

Beam: Mk B

Span: 3.4 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
U D	floor dead	0.7*2.6	0		L	3.09	3.09	3.2
U L	Floor live	1.5*2.6	0		L	6.63	6.63	6.8
U L	Pitched roof live	0.6*2.6	0		L	2.65	2.65	2.7
U D	Pitched roof dead	1.3*2.6	0		L	5.75	5.75	5.9
U D	wall	4.2*2.5	0		L	17.85	17.85	18.3
U D	o.w.	0.5	0		L	0.85	0.85	0.9
Total load (unfactored):						73.6 kN	36.82	36.82
							36.82	37.7
Dead/Permanent (unfactored):						55.1 kN	27.54	27.54
Live/Variable (unfactored):						18.6 kN	9.28	9.28
Total load (factored):						106.8 kN	53.41	53.41

Load types: U:UDL; Load positions are measured in m. from R1; Load durations: D: Dead; L: Live

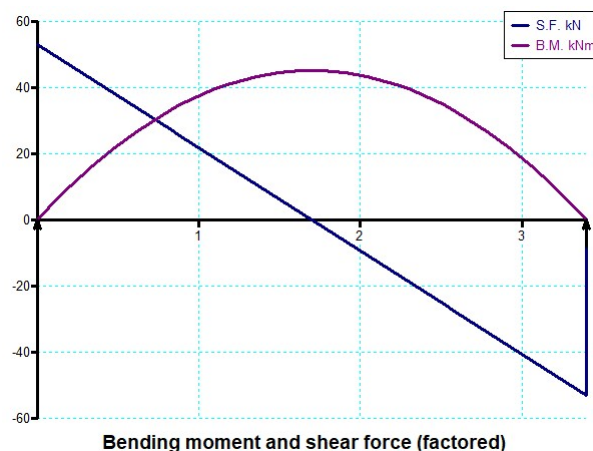
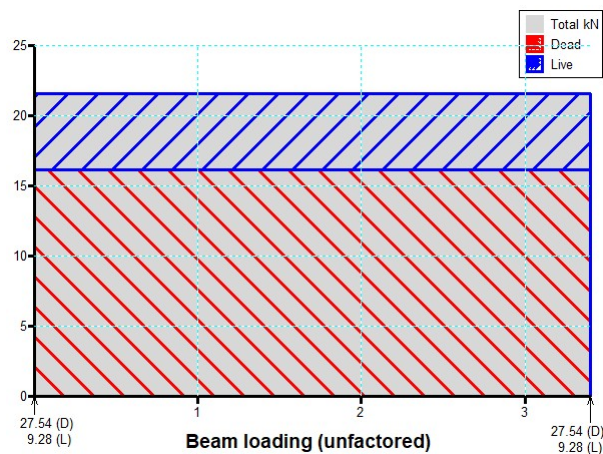
Maximum B.M. = 45.4 kNm (factored) at 1.70 m. from R1

Maximum S.F. = 53.4 kN (factored) at R1

Mid-span deflections: Dead: $28.2 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $9.5 \times 10^8 / EI$

Total: $37.7 \times 10^8 / EI$



Beam calculation to BS5950-1:2000 using S355 steel

SECTION SIZE : 203 x 203 x 46 UC S355 (semi-compact)

D=203.2 mm B=203.6 mm t=7.2 mm T=11.0 mm $I_x=4,570 \text{ cm}^4$ $r_y=5.13 \text{ cm}$ $S_x=497 \text{ cm}^3$ $x=17.7$

Section classification: T = 11.0mm $p_y = 355 \text{ N/mm}^2$ $e = \sqrt{(275/355)} = 0.88$

(Table 11)

Flange: $b/T = 101.8/11.0 = 9.25$ (≤ 15 Class 3 semi-compact)

Web: $d/t = 160.8/7.2 = 22.3$ (≤ 80 Class 1 plastic)

Section classification is Class 3, semi-compact $S_{x,eff} = 490 \text{ cm}^3$

Shear

Maximum S.F. = 53 kN

Shear capacity = $0.6 p_y t D = 0.6 \times 355 \times 7.2 \times 203.2/1000 = 312 \text{ kN OK}$

