Structural Calculations
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March 2015

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The span quoted is solely for the purpose of producing these structural calculations. Measurements must be taken on site before ordering any materials.

Beams specified for load bearing walls of cavity construction, are often two beams, one for each skin of brick/blockwork. Check the comments at the bottom of the page for each beam specified, before ordering any materials.

Loading Data

9"BRICKWORK:	
215mm Brickwork	=4.80kN/m2
Plaster	=0.60kN/m2
Total Load	**=5.40kN/m2**
BRICKWORK PARTITION:	
100mm Brickwork	=2.10kN/m2
2 No. Plaster Faces	=0.60kN/m2
Total Load	**=2.70kN/m2**
BLOCKWORK PARTITION:	
100mm Blockwork	=1.00kN/m2
2 No. Plaster Faces	=0.50kN/m2
Total Load	**=1.50kN/m2**
TILE HANGING TO TIMBER FRAME:	
Concrete Tiles	=0.55kN/m2
Battens & Felt	=0.10kN/m2
Timber Studs	=0.10kN/m2
Plasterboard	=0.15kN/m2
Insulation	=0.05kN/m2
Plaster	=0.25kN/m2
Total Load	**=1.20kN/m2**
TIMBER STUD PARTITION:	
2 No. Plasterboard Faces	=0.30kN/m2
Timber Studs	=0.10kN/m2
2 No. Plaster Faces	=0.30kN/m2
Insulation	=0.05kN/m2
Total Load	**=0.75kN/m2**

PITCHED ROOF:

Concrete Tiles	=0.60kN/m2
Battens & Felt	=0.10kN/m2
Rafters	=0.15kN/m2
Total Dead Load	=0.85kN/m2
Imposed Load	=0.75kN/m2
Total Load	**=1.60kN/m2**

ROOF SPACE:

Joists & Insulation	=0.15kN/m2
Ceiling	=0.15kN/m2
Total Dead Load	=0.30kN/m2
Imposed Load	=0.25kN/m2
Total Load	**=0.55kN/m2**

SLOPING CEILING:

Plasterboard	=0.15kN/m2
Insulation	=0.10kN/m2
Total Dead Load	=0.25kN/m2
Total Load	**=0.45kN/m2**

FLAT ROOF:

Chipping & Felt	=0.35kN/m2
Boards, Joists & Firings	=0.30kN/m2
Ceiling & Insulation	=0.15kN/m2
Total Dead Load	=0.80kN/m2
Imposed Load	=0.75kN/m2
Total Load	**=1.55kN/m2**

TIMBER ROOF:

Boards & Joists	=0.35kN/m2
Ceiling	=0.15kN/m2
Total Dead Load	=0.50kN/m2
Imposed Load	=1.50kN/m2
Total Load	**=2.00kN/m2**

EXTERNAL RENDER WALL:

Render 2 No. Skins	=0.30kN/m2
100mm Blockwork	=2.00kN/m2
Insulation	=0.05kN/m2
Plaster	=0.25kN/m2
Total Load	**=2.60kN/m2**

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Site: Removed to protect client confidentiality	**Made by KR**
Job:	**Page 1**
MEASUREMENTS TO BE TAKEN ON SITE BEFORE ORDERING MATERIALS	**File copy**

ProSteel 5.41i 532184 **Noname.PS5** **Printed 5 Jan 2018 11:18**

Beam: Ridge Beam **Span: 4.7 m.**

		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*
U	D	o.w.	0.4	0		L	0.94	0.94
U	D	FLAT ROOF	1.55*1.80	0		L	6.56	6.56
U	D	PITCHED ROOF	1.60*2.00	0		L	7.52	7.52
				Unfactored reactions (kN)		**Total:**	**15.02**	**15.02**
						Dead:	*15.02*	*15.02*
						Live:	*0.00*	*0.00*
Total load: 30.03/42.05 kN		*Unfactored/Factored*				**Factored reactions:**	**21.02**	**21.02**

Load types: U:UDL D: Dead; L: Live (positions in m. from R1)

Maximum B.M. (factored) = 24.7 kNm at 2.35 m. from R1

Maximum S.F. (factored) = 21.0 kN at R1

Live load deflection = 0.00 x 10^8/EI at R2 *(E in N/mm², I in cm⁴)*

Total deflection = 40.6 x 10^8/EI at 2.35 m. from R1

Beam calculation to BS5950-1:2000 using S275 steel

SECTION SIZE : **152 x 152 x 37 UC S275 (compact)**

D=161.8 mm B=154.4 mm t=8.0 mm T=11.5 mm I_x=2,210 cm^4 r_y=3.87 cm S_x=309 cm^3

Shear capacity = 0.6 p_y.t.D = 0.6 x 275 x 8.0 x 161.8/1000 = 214 kN (>=21.0) OK

Maximum moment = 24.70 kNm at 2.35 m. from R1

Moment capacity, M_c = p_y.S_x = 275 x 309/1000 = 84.97 kNm OK

Beam is laterally restrained at supports only: effective length = 1.0L

Effective length (L_E) = 4.70m
Buckling parameter (u) = 0.849
β_w = 1.000 (Class 1/2 compact)
Bending strength, p_b = 192.4 N/mm^2

Slenderness, λ (L_E/r_y) = 4.70 x 100/3.87 = 121.4
Slenderness factor (v) = 0.663 (x = 13.3; λ/x = 9.13)
Equivalent slenderness (λ_{LT}) = u.v.λ.$\sqrt{\beta_w}$ = 68.38

Maximum moment within segment, M_x = 24.70 kNm
Equivalent uniform moment factor, m_{LT} = 0.925 (M_2=18.5, M_3=24.7, M_4=18.5)
Equivalent uniform moment = 0.925 x 24.70 = 22.85 kNm
Buckling resistance moment, M_b = p_b.S_x = 192.4 x 309/1000 = 59.45 kNm OK

Check unstiffened web capacity with load of 21.02 kN
C1 = 84.0 kN; C2 = 2.20 kN/mm; C4 = 712; K = min{0.5+(a_e/1.4d),1.0}; p_{yw} = 275N/mm^2
(for derivation of C factors see Steelwork Design Guide to BS5950-1:2000 6th ed.)
Bearing capacity, P_w = C1+b_1C2 (b_e taken as zero) Buckling capacity, P_x = K$\sqrt{(C4.P_w)}$
Unstiffened web bearing capacity, P_w = 84.0 kN: no minimum stiff bearing length required

LL deflection = 0.000 x 1e8/205,000 x 2210.000 = 0.0 mm OK
TL deflection = 40.60 x 1e8/205,000 x 2210 = 9.0 mm (L/525)

Bearing details

152x152x37 UC stiff bearing length, b_1 = t + 1.6r + 2T = 43.2 mm

Local design strength of masonry (factored) = 0.700 N/mm^2 (User-entered value)

