KCR DESIGN
PLANS TO INSPIRE

Structural Calculations
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August 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**

Beam: **Beam A** **Span: 2.7 m.**

		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*
U	D	o.w.	0.2	0		L	0.27	0.27
U	D	PITCHED ROOF	1.60*5.0	0		L	10.80	10.80
				Unfactored reactions (kN)		**Total:**	**11.07**	**11.07**
						Dead:	*11.07*	*11.07*
						Live:	*0.00*	*0.00*
		Total load: 22.14/31.00 kN	*Unfactored/Factored*		**Factored reactions:**		**15.50**	**15.50**

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

Maximum B.M. (factored) = 10.5 kNm at 1.35 m. from R1

Maximum S.F. (factored) = 15.5 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 = 5.67 x 10^8/EI at 1.35 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 (>=15.5) OK

Maximum moment = 10.46 kNm at 1.35 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_E) = 2.70m
Buckling parameter (u) = 0.889
β_w = 1.000 (Class 1/2 compact)
Bending strength, p_b = 152.2 N/mm^2

Slenderness, λ (L_E/r_y) = 2.70 x 100/2.10 = 128.6
Slenderness factor (v) = 0.751 (x = 19.6; λ/x = 6.56)
Equivalent slenderness (λ_{LT}) = u.v.λ.$\sqrt{\beta_w}$ = 85.79

Maximum moment within segment, M_x = 10.46 kNm
Equivalent uniform moment factor, m_{LT} = 0.925 (M_2=7.85, M_3=10.5, M_4=7.85)
Equivalent uniform moment = 0.925 x 10.46 = 9.677 kNm
Buckling resistance moment, $M_b = p_b.S_x$ = 152.2 x 123/1000 = 18.72 kNm OK

Check unstiffened web capacity with load of 15.50 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, $P_x = K\sqrt{(C4.P_w)}$
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 = 5.674 x 1e8/205,000 x 834 = 3.3 mm (L/814)

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: 225 x 100 mm bearing plate

Factored reaction = 11.07 x 1.4 + 0.00 x 1.6 = 15.50 kN

10 mm m.s. bearing plate, size 225 x 100 mm
Bearing plate projection beyond stiff bearing length = (225-32.1)/2 = 96.5mm
Factored stress under plate = 15.50 x 1000/225 x 100 = 0.69 N/mm^2
Required plate thickness = $\sqrt{(3x0.69x96.5x96.5/275)}$ = 8.36 mm: use 10mm
Factored bending stress in plate = 0.69x96.5x(96.5/2)/(10x10/6) = 192.3 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: 2.7 m.**

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U	D	o.w.	0.2	0		L	0.27	0.27
U	D	PITCHED ROOF	1.60*5.0	0		L	10.80	10.80
					Unfactored reactions (kN)	**Total:**	**11.07**	**11.07**
						Dead:	*11.07*	*11.07*
						Live:	*0.00*	*0.00*
						Factored reactions:	**15.50**	**15.50**

Total load: 22.14/31.00 kN *Unfactored/Factored*

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

Maximum B.M. (factored) = 10.5 kNm at 1.35 m. from R1

Maximum S.F. (factored) = 15.5 kN at R1

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

Total deflection = 5.67 x 10^8/EI at 1.35 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 (>=15.5) OK

Maximum moment = 10.46 kNm at 1.35 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) = 2.70m
Slenderness, λ (L_F/r_v) = 2.70 x 100/2.10 = 128.6
Buckling parameter (u) = 0.889
Slenderness factor (v) = 0.751 (x = 19.6; λ/x = 6.56)
β_w = 1.000 (Class 1/2 compact)
Equivalent slenderness (λ_{LT}) = u.v.λ.$\sqrt{\beta_w}$ = 85.79
Bending strength, p_b = 152.2 N/mm^2

Maximum moment within segment, M_x = 10.46 kNm
Equivalent uniform moment factor, m_{LT} = 0.925 (M_2=7.85, M_3=10.5, M_4=7.85)
Equivalent uniform moment = 0.925 x 10.46 = 9.677 kNm
Buckling resistance moment, M_b = $p_b.S_x$ = 152.2 x 123/1000 = 18.72 kNm OK

Check unstiffened web capacity with load of 15.50 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$\sqrt{(C4.P_w)}$
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 = 5.674 x 1e8/205,000 x 834 = 3.3 mm (L/814)

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: 225 x 100 mm bearing plate