Bending

Maximum B.M. = 45.4 kNm

Moment capacity, $M_c = p_y S_{x,eff} = 355 \times 490/1000 = 174 \text{ kNm OK}$

Lateral-torsional buckling

Beam is laterally restrained at supports only

Restraint condition at R1 and R2: Compression flange laterally restrained. Nominal torsional restraint. Both flanges free to rotate on plan (1.0L)

Effective length = $1.0L = 3.40 \text{ m}$. [Table 13]

Bending strength, $p_b = 296 \text{ N/mm}^2$

Maximum moment within segment, $M_x = 45.4 \text{ kNm}$

Equivalent uniform moment factor, $m_{LT} = 0.925$ ($M_2 = 34.0$, $M_3 = 45.4$, $M_4 = 34.0$)

Equivalent uniform moment = $0.925 \times 45.4 = 42.0 \text{ kNm}$

Buckling resistance moment, $M_b = p_b S_{x,eff} = 296 \times 490/1000 = 145 \text{ kNm OK}$

Web buckling and crushing have not been checked

Deflection

LL deflection = $9.50 \times 1e8 / (205,000 \times 4,570) = 1.0 \text{ mm}$ (L/3353) OK

TL deflection = $37.69 \times 1e8 / (205,000 \times 4,570) = 4.0 \text{ mm}$ (L/845)

Provide 8mm Top Plate, width to support brickwork over Plate welded by a 6mm fillet weld 150mm hit 100mm miss

Beam: Mk C

Span: 5.0 m.

+ RH Cantilever: 2.7 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	SpanDef
U L	roof	0.6*1.7	0		L	2.55	2.55	8
U D	roof	1.3*1.7	0		L	5.53	5.53	18
R D	wall	4.2*2.5	2.5		7.7	-1.09	55.69	-17
R D	o.w.	0.95	0.0		7.7	1.68	5.63	2
P D	Beam: Mk B : R1	27.54 [B/F]	2.5			13.77	13.77	72
P L	Beam: Mk B : R1	9.28 [B/F]	2.4			4.83	4.46	24
Total load (unfactored, all loads applied): 115 kN						27.26	87.62	107
Dead/Permanent (unfactored):						101 kN	19.89	80.62
Live/Variable (unfactored):						14 kN	7.38	7.01
								32

Worst case factored reactions:

R1: factored DL+LL on span, unfactored DL on RH cant

42.98

R2: factored DL+LL on span and RH cant

124.08

Check for uplift at R1 with worst case loading

Unfactored DL on span; factored DL+LL on RH cantilever

16.55

OK

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

Load durations: D: Dead; L: Live

Span: Maximum B.M. = 87.0 kNm at 2.50 m. from R1 (factored) (Cant DLs factored by 1.0, Cant LLs ignored)

Maximum S.F. = -80.8 kN at R2 (factored)

Mid-span deflections: Dead = $75 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live = $32 \times 10^8 / EI$ (Live loads on span only)

Total = $107 \times 10^8 / EI$

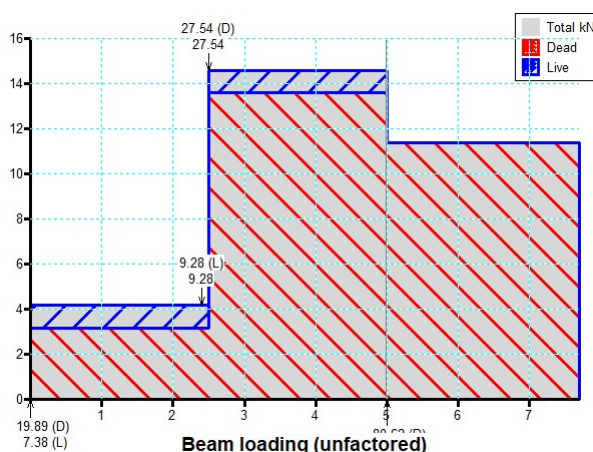
RH cantilever: B.M. at root = -58.4 kNm (factored)

