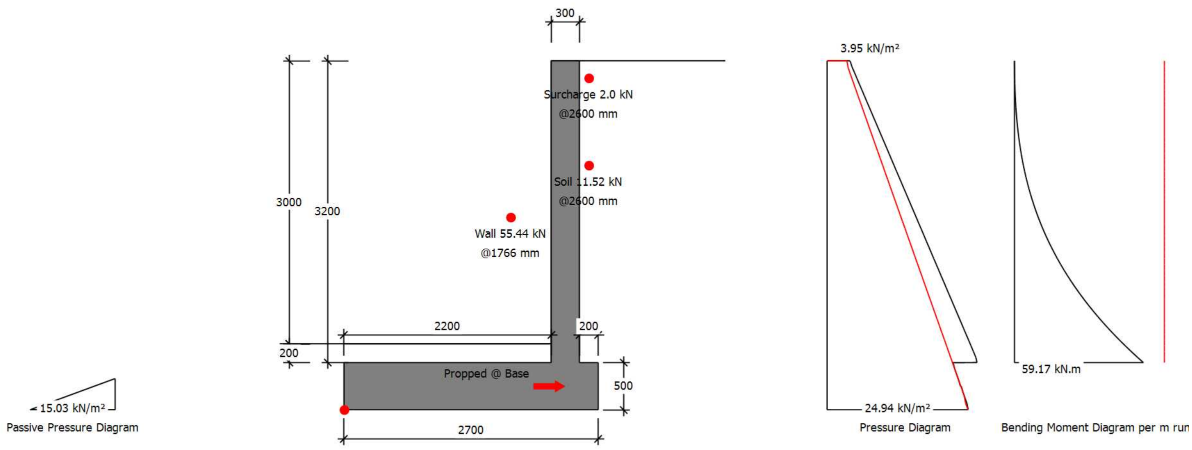
MasterKey : Retaining Wall Design to BS 8002 : 1994 and BS 8110 : 1997
MAIN RETAINING WALL
Reinforced Concrete Retaining Wall with Reinforced Base



Summary of Design Data

Notes	All dimensions are in mm and all forces are per metre run
Material Densities (kN/m³)	Soil 18.00, Concrete 24.00
Concrete grade	fcu 30 N/mm², Permissible tensile stress 0.250 N/mm²
Concrete covers (mm)	Wall inner cover 30 mm, Wall outer cover 30 mm, Base cover 50 mm
Reinforcement design	fy 500 N/mm² designed to BS 8110: 1997
Surcharge and Water Table	Surcharge 10.00 kN/m², Fully drained
Unplanned excavation depth	Front of wall 370 mm

† The Engineer must satisfy him/herself to the reinforcement detailing requirements of the relevant codes of practice

Additional Loads

Wall Propped at Base Level	Therefore no sliding check is required
† Dimensions	

Soil Properties

Bearing pressure	Premissable service pressure @ front 100.00 kN/m², @ back 100.00 kN/m²
Back Soil Friction and Cohesion	ϕ = Atn(Tan(30)/1.2) = 25.69°
Base Friction and Cohesion	δ = Atn(0.75xTan(Atn(Tan(30)/1.2))) = 19.84°
Front Soil Friction and Cohesion	ϕ = Atn(Tan(30)/1.2) = 25.69°

Loading Cases

G_{Soil}- Soil Self Weight, G_{Wall}- Wall & Base Self Weight, Fv_{Heel}- Vertical Loads over Heel,
P_a- Active Earth Pressure, $P_{surcharge}$- Earth pressure from surcharge, P_p- Passive Earth Pressure

Case 1: Geotechnical Design	$1.00\ G_{Soil}+1.00\ G_{Wall}+1.00\ Fv_{Heel}+1.00\ P_a+1.00\ P_{surcharge}+1.00\ P_p$
Case 2: Structural Ultimate Design	$1.40\ G_{Soil}+1.40\ G_{Wall}+1.60\ Fv_{Heel}+1.00\ P_a+1.00\ P_{surcharge}+1.00\ P_p$

Geotechnical Design

Wall Stability - Virtual Back Pressure

Case 1 Overturning/Stabilising	72.444/133.036	0.545	OK

Wall Sliding - Virtual Back Pressure

$Fx/(Rx_{Friction}+Rx_{Passive})$	0.000/(24.884+2.487)	0.000	OK
Prop Reaction Case 2 (Service)	52.5 kN @ Base		

