2 P S Design		PAGE:	1
PROJECT:	THE MALTHOUSE, BEAUWORTH	CALCS BY:	KP
ELEMENT:	STRUCTURAL CALCULATIONS	DATE:	08/09/2020

Reference drawings

Fowler Architecture drawing numbers 191125/201 to 204 inclusive.

General loading data

Pitched roof load

Weight of rafters = 0.15 x 0.05 x 6 x 1000/400 = 0.12 kN/m²

Weight of tiles or thatch, = say 0.7 kN/m²

Weight of battens, felt, insulation etc = say 0.18 kN/m²

Weight of plasterboard and skim coat = 0.2 kN/m²

Total roof dead load = 0.12 + 0.7 + 0.18 + 0.2 = 1.2 kN/m²

Roof imposed load (snow) = 0.3 kN/m²

Total roof load (unfactored) = 1.2 + 0.3 = 1.5 kN/m²

Roof load at ULS = (1.4 x 1.2) + (1.6 x 0.3) = 2.2 kN/m²

Floor load

Weight of floor joists = 0.15 kN/m²

Weight of floor boards = 0.12 kN/m²

Weight of plasterboard ceiling and skim coat = 0.3 kN/m²

Total dead load = 0.15 + 0.12 + 0.3 = 0.57, say 0.6 kN/m²

Floor imposed load = 1.5 kN/m²

Total floor load (unfactored) = 0.6 + 1.5 = 2.1 kN/m²

Floor loading at ULS = (1.4 x 0.6) + (1.6 x 1.5) = 3.3 kN/m²

Ceiling load

Weight of 50 x 100 ceiling joists = 0.05 x 0.1 x 6 x 1000/400 = 0.1 kN/m^2

Weight of plasterboard ceiling and skim coat = 0.3 kN/m^2

Total dead load = 0.1 + 0.3 = 0.4 kN/m^2

Imposed load = 0.25 kN/m^2

Total ceiling load (unfactored) = 0.4 + 0.25 = 0.65 kN/m^2

Ceiling loading at ULS = (1.4 x 0.4) + (1.6 x 0.25) = 1.0 kN/m^2

Cavity wall load

Weight of 100 thick leaf of bricks or blocks = 2.2 kN/m^2

Weight of plaster lining = 0.3 kN/m^2

Total weight = 2.2 + 0.3 = 2.5 kN/m^2

Weight at ULS = 1.4 x 2.5 = 3.5 kN/m^2

Note
Dimensions in these calculations are for structural purposes only, having been scaled from copies of drawings. The building contractor is to obtain detailed dimensions from site measurements.

The contractor is responsible for ensuring the stability of the structure at all times and that the works are carried out in accordance with the Construction, Design and Management Regulations.

These calculations only apply to the elements included in these documents. If any discrepancies are found on site the Engineer is to be informed.

Load on rafters

Udl from pitched roof

Load = 1.5 x 0.4 = 0.6 kN/m

Maximum span between centres of supports = 3.0 m

From page 4 use 50 x 150 C16 timbers at 400 mm centres

Load on existing beam over sitting room

Point load from floor beam

Load = 2.1 x 4.3 x 0.5 x 3.1 x 0.5 = 7.0 kN

Udl from floor

Load = 2.1 x 2.6 x 0.5 = 2.8 kN/m

Udl from lathe and plaster wall

Load = 0.6 x 2.4 = 1.5 kN/m

From beam self weight

Load at ULS = 0.14 x 0.3 x 6 = 0.3 kN/m

Total udl on beam = 2.8 + 1.5 + 0.3 = 4.6 kN/m

Maximum span between centres of supports = 4.4 m

Load on post = 4.6 x 4.4 / 2 + 7.0 = 17.1 kN

Post is to be removed and beam is to be supported from purlins above

By inspection use 127 x 76 x 13 kg/m UB

Nominal tensile capacity of M16 bolt is 30.1 kN therefore use M16 bar grade 8.8 threaded bar.

2 P S Design

PROJECT:	THE MALTHOUSE, BEAUWORTH	SHEET NO	4
ELEMENT:	RAFTERS	CALCS BY	KP
		DATE	08/09/2020

Reference (to BS5268 Part 2 1996)					
	Timber Type and Geometry				
	Span, L =	3.000	m		
	Section width, b =	50	mm		
	Section depth, d =	150	mm		
	Timber grade =	C16			
	Loading				
	UDL per rafter, w =	0.60	kN/m		
	Point load, P =	0.00	kN		
	Point of action of P from LH end, a =	0.00	m		
	Moment, $M = wL^2/8 + Pa(L-a)/L$ =	0.68	kNm		
	Shear force, $F_v = wL/2 + P(L-a)/L$ =	0.90	kN		
	Modification factors				
	Load duration factor, K_3 =	1.25			
cl 2.10.6	Depth factor, $K_7 = (300/depth)^{0.11}$ =	1.08			
cl 2.9	Load sharing factor, K_8 =	1.10			
	Total modification factor =	1.48			
	Grade stresses			**Modified stresses**	
Table 7	Bending parallel to grain	5.30	N/mm^2	7.86	N/mm^2
Table 7	Shear parallel to grain	0.67	N/mm^2	0.99	N/mm^2
Table 7	Bearing perpendicular to grain	1.70	N/mm^2	2.52	N/mm^2
Table 7	$E_{(mean)}$	8800	N/mm^2		
	Check for Bending:				
	Section modulus, $Z = bd^2/6$ =	187500	mm^3		
	Bending stress = M/Z =	3.60	N/mm^2		
	Permissible stress =	7.86	N/mm^2		OK
	Check for Deflection				
	Moment of Inertia, $I = bd^3/12$ =	14062500	mm^4		
	Area = bd =	7500	mm^2		
	UDL bending defl'n = $5wL^4/384EI$ =	5.11	mm		
	Point bending defl'n = $PL^3/48EI$ =	0.00	mm		
	UDL Shear defl'n = $12wL^2/5EA$ =	0.20	mm		
	Point shear defl'n = 24PL/5EA =	0.00	mm		
	Total deflection =	5.31	mm		
cl 2.10.7	Permissible deflection = 0.003 x span =	9.00	mm		OK
	Check for Shear				
	Shear stress = $3F_v/2A$ =	0.18	N/mm^2		
	Permissible stress =	0.99	N/mm^2		OK
	Check for Lateral Buckling				
Table 16	Max depth:breadth ratio =	5.00			
	Actual d/b =	3.00			OK
	Check for Bearing				
	Bearing width, b_w =	100	mm		
	Bearing stress = $F_v/(b_w \times b)$ =	0.18	N/mm^2		
	Permissible stress =	2.52	N/mm^2		OK