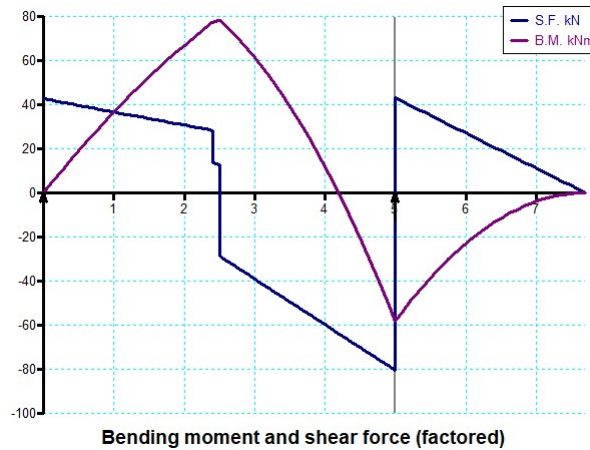
S.F. at root = 43.3 kN (factored)

End deflections: Dead = $20.20 \times 10^8 / EI \times (E$ in N/mm^2 , I in cm^4)

Live = $0.00 \times 10^8 / EI$ (Live loads on cantilever only)

Total = $20.20 \times 10^8 / EI$





Beam calculation to BS5950-1:2000 using S355 steel

SECTION SIZE : 203 x 203 x 46 UC S355 (semi-compact)

D=203.2 mm B=203.6 mm t=7.2 mm T=11.0 mm $I_x=4,570 \text{ cm}^4$ $r_y=5.13 \text{ cm}$ $S_x=497 \text{ cm}^3$ $x=17.7$

Section classification: T = 11.0mm $p_y = 355 \text{ N/mm}^2$ $e = \sqrt{(275/355)} = 0.88$

(Table 11)

Flange: $b/T = 101.8/11.0 = 9.25$ (≤ 15 Class 3 semi-compact)

Web: $d/t = 160.8/7.2 = 22.3$ (≤ 80 Class 1 plastic)

Section classification is Class 3, semi-compact $S_{x,eff} = 490 \text{ cm}^3$

Shear

Maximum S.F. = 81 kN

Shear capacity = $0.6 p_y \cdot t \cdot D = 0.6 \times 355 \times 7.2 \times 203.2/1000 = 312 \text{ kN OK}$

- Span -

Bending

Maximum B.M. = 87.0 kNm (span loaded, unfactored DLs only on cant)

Moment capacity, $M_c = p_y \cdot S_{x,eff} = 355 \times 490/1000 = 174 \text{ kNm OK}$

Lateral-torsional buckling

Beam is laterally restrained at supports only

Restraint condition at R1 and R2: Compression flange laterally restrained. Nominal torsional restraint. Both flanges free to rotate on plan (1.0L)

Effective length = $1.0L = 5.00 \text{ m}$. [Table 13]

Bending strength, $p_b = 239 \text{ N/mm}^2$

Maximum moment within segment, $M_x = 87.0 \text{ kNm}$

Equivalent uniform moment factor, $m_{LT} = 0.851$ ($M_2 = 49.0$, $M_3 = 87.0$, $M_4 = 38.9$)

Equivalent uniform moment = $0.851 \times 87.0 = 74.1 \text{ kNm}$

Buckling resistance moment, $M_b = p_b \cdot S_{x,eff} = 239 \times 490/1000 = 117 \text{ kNm OK}$

Web buckling and crushing have not been checked

Deflection

LL deflection = $32.4 \times 1e8 / (205,000 \times 4,570) = 3.5 \text{ mm}$ (L/1445) OK

TL deflection = $107.4 \times 1e8 / (205,000 \times 4,570) = 11.5 \text{ mm}$ (L/436)

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- Cantilever at R2 -

Bending

Root moment = 41.7 kNm

Moment capacity, $M_c = p_y \cdot S_{x, \text{eff}} = 355 \times 490/1000 = 174 \text{ kNm}$ OK