Soil Pressure

Virtual Back	52.323/100 kN/m², Length under pressure 2.636 m	0.523	OK
Wall Back	67.848/100 kN/m², Length under pressure 2.033 m	0.678	OK

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Structural Design

Prop Reaction

Maximum Prop Reaction (Ultimate)	60.8 kN @ Base		

Wall Design (Inner Steel)

Critical Section	Critical @ 0 mm from base, Case 2		
Steel Provided (Cover)	Main H10@125 (30 mm) Dist. H10@200 (40 mm)	628 mm²	OK
Compression Steel Provided (Cover)	Main H10@300 (30 mm) Dist. H10@300 (40 mm)	262 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	265 mm, 1000 mm, 628 mm², 500 N/mm², 30.0 N/mm²	252 mm	
Mr=fn(above,As',d',x,x/d)	262 mm², 35 mm, 23 mm, 0.09	68.8 kN.m	
Moment Capacity Check (M/Mr)	M 59.2 kN.m, Mr 68.8 kN.m	0.860	OK
Shear Capacity Check	F 49.0 kN, vc 0.461 N/mm², Fvr 122.1 kN	0.40	OK

Base Top Steel Design

Steel Provided (Cover)	Main H10@100 (50 mm) Dist. H10@100 (60 mm)	785 mm²	OK
Compression Steel Provided (Cover)	Main H10@100 (50 mm) Dist. H10@100 (60 mm)	785 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	445 mm, 1000 mm, 785 mm², 500 N/mm², 30 N/mm²	423 mm	
Mr=fn(above,As',d',x,x/d)	785 mm², 55 mm, 28 mm, 0.06	144.4 kN.m	
Moment Capacity Check (M/Mr)	M 2.0 kN.m, Mr 144.4 kN.m	0.014	OK
Shear Capacity Check	F 20.1 kN, vc 0.367 N/mm², Fvr 163.2 kN	0.12	OK

Base Bottom Steel Design

Steel Provided (Cover)	Main H10@100 (50 mm) Dist. H10@100 (60 mm)	785 mm²	OK
Compression Steel Provided (Cover)	Main H10@100 (50 mm) Dist. H10@100 (60 mm)	785 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	445 mm, 1000 mm, 785 mm², 500 N/mm², 30 N/mm²	423 mm	
Mr=fn(above,As',d',x,x/d)	785 mm², 55 mm, 28 mm, 0.06	144.4 kN.m	
Moment Capacity Check (M/Mr)	M 73.4 kN.m, Mr 144.4 kN.m	0.508	OK
Shear Capacity Check	F 52.1 kN, vc 0.367 N/mm², Fvr 163.2 kN	0.32	OK

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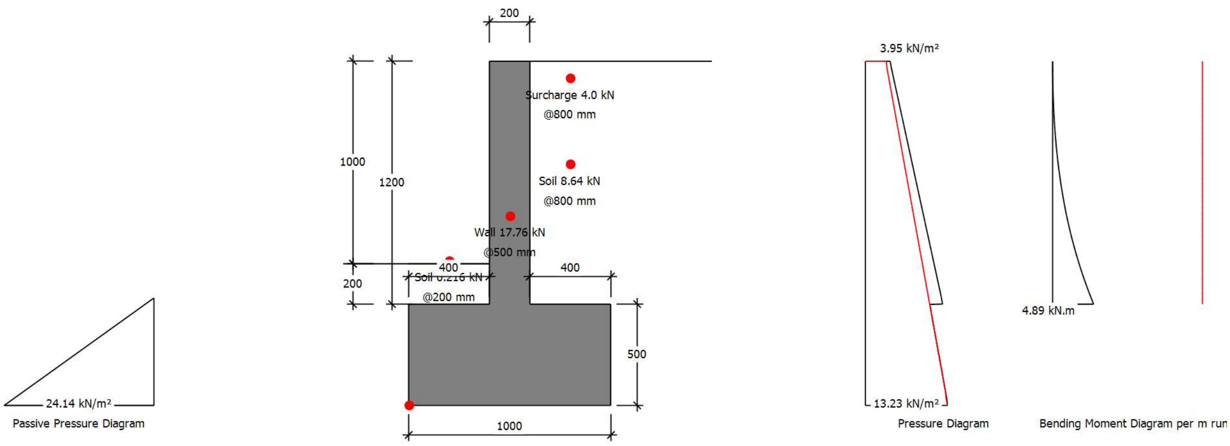
MasterKey : Retaining Wall Design to BS 8002 : 1994 and BS 8110 : 1997
GARDEN RETAINING WALL
Reinforced Concrete Retaining Wall with Reinforced Base



