ProSteel 7 Project Summary

Site address: 80 AppleTtree Avenue
Job: ASB\024\080\Appl
Client:
Job number:

ITEMS:

1: Beam: RIDGE BEAM RB1
Span: 6.1 m.
Reactions (unfactored/factored): R1: 17.46/26.51 kN; R2: 17.33/26.30 kN
Use 203 x 203 x 46 UC S355
Bearing R1: 20 mm m.s. bearing plate, size 450 x 100 mm
Bearing R2: 20 mm m.s. bearing plate, size 450 x 100 mm

2: Beam: SB1
Span: 6.1 m.
Reactions (unfactored/factored): R1: 20.29/31.62 kN; R2: 20.25/31.55 kN
Use 203 x 203 x 46 UC S355
Bearing R1: 25 mm m.s. bearing plate, size 550 x 100 mm
Bearing R2: 25 mm m.s. bearing plate, size 550 x 100 mm

3: Beam: SB2
Span: 6.1 m.
Reactions (unfactored/factored): R1: 29.83/46.35 kN; R2: 29.58/45.92 kN
Use 203 x 203 x 46 UC S355
Bearing R1: 30 mm m.s. bearing plate, size 750 x 100 mm
Bearing R2: Not specified

4: Beam: LINTEL L1 (ASSUME EXISTING)
Span: 1.0 m.
Reactions (unfactored/factored): R1: 11.01/16.44 kN; R2: 11.01/16.44 kN
Use 100 x 100 x 5 HF SHS S355 *(14.7kg/m)*

5: Beam: LINTEL L2
Span: 1.2 m.
Reactions (unfactored/factored): R1: 15.54/23.38 kN; R2: 35.38/53.49 kN
Use 100 x 100 x 5 HF SHS S355 *(14.7kg/m)*

Beam: **RIDGE BEAM RB1** **Span: 6.1 m.**

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O	D	o.w.	0.5	0		L	1.52	1.52	9
U	L	Flat Roof A	0.75*(2.9/2)	0		L	3.32	3.32	20
U	D	Flat Roof A	0.85*(2.9/2)	0		L	3.76	3.76	22
V	L	Pitch Roof	0.75*(3.1/2)	0	0.75*(0.1/2)	3.0	1.50	0.30	4
V	D	Pitch Roof	0.75*(3.1/2)	0	0.75*(0.1/2)	3.0	1.50	0.30	4
V	L	Pitch Roof	0.75*(0.1/2)	3.0	0.75*(3.1/2)	6.1	0.32	1.54	4
V	D	Pitch Roof	0.75*(0.1/2)	3.0	0.75*(3.1/2)	6.1	0.32	1.54	4
P	L	RT4:R2	10.26	3.0			5.21	5.05	48
					Total load (unfactored):	**34.8 kN**	**17.46**	**17.33**	115
				Dead/Permanent (unfactored):		14.2 kN	7.10	7.12	39
				Live/Variable (unfactored):		20.6 kN	10.35	10.20	76
				Total load (factored):		**52.8 kN**	**26.51**	**26.30**	

Load types: O:Beam o.w.; U:UDL; V:Variable load; P:Point load; Load positions: m. from R1
Load durations: D: Dead; L: Live

Maximum B.M. = 50.2 kNm (factored) at 3.00 m. from R1

Maximum S.F. = 26.5 kN (factored) at R1

Mid-span deflections: Dead: 39 x 10^8/EI *(E in N/mm^2, I in cm^4)*
Live: 76 x 10^8/EI
Total: 115 x 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 cm^4 r_y=5.13 cm S_x=497 cm^3 x=17.7

Section classification: T = 11.0mm p_y = 355 N/mm^2 e= √(275/355) = 0.88
(Table 11) Flange: b/T = 101.8/11.0 = 9.25 (<=15e Class 3 semi-compact)
Web: d/t = 160.8/7.2 = 22.3 (<=80e Class 1, plastic)
Section classification is Class 3, semi-compact $S_{x,eff}$ = 490 cm^3

Shear

Maximum S.F. = 27 kN

Shear capacity = 0.6 p_y.t.D = 0.6 x 355 x 7.2 x 203.2/1000 = 312 kN OK

Bending

Maximum B.M. = 50.2 kNm

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

Lateral-torsional buckling

Beam is laterally restrained at supports only

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

Effective length = 1.2L+2D = 7.73 m. [Table 13]