Lateral-torsional buckling

Maximum moment within segment = 41.7 kNm

Equivalent uniform moment = 41.7 kNm ($m = 1.00$)

Buckling resistance moment, $M_b = 497 \times 189.9/1000 = 94.4 \text{ kNm}$ ($p_b = 190$) OK

Deflection

LL deflection = $0.00 \times 1e8 / (205,000 \times 4,570) = 0.0 \text{ mm}$ OK

TL deflection = $20.20 \times 1e8 / (205,000 \times 4,570) = 2.2 \text{ mm}$ ($L/1252$)

Notched beam flexible end plate connection check (based on SCI/BCSA Simple Connections 'Green Book' P212, 2002 edition)

Location:

Factored shear at support, $F_v = 53.4$ kN

Beam: S355 203x203x46 UC connects to web of S355 203x203x46 UC

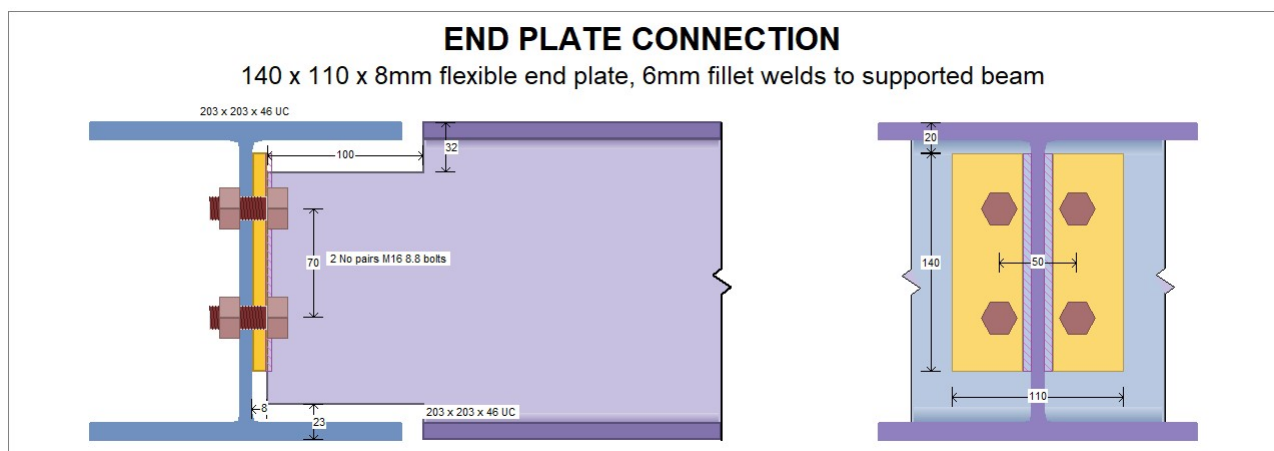
203x203x46 UC: D = 203.2 mm B = 203.6 mm t = 7.2 mm T = 11.0 mm r = 10.2 mm d = 160.8 mm $A = 58.7 \text{ cm}^2$

203x203x46 UC: D = 203.2 mm B = 203.6 mm t = 7.2 mm T = 11.0 mm r = 10.2 mm d = 160.8 mm $A = 58.7 \text{ cm}^2$

Notch dimensions: Top: 100 x 32 mm Bottom: 100 x 23mm

Use 140 x 110 x 8mm end plate with 2No pairs of M16 8.8 bolts

Bolt centres: 70mm (v) x 50mm (h)



Check 1: Recommended detailing requirements

All requirements satisfied

Check 2: Capacity of welds connecting end plate to supported beam web

Use 6mm fillet welds $a = 0.7s = 4.2\text{mm}$

Effective weld length, $l_{we} = 2 \times (140 - 2 \times 6) = 256\text{mm}$

Weld capacity = $220 \times 256 \times 4.2/1000 = 237 \text{ kN OK}$

Check 3: N/A

Check 4: Shear capacity of supported beam web

$A_v = 0.9 \times 140 \times 7.2 = 907 \text{ mm}^2$

Web capacity = $0.6 p_y A_v = 0.6 \times 355 \times 907/1000 = 193 \text{ kN OK}$