Hence section selected is OK **50 x 150** **C16**

Load on purlins over bedrooms 3 and 4

Point load from beam over sitting room

Load = 17.1 x 1.5 / 2 = 12.9 kN

Udl from pitched roof

Load = 2.2 x 2.8 = 6.2 kN/m

Udl from ceiling

Load = 1.0 x 1.8 x 0.5 = 0.9 kN/m

From beam self weight

Load at ULS = 1.4 x 0.3 = 0.4 kN/m

Total udl on beam = 6.2 + 0.9 + 0.4 = 7.5 kN/m

Design Moment and Shear Force

Maximum span between centres of supports = 7.1 m

Maximum bending moment, M = 7.5 x 7.1 x 7.1 / 8 + 12.9 x 2.7 x 4.4 / 7.1 = 68.9 kNm

Maximum shear force, Fv = 7.5 x 7.1 / 2 + 12.9 x 4.4 / 7.1 = 34.7 kN

From page 6 use 305 x 165 x 54 kg/m UB

2 P S Design

PROJECT:	THE MALTHOUSE, BEAWORTH	PAGE:	6
		BY:	KP
ELEMENT:	PURLIN OVER BEDS 3 AND 4	DATE:	08/09/2020

Reference		
(to BS 5950: Part 1: 2000)	Design bending moment, M =	68.90 kNm
	Design shear force, F_v =	34.70 kN
	Span between lateral supports, L =	7.100 m
	Steel grade selected =	S275
	Section selected =	305 x 165 x 54 UB
Table 9	Steel strength, p_y =	275 N/mm^2
Table 11	$\varepsilon = (275/p_y)^{0.5}$ =	1.00
	Flange thickness, T =	13.7 mm
	Web thickness, t =	7.9 mm
Section tables	Overall depth, D =	310.4 mm
	b/T =	6.09
	d/t =	33.60
Table 11	Section is Class	1 (plastic)
4.2.3	Shear area, Av = t.D =	2452 mm^2
4.2.3	Shear capacity, $P_v = 0.6p_yA_v$ =	404.61 kN
	Shear ratio F_v/P_v =	0.09
	The shear load is	Low
4.2.3	d/t < 70, hence no need to check for web shear buckling	
Section tables	Plastic section modulus, S =	846 cm^3
	Elastic section modulus, Z =	754 cm^3
4.2.5.2	For class 1 and 2 sections moment capacity, $M_c = p_yS$ =	232.65 kNm
4.2.5.1	Moment capacity limit for simply supported beam = $1.2p_yZ$ =	248.82 kNm
	Moment capacity is lesser of above values, hence M_c =	232.65 kNm
4.2.2	Section is not fully restrained against lateral torsional buckling between supports	
Table 13	Effective length, L_E =	1.2 L
	+	0.0 D
	Hence effective length, L_E =	8520 mm
	Radius of gyration, r_y =	3.93 cm
4.3.6.9	Ratio β_W =	1.0
	$\beta_W^{0.5}.L_E/r_y$ =	216.8
	D/T =	22.7
Table 20	Bending strength, p_b =	89 N/mm^2
4.3.7	Buckling resistance moment, $M_b = p_bS$ =	75.22 kNm

Bedroom 4 purlin support post

Assume column is 100 x 100 x 4.0 SHS, held in position at both ends and in direction at one end.

Length of column between ends = 5.0 m

End reaction from purlin = 34.7 kN, assumed to act 100 mm from the column face.

Distance from column x axis to beam load point = 100 + (100 / 2) = 150 mm

Moment in x axis from beam load = 34.7 x 0.15 = 5.2 kNm

Self weight of column = 5.0 x 0.12 x 1.4 = 0.9 kN

Hence design loads are:

F = 34.7 + 0.9 = 35.6 kN

M_x = 5.2 kNm

From pages 8 and 9 use 100 x 100 x 4.0 SHS

Post foundations

Safe ground bearing pressure assumed to be 100 kN/m^2

Design load = 35.6 / 1.5 = 23.8 kN

Therefore provide 450 x 550 wide mass concrete foundation

2 P S Design

PROJECT:	THE MALTHOUSE, BEAUWORTH	BY:	KP
ELEMENT:	SUPPORT POST TO BED 4 PURLINS	DATE:	08/09/20

Reference (to BS 5950: Part 1: 2000)				
	Column loads and geometry			
	Design bending moment, M_x =	**5.2**	**kNm**	
	Design bending moment, M_y =	**0.0**	**kNm**	
	Design shear force, F_{vx} =	**0.0**	**kN**	
	Design shear force, F_{vy} =	**0.0**	**kN**	
	Axial Force, F_c =	**35.6**	**kN**	
4.7.1.1	Segment length, L =	5.000	m	
	Steel grade selected =	S275		
	Section selection and classification			
	Section selected =	100 x 100 x 4.0 SHS		
Table 9	Steel strength, p_y =	275	N/mm^2	
Table 12 note b	$\varepsilon = (275/p_y)^{0.5}$ =	1.00		
Section tables	Local buckling ratio, d/t =	22.0		
Table 12	Since d/t < 56e section is class 1 (plastic)			
	Shear capacity			
	Cross section area, A_g =	15.2	cm^2	
Section tables	Depth of section, D =	100.0	mm	
	Width of section, B =	100.0	mm	
4.2.3 c)	Shear area, $A_v = AD/(D + B)$ =	760	mm^2	
4.2.3	Shear capacity, $P_v = 0.6p_y.A_v$ =	**125.4**	**kN**	OK
	Shear ratio for x-axis, F_{vx}/P_v =	0.00		
	Shear ratio for y-axis, F_{vy}/P_v =	0.00		
4.2.5.2	Since both shear ratios < 0.6, shear load is low in both directions			
	Cross-section capacity			
Section tables	Plastic section modulus, S =	54.4	cm^3	
Section tables	Elastic section modulus, Z =	46.4	cm^3	
4.2.5.2	For class 1 and 2 sections, $M_c = p_y S$ =	15.0	kNm	
4.2.5.1	Moment capacity general limit = $1.5p_y Z$ =	19.1	kNm	
	Hence $M_{cx} = M_{cy}$ =	**15.0**	**kNm**	OK
Table 22	Effective length, x-axis, L_{Ex} =	0.9	L	
Section tables	Radius of gyration, r_y =	3.91	cm	
	L_{Ex}/r_y =	108.7		
Table 24	Compressive strength, for x-axis fixity, p_{cx} =	139	N/mm^2	
4.7.4	Compressive resistance, for x-axis fixity, $P_{cx} = A_g.p_{cx}$ =	211.3	kN	
Table 22	Effective length, y-axis, L_{Ey} =	0.9	L	
Section tables	Radius of gyration, r_x =	3.91	cm	

	L_{Ey}/r_x =	108.7		
Table 24	Compressive strength, for y-axis fixity, p_{cy} =	139	N/mm^2	
4.7.4	Compressive resistance, for y-axis fixity, $P_{cy} = A_g.p_{cy}$ =	211.3	kN	
	Minimum compressive resistance, P_c =	211.3	kN	
	$A_g.p_y$ =	418.0	kN	
	$F_c/A_g.p_y$ =	0.09		
	M_x/M_{cx} =	0.35		
	M_y/M_{cy} =	0.00		
4.8.3.2	Cross section check, $F_c/A_g.p_y + M_x/M_{cx} + M_y/M_{cy}$ =	**0.43**		<1 so OK
	<u>Member buckling resistance</u>			
Table 26	Equivalent uniform moment factor, m_x =	0.95		
Table 26	Equivalent uniform moment factor, m_y =	0.95		
	F_c/P_c =	0.17		
	$m_x M_x/p_y Z$ =	0.39		
	$m_y M_y/p_y Z$ =	0.00		
4.8.3.3.1	Buckling check 1 - $F_c/P_c + m_x M_x/p_y Z + m_y M_y/p_y Z$ =	**0.56**		< 1 so OK
	F_c/P_{cy} =	0.17		
4.3.6.1	For SHS sections $M_b = M_c$ =	15.0	kNm	
Table 18	Equivalent uniform moment factor, m_{LT} =	0.93		
	Maximum major axis moment, M_{LT} =	5.2	kN	
	$m_{LT} M_{LT}/M_b$ =	0.32		
	$m_y M_y/p_y Z$ =	0.00		
4.8.3.3.1	Buckling check 2 - $F_c/P_{cy} + m_{LT} M_{LT}/M_b + m_y M_y/p_y Z$ =	**0.49**		<1 so OK

Load on purlins over bedroom 2

Udl from pitched roof

Load = 2.2 x 2.5 = 5.5 kN/m

Udl from ceiling

Load = 1.0 x 2.2 x 0.5 = 1.1 kN/m

From beam self weight

Load at ULS = 1.4 x 0.3 = 0.4 kN/m

Total udl on beam = 5.5 + 1.1 + 0.4 = 7.0 kN/m

Design Moment and Shear Force

Maximum span between centres of supports = 5.8 m

Maximum bending moment, M = 7.0 x 5.8 x 5.8 / 8 = 29.5 kNm

Maximum shear force, Fv = 7.0 x 5.8 / 2 = 20.3 kN

From page 11 use 254 x 146 x 31 kg/m UB

Reference			
(to BS 5950: Part 1: 2000)	Design bending moment, M =	29.50	kNm
	Design shear force, F_v =	20.30	kN
	Span between lateral supports, L =	5.800	m
	Steel grade selected =	S275	
	Section selected =	254 x 146 x 31 UB	
Table 9	Steel strength, p_y =	275	N/mm^2
Table 11	$\varepsilon = (275/p_y)^{0.5}$ =	1.00	
	Flange thickness, T =	8.6	mm
	Web thickness, t =	6.0	mm
Section tables	Overall depth, D =	251.4	mm
	b/T =	8.49	
	d/t =	36.50	
Table 11	Section is Class	1	(plastic)
4.2.3	Shear area, Av = t.D =	1508	mm^2
4.2.3	Shear capacity, $P_v = 0.6p_yA_v$ =	248.89	kN
	Shear ratio F_v/P_v =	0.08	
	The shear load is	Low	
4.2.3	d/t < 70, hence no need to check for web shear buckling		
Section tables	Plastic section modulus, S =	393	cm^3
	Elastic section modulus, Z =	351	cm^3
4.2.5.2	For class 1 and 2 sections moment capacity, $M_c = p_yS$ =	108.08	kNm
4.2.5.1	Moment capacity limit for simply supported beam = $1.2p_yZ$ =	115.83	kNm
	Moment capacity is lesser of above values, hence M_c =	108.08	kNm
4.2.2	Section is not fully restrained against lateral torsional buckling between supports		
Table 13	Effective length, L_E =	1.2	L
	+	0.0	D
	Hence effective length, L_E =	6960	mm
	Radius of gyration, r_y =	3.36	cm
4.3.6.9	Ratio β_W =	1.0	
	$\beta_W^{0.5}.L_E/r_y$ =	207.1	
	D/T =	29.2	
Table 20	Bending strength, p_b =	79	N/mm^2
4.3.7	Buckling resistance moment, $M_b = p_bS$ =	31.23	kNm