Bending strength, p_b = 172 N/mm^2

Maximum moment within segment, M_x = 50.2 kNm

Equivalent uniform moment factor, m_{LT} = 0.888 (M_2 = 32.2, M_3 = 49.8, M_4 = 31.8)

Equivalent uniform moment = 0.888 x 50.2 = 44.5 kNm

Buckling resistance moment, M_b = p_b.$S_{x,eff}$ = 172 x 490/1000 = 84.2 kNm OK

Web capacity

Check unstiffened web capacities with loads of 26.5 kN and 26.3 kN

$C1 = 108$ kN; $C2 = 2.56$ kN/mm; $C4 = 399$; $K = \min\{0.5+(a_e/1.4d),1.0\}$; $p_{yw} = 355$N/mm^2

For derivation of C factors see SCI Steelwork Design Guide to BS5950-1:2000 6th ed. (P325)

R1: Minimum required stiff bearing length, $b_1 = 0$mm: $a_e = 0$mm; $d = 160.8$mm ; $K = 0.500$

Bearing capacity, $P_w = C1 + b_1.C2 = 108$ kN

Buckling capacity, $P_x = K\sqrt{(C4.P_w)} = 0.500\sqrt{(399 \times 108)} = 104$ kN ¬

R2: Minimum required stiff bearing length, $b_1 = 0$mm: $a_e = 0$mm; $d = 160.8$mm ; $K = 0.500$

Bearing capacity, $P_w = C1 + b_1.C2 = 108$ kN

Buckling capacity, $P_x = K\sqrt{(C4.P_w)} = 0.500\sqrt{(399 \times 108)} = 104$ kN ¬

Deflection

LL deflection = 76.1 x 1e8/(205,000 x 4,570) = 8.1 mm (L/751) OK

TL deflection = 115.3 x 1e8/(205,000 x 4,570) = 12.3 mm (L/496)

Bearings

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

R1: 450 x 100 x 20 mm S275 bearing plate

Factored reaction = 1.4 x 7.10 + 1.6 x 10.35 = 26.5 kN

Local design strength of masonry (factored) = 0.630 N/mm^2 (User-entered value)

Factored stress under plate = 26.5 x 1000/450 x 100 = 0.59 N/mm^2 OK

Bearing plate projection beyond stiff bearing length = (450-45.5)/2 = 202mm

Required plate thickness = $\sqrt{(3 \times 0.59 \times 202 \times 202/265)} = 16.5$ mm: use 20mm

Factored bending stress in plate = 0.59 x 202 x (202/2)/(20 x 20/6) = 181 N/mm^2 (p_y=265 N/mm^2)

R2: 450 x 100 x 20 mm S275 bearing plate

Factored reaction = 1.4 x 7.12 + 1.6 x 10.20 = 26.3 kN

Local design strength of masonry (factored) = 0.630 N/mm^2 (User-entered value)

Factored stress under plate = 26.3 x 1000/450 x 100 = 0.58 N/mm^2 OK

Bearing plate projection beyond stiff bearing length = (450-45.5)/2 = 202mm

Required plate thickness = $\sqrt{(3 \times 0.58 \times 202 \times 202/265)} = 16.4$ mm: use 20mm

Factored bending stress in plate = 0.58 x 202 x (202/2)/(20 x 20/6) = 179 N/mm^2 (p_y=265 N/mm^2)

Beam: **SB1** **Span: 6.1 m.**

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O	D	o.w.	0.5	0		L	1.52	1.52	9
R	L	Floor A	1.31/0.4	0		1.6	4.55	0.69	9
R	D	Floor A	0.44/0.4	0		1.6	1.53	0.23	3
R	L	Floor B	1.5*(2.7/2)	1.6		4.4	2.88	2.79	24
R	D	Floor B	0.5*(2.7/2)	1.6		4.4	0.96	0.93	8
R	L	Floor A	1.31/0.4	4.4		6.1	0.78	4.79	10
R	D	Floor A	0.44/0.4	4.4		6.1	0.26	1.61	4
P	L	T3:R1	3.58	0.6			3.23	0.35	5
P	L	T4:R1	4.17	1.7			3.01	1.16	15
P	L	T5:R1	4.17	4.4			1.16	3.01	15
P	L	T6:R1	3.58	5.4			0.41	3.17	6
				Total load (unfactored):		**40.5 kN**	**20.29**	**20.25**	108
				Dead/Permanent (unfactored):		8.6 kN	4.28	4.29	24
				Live/Variable (unfactored):		32.0 kN	16.02	15.96	84
				Total load (factored):		**63.2 kN**	**31.62**	**31.55**	