R1: 350 x 100 mm bearing plate

Factored reaction = 15.02 x 1.4 + 0.00 x 1.6 = 21.02 kN

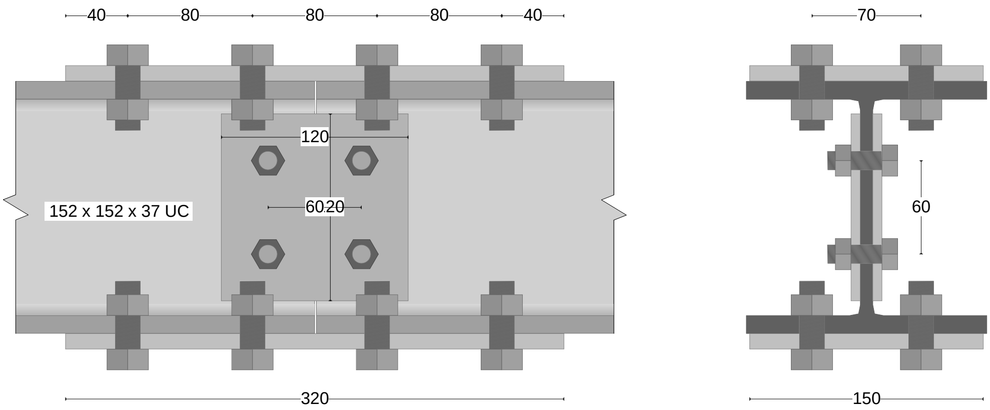
15 mm m.s. bearing plate, size 350 x 100 mm
Bearing plate projection beyond stiff bearing length = (350-43.2)/2 = 153.4mm
Factored stress under plate = 21.02 x 1000/350 x 100 = 0.60 N/mm^2
Required plate thickness = $\sqrt{(3x0.60x153x153/275)}$ = 12.4 mm: use 15mm
Factored bending stress in plate = 0.60x153x(153/2)/(15x15/6) = 188.5 N/mm^2 (p_y=275 N/mm^2)

R2 as R1

Encase beam to provide half-hour fire resistance as per specification.

Flange plate splice calculation

Beam: Ridge Beam



Flange plates: 320 x 150 x 10, M16 8.8 bolts (16). Web plates: 120 x 120 x 6, M12 8.8 bolts (4)

Beam span: 4.7 m. Section: 152 x 152 x 37 UC S275

Splice 1: 2.3 m. from R1. BM: 24.7 kNm SF: 0.447 kN (factored)

Section dims: D = 161.8 B = 154.4 T = 11.5 t = 8.0 d = 123.6 r = 7.6

Use S275 flange plates 320 x 150 x 10 mm,

Use 2 pairs of M16 8.8 bolts at 70 mm cross centres; inmost pairs of bolts 80 mm apart, 1 further sets of bolts at 80 mm pitch

Web plates: 120 x 120 x 6 mm, 2 pairs of M12 bolts at 60 mm (H) and 60 mm (V)

Basic detailing checks

Flange/Flange Plates: Using M16 bolts, washer diameter 30 mm

Check flange bolt washer clear beam web: pitch (70 mm) >= 54 mm OK
Flange plate bolt edge distance (40.0 mm) >= 1.25D (20 mm) OK
Flange bolt edge distance (42.2 mm) >= 1.25D (20 mm) OK
Plate plate bolt end distance (40 mm) >= 1.25D (20 mm) OK
Flange bolt end distance (40 mm) >= 1.25D (20 mm) OK
Flange bolt spacing (80 mm) >= 2.5D (40 mm) OK

Web/Web Plates: Using M12 bolts, washer diameter 24 mm

Check web plate clears beam roots: plate height, 120 mm < d, 123.6 mm OK
Web plate bolt end distance H (30 mm) >= 2.0D (24 mm) OK
Web plate bolt end distance V (30 mm) >= 2.0D (24 mm) OK
Web plate bolt pitch (60 mm) >= 2.5D (30 mm) OK
Web bolt end clearance (30 mm) >= 2.0D (24 mm) OK
All dimensional checks satisfied

Flange plates

Check splice with BM = 24.7 kNm

Axial force in beam flanges = 24.7 x 1000/(161.8-11.5) = 164 kN

Beam flange area, gross = 1,776 mm^2, net = 1,362 mm^2; Flange plate area: gross = 1,500 mm^2, net = 1,140 mm^2

Axial capacities:	Plate: 1,500 x 275/1000 = 412.5 kN Plate net: 1.2 x 1,140 x 275/1000 = 376.2 kN <<< OK Beam flange: 1,776 x 275/1000 = 488.3 kN Beam flange net: 1.2 x 1,362 x 275/1000 = 449.3 kN
Plate block shear:	0.6 x 275 x 10 x [120 + 1.2 x (110 - 2.5 x 18)]/1000 = 327 kN OK (K_e = 1.2; k = 2.5)
Strut action moment:	Flange effective area: 1,634 mm^2 Plate effective area: 1,368 mm^2 Flange modulus, Z_v: 36.0cm^3 Plate modulus, Z_v: 29.1 cm^3 Moment from strut action should be checked if beam is not restrained

Bolt capacity:	Shear: 157 x 375/1000 = 58.9 kN Bearing on plate: 16 x 10 x 460/1000 = 73.6 kN Bearing on flange: 16 x 11.5 x 460/1000 = 84.6 kN
Bolt group capacity:	Flange: 4 x 58.9 = 235.5 kN OK Plate: 4 x 58.9 = 235.5 kN OK

Splice moment capacity = 35.4 kNm (limited by flange bolt group)

Web plates

Check splice with SF = 0.447 kN

Plate shear capacity:	Gross area = 2 x 120 x 6 = 1,440 mm^2 Net area = 1,440 - (2 x 2 x 14 x 6) = 1,104 mm^2 Net shear capacity limit = 0.85 x A_v/K_e = 0.85 x 1,440/1.2 = 1,020 mm^2 Reduction for bolt holes not required Shear capacity = 0.6 x 1,440 x 275/1000 = 237.6 kN OK
Plate block shear:	2 x 0.6 x 275 x 6 x [61 + 1.2 x (60.0 - 0.5 x 18)]/1000 = 247 kN OK (K_e = 1.2; k = 0.5)
Bolt capacity:	Shear: 2 x 84.3 x 375/1000 = 63.2 kN Bearing on plates: 2 x 12 x 6 x 460/1000 = 66.2 kN Bearing on beam web: 12 x 8.0 x 460/1000 = 44.2 kN <<<
Load per bolt	Shear on each bolt = 0.447/2 = 0.224 kN Modulus of web bolt group = n(n + 1)p/6 = 2 x 3 x 60/6 = 60.0 mm Moment load on outermost bolts = 0.447 x 30/60.0 = 0.224 kN Load on outermost bolts = $\sqrt{(0.224^2 + 0.224^2)}$ = 0.316 kN OK Web plate bolts are adequate