Load on purlins over bedroom 1

Udl from pitched roof

Load = 2.2 x 3.0 = 6.6 kN/m

Udl from ceiling

Load = 1.0 x 2.6 x 0.5 = 1.3 kN/m

From beam self weight

Load at ULS = 1.4 x 0.3 = 0.4 kN/m

Total udl on beam = 6.6 + 1.3 + 0.4 = 8.3 kN/m

Design Moment and Shear Force

Maximum span between centres of supports = 6.2 m

Maximum bending moment, M = 8.3 x 6.2 x 6.2 / 8 = 40.0 kNm

Maximum shear force, Fv = 8.3 x 6.2 / 2 = 25.8 kN

From page 13 use 254 x 146 x 37 kg/m UB

By inspection use same size over bathroom and landing

2 P S Design	PAGE: 13
PROJECT: THE MALTHOUSE, BEAWORTH	BY: KP
ELEMENT: PURLINS OVER BED 1	DATE: 08/09/2020

Reference (to BS 5950: Part 1: 2000)			
	Design bending moment, M =	40.00	kNm
	Design shear force, F_v =	25.80	kN
	Span between lateral supports, L =	6.200	m
	Steel grade selected =	S275	
	Section selected =	254 x 146 x 37 UB	
Table 9	Steel strength, p_y =	275	N/mm^2
Table 11	$\varepsilon = (275/p_y)^{0.5}$ =	1.00	
	Flange thickness, T =	10.9	mm
	Web thickness, t =	6.3	mm
Section tables	Overall depth, D =	256.0	mm
	b/T =	6.72	
	d/t =	34.80	
Table 11	Section is Class	1	(plastic)
4.2.3	Shear area, Av = t.D =	1613	mm^2
4.2.3	Shear capacity, $P_v = 0.6p_yA_v$ =	266.11	kN
	Shear ratio F_v/P_v =	0.10	
	The shear load is	Low	
4.2.3	d/t < 70, hence no need to check for web shear buckling		
Section tables	Plastic section modulus, S =	483	cm^3
	Elastic section modulus, Z =	433	cm^3
4.2.5.2	For class 1 and 2 sections moment capacity, $M_c = p_yS$ =	132.83	kNm
4.2.5.1	Moment capacity limit for simply supported beam = $1.2p_yZ$ =	142.89	kNm
	Moment capacity is lesser of above values, hence M_c =	132.83	kNm
4.2.2	Section is not fully restrained against lateral torsional buckling between supports		
Table 13	Effective length, L_E =	1.2	L
	+	0.0	D
	Hence effective length, L_E =	7440	mm
	Radius of gyration, r_y =	3.48	cm
4.3.6.9	Ratio β_W =	1.0	
	$\beta_W^{0.5}.L_E/r_y$ =	213.8	
	D/T =	23.5	
Table 20	Bending strength, p_b =	88	N/mm^2
4.3.7	Buckling resistance moment, $M_b = p_bS$ =	42.52	kNm

Check on cantilever purlins over bedroom 2

Point load from landing purlin

Load = 2.2 x 2.1 x 3.2 x 0.5 = 7.4 kN

Udl from pitched roof

Load = 2.2 x 2.1 = 4.6 kN/m

Udl from ceiling

Load = 1.0 x 2.6 x 0.5 = 1.3 kN/m

From beam self weight

Load at ULS = 1.4 x 0.3 = 0.4 kN/m

Total udl on beam = 4.6 + 1.3 + 0.4 = 6.3 kN/m

Design Moment and Shear Force

Maximum cantilever span = 1.4 m

Maximum bending moment, M = 6.3 x 1.4 x 1.4 / 2 + 7.4 x 1.4 = 16.6 kNm

Maximum shear force, Fv = 6.3 x 1.4 + 7.4 = 16.2 kN

By comparison to bed 2 purlin calculations use 254 x 146 x 31 kg/m UB

Load on beam to support bedroom 2 purlin

Point load from purlins

Load = 20.3 + 16.2 = 36.5 kN

Design Moment and Shear Force

Maximum cantilever span = 0.6 m

Maximum bending moment, M = 36.5 x 0.6 = 21.9 kNm

Maximum shear force, Fv = 36.5 kN

From page 16 use 152 x 89 x 16 kg/m UB

Bedroom 2 purlin support post

Assume column is 70 x 70 x 3.6 SHS, held in position at both ends and in direction at one end.

Length of column between ends = 3.1 m

End reaction from purlin = 36.5 kN, assumed to act 100 mm from the column face.