Summary of Design Data

Notes	All dimensions are in mm and all forces are per metre run
Material Densities (kN/m³)	Soil 18.00, Concrete 24.00
Concrete grade	fcu 30 N/mm², Permissible tensile stress 0.250 N/mm²
Concrete covers (mm)	Wall inner cover 30 mm, Wall outer cover 30 mm, Base cover 50 mm
Reinforcement design	fy 500 N/mm² designed to BS 8110: 1997
Surcharge and Water Table	Surcharge 10.00 kN/m², Fully drained
Unplanned excavation depth	Front of wall 170 mm

† The Engineer must satisfy him/herself to the reinforcement detailing requirements of the relevant codes of practice

Soil Properties

Bearing pressure	Premissable service pressure @ front 100.00 kN/m², @ back 100.00 kN/m²
Back Soil Friction and Cohesion	ϕ = Atn(Tan(30)/1.2) = 25.69°
Base Friction and Cohesion	δ = Atn(0.75xTan(Atn(Tan(30)/1.2))) = 19.84°
Front Soil Friction and Cohesion	ϕ = Atn(Tan(30)/1.2) = 25.69°

Loading Cases

G_{Soil}- Soil Self Weight, G_{Wall}- Wall & Base Self Weight, Fv_{Heel}- Vertical Loads over Heel,
P_a- Active Earth Pressure, $P_{surcharge}$- Earth pressure from surcharge, P_p- Passive Earth Pressure

Case 1: Geotechnical Design	$1.00\ G_{Soil}+1.00\ G_{Wall}+1.00\ Fv_{Heel}+1.00\ P_a+1.00\ P_{surcharge}+1.00\ P_p$
Case 2: Structural Ultimate Design	$1.40\ G_{Soil}+1.40\ G_{Wall}+1.60\ Fv_{Heel}+1.00\ P_a+1.00\ P_{surcharge}+1.00\ P_p$

Geotechnical Design

Wall Stability - Virtual Back Pressure

Case 1 Overturning/Stabilising	9.588/19.035	0.504	OK

Wall Sliding - Virtual Back Pressure

$Fx/(Rx_{Friction}+ Rx_{Passive})$	14.105/(11.048+6.410)	0.808	OK

Soil Pressure

Virtual Back	66.144/100 kN/m², Length under pressure 0.926 m	0.661	OK
Wall Back	69.959/100 kN/m², Length under pressure 0.875 m	0.700	OK

Structural Design

Wall Design (Inner Steel)

Critical Section	Critical @ 0 mm from base, Case 2		
Steel Provided (Cover)	Main H10@300 (30 mm) Dist. H10@300 (40 mm)	262 mm²	OK
Compression Steel Provided (Cover)	Main H10@300 (30 mm) Dist. H10@300 (40 mm)	262 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	165 mm, 1000 mm, 262 mm², 500 N/mm², 30.0 N/mm²	157 mm	

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Mr=fn(above,As',d',x,x/d)	262 mm², 35 mm, 9 mm, 0.06	17.9 kN.m	
Moment Capacity Check (M/Mr)	M 4.9 kN.m, Mr 17.9 kN.m	0.274	OK
Shear Capacity Check	F 9.8 kN, vc 0.454 N/mm², Fvr 74.9 kN	0.13	OK

Base Top Steel Design

Steel Provided (Cover)	Main H10@100 (50 mm) Dist. H10@100 (60 mm)	785 mm²	OK
Compression Steel Provided (Cover)	Main H10@100 (50 mm) Dist. H10@100 (60 mm)	785 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	445 mm, 1000 mm, 785 mm², 500 N/mm², 30 N/mm²	423 mm	
Mr=fn(above,As',d',x,x/d)	785 mm², 55 mm, 28 mm, 0.06	144.4 kN.m	
Moment Capacity Check (M/Mr)	M 3.2 kN.m, Mr 144.4 kN.m	0.022	OK
Shear Capacity Check	F 14.4 kN, vc 0.367 N/mm², Fvr 163.2 kN	0.09	OK