Factored reaction = 11.07 x 1.4 + 0.00 x 1.6 = 15.50 kN

10 mm m.s. bearing plate, size 225 x 100 mm
Bearing plate projection beyond stiff bearing length = (225-32.1)/2 = 96.5mm
Factored stress under plate = 15.50 x 1000/225 x 100 = 0.69 N/mm^2
Required plate thickness = $\sqrt{(3x0.69x96.5x96.5/275)}$ = 8.36 mm: use 10mm
Factored bending stress in plate = 0.69x96.5x(96.5/2)/(10x10/6) = 192.3 N/mm^2 (p_y=275 N/mm^2)

R2 as R1

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

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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**

SuperBeam 4.57f 452185 **Noname.SBW** **Printed 4 Jan 2018 16:38**

Beam: **Beam A** **Span: 2.7 m.**

		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*
U	T	o.w.	0.15	0		L	0.20	0.20
U	T	PITCHED ROOF	1.60*0.7	0		L	1.51	1.51
							1.71	1.71

Total load: 3.43 kN

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

Maximum B.M. = 1.16 kNm at 1.35 m. from R1

Maximum S.F. = 1.71 kN at R1

Total deflection = 0.879 x 10^8/EI at 1.35 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 2 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 1.14 = 6,612 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, $\sigma_{m,a}$ = 1.16 x 1000/316.9 = 3.65 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 = 1.71 x 1000 x 3/2 x 50 x 195 = 0.26 N/mm^2 OK

Deflection

Bending deflection = 0.879 x 10^8/6,612 x 3,090 = 4.30 mm

Mid-span shear deflection = 1.2 x 1.16 x 10^6/((E/16) x 50 x 195) = 0.34 mm

Total deflection = 4.30 + 0.34 = 4.65 mm (0.0017 L) <=0.003L OK

Beam: **Beam B** **Span: 2.7 m.**

		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*
U	T	o.w.	0.2	0		L	0.27	0.27
U	T	FLAT ROOF	1.55*0.8	0		L	1.67	1.67
U	T	TIMBER STUD PARTITOIN	0.75*2.40	0		L	2.43	2.43
U	T	TIMBER FLOOR	2.00*0.4	0		L	1.08	1.08
							5.45	5.45

Total load: 10.91 kN

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

Maximum B.M. = 3.68 kNm at 1.35 m. from R1

Maximum S.F. = 5.45 kN at R1

Total deflection = 2.80 x 10^8/EI at 1.35 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 + 8 x 170 flitch plate *14.3 kg/m approx*

z = 316.9 cm^3 I = 3,090 cm^4 Flitch plate z = 38.5 cm^3 I = 328 cm^4

Timber grade: C16 2 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

Loading will be carried by the timber members and flitch plate in proportion to their EI values. Checks are made using the mean and minimum E-values for timber to produce worst case stresses on timber and steel members respectively. See TRADA guidance document GD9, 2008, for more information.

EI_{steel} = 205,000 x 328 x 10^4 = 671 x 10^9 Nmm^2

Calculate K_{8A} *(modified K8 as per TRADA GD9)*

Using E_{mean}, EI_{timber} = 8,800 x 3,090 x 10^4 = 272 x 10^9 Nmm^2

Timber carries 272/(272+671) = 0.288 of total load (in worst case)

K_{8A} = 1.04 (EI_{steel} >= $0.2EI_{total}$ and EI_{steel} <= $0.8EI_{total}$)

Calculate effect of bolt holes

M16 bolts, centres offset 0 mm from beam centre line: assume 17 mm holes
To allow for holes factor bending stresses by 1.0 (timber) and 1.0 (steel)

Bending

Permissible bending stress, $\sigma_{m,adm} = \sigma_{m,g}.K_3.K_7.K_{8A}$ = 5.3 x 1.00 x 1.049 x 1.04 = 5.78 N/mm^2

Applied bending stress, $\sigma_{m,a}$ = 0.288 x 3.68 x 1.000 x 1000/316.9 = 3.35 N/mm^2 OK

Shear

Permissible shear stress, τ_{adm} = 0.67 x 1.04 = 0.70 N/mm^2

Applied shear stress, τ_a = 0.288 x 5.454 x 1000 x 3/(2 x 50 x 195) = 0.24 N/mm^2 OK

Deflection:

Using E_{min} x K_9 (2 members) Timber EI = 5,800 x 1.14 x 3,090 x 10^4 = 204 x 10^9 Nmm^2

Timber carries 204/(204+671) = 0.233 of total load (average case)

Bending deflection = 0.233 x 2.80 x 10^8/(6,611 x 3,090) = 3.19 mm

Mid-span shear deflection = 0.233 x 1.2 x 3.68 x 10^6/(E/16) x 50 x 195 = 0.26 mm