Load types: O:Beam o.w.; R:Part UDL; P:Point load; Load positions: m. from R1
Load durations: D: Dead; L: Live

Maximum B.M. = 41.33 kNm (factored) at 3.08 m. from R1

Maximum S.F. = 31.6 kN (factored) at R1

Mid-span deflections: Dead: 24 x 10^8/EI *(E in N/mm^2, I in cm^4)*
Live: 84 x 10^8/EI
Total: 108 x 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 cm^4 r_y=5.13 cm S_x=497 cm^3 x=17.7

Section classification: T = 11.0mm p_y = 355 N/mm^2 e= $\sqrt{}$(275/355) = 0.88
(Table 11) Flange: b/T = 101.8/11.0 = 9.25 (<=15e Class 3 semi-compact)
Web: d/t = 160.8/7.2 = 22.3 (<=80e Class 1, plastic)
Section classification is Class 3, semi-compact $S_{x,eff}$ = 490 cm^3

Shear

Maximum S.F. = 32 kN

Shear capacity = 0.6 p_y.t.D = 0.6 x 355 x 7.2 x 203.2/1000 = 312 kN OK

Bending

Maximum B.M. = 41.3 kNm

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

Lateral-torsional buckling

Beam is laterally restrained at supports only

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

Effective length = 1.2L+2D = 7.73 m. [Table 13]

Bending strength, p_b = 172 N/mm^2

Maximum moment within segment, M_x = 41.3 kNm

Equivalent uniform moment factor, m_{LT} = 0.950 (M_2 = 34.2, M_3 = 41.3, M_4 = 34.7)

Equivalent uniform moment = 0.950 x 41.3 = 39.3 kNm

Buckling resistance moment, M_b = p_b.$S_{x,eff}$ = 172 x 490/1000 = 84.2 kNm OK

Web capacity

Check unstiffened web capacities with loads of 31.6 kN and 31.5 kN

C1 = 108 kN; C2 = 2.56 kN/mm; C4 = 399; K = min{0.5+(a_e/1.4d),1.0}; p_{yw} = 355N/mm^2

For derivation of C factors see SCI Steelwork Design Guide to BS5950-1:2000 6th ed. (P325)

R1: Minimum required stiff bearing length, b_1 = 0mm: a_e = 0mm; d = 160.8mm ; K = 0.500

Bearing capacity, P_w = C1 + b_1.C2 = 108 kN

Buckling capacity, P_x = K√(C4.P_w) = 0.500√(399 x 108) = 104 kN ¬

R2: Minimum required stiff bearing length, b_1 = 0mm: a_e = 0mm; d = 160.8mm ; K = 0.500

Bearing capacity, P_w = C1 + b_1.C2 = 108 kN

Buckling capacity, P_x = K√(C4.P_w) = 0.500√(399 x 108) = 104 kN ¬

Deflection

LL deflection = 84.3 x 1e8/(205,000 x 4,570) = 9.0 mm (L/678) OK

TL deflection = 108.0 x 1e8/(205,000 x 4,570) = 11.5 mm (L/529)

Bearings

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

R1: 550 x 100 x 25 mm S275 bearing plate

Factored reaction = 1.4 x 4.28 + 1.6 x 16.02 = 31.6 kN

Local design strength of masonry (factored) = 0.630 N/mm^2 (User-entered value)

Factored stress under plate = 31.6 x 1000/550 x 100 = 0.57 N/mm^2 OK

Bearing plate projection beyond stiff bearing length = (550-45.5)/2 = 252mm

Required plate thickness = √(3 x 0.57 x 252 x 252/265) = 20.3 mm: use 25mm

Factored bending stress in plate = 0.57 x 252 x (252/2)/(25 x 25/6) = 176 N/mm^2 (p_y=265 N/mm^2)