Check 5: Shear and bending interaction at notch

Shear capacity at notch, $P_{vN} = 0.6 p_y A_v$

$P_{vN} = 0.6 \times 355 \times 0.9 \times (203.2 - 32 - 32) \times 7.2/1000 = 205 \text{ kN}$ ($F_v \leq 0.75 P_{vN}$) OK

Moment at notch = $53.4 \times (100+10)/1000 = 5.87 \text{ kNm}$

Moment capacity at notch, $M_{cN} = p_y Z_N = 355 \times 26.4/1000 = 9.36 \text{ kNm OK}$

Bending+shear interaction load capacity = 84.9 kN OK

Moment at notch = $84.9 \times (100+10)/1000 = 9.34 \text{ kNm}$

Moment capacity at notch, $M_{cN} = p_y Z_N = 355 \times 26.4/1000 = 9.36 \text{ kNm OK}$

Check 6: Local stability of notched beam

Not checked

Check 7: Overall stability of notched beam

Not checked

Check 8: Capacity of bolt group connection to supporting beam web

Bolt shear capacity, P_s , per bolt = $157 \times 375/1000 = 58.9$ kN

Shear capacity of bolt group, $SPS = 4 \times 58.9 = 236$ kN OK $\rightarrow$

Bolt bearing capacity per bolt = $16 \times 8 \times 460/1000 = 58.9$ kN

Bearing capacity of end plate at support = $2 \times 2 \times 58.9 = 236$ kN OK

Check 9: Shear and bearing capacity of plate at support

Plain shear capacity, $P_v = 0.6 \cdot p_y \cdot A_v$

$$A_v = 0.9 \times 140 \times 8 = 1,008 \text{ mm}^2$$

$$P_v = 0.6 \times 275 \times 1,008/1000 = 166 \text{ kN (x2) OK } \rightarrow$$

Plain shear capacity, $P_{v,net} = 0.7 \cdot p_y \cdot K_e \cdot A_{v,net}$

$$A_{v,net} = 1,008 - 2 \times 18 \times 8 = 720 \text{ mm}^2$$

$$P_{v,net} = 0.7 \times 275 \times 1.2 \times 720/1000 = 166 \text{ kN (x2) OK}$$

Block shear capacity, $P_r = 0.6 \cdot p_y \cdot t \cdot (L_v + K_e(L_t - 0.5D_h))$

$$P_r = 0.6 \times 275 \times 8 \times (105 + 1.2 \times (30 - 0.5 \times 18)) / 1000 = 172 \text{ kN (x2) OK}$$

Check 10: Local capacity of supporting beam web

Shear capacity per bolt line, gross, $P_v = 0.6 \cdot p_y \cdot A_v$

$$A_v = (55 + (2 - 1) \times 70 + 25) \times 7.2 = 1,080 \text{ mm}^2$$

$$P_v = 0.6 \times 355 \times 1,080/1000 = 230 \text{ kN (x2) OK}$$

Shear capacity per bolt line, net, $P_{v,net} = 0.7 \cdot p_y \cdot K_e \cdot A_{v,net}$

$$A_{v,net} = 1,080 - 2 \times 18 \times 7.2 = 821 \text{ mm}^2$$

$$P_{v,net} = 0.7 \times 355 \times 1.1 \times 821/1000 = 224 \text{ kN (x2) OK}$$

Bearing capacity of supporting beam web /bolt = $16 \times 7.2 \times 550/1000 = 63.4$ kN

Bearing capacity per bolt line = $2 \times 63.4/1000 = 127$ kN (x2) OK $\rightarrow$

Further checking is needed if this web supports connections on both sides

Summary:

Check	Cap kN
1 Basic detailing requirements	OK
2 Capacity of welds connecting end plate to supported beam web	236.5
3 N/A	N/A
4 Shear strength of beam web at end plate	193.2
5 Shear and bending interaction at notch	84.9
6 Local stability of notched beam	Not checked
7 Local shear capacity of supporting member	Not checked
8 Shear capacity of bolt group connection to support	235.5
9 Shear and bearing capacity of end plate	332.6
10 Local capacity of supporting member	253.4

Critical design check: 5 - Shear and bending interaction at notch

Connection capacity = **84.9 kN** OK

Beam: Floor Beam

Span: 3.5 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
U D	floor dead	0.7*2.0	0		L	2.45	2.45	2.74
U L	Floor live	1.5*2.0	0		L	5.25	5.25	5.86
U D	o.w.	0.2	0		L	0.35	0.35	0.39
U D	partition	1.0*2.5	0		L	4.38	4.38	4.88
Total load (unfactored):						24.85 kN	12.43	12.43
Dead/Permanent (unfactored):						14.35 kN	7.18	7.18
Live/Variable (unfactored):						10.50 kN	5.25	5.25
Total load (factored):						36.89 kN	18.44	18.44

Load types: U:UDL; Load positions are measured in m. from R1; Load durations: D: Dead; L: Live

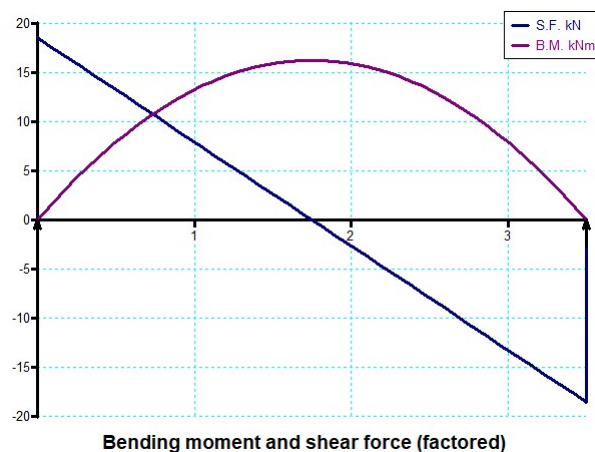
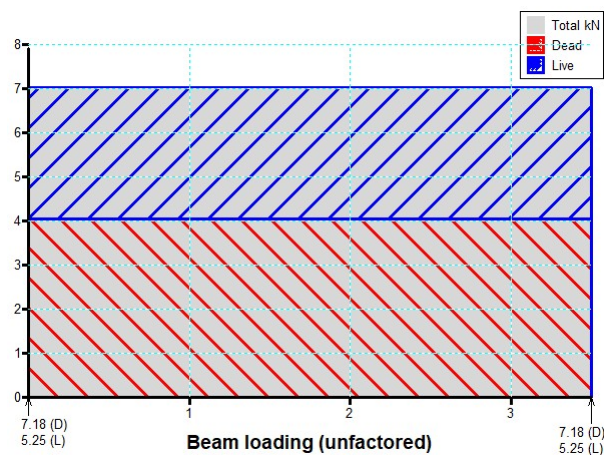
Maximum B.M. = 16.14 kNm (factored) at 1.75 m. from R1

Maximum S.F. = 18.44 kN (factored) at R1

Mid-span deflections: Dead: $8.01 \times 10^{-8}/EI$ (E in N/mm^2 , I in cm^4)

Live: $5.86 \times 10^{-8}/EI$

Total: $13.87 \times 10^{-8}/EI$



Beam calculation to BS5950-1:2000 using S355 steel

SECTION SIZE : 152 x 89 x 16 UB S355 (compact)

D=152.4 mm B=88.7 mm t=4.5 mm T=7.7 mm $I_x=834 \text{ cm}^4$ $r_y=2.10 \text{ cm}$ $S_x=123 \text{ cm}^3$ $x=19.6$

Section classification: T = 7.7mm $p_y = 355 \text{ N/mm}^2$ $e = \sqrt{(275/355)} = 0.88$

(Table 11)

Flange: $b/T = 44.3/7.7 = 5.76$ (≤ 9 Class 1 plastic)