Splice shear capacity = 62.5 kN (limited by web bolt group capacity)

Beam: **Ashler Beam** **Span: 5.0 m.**

		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*
U	D	o.w.	0.5	0		L	1.25	1.25
U	D	PITCHED ROOF	1.60*2.0	0		L	8.00	8.00
U	D	TIMBER STUD PARTITION	0.75*1.2	0		L	2.25	2.25
U	D	TIMBER FLOOR	2.0*1.7	0		L	8.50	8.50
				Unfactored reactions (kN)		**Total:**	**20.00**	**20.00**
						Dead:	*20.00*	*20.00*
						Live:	*0.00*	*0.00*
		Total load: 40.00/56.00 kN *Unfactored/Factored*				**Factored reactions:**	**28.00**	**28.00**

Load types: U:UDL D: Dead; L: Live (positions in m. from R1)

Maximum B.M. (factored) = 35.0 kNm at 2.50 m. from R1

Maximum S.F. (factored) = 28.0 kN at R1

Live load deflection = 0.00 x 10^8/EI at R2 *(E in N/mm^2, I in cm^4)*

Total deflection = 65.1 x 10^8/EI at 2.50 m. from R1

Beam calculation to BS5950-1:2000 using S275 steel

SECTION SIZE : **203 x 203 x 46 UC S275 (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

Shear capacity = 0.6 p_y.t.D = 0.6 x 275 x 7.2 x 203.2/1000 = 241 kN (>=28.0) OK

Maximum moment = 35.00 kNm at 2.50 m. from R1

Moment capacity, $M_c = p_y.S_x$ = 275 x 497/1000 = 136.7 kNm OK

Beam is laterally restrained at supports only: effective length = 1.0L

Effective length (L_E) = 5.00m
Slenderness, λ (L_E/r_y) = 5.00 x 100/5.13 = 97.47
Buckling parameter (u) = 0.846
Slenderness factor (v) = 0.794 (x = 17.7; λ/x = 5.51)
β_w = 1.000 (Class 1/2 compact)
Equivalent slenderness (λ_{LT}) = u.v.λ.$\sqrt{\beta_w}$ = 65.47
Bending strength, p_b = 199.6 N/mm^2

Maximum moment within segment, M_x = 35.00 kNm
Equivalent uniform moment factor, m_{LT} = 0.925 (M_2=26.3, M_3=35.0, M_4=26.3)
Equivalent uniform moment = 0.925 x 35.00 = 32.38 kNm
Buckling resistance moment, $M_b = p_b.S_x$ = 199.6 x 497/1000 = 99.19 kNm OK

Check unstiffened web capacity with load of 28.00 kN
C1 = 84.0 kN; C2 = 1.98 kN/mm; C4 = 399; K = min{0.5+(a_e/1.4d),1.0}; p_{yw} = 275N/mm^2
(for derivation of C factors see Steelwork Design Guide to BS5950-1:2000 6th ed.)
Bearing capacity, P_w = C1+b_1C2 (b_e taken as zero) Buckling capacity, P_x = K$\sqrt{(C4.P_w)}$
Unstiffened web bearing capacity, P_w = 84.0 kN: no minimum stiff bearing length required

LL deflection = 0.000 x 1e8/205,000 x 4570.000 = 0.0 mm OK
TL deflection = 65.10 x 1e8/205,000 x 4570 = 6.9 mm (L/720)

Bearing details

203x203x46 UC stiff bearing length, b_1 = t + 1.6r + 2T = 45.5 mm

Local design strength of masonry (factored) = 0.700 N/mm^2 (User-entered value)

R1: 450 x 100 mm bearing plate

Factored reaction = 20.00 x 1.4 + 0.00 x 1.6 = 28.00 kN

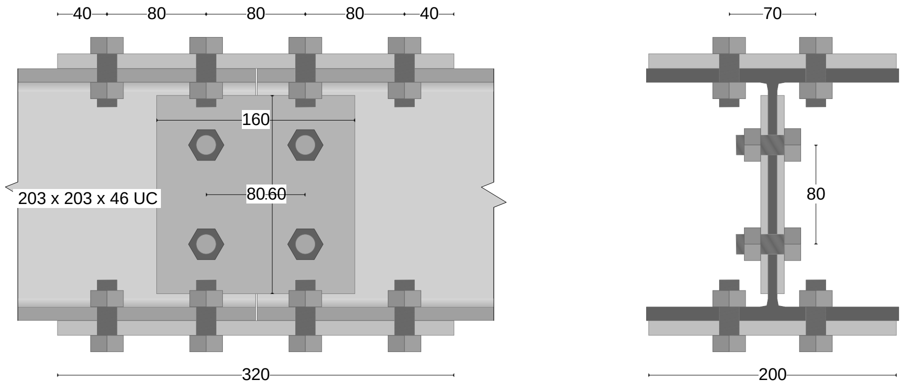
20 mm m.s. bearing plate, size 450 x 100 mm
Bearing plate projection beyond stiff bearing length = (450-45.5)/2 = 202.2mm
Factored stress under plate = 28.00 x 1000/450 x 100 = 0.62 N/mm^2
Required plate thickness = $\sqrt{(3x0.62x202x202/265)}$ = 17.0 mm: use 20mm
Factored bending stress in plate = 0.62x202x(202/2)/(20x20/6) = 190.9 N/mm^2 (p_y=265 N/mm^2)

R2 as R1

Encase beam to provide half-hour fire resistance as per specification.

Flange plate splice calculation

Beam: Ashler Beam



Flange plates: 320 x 200 x 12, M16 8.8 bolts (16). Web plates: 160 x 160 x 6, M16 8.8 bolts (4)

Beam span: 5.0 m. Section: 203 x 203 x 46 UC S275

Splice 1: 2.5 m. from R1. BM: 35.0 kNm SF: 0.000 kN (factored)

Section dims: D = 203.2 B = 203.6 T = 11.0 t = 7.2 d = 160.8 r = 10.2

Use S275 flange plates 320 x 200 x 12 mm,

Use 2 pairs of M16 8.8 bolts at 70 mm cross centres; inmost pairs of bolts 80 mm apart, 1 further sets of bolts at 80 mm pitch

Web plates: 160 x 160 x 6 mm, 2 pairs of M16 bolts at 80 mm (H) and 80 mm (V)