Total deflection = 3.19 + 0.26 = 3.45 mm (0.0013 L) OK

Mid-span creep deflection:

Note that this calculation simplifies the Annex K calculation by taking all live loads as the leading live load rather than just the primary one if more than one

Service class 1 (dry) assumed: k_{def} = 0.6 ψ_2 = 0.3 (domestic) $Defl_{dead}$ = 0.70 $Defl_{live}$ = 2.10

Loads are assumed to be 25.0% dead; 75.0% live

$E_{fin} = E_{inst}$ x ($Defl_{dead}$ + $Defl_{live}$)/($Defl_{dead}$ (1+ k_{def}) + $Defl_{live}$ (1 + $\psi_2.k_{def}$)) = E_{inst} x 0.778
$E_{min,fin}$ = 5,800 x 1.14 x 0.778 = 5,146 N/mm^2
Timber $E_{min,fin}$ I = 5,146 x 3,090 x 10^4 = 159 x 10^9 Nmm^2

Long term/instantaneous deflection = (204 + 671) / (159 + 671) = 1.05

Final deflection = 3.45 x 1.05 = 3.64 mm (0.0012 L) OK

Check flitch plate:

Using $E_{min\ fin}$ for timber, flitch plate carries 671/(159 + 671) = 0.809 of total load

Per TRADA GD9 factor load by 1.10 to allow for slip and shear deflection in plate

Flitch plate f_{bc} = 0.809 x 3.68 x 1.10 x 1.000 x 1000/38.5 = 85.0 N/mm^2 OK

Bolting:

Use M16 4.6 bolts. Bolt numbers are calculated assuming worst case load on flitch plate

Load capacity per bolt in double shear = 3.69kN (BS5268 eq. G.7 - limiting value)

(G.7: 3.69kN; G.8: 32.0kN; G.9: 7.40kN; G.10: 10.2kN)

F_d=1350; $M_{y,d}$=196,608Nmm; p_k=310kg/m^3; K_{90}=1.59; $f_{h,0,d}$=9.895; $f_{h,1,d}$=6.223; Β and K_a taken as 1.0

Bearings: R1 (5.45kN): Required number of bolts = 0.789 x 5.45/3.69 = 1.17 i.e. 2 bolts min.
R2 (5.45kN): Required number of bolts = 0.789 x 5.45/3.69 = 1.17 i.e. 2 bolts min.

For load transference a minimum of 3 bolts are also required across the span

Recommended bolting pattern across span: Bolts at max 487 mm max c/s, alternately set 0mm above and below centre line of beam with an additional centred bolt at each significant point load position.

Beam: **Beam C** **Span: 2.7 m.**

		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*
U	T	o.w.	0.2	0		L	0.27	0.27
U	T	PITCHED ROOF	1.60*1.00	0		L	2.16	2.16
U	T	FLAT ROOF	1.55*0.8	0		L	1.67	1.67
U	T	ROOF SPACE	0.55*1.0	0		L	0.74	0.74
							4.85	4.85

Total load: 9.69 kN

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

Maximum B.M. = 3.27 kNm at 1.35 m. from R1

Maximum S.F. = 4.85 kN at R1

Total deflection = 2.48 x 10^8/EI at 1.35 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 147 C16 + 10 x 122 flitch plate *12.3 kg/m approx*

z = 180.1 cm^3 I = 1,324 cm^4 Flitch plate z = 24.8 cm^3 I = 151 cm^4

Timber grade: C16 2 members acting together: K_8 = 1.1

K_3 (loading duration factor) = 1.00 K_7 (depth factor) = 1.082 K_8 (load sharing factor) = 1.1

Loading will be carried by the timber members and flitch plate in proportion to their EI values. Checks are made using the mean and minimum E-values for timber to produce worst case stresses on timber and steel members respectively. See TRADA guidance document GD9, 2008, for more information.

EI_{steel} = 205,000 x 151 x 10^4 = 310 x 10^9 Nmm^2

Calculate K_{8A} *(modified K8 as per TRADA GD9)*

Using E_{mean}, EI_{timber} = 8,800 x 1,324 x 10^4 = 116 x 10^9 Nmm^2

Timber carries 116/(116+310) = 0.273 of total load (in worst case)

K_{8A} = 1.04 (EI_{steel} >= $0.2EI_{total}$ and EI_{steel} <= $0.8EI_{total}$)

Calculate effect of bolt holes

M16 bolts, centres offset 0 mm from beam centre line: assume 17 mm holes
To allow for holes factor bending stresses by 1.0 (timber) and 1.0 (steel)