Distance from column x axis to beam load point = 100 + (70 / 2) = 135 mm

Moment in x axis from beam load = 36.5 x 0.135 = 5.0 kNm

Self weight of column = 3.1 x 0.08 x 1.4 = 0.4 kN

Hence design loads are:

F = 36.5 + 0.4 = 36.9 kN

M_x = 5.0 kNm

From pages 17 and 18 use 70 x 70 x 5.0 SHS

Reference (to BS 5950: Part 1: 2000)		
	Design bending moment, M =	21.90 kNm
	Design shear force, F_v =	36.50 kN
	Span between lateral supports, L =	0.600 m
	Steel grade selected =	S275
	Section selected =	152 x 89 x 16 UB
Table 9	Steel strength, p_y =	275 N/mm^2
Table 11	$\varepsilon = (275/p_y)^{0.5}$ =	1.00
	Flange thickness, T =	7.7 mm
	Web thickness, t =	4.5 mm
Section tables	Overall depth, D =	152.4 mm
	b/T =	5.76
	d/t =	27.10
Table 11	Section is Class	1 (plastic)
4.2.3	Shear area, Av = t.D =	686 mm^2
4.2.3	Shear capacity, $P_v = 0.6p_yA_v$ =	113.16 kN
	Shear ratio F_v/P_v =	0.32
	The shear load is	Low
4.2.3	d/t < 70, hence no need to check for web shear buckling	
Section tables	Plastic section modulus, S =	123 cm^3
	Elastic section modulus, Z =	109 cm^3
4.2.5.2	For class 1 and 2 sections moment capacity, $M_c = p_yS$ =	33.83 kNm
4.2.5.1	Moment capacity limit for simply supported beam = $1.2p_yZ$ =	35.97 kNm
	Moment capacity is lesser of above values, hence M_c =	33.83 kNm
4.2.2	Section is not fully restrained against lateral torsional buckling between supports	
Table 13	Effective length, L_E =	1.2 L
	+	0.0 D
	Hence effective length, L_E =	720 mm
	Radius of gyration, r_y =	2.10 cm
4.3.6.9	Ratio β_W =	1.0
	$\beta_W^{0.5}.L_E/r_y$ =	34.3
	D/T =	19.8
Table 20	Bending strength, p_b =	275 N/mm^2
4.3.7	Buckling resistance moment, $M_b = p_bS$ =	33.83 kNm

Reference (to BS 5950: Part 1: 2000)				
	Column loads and geometry			
	Design bending moment, M_x =	**5.0**	**kNm**	
	Design bending moment, M_y =	**0.0**	**kNm**	
	Design shear force, F_{vx} =	**0.0**	**kN**	
	Design shear force, F_{vy} =	**0.0**	**kN**	
	Axial Force, F_c =	**36.9**	**kN**	
4.7.1.1	Segment length, L =	3.100	m	
	Steel grade selected =	S275		
	Section selection and classification			
	Section selected =	70 x 70 x 5.0 SHS		
Table 9	Steel strength, p_y =	275	N/mm^2	
Table 12 note b	$\varepsilon = (275/p_y)^{0.5}$ =	1.00		
Section tables	Local buckling ratio, d/t =	11.0		
Table 12	Since d/t < 56e section is class 1 (plastic)			
	Shear capacity			
	Cross section area, A_g =	12.7	cm^2	
Section tables	Depth of section, D =	70.0	mm	
	Width of section, B =	70.0	mm	
4.2.3 c)	Shear area, $A_v = AD/(D + B)$ =	635	mm^2	
4.2.3	Shear capacity, $P_v = 0.6p_y.A_v$ =	**104.8**	**kN**	OK
	Shear ratio for x-axis, F_{vx}/P_v =	0.00		
	Shear ratio for y-axis, F_{vy}/P_v =	0.00		
4.2.5.2	Since both shear ratios < 0.6, shear load is low in both directions			
	Cross-section capacity			
Section tables	Plastic section modulus, S =	30.8	cm^3	
Section tables	Elastic section modulus, Z =	25.3	cm^3	
4.2.5.2	For class 1 and 2 sections, $M_c = p_y S$ =	8.5	kNm	
4.2.5.1	Moment capacity general limit = $1.5p_y Z$ =	10.4	kNm	
	Hence $M_{cx} = M_{cy}$ =	**8.5**	**kNm**	OK
Table 22	Effective length, x-axis, L_{Ex} =	0.9	L	
Section tables	Radius of gyration, r_y =	2.64	cm	
	L_{Ex}/r_y =	99.8		
Table 24	Compressive strength, for x-axis fixity, p_{cx} =	157	N/mm^2	
4.7.4	Compressive resistance, for x-axis fixity, $P_{cx} = A_g.p_{cx}$ =	199.4	kN	
Table 22	Effective length, y-axis, L_{Ey} =	0.9	L	
Section tables	Radius of gyration, r_x =	2.64	cm	

	L_{Ey}/r_x =	99.8		
Table 24	Compressive strength, for y-axis fixity, p_{cy} =	157	N/mm^2	
4.7.4	Compressive resistance, for y-axis fixity, $P_{cy} = A_g.p_{cy}$ =	199.4	kN	
	Minimum compressive resistance, P_c =	199.4	kN	
	$A_g.p_y$ =	349.3	kN	
	$F_c/A_g.p_y$ =	0.11		
	M_x/M_{cx} =	0.59		
	M_y/M_{cy} =	0.00		
4.8.3.2	Cross section check, $F_c/A_g.p_y + M_x/M_{cx} + M_y/M_{cy}$ =	**0.70**		<1 so OK
	<u>Member buckling resistance</u>			
Table 26	Equivalent uniform moment factor, m_x =	0.95		
Table 26	Equivalent uniform moment factor, m_y =	0.95		
	F_c/P_c =	0.19		
	$m_x M_x/p_y Z$ =	0.68		
	$m_y M_y/p_y Z$ =	0.00		
4.8.3.3.1	Buckling check 1 - $F_c/P_c + m_x M_x/p_y Z + m_y M_y/p_y Z$ =	**0.87**		< 1 so OK
	F_c/P_{cy} =	0.19		
4.3.6.1	For SHS sections $M_b = M_c$ =	8.5	kNm	
Table 18	Equivalent uniform moment factor, m_{LT} =	0.93		
	Maximum major axis moment, M_{LT} =	5.0	kN	
	$m_{LT} M_{LT}/M_b$ =	0.55		
	$m_y M_y/p_y Z$ =	0.00		
4.8.3.3.1	Buckling check 2 - $F_c/P_{cy} + m_{LT} M_{LT}/M_b + m_y M_y/p_y Z$ =	**0.73**		<1 so OK

Load on beam to support bedroom 1 purlin

Point load from purlin

Load = 8.3 x 9.8 x 0.5 = 40.7 kN

Design Moment and Shear Force

Maximum span between centres of supports = 4.4 m

Maximum bending moment, M = 40.7 x 4.4 / 4 = 44.8 kNm

Maximum shear force, Fv = 40.7 / 2 = 20.4 kN

From page 20 use 254 x 146 x 37 kg/m UB

Bedroom 1 purlin support post

Assume column is 70 x 70 x 3.6 SHS, held in position at both ends and in direction at one end.