Basic detailing checks

Flange/Flange Plates: Using M16 bolts, washer diameter 30 mm

Check flange bolt washer clear beam web: pitch (70 mm) >= 58 mm OK
Flange plate bolt edge distance (65.0 mm) >= 1.25D (20 mm) OK
Flange bolt edge distance (66.8 mm) >= 1.25D (20 mm) OK
Plate plate bolt end distance (40 mm) >= 1.25D (20 mm) OK
Flange bolt end distance (40 mm) >= 1.25D (20 mm) OK
Flange bolt spacing (80 mm) >= 2.5D (40 mm) OK

Web/Web Plates: Using M16 bolts, washer diameter 30 mm

Check web plate clears beam roots: plate height, 160 mm < d, 160.8 mm OK
Web plate bolt end distance H (40 mm) >= 2.0D (32 mm) OK
Web plate bolt end distance V (40 mm) >= 2.0D (32 mm) OK
Web plate bolt pitch (80 mm) >= 2.5D (40 mm) OK
Web bolt end clearance (40 mm) >= 2.0D (32 mm) OK
All dimensional checks satisfied

Flange plates

Check splice with BM = 35.0 kNm

Axial force in beam flanges = 35.0 x 1000/(203.2-11.0) = 182 kN

Beam flange area, gross = 2,240 mm^2, net = 1,844 mm^2; Flange plate area: gross = 2,400 mm^2, net = 1,968 mm^2

Axial capacities:	Plate: 2,400 x 275/1000 = 660.0 kN Plate net: 1.2 x 1,968 x 275/1000 = 649.4 kN Beam flange: 2,240 x 275/1000 = 615.9 kN Beam flange net: 1.2 x 1,844 x 275/1000 = 608.4 kN <<< OK
Plate block shear:	0.6 x 275 x 12 x [120 + 1.2 x (135 - 2.5 x 18)]/1000 = 451 kN OK (K_e = 1.2; k = 2.5)
Strut action moment:	Flange effective area: 2,212 mm^2 Plate effective area: 2,362 mm^2 Flange modulus, Z_y: 66.8cm^3 Plate modulus, Z_y: 69.9 cm^3 Check for strut action moment not required

Bolt capacity: Shear: 157 x 375/1000 = 58.9 kN
Bearing on plate: 16 x 12 x 460/1000 = 88.3 kN
Bearing on flange: 16 x 11.0 x 460/1000 = 81.0 kN

Bolt group capacity: Flange: 4 x 58.9 = 235.5 kN OK
Plate: 4 x 58.9 = 235.5 kN OK

Splice moment capacity = 45.3 kNm (limited by flange bolt group)

Web plates

Check splice with SF = 0.000 kN

Plate shear capacity: Gross area = 2 x 160 x 6 = 1,920 mm^2
Net area = 1,920 - (2 x 2 x 18 x 6) = 1,488 mm^2
Net shear capacity limit = 0.85 x A_v/K_e = 0.85 x 1,920/1.2 = 1,360 mm^2
Reduction for bolt holes not required
Shear capacity = 0.6 x 1,920 x 275/1000 = 316.8 kN OK

Plate block shear: 2 x 0.6 x 275 x 6 x [81 + 1.2 x (80.0 - 0.5 x 18)]/1000 = 329 kN OK (K_e = 1.2; k = 0.5)

Bolt capacity: Shear: 2 x 157 x 375/1000 = 118 kN
Bearing on plates: 2 x 16 x 6 x 460/1000 = 88.3 kN
Bearing on beam web: 16 x 7.2 x 460/1000 = 53.0 kN <<<

Load per bolt Shear on each bolt = 0.000/2 = 0.000 kN
Modulus of web bolt group = n(n + 1)p/6 = 2 x 3 x 80/6 = 80.0 mm
Moment load on outermost bolts = 0.000 x 40/80.0 = 0.000 kN
Load on outermost bolts = $\sqrt{(0.000^2 + 0.000^2)}$ = 0.000 kN OK
Web plate bolts are adequate

Splice shear capacity = 74.9 kN (limited by web bolt group capacity)

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Beam: **Beam A** **Span: 2.4 m.**

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U	D	o.w.	0.15	0		L	0.18	0.18
U	D	FLAT ROOF	1.55*1.80	0		L	3.35	3.35
U	D	TILE HANGING TO TIMBER F	1.2*2.4	0		L	3.46	3.46
U	D	TIMBER FLOOR	2.0*1.8	0		L	4.32	4.32
				Unfactored reactions (kN)		**Total:**	**11.30**	**11.30**
						Dead:	*11.30*	*11.30*
						Live:	*0.00*	*0.00*
		Total load: 22.61/31.65 kN *Unfactored/Factored*				**Factored reactions:**	**15.83**	**15.83**

Load types: U:UDL D: Dead; L: Live (positions in m. from R1)

Maximum B.M. (factored) = 9.50 kNm at 1.20 m. from R1

Maximum S.F. (factored) = 15.8 kN at R1

Live load deflection = 0.00 x 10^8/EI at R2 *(E in N/mm², I in cm⁴)*

Total deflection = 4.07 x 10^8/EI at 1.20 m. from R1

Beam calculation to BS5950-1:2000 using S275 steel

SECTION SIZE : **127 x 76 x 13 UB S275 (compact)**

D=127.0 mm B=76.0 mm t=4.0 mm T=7.6 mm I_x=473 cm^4 r_y=1.84 cm S_x=84.0 cm^3

Shear capacity = 0.6 p_y.t.D = 0.6 x 275 x 4.0 x 127.0/1000 = 83.8 kN (>=15.8) OK

Maximum moment = 9.495 kNm at 1.20 m. from R1

Moment capacity, M_c = p_y.S_x = 275 x 84.0/1000 = 23.10 kNm OK

Beam is laterally restrained at supports only: effective length = 1.0L

Effective length (L_E) = 2.40m
Buckling parameter (u) = 0.896
β_w = 1.000 (Class 1/2 compact)
Bending strength, p_b = 161.2 N/mm^2

Slenderness, λ (L_E/r_y) = 2.40 x 100/1.84 = 130.4
Slenderness factor (v) = 0.698 (x = 16.3; λ/x = 8.00)
Equivalent slenderness (λ_{LT}) = u.v.λ.$\sqrt{\beta_w}$ = 81.63

Maximum moment within segment, M_x = 9.495 kNm
Equivalent uniform moment factor, m_{LT} = 0.925 (M_2=7.12, M_3=9.50, M_4=7.12)
Equivalent uniform moment = 0.925 x 9.495 = 8.783 kNm
Buckling resistance moment, M_b = p_b.S_x = 161.2 x 84.0/1000 = 13.54 kNm OK

Check unstiffened web capacity with load of 15.83 kN
C1 = 33.4 kN; C2 = 1.10 kN/mm; C4 = 114; K = min{0.5+(a_e/1.4d),1.0}; p_{yw} = 275N/mm^2
(for derivation of C factors see Steelwork Design Guide to BS5950-1:2000 6th ed.)
Bearing capacity, P_w = C1+b_1C2 (b_e taken as zero) Buckling capacity, P_x = K$\sqrt{(C4.P_w)}$
With b_1=0, unstiffened web buckling capacity, P_x = 30.9 kN: no minimum stiff bearing length required