Bending

Permissible bending stress, $\sigma_{m,adm} = \sigma_{m,g}.K_3.K_7.K_{8A}$ = 5.3 x 1.00 x 1.082 x 1.04 = 5.96 N/mm^2

Applied bending stress, $\sigma_{m,a}$ = 0.273 x 3.27 x 1.000 x 1000/180.1 = 4.96 N/mm^2 OK

Shear

Permissible shear stress, τ_{adm} = 0.67 x 1.04 = 0.70 N/mm^2

Applied shear stress, τ_a = 0.273 x 4.846 x 1000 x 3/(2 x 50 x 147) = 0.27 N/mm^2 OK

Deflection:

Using E_{min} x K_9 (2 members) Timber EI = 5,800 x 1.14 x 1,324 x 10^4 = 87 x 10^9 Nmm^2

Timber carries 87/(87+310) = 0.220 of total load (average case)

Bending deflection = 0.220 x 2.48 x 10^8/(6,611 x 1,324) = 6.25 mm

Mid-span shear deflection = 0.220 x 1.2 x 3.27 x 10^6/(E/16) x 50 x 147 = 0.28 mm

Total deflection = 6.25 + 0.28 = 6.53 mm (0.0024 L) OK

Mid-span creep deflection:

Note that this calculation simplifies the Annex K calculation by taking all live loads as the leading live load rather than just the primary one if more than one

Service class 1 (dry) assumed: k_{def} = 0.6 ψ_2 = 0.3 (domestic) $Defl_{dead}$ = 0.62 $Defl_{live}$ = 1.86

Loads are assumed to be 25.0% dead; 75.0% live

$E_{fin} = E_{inst}$ x ($Defl_{dead}$ + $Defl_{live}$)/($Defl_{dead}$ (1+ k_{def}) + $Defl_{live}$ (1 + $\psi_2.k_{def}$)) = E_{inst} x 0.778

$E_{min,fin}$ = 5,800 x 1.14 x 0.778 = 5,146 N/mm^2

Timber $E_{min,fin}$ I = 5,146 x 1,324 x 10^4 = 68 x 10^9 Nmm^2

Long term/instantaneous deflection = (87 + 310) / (68 + 310) = 1.05

Final deflection = 6.53 x 1.05 = 6.87 mm (0.0023 L) OK

Check flitch plate:

Using $E_{min\ fin}$ for timber, flitch plate carries 310/(68 + 310) = 0.820 of total load

Per TRADA GD9 factor load by 1.10 to allow for slip and shear deflection in plate

Flitch plate f_{bc} = 0.820 x 3.27 x 1.10 x 1.000 x 1000/24.8 = 118.9 N/mm^2 OK

Bolting:

Use M16 4.6 bolts. Bolt numbers are calculated assuming worst case load on flitch plate

Load capacity per bolt in double shear = 3.69kN (BS5268 eq. G.7 - limiting value)

(G.7: 3.69kN; G.8: 40.0kN; G.9: 7.40kN; G.10: 10.2kN)

F_d=1350; $M_{y,d}$=196,608Nmm; p_k=310kg/m^3; K_{90}=1.59; $f_{h,0,d}$=9.895; $f_{h,1,d}$=6.223; B and K_a taken as 1.0

Bearings: R1 (4.85kN): Required number of bolts = 0.802 x 4.85/3.69 = 1.05 i.e. 2 bolts min.
R2 (4.85kN): Required number of bolts = 0.802 x 4.85/3.69 = 1.05 i.e. 2 bolts min.

For load transference a minimum of 3 bolts are also required across the span

Recommended bolting pattern across span: Bolts at max 367 mm max c/s, alternately set 0mm above and below centre line of beam with an additional centred bolt at each significant point load position.

Beam: **Ridge beam** **Span: 2.7 m.**

		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*
U	T	o.w.	0.15	0		L	0.20	0.20
U	T	PITCHED ROOF	1.60*1.00	0		L	2.16	2.16
U	T	PITCHED ROOF	1.60*1.00	0		L	2.16	2.16
							4.52	4.52

Total load: 9.05 kN

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

Maximum B.M. = 3.05 kNm at 1.35 m. from R1

Maximum S.F. = 4.52 kN at R1

Total deflection = 2.32 x 10^8/EI at 1.35 m. from R1 *(E in N/mm², I in cm⁴)*

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

Use 50 x 195 C16 + 8 x 170 flitch plate *14.3 kg/m approx*

z = 316.9 cm^3 I = 3,090 cm^4 Flitch plate z = 38.5 cm^3 I = 328 cm^4

Timber grade: C16 2 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

Loading will be carried by the timber members and flitch plate in proportion to their EI values. Checks are made using the mean and minimum E-values for timber to produce worst case stresses on timber and steel members respectively. See TRADA guidance document GD9, 2008, for more information.