Length of column between ends = 3.1 m

End reaction from beam = 20.4 kN, assumed to act 100 mm from the column face.

End reaction from purlin = 8.3 x 7.8 / 2 = 32.4 kN

Distance from column x axis to beam load point = 100 + (70 / 2) = 135 mm

Moment in x axis from beam load = (32.4 – 20.4) x 0.135 = 1.6 kNm

Self weight of column = 3.1 x 0.08 x 1.4 = 0.4 kN

Hence design loads are:

F = 20.4 + 32.4 + 0.4 = 53.2 kN

M_x = 1.6 kNm

From pages 21 and 22 use 70 x 70 x 3.6 SHS

Reference (to BS 5950: Part 1: 2000)		
	Design bending moment, M =	44.80 kNm
	Design shear force, F_v =	20.40 kN
	Span between lateral supports, L =	4.400 m
	Steel grade selected =	S275
	Section selected =	254 x 146 x 37 UB
Table 9	Steel strength, p_y =	275 N/mm^2
Table 11	$\varepsilon = (275/p_y)^{0.5}$ =	1.00
	Flange thickness, T =	10.9 mm
	Web thickness, t =	6.3 mm
Section tables	Overall depth, D =	256.0 mm
	b/T =	6.72
	d/t =	34.80
Table 11	Section is Class	1 (plastic)
4.2.3	Shear area, Av = t.D =	1613 mm^2
4.2.3	Shear capacity, $P_v = 0.6p_yA_v$ =	266.11 kN
	Shear ratio F_v/P_v =	0.08
	The shear load is	Low
4.2.3	d/t < 70, hence no need to check for web shear buckling	
Section tables	Plastic section modulus, S =	483 cm^3
	Elastic section modulus, Z =	433 cm^3
4.2.5.2	For class 1 and 2 sections moment capacity, $M_c = p_yS$ =	132.83 kNm
4.2.5.1	Moment capacity limit for simply supported beam = $1.2p_yZ$ =	142.89 kNm
	Moment capacity is lesser of above values, hence M_c =	132.83 kNm
4.2.2	Section is not fully restrained against lateral torsional buckling between supports	
Table 13	Effective length, L_E =	1.2 L
	+	0.0 D
	Hence effective length, L_E =	5280 mm
	Radius of gyration, r_y =	3.48 cm
4.3.6.9	Ratio β_W =	1.0
	$\beta_W^{0.5}.L_E/r_y$ =	151.7
	D/T =	23.5
Table 20	Bending strength, p_b =	121 N/mm^2
4.3.7	Buckling resistance moment, $M_b = p_bS$ =	58.66 kNm

2 P S Design		PAGE:	21
PROJECT:	THE MALTHOUSE, BEAUWORTH	BY:	KP
ELEMENT:	SUPPORT POST TO BED 1 PURLINS	DATE:	08/09/20

Reference (to BS 5950: Part 1: 2000)				
	Column loads and geometry			
	Design bending moment, M_x =	**1.6**	**kNm**	
	Design bending moment, M_y =	**0.0**	**kNm**	
	Design shear force, F_{vx} =	**0.0**	**kN**	
	Design shear force, F_{vy} =	**0.0**	**kN**	
	Axial Force, F_c =	**53.2**	**kN**	
4.7.1.1	Segment length, L =	3.100	m	
	Steel grade selected =	S275		
	Section selection and classification			
	Section selected =	70 x 70 x 3.6 SHS		
Table 9	Steel strength, p_y =	275	N/mm^2	
Table 12 note b	$\varepsilon = (275/p_y)^{0.5}$ =	1.00		
Section tables	Local buckling ratio, d/t =	16.4		
Table 12	Since d/t < 56e section is class 1 (plastic)			
	Shear capacity			
	Cross section area, A_g =	9.4	cm^2	
Section tables	Depth of section, D =	70.0	mm	
	Width of section, B =	70.0	mm	
4.2.3 c)	Shear area, $A_v = AD/(D + B)$ =	471	mm^2	
4.2.3	Shear capacity, $P_v = 0.6p_y.A_v$ =	**77.7**	**kN**	OK
	Shear ratio for x-axis, F_{vx}/P_v =	0.00		
	Shear ratio for y-axis, F_{vy}/P_v =	0.00		
4.2.5.2	Since both shear ratios < 0.6, shear load is low in both directions			
	Cross-section capacity			
Section tables	Plastic section modulus, S =	23.3	cm^3	
Section tables	Elastic section modulus, Z =	19.6	cm^3	
4.2.5.2	For class 1 and 2 sections, $M_c = p_y S$ =	6.4	kNm	
4.2.5.1	Moment capacity general limit = $1.5p_y Z$ =	8.1	kNm	
	Hence $M_{cx} = M_{cy}$ =	**6.4**	**kNm**	OK
Table 22	Effective length, x-axis, L_{Ex} =	0.9	L	
Section tables	Radius of gyration, r_y =	2.70	cm	
	L_{Ex}/r_y =	97.6		
Table 24	Compressive strength, for x-axis fixity, p_{cx} =	162	N/mm^2	
4.7.4	Compressive resistance, for x-axis fixity, $P_{cx} = A_g.p_{cx}$ =	152.6	kN	
Table 22	Effective length, y-axis, L_{Ey} =	0.9	L	
Section tables	Radius of gyration, r_x =	2.70	cm	

