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	Www	w.BEAM-DE	SIGNS.co.uk	Job re Sheet Made Date	f : BD110624 : Structure / 2 - By : Beam-Designs - SR : June 2011 /	
					:	
.oad to Be	Email er	nquiries@beam-designs.co.	uk			
out to be	am 1					
ounge 2.5	5/2*0.9=1.13	3 2.5/2*(0.5=0.63			
lat 0.7	7/2*0.6=0.2	1 0.7/2*0	0.75=0.26			
001						
otal 1.3	34 kN/m Do	ead 0.89 k	xN/m Live			
oad to Be	eam 2					
oint loads	from Beam	1				
			Bea	m 1		
7 C:\USERS	5\ADMINISTR/	ATOR\DESKTOP\TEE	BEAM NO1.\$5			
		Αχι	al with Mom	IENTS (M EMB	ER)	
			Bear	m 1		
	Ν	/lember 8 (l	N.3-N.7) @	Level 1 in L	oad Case 1	
lember	r Loadin	g and Membe	er Forces			
Loading	g Combinat:	ion : 1 UT + 1.3!	5 D1 + 1.5 L1			
D1 UDLY	Y -001.340 Y -000.890	(ki (ki	N/m) N/m)		_	
-						
						_
		Member Forces in	Load Case 1 and Ma	ximum Deflection 1	rom Load Case 3	
Mem	Node End1	Member Forces in Torque Moment	Load Case 1 and Ma Shear	aximum Deflection t Bending Moment	rom Load Case 3 Maximum	Maximum
Mem ber No.	Node End1 End2	Member Forces in Torque Moment (kN.m)	Load Case 1 and Ma Shear Force (kN)	aximum Deflection f Bending Moment (kN.m)	rom Load Case 3 Maximum Moment (kN.m @ m)	Maximum Deflection (mm @ m)
Mem ber No. 8	Node End1 End2 3 7	Member Forces in Torque Moment (kN.m) 0.000 0.000	Load Case 1 and Ma Shear Force (kN) 10.203	Eximum Deflection t Bending Moment (kN.m) 0.000	rom Load Case 3 Maximum Moment (kN.m @ m) 15.305 @ 3.000	Maximum Deflection (mm @ m) 14.33 @ 3.00
Mem ber No. 8	Node End1 End2 3 7	Member Forces in Torque Moment (kN.m) 0.000 0.000	Load Case 1 and Ma Shear Force (kN) 10.203 -10.203	aximum Deflection f Bending Moment (kN.m) 0.000 0.000	Trom Load Case 3 Maximum Moment (kN.m @ m) 15.305 @ 3.000	Maximum Deflection (mm @ m) 14.33 @ 3.00
Mem ber No. 8	Node End1 End2 3 7	Member Forces in Torque Moment (kN.m) 0.000 0.000	Load Case 1 and Ma Shear Force (kN) 10.203 -10.203 Area (EN 1993	Eximum Deflection f Bending Moment (kN.m) 0.000 0.000 0.000	Trom Load Case 3 Maximum Moment (kN.m @ m) 15.305 @ 3.000	Maximum Deflection (mm @ m) 14.33 @ 3.00
Mem ber No. 8 Class = Fn(Class = Fn(Node End1 End2 3 7 Cation ar	Member Forces in Torque Moment (kN.m) 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000	Load Case 1 and Ma Shear Force (kN) 10.203 -10.203 Area (EN 1993 30.58, 275, 0, 15.3, 0	Aximum Deflection f Bending Moment (kN.m) 0.000 0.000 3: 2006)	Tom Load Case 3 Maximum Moment (kN.m @ m) 15.305 @ 3.000 (Axial: Non-Slender)	Maximum Deflection (mm @ m) 14.33 @ 3.00 Class 1
Mem ber No. 8 Classific Class = Fn(Auto Design	Node End1 End2 3 7 Cation ar b/T,d/t,f _y ,N,M h Load Cases t Capaci	Member Forces in Torque Moment (kN.m) 0.000	Load Case 1 and Ma Shear Force (KN) 10.203 -10.203 -10.203 Area (EN 1993 30.58, 275, 0, 15.3, 0	aximum Deflection f Bending Moment (kN.m) 0.000 0.000 3: 2006)	Trom Load Case 3 Maximum Moment (kN.m @ m) 15.305 @ 3.000 (Axial: Non-Slender)	Maximum Deflection (mm @ m) 14.33 @ 3.00 Class 1