EI_{steel} = 205,000 x 328 x 10^4 = 671 x 10^9 Nmm^2

Calculate K_{8A} *(modified K8 as per TRADA GD9)*

Using E_{mean}, EI_{timber} = 8,800 x 3,090 x 10^4 = 272 x 10^9 Nmm^2

Timber carries 272/(272+671) = 0.288 of total load (in worst case)

K_{8A} = 1.04 (EI_{steel} >= $0.2EI_{total}$ and EI_{steel} <= $0.8EI_{total}$)

Calculate effect of bolt holes

M16 bolts, centres offset 0 mm from beam centre line: assume 17 mm holes
To allow for holes factor bending stresses by 1.0 (timber) and 1.0 (steel)

Bending

Permissible bending stress, $\sigma_{m,adm} = \sigma_{m,g}.K_3.K_7.K_{8A}$ = 5.3 x 1.00 x 1.049 x 1.04 = 5.78 N/mm^2

Applied bending stress, $\sigma_{m,a}$ = 0.288 x 3.05 x 1.000 x 1000/316.9 = 2.78 N/mm^2 OK

Shear

Permissible shear stress, τ_{adm} = 0.67 x 1.04 = 0.70 N/mm^2

Applied shear stress, τ_a = 0.288 x 4.523 x 1000 x 3/(2 x 50 x 195) = 0.20 N/mm^2 OK

Deflection:

Using E_{min} x K_9 (2 members) Timber EI = 5,800 x 1.14 x 3,090 x 10^4 = 204 x 10^9 Nmm^2

Timber carries 204/(204+671) = 0.233 of total load (average case)

Bending deflection = 0.233 x 2.32 x 10^8/(6,611 x 3,090) = 2.65 mm

Mid-span shear deflection = 0.233 x 1.2 x 3.05 x 10^6/(E/16) x 50 x 195 = 0.21 mm

Total deflection = 2.65 + 0.21 = 2.86 mm (0.0011 L) OK

Mid-span creep deflection:

Note that this calculation simplifies the Annex K calculation by taking all live loads as the leading live load rather than just the primary one if more than one

Service class 1 (dry) assumed: k_{def} = 0.6 ψ_2 = 0.3 (domestic) $Defl_{dead}$ = 0.58 $Defl_{live}$ = 1.74

Loads are assumed to be 25.0% dead; 75.0% live

$E_{fin} = E_{inst}$ x ($Defl_{dead}$ + $Defl_{live}$)/($Defl_{dead}$ (1+ k_{def}) + $Defl_{live}$ (1 + $\psi_2.k_{def}$)) = E_{inst} x 0.778
$E_{min,fin}$ = 5,800 x 1.14 x 0.778 = 5,146 N/mm^2
Timber $E_{min,fin}$ I = 5,146 x 3,090 x 10^4 = 159 x 10^9 Nmm^2

Long term/instantaneous deflection = (204 + 671) / (159 + 671) = 1.05

Final deflection = 2.86 x 1.05 = 3.02 mm (0.0010 L) OK

Check flitch plate:

Using $E_{min\ fin}$ for timber, flitch plate carries 671/(159 + 671) = 0.809 of total load

Per TRADA GD9 factor load by 1.10 to allow for slip and shear deflection in plate

Flitch plate f_{bc} = 0.809 x 3.05 x 1.10 x 1.000 x 1000/38.5 = 70.5 N/mm^2 OK

Bolting:

Use M16 4.6 bolts. Bolt numbers are calculated assuming worst case load on flitch plate

Load capacity per bolt in double shear = 3.69kN (BS5268 eq. G.7 - limiting value)

(G.7: 3.69kN; G.8: 32.0kN; G.9: 7.40kN; G.10: 10.2kN)

F_d=1350; $M_{y,d}$=196,608Nmm; p_k=310kg/m^3; K_{90}=1.59; $f_{h,0,d}$=9.895; $f_{h,1,d}$=6.223; B and K_a taken as 1.0

Bearings: R1 (4.52kN): Required number of bolts = 0.789 x 4.52/3.69 = 0.97 i.e. 1 bolt min.
R2 (4.52kN): Required number of bolts = 0.789 x 4.52/3.69 = 0.97 i.e. 1 bolt min.

For load transference a minimum of 2 bolts are also required across the span

Recommended bolting pattern across span: Bolts at max 487 mm max c/s, alternately set 0mm above and below centre line of beam with an additional centred bolt at each significant point load position.