Web: $d/t = 121.8/4.5 = 27.1$ (≤ 80 Class 1 plastic)

For design purposes section classification is Class 2, compact

Shear

Maximum S.F. = 18 kN

Shear capacity = $0.6 p_y t D = 0.6 \times 355 \times 4.5 \times 152.4 / 1000 = 146 \text{ kN OK}$

Bending

Maximum B.M. = 16.14 kNm

Moment capacity, $M_c = p_y S_x = 355 \times 123 / 1000 = 43.7 \text{ kNm OK}$

Lateral-torsional buckling

Beam is laterally restrained at supports only

Restraint condition at R1 and R2: Compression flange laterally restrained. Nominal torsional restraint. Both flanges free to rotate on plan (1.0L)

Effective length = $1.0L = 3.50 \text{ m}$. [Table 13]

Bending strength, $p_b = 137 \text{ N/mm}^2$

Maximum moment within segment, $M_x = 16.14 \text{ kNm}$

Equivalent uniform moment factor, $m_{LT} = 0.925$ ($M_2 = 12.1$, $M_3 = 16.1$, $M_4 = 12.1$)

Equivalent uniform moment = $0.925 \times 16.14 = 14.93 \text{ kNm}$

Buckling resistance moment, $M_b = p_b S_x = 137 \times 123 / 1000 = 16.80 \text{ kNm OK}$

Web buckling and crushing have not been checked

Deflection

LL deflection = $5.86 \times 10^8 / (205,000 \times 834) = 3.4 \text{ mm (L/1021) OK}$

TL deflection = $13.87 \times 10^8 / (205,000 \times 834) = 8.1 \text{ mm (L/431)}$

Beam: Mk A

Span: 4.2 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
U D	wall	4.2*2.5	0		L	22.05	22.05	42.5
U L	roof	0.6*1.8	0		L	2.27	2.27	4.4
U D	roof	1.3*1.8	0		L	4.91	4.91	9.5
U D	o.w.	0.5	0		L	1.05	1.05	2.0
P D	Beam: Floor Beam : R1	7.18 [B/F]	3.2			1.71	5.47	7.3
P L	Beam: Floor Beam : R1	5.25 [B/F]	3.2			1.25	4.00	5.4
Total load (unfactored):						73.0 kN	33.24	39.75
Dead/Permanent (unfactored):						63.2 kN	29.72	33.48
Live/Variable (unfactored):						9.8 kN	3.52	6.27
Total load (factored):						104.1 kN	47.24	56.90

Load types: U:UDL; P:Point load; Load positions are measured in m. from R1
Load durations: D: Dead; L: Live

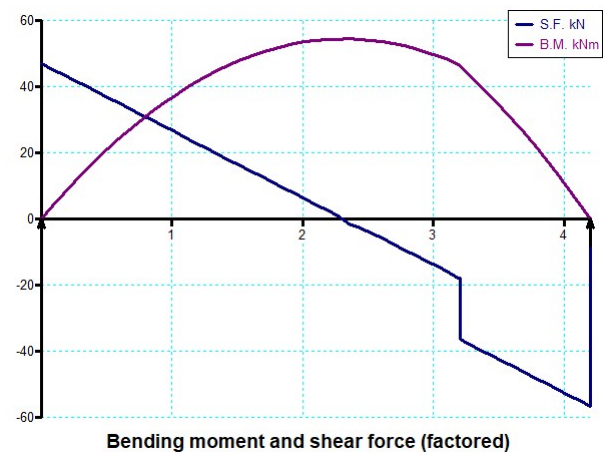
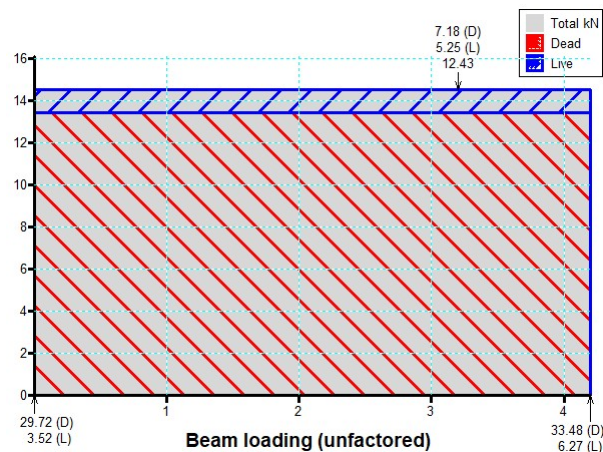
Maximum B.M. = 54.7 kNm (factored) at 2.31 m. from R1