	L_{Ey}/r_x =	97.6		
Table 24	Compressive strength, for y-axis fixity, p_{cy} =	162	N/mm^2	
4.7.4	Compressive resistance, for y-axis fixity, $P_{cy} = A_g.p_{cy}$ =	152.6	kN	
	Minimum compressive resistance, P_c =	152.6	kN	
	$A_g.p_y$ =	259.1	kN	
	$F_c/A_g.p_y$ =	0.21		
	M_x/M_{cx} =	0.25		
	M_y/M_{cy} =	0.00		
4.8.3.2	Cross section check, $F_c/A_g.p_y + M_x/M_{cx} + M_y/M_{cy}$ =	**0.46**		<1 so OK
	<u>Member buckling resistance</u>			
Table 26	Equivalent uniform moment factor, m_x =	0.95		
Table 26	Equivalent uniform moment factor, m_y =	0.95		
	F_c/P_c =	0.35		
	$m_x M_x/p_y Z$ =	0.28		
	$m_y M_y/p_y Z$ =	0.00		
4.8.3.3.1	Buckling check 1 - $F_c/P_c + m_x M_x/p_y Z + m_y M_y/p_y Z$ =	**0.63**		< 1 so OK
	F_c/P_{cy} =	0.35		
4.3.6.1	For SHS sections $M_b = M_c$ =	6.4	kNm	
Table 18	Equivalent uniform moment factor, m_{LT} =	0.93		
	Maximum major axis moment, M_{LT} =	1.6	kN	
	$m_{LT}M_{LT}/M_b$ =	0.23		
	$m_y M_y/p_y Z$ =	0.00		
4.8.3.3.1	Buckling check 2 - $F_c/P_{cy} + m_{LT}M_{LT}/M_b + m_y M_y/p_y Z$ =	**0.58**		<1 so OK

Load on new floor joists

Udl from floor

Load = 2.1 x 0.4 = 0.84 kN/m

Maximum span between centres of supports = 3.4 m

From page 24 use 50 x 150 C24 timbers at 400 mm centres

Load on beam over dining

Udl from new floor

Load = 3.3 x 6.0 x 0.5 = 9.9 kN/m

From beam self weight

Load at ULS = 1.4 x 0.5 = 0.7 kN/m

Total udl on beam = 9.9 + 0.7 = 10.6 kN/m

Design Moment and Shear Force

Maximum span between centres of supports = 6.5 m

Maximum bending moment, M = 10.6 x 6.5 x 6.5 / 8 = 56.0 kNm

Maximum shear force, Fv = 10.6 x 6.5 / 2 = 34.5 kN

From page 25 use 254 x 146 x 43 kg/m UB

Check deflection

Allowable = 6500 / 360 = 18 but limit to 14.0 mm

Actual = 5 x 10.6 x 6.5 x 6500^3 / 1.5 x 384 x 205 x 6540 x 10^4 = 12.3 mm

Therefore ok

2 P S Design		SHEET NO	24
PROJECT:	THE MALTHOUSE, BEAUWORTH	CALCS BY	KP
ELEMENT:	FLOOR JOISTS	DATE	08/09/2020

Reference (to BS5268 Part 2 1996)					
	Timber Type and Geometry				
	Span, L =	3.400	m		
	Section width, b =	50	mm		
	Section depth, d =	150	mm		
	Timber grade =	C24			
	Loading				
	UDL per joist, w =	0.84	kN/m		
	Point load, P =	0.00	kN		
	Point of action of P from LH end, a =	0.00	m		
	Moment, $M = wL^2/8 + Pa(L-a)/L$ =	1.21	kNm		
	Shear force, $F_v = wL/2 + P(L-a)/L$ =	1.43	kN		
	Modification factors				
	Load duration factor, K_3 =	1.00			
cl 2.10.6	Depth factor, $K_7 = (300/depth)^{0.11}$ =	1.08			
cl 2.9	Load sharing factor, K_8 =	1.10			
	Total modification factor =	1.19			
	Grade stresses			Modified stresses	
Table 7	Bending parallel to grain	7.50	N/mm^2	8.90	N/mm^2
Table 7	Shear parallel to grain	0.71	N/mm^2	0.84	N/mm^2
Table 7	Bearing perpendicular to grain	1.90	N/mm^2	2.26	N/mm^2
Table 7	$E_{(mean)}$	10800	N/mm^2		
	Check for Bending:				
	Section modulus, $Z = bd^2/6$ =	187500	mm^3		
	Bending stress = M/Z =	6.47	N/mm^2		
	Permissible stress =	8.90	N/mm^2	OK	
	Check for Deflection				
	Moment of Inertia, $I = bd^3/12$ =	14062500	mm^4		
	Area = bd =	7500	mm^2		
	UDL bending defl'n = $5wL^4/384EI$ =	9.62	mm		
	Point bending defl'n = $PL^3/48EI$ =	0.00	mm		
	UDL Shear defl'n = $12wL^2/5EA$ =	0.29	mm		
	Point shear defl'n = 24PL/5EA =	0.00	mm		
	Total deflection =	9.91	mm		
cl 2.10.7	Permissible deflection = 0.003 x span =	10.20	mm	OK	
	Check for Shear				
	Shear stress = $3F_v/2A$ =	0.29	N/mm^2		
	Permissible stress =	0.84	N/mm^2	OK	
	Check for Lateral Buckling				
Table 16	Max depth:breadth ratio =	5.00			
	Actual d/b =	3.00		OK	
	Check for Bearing				
	Bearing width, b_w =	100	mm		
	Bearing stress = $F_v/(b_w \times b)$ =	0.29	N/mm^2		
	Permissible stress =	2.26	N/mm^2	OK	
	Hence section selected is OK	**50 x 150**		**C24**	