R2: 550 x 100 x 25 mm S275 bearing plate

Factored reaction = 1.4 x 4.29 + 1.6 x 15.96 = 31.5 kN

Local design strength of masonry (factored) = 0.630 N/mm^2 (User-entered value)

Factored stress under plate = 31.5 x 1000/550 x 100 = 0.57 N/mm^2 OK

Bearing plate projection beyond stiff bearing length = (550-45.5)/2 = 252mm

Required plate thickness = √(3 x 0.57 x 252 x 252/265) = 20.3 mm: use 25mm

Factored bending stress in plate = 0.57 x 252 x (252/2)/(25 x 25/6) = 175 N/mm^2 (p_y=265 N/mm^2)

Beam: **SB2** **Span: 6.1 m.**

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O	D	o.w.	0.5	0		L	1.52	1.52	9
R	L	Floor A	1.21/0.4	0		1.6	4.21	0.63	9
R	D	Floor A	0.40/0.4	0		1.6	1.39	0.21	3
R	L	Floor B	1.5*(2.7/2)	1.6		4.4	2.88	2.79	24
R	D	Floor B	0.5*(2.7/2)	1.6		4.4	0.96	0.93	8
R	L	Floor A	1.21/0.4	4.4		6.1	0.72	4.43	10
R	D	Floor A	0.40/0.4	4.4		6.1	0.24	1.46	3
R	L	Floor C	1.5*(3.6/2)	0		5.1	8.01	5.76	46
R	D	Floor C	0.5*(3.6/2)	0		5.1	2.67	1.92	15
R	D	Stud Wall	0.5*2.2	5.1		6.1	0.09	1.01	1
P	L	T3:R2	4.72	0.6			4.26	0.46	7
P	L	T4:R2	1.87	1.7			1.35	0.52	7
P	L	T5:R2	1.87	4.4			0.52	1.35	7
P	L	T6:R2	4.72	5.4			0.54	4.18	8
P	L	T1:R1	2.88	5.1			0.47	2.41	6
				Total load (unfactored):		**59.4 kN**	**29.83**	**29.58**	161
				Dead/Permanent (unfactored):		13.9 kN	6.87	7.06	40
				Live/Variable (unfactored):		45.5 kN	22.96	22.53	122
				Total load (factored):		**92.3 kN**	**46.35**	**45.92**	

Load types: O:Beam o.w.; R:Part UDL; P:Point load; Load positions: m. from R1
Load durations: D: Dead; L: Live

Maximum B.M. = 62.8 kNm (factored) at 3.11 m. from R1

Maximum S.F. = 46.4 kN (factored) at R1

Mid-span deflections: Dead: 40 x 10^8/EI *(E in N/mm^2, I in cm^4)*
Live: 122 x 10^8/EI
Total: 161 x 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 cm^4 r_y=5.13 cm S_x=497 cm^3 x=17.7

Section classification: (Table 11)
T = 11.0mm p_y = 355 N/mm^2 e= $\sqrt{}$(275/355) = 0.88
Flange: b/T = 101.8/11.0 = 9.25 (<=15e Class 3 semi-compact)
Web: d/t = 160.8/7.2 = 22.3 (<=80e Class 1, plastic)
Section classification is Class 3, semi-compact $S_{x,eff}$ = 490 cm^3

Shear

Maximum S.F. = 46 kN

Shear capacity = 0.6 p_y.t.D = 0.6 x 355 x 7.2 x 203.2/1000 = 312 kN OK

Bending

Maximum B.M. = 62.8 kNm

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

Lateral-torsional buckling

Beam is laterally restrained at supports only

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

Effective length = 1.2L+2D = 7.73 m. [Table 13]

Bending strength, p_b = 172 N/mm^2

Maximum moment within segment, M_x = 62.8 kNm

Equivalent uniform moment factor, m_{LT} = 0.939 (M_2 = 49.1, M_3 = 62.8, M_4 = 51.0)