Maximum S.F. = -56.9 kN (factored) at R2

Mid-span deflections: Dead: $61.4 \times 10^8 / EI$ (E in N/mm^2 , I in cm^4)

Live: $9.7 \times 10^8 / EI$

Total: $71.1 \times 10^8 / EI$



Beam calculation to BS5950-1:2000 using S355 steel

SECTION SIZE : 200 x 150 x 10 HF RHS S355 (50.9kg/m) (compact)

$I_x = 3,570 \text{ cm}^4$ $r_y = 5.91 \text{ cm}$ $S_x = 436 \text{ cm}^3$ $Z_x = 357 \text{ cm}^3$ $A = 64.9 \text{ cm}^2$

Section classification: $T = 10.0\text{mm}$ $p_y = 355\text{ N/mm}^2$ $e = \sqrt{(275/355)} = 0.88$
(Table 12) Flange: $b/T = 120.0/10.0 = 12.0$ (≤ 28 Class 1 plastic)
Web: $d/t = 170.0/10.0 = 17.0$ (≤ 64 Class 1 plastic)
For design purposes section classification is Class 2, compact

Shear

Maximum S.F. = 57 kN

Shear capacity = $0.6 p_y A_D / (D+B) = 0.6 \times 355 \times (100 \times 64.9 \times 200) / (200+150) / 1000 = 790\text{ kN OK}$

Bending

Maximum B.M. = 54.7 kNm

Moment capacity, $M_c = 1.2 p_y Z_x = 1.2 \times 355 \times 357 / 1000 = 152\text{ kNm OK}$

Lateral-torsional buckling

Beam is laterally restrained at supports only

Restraint condition at R1 and R2: Compression flange laterally restrained. Nominal torsional restraint. Both flanges free to rotate on plan (1.0L)

Effective length = $1.0L = 4.20\text{ m}$. [Table 13]

Slenderness (L_E/r_y) = $4.20 \times 100 / 5.91 = 71.1$

Check for lateral torsional buckling not required when $L_E/r_y \leq 449$ (Table 15)

Web buckling and crushing have not been checked

Deflection

LL deflection = $9.7 \times 1e8 / (205,000 \times 3,570) = 1.3\text{ mm}$ (L/3159) OK

TL deflection = $71.1 \times 1e8 / (205,000 \times 3,570) = 9.7\text{ mm}$ (L/432)

Bearings

200 x 150 x 10 RHS stiff bearing length, b_1 taken as $B - 3t = 120\text{ mm}$

Masonry: 100mm 7.3N/mm² solid block (SF>2.0), class (iii) mortar, normal const/normal mfr

Local design strength (factored) = $6.4/3.5 = 1.83\text{ N/mm}^2$ (BS5628-1:2005 Table 2d/2e)

R1: 300 x 100 x 150 mm Padstone

Factored reaction = $1.4 \times 29.72 + 1.6 \times 3.52 = 47.2\text{ kN}$

Factored stress under padstone = $47.2 \times 1000 / 300 \times 100 = 1.57\text{ N/mm}^2\text{ OK}$

R2: 350 x 100 x 150 mm Padstone

Factored reaction = $1.4 \times 33.48 + 1.6 \times 6.27 = 56.9\text{ kN}$

Factored stress under padstone = $56.9 \times 1000 / 350 \times 100 = 1.63\text{ N/mm}^2\text{ OK}$

Provide 8mm Bottom Plate to support outer leaf of cavity wall. Plate welded by 6mm fillet weld 150mm hit 100mm miss