Mem ber No. 8 Classific Class = Fn(Auto Desigr Moment	Node End1 End2 3 7 Cation ar b/T,d/t,f _y ,N,M Load Cases t Capaci	Member Forces in Torque Moment (kN.m) 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 1 ty Check M.c. 0.000	Load Case 1 and Ma Shear Force (kN) 10.203 -10.203 -10.203 30.58, 275, 0, 15.3, 0 .y.Rd / 156.396 =	Aximum Deflection f Bending Moment (kN.m) 0.000 0.000 3: 2006)	rom Load Case 3 Maximum Moment (kN.m @ m) 15.305 @ 3.000 (Axial: Non-Slender) 0	Maximum Deflection (mm @ m) 14.33 @ 3.00 Class 1 Low Shear
Mem ber No. 8 Classific Class = Fn(Auto Desigr Moment Vy.Ed/Vpl.y.Rd Mc.y.Rd = fy.V My.Fd/Mc.y.Rd	Node End1 End2 3 7 Cation ar b/T,d/t,f _y ,N,M h Load Cases t Capaci	Member Forces in Torque Moment (kN.m) 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.001 1 ty Check M.c. 0.002 275 x 15.295	Load Case 1 and Ma Shear Force (KN) 10.203 -10.203 -10.203 Area (EN 1993 30.58, 275, 0, 15.3, 0 .y.Rd / 156.396 = 171.3/1 5 / 47.108 =	aximum Deflection f Bending Moment (kN.m) 0.000 0.000 3: 2006)	Trom Load Case 3 Maximum Moment (kN.m @ m) 15.305 @ 3.000 (Axial: Non-Slender) 0 47.108 kN.m 0.325	Maximum Deflection (mm @ m) 14.33 @ 3.00 Class 1 Low Shear OK
Mem ber No. 8 Classific Class = Fn(Auto Desigr Moment Vy.Ed/Vpl.y.Rd Mc.y.Rd = fy.V My.Ed/Mc.y.Rd	Node End1 End2 3 7 Cation ar b/T,d/t,f _y ,N,M h Load Cases t Capaci N _{pLy} / γ _{M0} ent Unife	Member Forces in Torque Moment (kN.m) 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 1 ty Check M.c. 0.002 275 x 15.295 orm Moment	Load Case 1 and Ma Shear Force (kN) 10.203 -10.203 -10.203 Area (EN 1993 30.58, 275, 0, 15.3, 0 .y.Rd / 156.396 = 171.3/1 5 / 47.108 = Factor C1	Aximum Deflection f Bending Moment (kN.m) 0.000 0.000 3: 2006)	rom Load Case 3 Maximum Moment (kN.m @ m) 15.305 @ 3.000 (Axial: Non-Slender) 0 47.108 kN.m 0.325	Maximum Deflection (mm @ m) 14.33 @ 3.00 Class 1 Low Shear OK
Mem ber No. 8 Classific Class = Fn(Auto Design Moment Vy.Ed/Vpl.y.Rd My.Ed/Vpl.y.Rd My.Ed/Mc.y.Rd Gquivale C1= fn(M1, 1)	Node End1 End2 3 7 Cation ar b/T,d/t,fy,N,M h Load Cases t Capaci N _{pLy} / γ _{M0} ent Unife M ₂ , M ₀ , ψ,μ)	Member Forces in Torque Moment (kN.m) 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 1 ty Check M.c. 0.002 275 x 15.295 orm Moment 0.0, 0.	Load Case 1 and Ma Shear Force (KN) 10.203 -10.203 -10.203 Area (EN 1993 30.58, 275, 0, 15.3, 0 .y.Rd / 156.396 = 171.3/1 5 / 47.108 = Factor C1 0, 15.3, 0.800, 300.000	aximum Deflection f Bending Moment (kN.m) 0.000 0.000 3: 2006)	Trom Load Case 3 Maximum Moment (kN.m @ m) 15.305 @ 3.000 (Axial: Non-Slender) 0 47.108 kN.m 0.325 1.127	Maximum Deflection (mm@m) 14.33 @ 3.00 Class 1 Low Shear OK Uniform