Base Bottom Steel Design

Steel Provided (Cover)	Main H10@100 (50 mm) Dist. H10@100 (60 mm)	785 mm²	OK
Compression Steel Provided (Cover)	Main H10@100 (50 mm) Dist. H10@100 (60 mm)	785 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	445 mm, 1000 mm, 785 mm², 500 N/mm², 30 N/mm²	423 mm	
Mr=fn(above,As',d',x,x/d)	785 mm², 55 mm, 28 mm, 0.06	144.4 kN.m	
Moment Capacity Check (M/Mr)	M 3.7 kN.m, Mr 144.4 kN.m	0.026	OK
Shear Capacity Check	F 17.1 kN, vc 0.367 N/mm², Fvr 163.2 kN	0.10	OK

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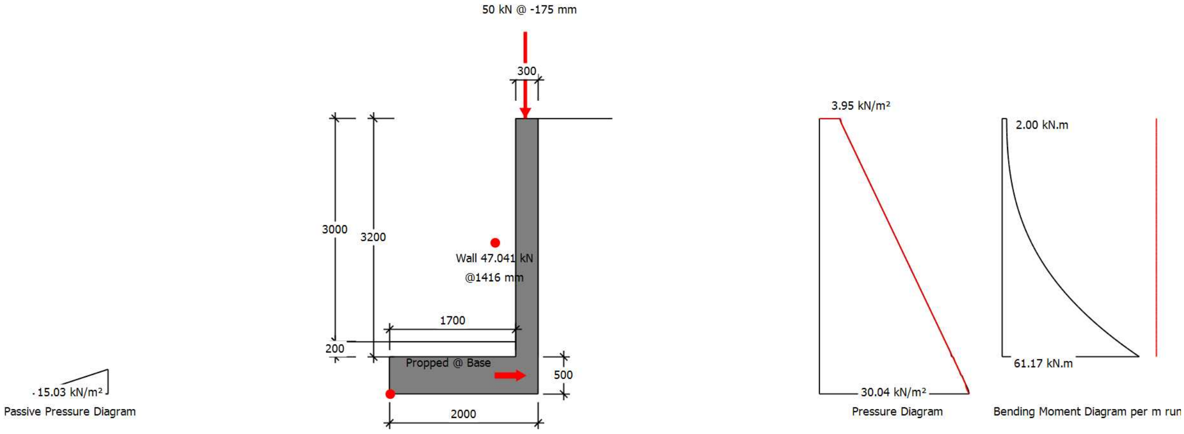
MasterKey : Retaining Wall Design to BS 8002 : 1994 and BS 8110 : 1997
MAIN RETAINING WALL- UNDER THE WALL
Reinforced Concrete Retaining Wall with Reinforced Base



Summary of Design Data

Notes	All dimensions are in mm and all forces are per metre run
Material Densities (kN/m³)	Soil 18.00, Concrete 24.00
Concrete grade	fcu 30 N/mm², Permissible tensile stress 0.250 N/mm²
Concrete covers (mm)	Wall inner cover 30 mm, Wall outer cover 30 mm, Base cover 50 mm
Reinforcement design	fy 500 N/mm² designed to BS 8110: 1997
Surcharge and Water Table	Surcharge 10.00 kN/m², Fully drained
Unplanned excavation depth	Front of wall 370 mm

† The Engineer must satisfy him/herself to the reinforcement detailing requirements of the relevant codes of practice

Additional Loads

Wall Propped at Base Level	Therefore no sliding check is required
Vertical Line Load	50 kN/m @ X -175 mm and Y 0 mm - Load type Live
† Dimensions	Ties, line loads and partial loads are measured from the inner top edge of the wall

Soil Properties

Bearing pressure	Premissable service pressure @ front 100.00 kN/m², @ back 100.00 kN/m²
Back Soil Friction and Cohesion	ϕ = Atn(Tan(30)/1.2) = 25.69°
Base Friction and Cohesion	δ = Atn(0.75xTan(Atn(Tan(30)/1.2))) = 19.84°
Front Soil Friction and Cohesion	ϕ = Atn(Tan(30)/1.2) = 25.69°