Equivalent uniform moment = 0.939 x 62.8 = 59.0 kNm

Buckling resistance moment, $M_b = p_b.S_{x,eff}$ = 172 x 490/1000 = 84.2 kNm OK

Web capacity

Check unstiffened web capacities with loads of 46.4 kN and 45.9 kN
C1 = 108 kN; C2 = 2.56 kN/mm; C4 = 399; K = min{0.5+(a_e/1.4d),1.0}; p_{yw} = 355N/mm ^{2}
For derivation of C factors see SCI Steelwork Design Guide to BS5950-1:2000 6th ed. (P325)

R1: Minimum required stiff bearing length, b_1 = 0mm: a_e = 0mm; d = 160.8mm ; K = 0.500
Bearing capacity, P_w = C1 + b_1.C2 = 108 kN
Buckling capacity, P_x = K√(C4.P_w) = 0.500√(399 x 108) = 104 kN ¬

R2: Minimum required stiff bearing length, b_1 = 0mm: a_e = 0mm; d = 160.8mm ; K = 0.500
Bearing capacity, P_w = C1 + b_1.C2 = 108 kN
Buckling capacity, P_x = K√(C4.P_w) = 0.500√(399 x 108) = 104 kN ¬

Deflection

LL deflection = 122 x 1e8/(205,000 x 4,570) = 13.0 mm (L/469) OK
TL deflection = 161 x 1e8/(205,000 x 4,570) = 17.2 mm (L/354)

Bearings

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

R1: 750 x 100 x 30 mm S275 bearing plate

Factored reaction = 1.4 x 6.87 + 1.6 x 22.96 = 46.4 kN

Local design strength of masonry (factored) = 0.630 N/mm ^{2} (User-entered value)

Factored stress under plate = 46.4 x 1000/750 x 100 = 0.62 N/mm ^{2} OK

Bearing plate projection beyond stiff bearing length = (750-45.5)/2 = 352mm
Required plate thickness = √(3 x 0.62 x 352 x 352/265) = 29.5 mm: use 30mm
Factored bending stress in plate = 0.62 x 352 x (352/2)/(30 x 30/6) = 256 N/mm ^{2} (p_y=265 N/mm ^{2})

R2: None

Beam: **LINTEL L1 (ASSUME EXISTING)** **Span: 1.0 m.**

	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O D	o.w.	0.15	0		L	0.08	0.08	0.00
V D	Brick Wall	4.55*0.8	0	4.55*1.2	0.5	1.67	0.61	0.03
V D	Brick Wall	4.55*1.2	0.5	4.55*0.8	1.0	0.61	1.67	0.03
P D	Bm: RIDGE BEAM RB1 : R2	7.12 [B/F]	0.5			3.56	3.56	0.15
P L	Bm: RIDGE BEAM RB1 : R2	10.20 [B/F]	0.5			5.10	5.10	0.21
			Total load (unfactored):		**22.03 kN**	**11.01**	**11.01**	0.43
			Dead/Permanent (unfactored):		11.82 kN	5.91	5.91	0.21
			Live/Variable (unfactored):		10.20 kN	5.10	5.10	0.21
			Total load (factored):		**32.88 kN**	**16.44**	**16.44**	

Load types: O:Beam o.w.; V:Variable load; P:Point load; Load positions: m. from R1
Load durations: D: Dead; L: Live

Maximum B.M. = 7.45 kNm (factored) at 0.50 m. from R1

Maximum S.F. = 16.44 kN (factored) at R1

Mid-span deflections: Dead: 0.21 x 10^8/EI *(E in N/mm^2, I in cm^4)*
Live: 0.21 x 10^8/EI
Total: 0.43 x 10^8/EI

Beam calculation to BS5950-1:2000 using S355 steel

SECTION SIZE : **100 x 100 x 5 HF SHS** S355 *(14.7kg/m) (compact)*

I_x = 279 cm^4 r_y = 3.86 cm S_x = 66.0 cm^3 A = 18.7 cm^2

Section classification: T = 5.0mm p_y = 355 N/mm^2 $\varepsilon = \sqrt{(275/355)}$ = 0.88
(Table 12) Flange: b/T = 85.0/5.0 = 17.0 (<=28ε Class 1 plastic)
Web: d/t = 85.0/5.0 = 17.0 (<=64ε Class 1, plastic)
For design purposes section classification is Class 2, compact