LL deflection = 0.000 x 1e8/205,000 x 473.000 = 0.0 mm OK
TL deflection = 4.069 x 1e8/205,000 x 473 = 4.2 mm (L/572)

Bearing details

127x76x13 UB stiff bearing length, b_1 = t + 1.6r + 2T = 31.4 mm

Local design strength of masonry (factored) = 0.700 N/mm^2 (User-entered value)

R1: 100 x 250 mm bearing plate

Factored reaction = 11.30 x 1.4 + 0.00 x 1.6 = 15.83 kN

5 mm m.s. bearing plate, size 100 x 250 mm
Bearing plate projection beyond stiff bearing length = (100-31.4)/2 = 34.3mm
Factored stress under plate = 15.83 x 1000/100 x 250 = 0.63 N/mm^2
Required plate thickness = $\sqrt{(3x0.63x34.3x34.3/275)}$ = 2.85 mm: use 5mm
Factored bending stress in plate = 0.63x34.3x(34.3/2)/(5x5/6) = 89.5 N/mm^2 (p_y=275 N/mm^2)

R2 as R1

Encase beam to provide half-hour fire resistance as per specification.

Beam: **Beam B** **Span: 2.2 m.**

		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*
U	D	o.w.	0.15	0		L	0.17	0.17
U	D	TIMBER FLOOR	2.0*1.4	0		L	3.08	3.08
U	D	TIMBER FLOOR	2.0*1.4	0		L	3.08	3.08
					Unfactored reactions (kN)	**Total:**	**6.32**	**6.32**
						Dead:	*6.32*	*6.32*
						Live:	*0.00*	*0.00*
		Total load: 12.65/17.71 kN	*Unfactored/Factored*			**Factored reactions:**	**8.85**	**8.85**

Load types: U:UDL D: Dead; L: Live (positions in m. from R1)

Maximum B.M. (factored) = 4.87 kNm at 1.10 m. from R1

Maximum S.F. (factored) = 8.85 kN at R1

Live load deflection = 0.00 x 10^8/EI at R2 *(E in N/mm², I in cm⁴)*

Total deflection = 1.75 x 10^8/EI at 1.10 m. from R1

Beam calculation to BS5950-1:2000 using S275 steel

SECTION SIZE : **127 x 76 x 13 UB S275 (compact)**

D=127.0 mm B=76.0 mm t=4.0 mm T=7.6 mm I_x=473 cm^4 r_y=1.84 cm S_x=84.0 cm^3

Shear capacity = 0.6 p_y.t.D = 0.6 x 275 x 4.0 x 127.0/1000 = 83.8 kN (>=8.85) OK

Maximum moment = 4.870 kNm at 1.10 m. from R1

Moment capacity, M_c = p_y.S_x = 275 x 84.0/1000 = 23.10 kNm OK

Beam is laterally restrained at supports only: effective length = 1.0L

Effective length (L_F) = 2.20m
Buckling parameter (u) = 0.896
β_w = 1.000 (Class 1/2 compact)
Bending strength, p_b = 171.0 N/mm²

Slenderness, λ (L_F/r_v) = 2.20 x 100/1.84 = 119.6
Slenderness factor (v) = 0.721 (x = 16.3; λ/x = 7.34)
Equivalent slenderness (λ_{LT}) = u.v.λ.$\sqrt{\beta_w}$ = 77.29

Maximum moment within segment, M_x = 4.870 kNm
Equivalent uniform moment factor, m_{LT} = 0.925 (M_2=3.65, M_3=4.87, M_4=3.65)
Equivalent uniform moment = 0.925 x 4.870 = 4.505 kNm
Buckling resistance moment, M_b = p_b.S_x = 171.0 x 84.0/1000 = 14.37 kNm OK

Check unstiffened web capacity with load of 8.855 kN
C1 = 33.4 kN; C2 = 1.10 kN/mm; C4 = 114; K = min{0.5+(a_e/1.4d),1.0}; p_{yw} = 275N/mm²
(for derivation of C factors see Steelwork Design Guide to BS5950-1:2000 6th ed.)
Bearing capacity, P_w = C1+b_1C2 (b_e taken as zero) Buckling capacity, P_x = K$\sqrt{(C4.P_w)}$
With b_1=0, unstiffened web buckling capacity, P_x = 30.9 kN: no minimum stiff bearing length required

LL deflection = 0.000 x 1e8/205,000 x 473.000 = 0.0 mm OK
TL deflection = 1.754 x 1e8/205,000 x 473 = 1.8 mm (L/1216)

Bearing details

127x76x13 UB stiff bearing length, b_1 = t + 1.6r + 2T = 31.4 mm

R1: 150 x 100 mm bearing plate

Factored reaction = 6.32 x 1.4 + 0.00 x 1.6 = 8.85 kN

5 mm m.s. bearing plate, size 150 x 100 mm
Local design strength of masonry (factored) = 0.700 N/mm² (User-ented value)
Bearing plate projection beyond stiff bearing length = (150-31.4)/2 = 59.3mm
Factored stress under plate = 8.85 x 1000/150 x 100 = 0.59 N/mm²
Required plate thickness = $\sqrt{(3x0.59x59.3x59.3/275)}$ = 4.76 mm: use 5mm
Factored bending stress in plate = 0.59x59.3x(59.3/2)/(5x5/6) = 249.3 N/mm² (p_y=275 N/mm²)

R2: None

Encase beam to provide half-hour fire resistance as per specification.

Beam: **Beam C** **Span: 4.0 m.**

		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*
U	D	o.w.	0.2	0		L	0.40	0.40
U	D	TIMBER FLOOR	2.0*0.40	0		L	1.60	1.60
P	D	Beam: Beam B : R1	6.32 [B/F]	1.0			4.74	1.58
P	L	Beam: Beam B : R1	0.00 [B/F]	1.0			0.00	0.00
				Unfactored reactions (kN)		**Total:**	**6.74**	**3.58**
						Dead:	*6.74*	*3.58*
						Live:	*0.00*	*0.00*
Total load: 10.32/14.45 kN		*Unfactored/Factored*		**Factored reactions:**			**9.44**	**5.01**

Load types: U:UDL P:PL D: Dead; L: Live (positions in m. from R1)