Reference (to BS 5950: Part 1: 2000)		
	Design bending moment, M =	56.00 kNm
	Design shear force, F_v =	34.50 kN
	Span between lateral supports, L =	1.000 m
	Steel grade selected =	S275
	Section selected =	254 x 146 x 43 UB
Table 9	Steel strength, p_y =	275 N/mm^2
Table 11	$\varepsilon = (275/p_y)^{0.5}$ =	1.00
	Flange thickness, T =	12.7 mm
	Web thickness, t =	7.2 mm
Section tables	Overall depth, D =	259.6 mm
	b/T =	5.80
	d/t =	30.40
Table 11	Section is Class	1 (plastic)
4.2.3	Shear area, Av = t.D =	1869 mm^2
4.2.3	Shear capacity, $P_v = 0.6p_yA_v$ =	308.40 kN
	Shear ratio F_v/P_v =	0.11
	The shear load is	Low
4.2.3	d/t < 70, hence no need to check for web shear buckling	
Section tables	Plastic section modulus, S =	566 cm^3
	Elastic section modulus, Z =	504 cm^3
4.2.5.2	For class 1 and 2 sections moment capacity, $M_c = p_yS$ =	155.65 kNm
4.2.5.1	Moment capacity limit for simply supported beam = $1.2p_yZ$ =	166.32 kNm
	Moment capacity is lesser of above values, hence M_c =	155.65 kNm
4.2.2	Section is not fully restrained against lateral torsional buckling between supports	
Table 13	Effective length, L_E =	1.2 L
	+	0.0 D
	Hence effective length, L_E =	1200 mm
	Radius of gyration, r_y =	3.52 cm
4.3.6.9	Ratio β_W =	1.0
	$\beta_W^{0.5}.L_E/r_y$ =	34.1
	D/T =	20.4
Table 20	Bending strength, p_b =	275 N/mm^2
4.3.7	Buckling resistance moment, $M_b = p_bS$ =	155.65 kNm

Load on beam over wc

Point load from purlin posts

Load = 36.9 kN

Design Moment and Shear Force

Maximum span between centres of supports = 3.7 m

Maximum bending moment, M = 36.9 x 1.6 = 59.0 kNm

Maximum shear force, Fv = 36.9 kN

From page 27 use 254 x 146 x 37 kg/m UB

Reference (to BS 5950: Part 1: 2000)			
	Design bending moment, M =	59.00	kNm
	Design shear force, F_v =	36.90	kN
	Span between lateral supports, L =	3.700	m
	Steel grade selected =	S275	
	Section selected =	254 x 146 x 37 UB	
Table 9	Steel strength, p_y =	275	N/mm^2
Table 11	$\varepsilon = (275/p_y)^{0.5}$ =	1.00	
	Flange thickness, T =	10.9	mm
	Web thickness, t =	6.3	mm
Section tables	Overall depth, D =	256.0	mm
	b/T =	6.72	
	d/t =	34.80	
Table 11	Section is Class	1	(plastic)
4.2.3	Shear area, Av = t.D =	1613	mm^2
4.2.3	Shear capacity, $P_v = 0.6p_yA_v$ =	266.11	kN
	Shear ratio F_v/P_v =	0.14	
	The shear load is	Low	
4.2.3	d/t < 70, hence no need to check for web shear buckling		
Section tables	Plastic section modulus, S =	483	cm^3
	Elastic section modulus, Z =	433	cm^3
4.2.5.2	For class 1 and 2 sections moment capacity, $M_c = p_yS$ =	132.83	kNm
4.2.5.1	Moment capacity limit for simply supported beam = $1.2p_yZ$ =	142.89	kNm
	Moment capacity is lesser of above values, hence M_c =	132.83	kNm
4.2.2	Section is not fully restrained against lateral torsional buckling between supports		
Table 13	Effective length, L_E =	1.2	L
	+	0.0	D
	Hence effective length, L_E =	4440	mm
	Radius of gyration, r_y =	3.48	cm
4.3.6.9	Ratio β_W =	1.0	
	$\beta_W^{0.5}.L_E/r_y$ =	127.6	
	D/T =	23.5	
Table 20	Bending strength, p_b =	142	N/mm^2
4.3.7	Buckling resistance moment, $M_b = p_bS$ =	68.64	kNm

Beam End Bearing (Reference to BS 5628 Part 1)

The beams have their ends bearings on the internal and external leafs of the cavity walls.

The bearings on the walls are assumed to be Bearing Type 2 as shown at Figure 4 of BS 5628.

End reactions for beams generally

Maximum wall reactions = 53.2 kN

For an end bearing at the wall of 100 mm, and a wall width of 100 mm, the bearing stress under the beams = 53.2 x 1000 / 100 x 100 = 5.32 N/mm²

For a Type 2 bearing the permissible stress = $1.5f_k/\gamma_m$

$\gamma_m = 3.5$ *hence permissible stress =* $1.5/3.5\ f_k = 0.42\ f_k$

Hence wall strength required = 5.32 / 0.42 = 12.67 N/mm^2

Use a concrete padstone.

Bearing stress under the padstone = 53.2 x 1000 / 100 x 440 = 1.21 N/mm^2

Wall strength required under padstone = 1.21 / 0.42 = 2.88 N/mm^2

Hence, providing supporting walls are constructed with standard format solid concrete blocks or bricks (minimum $f_k = 3.0\ N/mm^2$*) the arrangement is OK.*

Use concrete padstones 100 x 440 x 215 deep