Loading Cases

G_{Wall}- Wall & Base Self Weight, Fv_{Heel}- Vertical Loads over Heel, P_a- Active Earth Pressure, $P_{surcharge}$- Earth pressure from surcharge, P_p- Passive Earth Pressure

Case 1: Geotechnical Design	$1.00\ G_{Wall}+1.00\ Fv_{Heel}+1.00\ P_a+1.00\ P_{surcharge}+1.00\ P_p$
Case 2: Structural Ultimate Design	$1.40\ G_{Wall}+1.60\ Fv_{Heel}+1.00\ P_a+1.00\ P_{surcharge}+1.00\ P_p$

Geotechnical Design

Wall Stability - Virtual Back Pressure

Case 1 Overturning/Stabilising	87.285/157.876	0.553	OK

Wall Sliding - Virtual Back Pressure

$Fx/(Rx_{Friction}+Rx_{Passive})$	0.000/(35.017+2.487)	0.000	OK
Prop Reaction Case 2 (Service)	63.3 kN @ Base		

Soil Pressure

Virtual Back (No uplift)	Max(88.196/100, 8.840/100) kN/m²	0.882	OK
Wall Back (No uplift)	Max(87.621/100, 9.415/100) kN/m²	0.876	OK

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Structural Design

Prop Reaction

Maximum Prop Reaction (Ultimate)	63.2 kN @ Base		

Wall Design (Inner Steel)

Critical Section	Critical @ 0 mm from base, Case 2		
Steel Provided (Cover)	Main H10@125 (30 mm) Dist. H10@200 (40 mm)	628 mm²	OK
Compression Steel Provided (Cover)	Main H10@300 (30 mm) Dist. H10@300 (40 mm)	262 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	265 mm, 1000 mm, 628 mm², 500 N/mm², 30.0 N/mm²	252 mm	
Mr=fn(above,As',d',x,x/d)	262 mm², 35 mm, 23 mm, 0.09	68.8 kN.m	
Moment Capacity Check (M/Mr)	M 61.2 kN.m, Mr 68.8 kN.m	0.889	OK
Wall Axial Design (N/Ncap)	N 112.3 kN, Ncap 3600.0 kN	0.031	OK
Wall Slenderness λ	Leff/tk =2.00x3200.0/300.0	21.3	OK
Kmin = (Nuz-N)/(Nuz-Nbal)	Min(1.0, 4000.0 - 112.3)/(4000.0 - 1687.9)	1.0	
M_{add}= N.Kmin.h.λ^2/2000	112.3x1.0x300.0x21.3²/2000	7.6kN.m	
(M+Madd)/Mr_{Axial}	M+Madd 68.8 kN, Mr_{Axail}83.8 kN.m	0.820	OK
Shear Capacity Check	F 49.0 kN, vc 0.461 N/mm², Fvr 122.1 kN	0.40	OK

Base Top Steel Design

Steel Provided (Cover)	Main H10@100 (50 mm) Dist. H10@100 (60 mm)	785 mm²	OK
Compression Steel Provided (Cover)	Main H10@100 (50 mm) Dist. H10@100 (60 mm)	785 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	445 mm, 1000 mm, 785 mm², 500 N/mm², 30 N/mm²	423 mm	
Mr=fn(above,As',d',x,x/d)	785 mm², 55 mm, 28 mm, 0.06	144.4 kN.m	
Moment Capacity Check (M/Mr)	M 0.0 kN.m, Mr 144.4 kN.m	0.000	OK
Shear Capacity Check	F 0.0 kN, vc 0.367 N/mm², Fvr 163.2 kN	0.00	OK

Base Bottom Steel Design

Steel Provided (Cover)	Main H10@100 (50 mm) Dist. H10@100 (60 mm)	785 mm²	OK
Compression Steel Provided (Cover)	Main H10@100 (50 mm) Dist. H10@100 (60 mm)	785 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	445 mm, 1000 mm, 785 mm², 500 N/mm², 30 N/mm²	423 mm	
Mr=fn(above,As',d',x,x/d)	785 mm², 55 mm, 28 mm, 0.06	144.4 kN.m	
Moment Capacity Check (M/Mr)	M 75.0 kN.m, Mr 144.4 kN.m	0.519	OK
Shear Capacity Check	F 92.9 kN, vc 0.367 N/mm², Fvr 163.2 kN	0.57	OK