Maximum B.M. (factored) = 8.74 kNm at 1.00 m. from R1

Maximum S.F. (factored) = 9.44 kN at R1

Live load deflection = 0.00 x 10^8/EI at R2 *(E in N/mm^2, I in cm^4)*

Total deflection = 9.19 x 10^8/EI at 1.87 m. from R1

Beam calculation to BS5950-1:2000 using S275 steel

SECTION SIZE : **152 x 89 x 16 UB S275 (compact)**

D=152.4 mm B=88.7 mm t=4.5 mm T=7.7 mm I_x=834 cm^4 r_y=2.10 cm S_x=123 cm^3

Shear capacity = 0.6 p_y.t.D = 0.6 x 275 x 4.5 x 152.4/1000 = 113 kN (>=9.44) OK

Maximum moment = 8.741 kNm at 1.00 m. from R1

Moment capacity, M_c = p_y.S_x = 275 x 123/1000 = 33.83 kNm OK

Beam is laterally restrained at supports only: effective length = 1.0L

Effective length (L_F) = 4.00m
Slenderness, λ (L_F/r_v) = 4.00 x 100/2.10 = 190.5
Buckling parameter (u) = 0.889
Slenderness factor (v) = 0.647 (x = 19.6; λ/x = 9.72)
β_w = 1.000 (Class 1/2 compact)
Equivalent slenderness (λ_{LT}) = u.v.λ.$\sqrt{\beta_w}$ = 109.5
Bending strength, p_b = 109.8 N/mm^2

Maximum moment within segment, M_x = 8.741 kNm
Equivalent uniform moment factor, m_{LT} = 0.837 (M_2=8.74, M_3=7.23, M_4=4.31)
Equivalent uniform moment = 0.837 x 8.741 = 7.320 kNm
Buckling resistance moment, M_b = p_b.S_x = 109.8 x 123/1000 = 13.50 kNm OK

Check unstiffened web capacities with loads of 9.441 kN and 5.014 kN
C1 = 37.9 kN; C2 = 1.24 kN/mm; C4 = 129; K = min{0.5+(a_e/1.4d),1.0}; p_{yw} = 275N/mm^2
(for derivation of C factors see Steelwork Design Guide to BS5950-1:2000 6th ed.)
Bearing capacity, P_w = C1+b_1C2 (b_e taken as zero) Buckling capacity, P_x = K$\sqrt{}$(C4.P_w)
R1: With b_1=0, unstiffened web buckling capacity, P_x = 34.9 kN: no minimum stiff bearing length required
R2: With b_1=0, unstiffened web buckling capacity, P_x = 34.9 kN: no minimum stiff bearing length required

LL deflection = 0.000 x 1e8/205,000 x 834.000 = 0.0 mm OK
TL deflection = 9.189 x 1e8/205,000 x 834 = 5.4 mm (L/744)

Bearing details

152x89x16 UB stiff bearing length, b_1 = t + 1.6r + 2T = 32.1 mm

Local design strength of masonry (factored) = 0.700 N/mm^2 (User-entered value)

R1: 150 x 100 mm bearing plate

Factored reaction = 6.74 x 1.4 + 0.00 x 1.6 = 9.44 kN

5 mm m.s. bearing plate, size 150 x 100 mm
Bearing plate projection beyond stiff bearing length = (150-32.1)/2 = 59mm
Factored stress under plate = 9.44 x 1000/150 x 100 = 0.63 N/mm^2
Required plate thickness = $\sqrt{}$(3x0.63x59.0x59.0/275) = 4.89 mm: use 5mm
Factored bending stress in plate = 0.63x59.0x(59.0/2)/(5x5/6) = 262.7 N/mm^2 (p_y=275 N/mm^2)

R2: 89 x 100 mm bearing plate

Factored reaction = 3.58 x 1.4 + 0.00 x 1.6 = 5.01 kN

5 mm m.s. bearing plate, size 89 x 100 mm
Bearing plate projection beyond stiff bearing length = (89-32.1)/2 = 28.5mm
Factored stress under plate = 5.01 x 1000/89 x 100 = 0.56 N/mm^2
Required plate thickness = √(3x0.56x28.5x28.5/275) = 2.23 mm: use 5mm
Factored bending stress in plate = 0.56x28.5x(28.5/2)/(5x5/6) = 54.8 N/mm^2 (p_y=275 N/mm^2)

Encase beam to provide half-hour fire resistance as per specification.

Notched beam web cleat connection check (based on SCI/BCSA Simple Connections, 2002 edition)

Location:

Factored shear at support, F_v = 8.855 kN

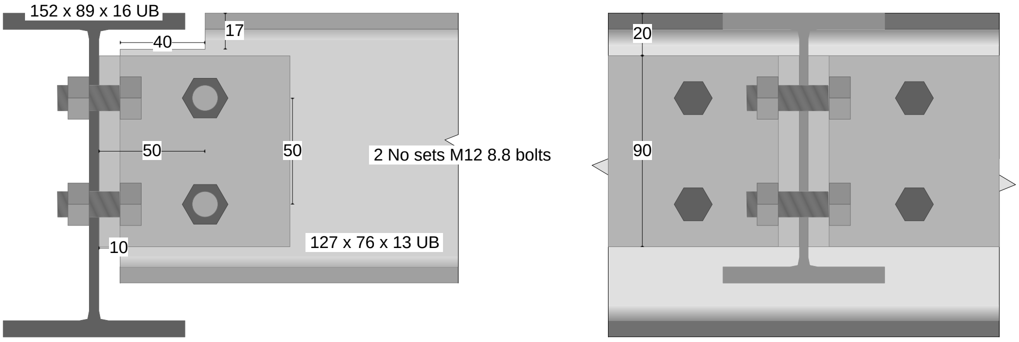
Beam: S275 127x76x13 UB connection to web of S275 152x89x16 UB

Notch dimensions: Top: 40 x 17mm

Use 2no 90mm long 90x90x10 angle cleats, 2no sets of M12 8.8 bolts at 50 mm centres

ANGLE CLEAT CONNECTION

90mm long 90 x 90 x 10mm angle cleats



Check 1: Recommended detailing requirements
All requirements satisfied

Check 2: Capacity of cleat/supported beam bolt group

Distance from face of supporting member to centre line of bolts, a = 50 mm, pitch, p_1 = 50 mm

Vertical load per bolt = $F_{sv} = F_v/n$ = 8.85/2 = 4.43 kN
Horizontal load on top and bottom bolts (from moment), $F_{sm} = B.F_v$
where $B = 6a/(n(n + 1).p_1)$ = 6 x 50/(2 x 3 x 50) = 1.00 F_{sm} = 1.00 x 8.85 = 8.85 kN
Resultant load on top and bottom bolts = $\sqrt{(4.43^2 + 8.85^2)}$ = 9.90 kN

Bolt shear capacity, P_s, per bolt = 2 x 84.3 x 375/1000 = 63.2 kN (double shear)