Shear

Maximum S.F. = 16 kN

Shear capacity = 0.6 p_y.AD/(D+B) = 0.6 x 355 x (100 x 18.7 x 100)/(100+100)/1000 = 199 kN OK

Bending

Maximum B.M. = 7.45 kNm

Moment capacity, M_c = p_y.S_x = 355 x 66.0/1000 = 23.43 kNm OK

Lateral-torsional buckling

Beam is laterally restrained at supports only

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

Effective length = 1.2L+2D = 1.40 m. [Table 13]

Slenderness (L_E/r_y) = 1.40 x 100/3.86 = 36.3

Check for lateral torsional buckling not required (Table 15)

Web buckling and crushing have not been checked

Deflection

LL deflection = 0.213 x 1e8/(205,000 x 279) = 0.4 mm (L/2691) OK
TL deflection = 0.425 x 1e8/(205,000 x 279) = 0.7 mm (L/1344)

Beam: LINTEL L2 **Span: 1.2 m.**

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O	D	o.w.	0.15	0		L	0.09	0.09	0.00
R	L	Floor	1.5*(0.4/2)	0		0.8	0.16	0.08	0.01
R	D	Floor	0.5*(0.4/2)	0		0.8	0.05	0.03	0.00
V	D	Brick Wall	4.55*1.3	0	4.55*2.3	1.2	4.46	5.37	0.22
P	D	Beam: SB2 : R2	7.06 [B/F]	0.8			2.35	4.70	0.22
P	L	Beam: SB2 : R2	22.53 [B/F]	0.8			7.51	15.02	0.69
P	D	Beam: LINTEL L1 : R1	5.91 [B/F]	1.1			0.49	5.42	0.05
P	L	Beam: LINTEL L1 : R1	5.10 [B/F]	1.1			0.43	4.68	0.05
				Total load (unfactored):		**50.9 kN**	**15.54**	**35.38**	1.24
			Dead/Permanent (unfactored):			23.1 kN	7.45	15.61	0.50
			Live/Variable (unfactored):			27.9 kN	8.09	19.77	0.74
			Total load (factored):			**76.9 kN**	**23.38**	**53.49**	

Load types: O:Beam o.w.; R:Part UDL; V:Variable load; P:Point load; Load positions: m. from R1
Load durations: D: Dead; L: Live

Maximum B.M. = 15.33 kNm (factored) at 0.80 m. from R1

Maximum S.F. = -53.5 kN (factored) at R2

Mid-span deflections: Dead: 0.50 x 10^8/EI *(E in N/mm^2, I in cm^4)*
Live: 0.74 x 10^8/EI
Total: 1.24 x 10^8/EI

Beam calculation to BS5950-1:2000 using S355 steel

SECTION SIZE : **100 x 100 x 5 HF SHS** S355 *(14.7kg/m) (compact)*

I_x = 279 cm^4 r_y = 3.86 cm S_x = 66.0 cm^3 A = 18.7 cm^2

Section classification: T = 5.0mm p_y = 355 N/mm^2 e= √(275/355) = 0.88
(Table 12) Flange: b/T = 85.0/5.0 = 17.0 (<=28ε Class 1 plastic)
Web: d/t = 85.0/5.0 = 17.0 (<=64ε Class 1, plastic)
For design purposes section classification is Class 2, compact

Shear

Maximum S.F. = 53 kN

Shear capacity = 0.6 p_y.AD/(D+B) = 0.6 x 355 x (100 x 18.7 x 100)/(100+100)/1000 = 199 kN OK

Bending

Maximum B.M. = 15.33 kNm

Moment capacity, M_c = p_y.S_x = 355 x 66.0/1000 = 23.43 kNm OK

Lateral-torsional buckling

Beam is laterally restrained at supports only

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

Effective length = 1.2L+2D = 1.64 m. [Table 13]

Slenderness (L_E/r_y) = 1.64 x 100/3.86 = 42.5

Check for lateral torsional buckling not required (Table 15)

Web buckling and crushing have not been checked

Deflection

LL deflection = 0.74 x 1e8/(205,000 x 279) = 1.3 mm (L/925) OK
TL deflection = 1.24 x 1e8/(205,000 x 279) = 2.2 mm (L/554)