End distances, e, in load direction: Beam: e = 45 mm; Cleats: e = 45 mm
Bearing capacity, $P_{bs} = t.\min(D,e/2).p_{bs}$ p_{bs} S275: 460 N/mm^2; S355: 550 N/mm^2
Bearing capacity of cleats, P_{bs}, per bolt = 2 x 10 x 12 x 460/1000 = 110.4 kN
Bearing capacity of beam web, P_{bs}, per bolt = 4.0 x 12 x 460/1000 = 22.1 kN

Bolt capacity = P_{bs} = 22.1 kN/bolt (bearing capacity of beam web governs)

Bolt group capacity (single line) = $n.P_{bs}/\sqrt{(1 + (Bn)^2)}$ = 2 x 22.1/$\sqrt{(1 + (1.00 \times 2)^2)}$ = 19.7 kN OK

Check 3: Shear capacity of cleat legs at bolt line
Plain shear capacity of each cleat, P_v = 122.4 kN (x2) OK
Block shear capacity of each cleat, P_r = 180.8 kN (x2) OK

Check 4: Capacity of supported beam at bolt line
Plain shear capacity of beam web, P_v = 72.6 kN OK (F_v<=0.75P_v
Block shear capacity of beam web, P_r = 74.3 kN OK

Check 5: Shear and bending interaction at notch
Shear capacity at notch, P_{vN} = 72.6 kN (F_v<=0.75P_{vN})

Moment at notch = 8.85 x (40+10)/1000 = 0.443 kNm
Moment capacity at notch, $M_{cN} = p_y.Z_N$ = 275 x 13.0/1000 = 3.58 kNm OK

Moment-determined capacity = 3.58/((40+10)/1000) = 71.6 kN

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 support
Bolt shear capacity, P_s, per bolt = 84.3 x 375/1000 = 31.6 kN
Shear capacity of bolt group, ΣPS = 4 x 31.6 = 126.5 kN OK
Bolt bearing capacity per bolt = 12 x 10 x 460/1000 = 55.2 kN
Top pair bolt bearing capacity = 20.0 [e/2] x 10 x 460/1000 = 46.0 kN
Bearing capacity of cleats at support = 2 x 46.0 + 2 x (2 - 1) x 55.2 = 202.4 kN OK

Check 9: Shear and bearing capacity of cleats at support
Plain shear capacity, P_v = 0.7 x 275 x 1.2 x 530/1000 = 122.4 kN (x2) OK
Block shear capacity, P_r = 180.8 kN (x2) OK

Check 10: Local capacity of supporting beam web
Shear capacity, $P_v = 0.6.p_v.A_v$ = 0.6 x 275 x 518/1000 = 85.4 kN (x2) OK
Further checking is needed if this web supports connections on both sides

Summary:

Check		**Cap kN**
1	Basic detailing requirements	OK
2	**Strength of bolt group to cleat/beam web**	**19.7**
3	Shear and bearing capacity of cleat leg at bolt line	244.9
4	Capacity of supported beam at bolt line	72.6
5	Shear and bending interaction at notch	71.6
6	Local stability of notched beam	Not checked
7	Overall stability of notched beam	Not checked
8	Shear capacity of bolt group connection to support	126.5
9	Shear and bearing capacity of cleats at support	244.9
10	Local capacity of supporting member	170.8

Critical design check: 2 - Strength of bolt group to cleat/beam web

Connection capacity = **19.7 kN** OK

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

		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*
U	T	o.w.	0.1	0		L	0.17	0.17
U	T	TIMBER FLOOR	2.00*0.40	0		L	1.36	1.36
							1.53	1.53

Total load: 3.06 kN

Load types: U:UDL T: Total (positions in m. from R1)

Maximum B.M. = 1.30 kNm at 1.70 m. from R1

Maximum S.F. = 1.53 kN at R1

Total deflection = 1.57 x 10^8/EI at 1.70 m. from R1 *(E in N/mm^2, I in cm^4)*

Timber beam calculation to BS5268 Part 2: 2002 using C16 timber

Use 50 x 195 C16 *3.6 kg/m approx*

z = 316.9 cm^3 I = 3,090 cm^4

Timber grade: C16 4 members acting together: K_8 = 1.1

K_3 (loading duration factor) = 1.00 K_7 (depth factor) = 1.049 K_8 (load sharing factor) = 1.1

E = 5,800 x 1.24 = 7,192 N/mm^2 ($E_{min}.K_9$)

Bending

Permissible bending stress, $\sigma_{m,adm} = \sigma_{m,g}.K_3.K_7.K_8$ = 5.3 x 1.00 x 1.049 x 1.1 = 6.11 N/mm^2

Applied bending stress, $\sigma_{m,a}$ = 1.30 x 1000/316.9 = 4.10 N/mm^2 OK

Shear

Permissible shear stress, $\tau_{adm,//} = \tau_{g,//}.K_3.K_8$ = 0.67 x 1.00 x 1.1 = 0.74 N/mm^2

Applied shear stress, τ_a = 1.53 x 1000 x 3/2 x 50 x 195 = 0.24 N/mm^2 OK

Deflection

Bending deflection = 1.57 x 10^8/7,192 x 3,090 = 7.05 mm

Mid-span shear deflection = 1.2 x 1.30 x 10^6/((E/16) x 50 x 195) = 0.36 mm

Total deflection = 7.05 + 0.36 = 7.40 mm (0.0022 L) <=0.003L OK

Beam: **Flat roof joists** **Span: 3.8 m.**

		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*
U	T	o.w.	0.1	0		L	0.19	0.19
U	T	FLAT ROOF	1.55*0.40	0		L	1.18	1.18
							1.37	1.37

Total load: 2.74 kN

Load types: U:UDL T: Total (positions in m. from R1)

Maximum B.M. = 1.30 kNm at 1.90 m. from R1

Maximum S.F. = 1.37 kN at R1

Total deflection = 1.95 x 10^8/EI at 1.90 m. from R1 *(E in N/mm^2, I in cm^4)*

Timber beam calculation to BS5268 Part 2: 2002 using C16 timber

Use 50 x 195 C16 *3.6 kg/m approx*

z = 316.9 cm^3 I = 3,090 cm^4

Timber grade: C16 4 members acting together: K_8 = 1.1

K_3 (loading duration factor) = 1.00 K_7 (depth factor) = 1.049 K_8 (load sharing factor) = 1.1

E = 5,800 x 1.24 = 7,192 N/mm^2 ($E_{min}.K_9$)

Bending

Permissible bending stress, $\sigma_{m,adm} = \sigma_{m,g}.K_3.K_7.K_8$ = 5.3 x 1.00 x 1.049 x 1.1 = 6.11 N/mm^2

Applied bending stress, $\sigma_{m,a}$ = 1.30 x 1000/316.9 = 4.10 N/mm^2 OK

Shear

Permissible shear stress, $\tau_{adm,//} = \tau_{g,//}.K_3.K_8$ = 0.67 x 1.00 x 1.1 = 0.74 N/mm^2

Applied shear stress, τ_a = 1.37 x 1000 x 3/2 x 50 x 195 = 0.21 N/mm^2 OK

Deflection

Bending deflection = 1.95 x 10^8/7,192 x 3,090 = 8.80 mm

Mid-span shear deflection = 1.2 x 1.30 x 10^6/((E/16) x 50 x 195) = 0.36 mm

Total deflection = 8.80 + 0.36 = 9.15 mm (0.0024 L) <=0.003L OK

Beam: **Floor joists alternative** **Span: 3.4 m.**

		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*
U	T	o.w.	0.1	0		L	0.17	0.17
U	T	TIMBER FLOOR	2.00*0.40	0		L	1.36	1.36
							1.53	1.53

Total load: 3.06 kN

Load types: U:UDL T: Total (positions in m. from R1)

Maximum B.M. = 1.30 kNm at 1.70 m. from R1

Maximum S.F. = 1.53 kN at R1

Total deflection = 1.57 x 10^8/EI at 1.70 m. from R1 *(E in N/mm^2, I in cm^4)*

Timber beam calculation to BS5268 Part 2: 2002 using C24 timber

Use 75 x 150 C24 *4.7 kg/m approx*

z = 281.3 cm^3 I = 2,109 cm^4

Timber grade: C24 4 members acting together: K_8 = 1.1

K_3 (loading duration factor) = 1.00 K_7 (depth factor) = 1.079 K_8 (load sharing factor) = 1.1

E = 7,200 x 1.24 = 8,928 N/mm^2 ($E_{min}.K_9$)

Bending

Permissible bending stress, $\sigma_{m,adm} = \sigma_{m,g}.K_3.K_7.K_8$ = 7.5 x 1.00 x 1.079 x 1.1 = 8.90 N/mm^2

Applied bending stress, $\sigma_{m,a}$ = 1.30 x 1000/281.3 = 4.62 N/mm^2 OK

Shear

Permissible shear stress, $\tau_{adm,//} = \tau_{g,//}.K_3.K_8$ = 0.71 x 1.00 x 1.1 = 0.78 N/mm^2

Applied shear stress, τ_a = 1.53 x 1000 x 3/2 x 75 x 150 = 0.20 N/mm^2 OK

Deflection

Bending deflection = 1.57 x 10^8/8,928 x 2,109 = 8.31 mm

Mid-span shear deflection = 1.2 x 1.30 x 10^6/((E/16) x 75 x 150) = 0.25 mm

Total deflection = 8.31 + 0.25 = 8.56 mm (0.0025 L) <=0.003L OK

Beam: **Beam for ridge post** **Span: 3.4 m.**

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U	T	o.w.	0.2	0		L	0.34	0.34
U	T	TIMBER FLOOR	2.00*0.40	0		L	1.36	1.36
U	T	TIMBER STUD PARTITOIN	0.75*1.80	0		L	2.30	2.30
P	T	Ridge post	21	0.2			19.76	1.24
							23.76	5.23

Total load: 28.99 kN

Load types: U:UDL P:PL T: Total (positions in m. from R1)

Maximum B.M. = 5.82 kNm at 1.17 m. from R1

Maximum S.F. = 23.8 kN at R1

Total deflection = 7.14 x 10^8/EI at 1.59 m. from R1 *(E in N/mm^2, I in cm^4)*

Steel calculation to BS449 Part 2 using S275 (Grade 43) steel

SECTION SIZE : **127 x 76 x 13 UB Grade 43**

D=127.0 mm B=76.0 mm t=4.0 mm T=7.6 mm I_x=473 cm^4 r_y=1.84 cm Z_x=75.0 cm^3

L_E/r_y = 3.40x100/1.84 = 185 D/T = 16.7

Permissible bending stress, p_{bc} = 95.3 N/mm^2 (Table 3a)

Actual bending stress, f_{bc} = 5.820 x 1000/75.0 = 77.6 N/mm^2 OK

Maximum shear in web, f_s = 23.76 x 1000/(4.0 x 127.0) = 46.8 N/mm^2 OK

Check unstiffened web capacities with loads of 23.76 kN and 5.230 kN

Stiffeners are required where reactions are taken on bottom flange and minimum stiff bearing length(s) shown below cannot be provided

Bearing: p_h = 210N/mm^2 (Table 9); C1 = 22.1 kN; C2 = 0.840 kN/mm

Buckling: p_c = 149N/mm^2 (Table 17a); C1 = 37.8 kN; C2 = 0.596 kN/mm

R1: Minimum required stiff bearing length, L_h = 2mm

Bearing capacity, P_w = 22.1 + 2 x 0.840 = 23.8kN

With L_h = 2mm, buckling capacity, P_x = 37.8 + 2 x 0.596 = 39.0kN

R2: Unstiffened web bearing capacity, P_w = 22.1kN: no minimum stiff bearing length required

Total deflection = 7.14 x 1e8/(205,000 x 473) = 7.4 mm (L/462) OK

Combined bending and shear check (14.c): $(f_{bc}/p_{bc})^2 + (f_s/p_s)^2$ = 0.663 at 1.16 m. (<=1.25 OK)

Bearing details *(bearing plate sizing to BS5950-1:2000)*

127x76x13 UB stiff bearing length, b_1 = t + 1.6r + 2T = 31.4 mm

Factor reactions by 1.55 (user selected value)

Local design strength of masonry (factored) = 0.700 N/mm^2 (User-entered value)

R1: 550 x 100 mm bearing plate

Factored reaction = 23.76 x 1.55 = 36.83 kN

25 mm m.s. bearing plate, size 550 x 100 mm
Bearing plate projection beyond stiff bearing length = (550-31.4)/2 = 259.3mm
Factored stress under plate = 1.55 x 23.76 x 1000/550 x 100 = 0.67 N/mm^2
Required plate thickness = √(3x0.67x259x259/265) = 22.6 mm: use 25mm
Factored bending stress in plate = 0.67x259x(259/2)/(25x25/6) = 216.1 N/mm^2 (p_y=265 N/mm^2)

R2: 125 x 100 mm bearing plate

Factored reaction = 5.23 x 1.55 = 8.11 kN

5 mm m.s. bearing plate, size 125 x 100 mm
Bearing plate projection beyond stiff bearing length = (125-31.4)/2 = 46.8mm
Factored stress under plate = 1.55 x 5.23 x 1000/125 x 100 = 0.65 N/mm^2
Required plate thickness = √(3x0.65x46.8x46.8/275) = 3.94 mm: use 5mm
Factored bending stress in plate = 0.65x46.8x(46.8/2)/(5x5/6) = 170.6 N/mm^2 (p_y=275 N/mm^2)