Mem ber No. 8 Classific Class = Fn(Auto Design Moment Vy.Ed/Vpl.y.Rd My.Ed/Mc.y.Rd Cquivale C1 = fn(M1, I)	Node End1 End2 3 7 Cation ar b/T,d/t,fy,N,M h Load Cases t Capaci NpLy/ ΥM0 ent Unife M2, Mo, Ψ/μ) Buckling	Member Forces in Torque Moment (kN.m) 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 1 ty Check M.c. 0.002 275 x 15.295 orm Moment 0.0, 0. g Check M.b.F	Load Case 1 and Ma Shear Force (kN) 10.203 -10.203 -10.203 Area (EN 1993 30.58, 275, 0, 15.3, 0 .y.Rd / 156.396 = 171.3/1 5 / 47.108 = Factor C1 0, 15.3, 0.800, 300.000 Rd	aximum Deflection f Bending Moment (kN.m) 0.000 0.000 3: 2006)	rom Load Case 3 Maximum Moment (kN.m@m) 15.305 @ 3.000 (Axial: Non-Slender) 0 47.108 kN.m 0.325 1.127	Maximum Deflection (mm @ m) 14.33 @ 3.00 Class 1 Class 1 Low Shear OK Uniform
Mem ber No. 8 Classific Class = Fn(Auto Design Moment Vy,Ed/VpLy,Rd Mo,ed = fy.V My,Ed/Mc.y,Rd Cquivale C1= fn(M1, I .ateral Le = 1.00 L Marging = Fn(C1, r)	Node End1 End2 3 7 Cation ar b/T,d/t,fy,N,M h Load Cases t Capaci N _{pLy} / γM0 ent Unife M ₂ , M _o , ψ,μ) Buckling	Member Forces in Torque Moment (kN.m) 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 1 ty Check M.c. 0.002 275 x 15.295 orm Moment 0.0, 0. g Check M.b.F 1 x 6 = 1.127,	Load Case 1 and Ma Shear Force (KN) 10.203 -10.203 -10.203 Area (EN 1993 30.58, 275, 0, 15.3, 0 .y.Rd / 156.396 = 171.3/1 5 / 47.108 = Factor C1 0, 15.3, 0.800, 300.000 Rd = 6.000, 137.6, 4.408, 0	Aximum Deflection f Bending Moment 0.000 0.000 0.000 3: 2006) 0 0 0.009848, 210000	Trom Load Case 3 Maximum Moment (kN.m @ m) 15.305 @ 3.000 (Axial: Non-Slender) 0 47.108 kN.m 0.325 1.127 6 m 20.379 kN.m	Maximum Deflection (mm@m) 14.33 @ 3.00 Class 1 Low Shear OK Uniform
$\begin{tabular}{c}{llllllllllllllllllllllllllllllllll$	Node End1 End2 3 7 cation ar b/T,d/t,fy,N,M h Load Cases t Capaci MpLy/ YM0 ent Unife Le, I_z, It, Iw, E) /Mcr Arrence	Member Forces in Torque Moment (kN.m) 0.000 0.000 nd Effective A hy,M₂) 6.41, 3 1 ty Check M.c. 0.002 275 x 15.295 orm Moment 0.0, 0. g Check M.b.F 1 x 6 = 1.127, √ 171. 1 520	Load Case 1 and Ma Shear Force (kN) 10.203 -10.203 -10.203 Area (EN 1993 30.58, 275, 0, 15.3, 0 .y.Rd / 156.396 = 171.3/1 5/ 47.108 = Factor C1 0, 15.3, 0.800, 300.000 Rd = 6.000, 137.6, 4.408, 0 3 x 275 / 20.379 1.575	aximum Deflection f Bending Moment (kN.m) 0.000 0.000 3: 2006) 0 0.009848, 210000	rom Load Case 3 Maximum Moment (kN.m @ m) 15.305 @ 3.000 (Axial: Non-Slender) 0 47.108 kN.m 0.325 1.127 6 m 20.379 kN.m 1.520 0 41	Maximum Deflection (mm @ m) 14.33 @ 3.00 Class 1 Low Shear OK Uniform
$\begin{tabular}{lllllllllllllllllllllllllllllllllll$	Node End1 End2 3 7 Cation ar (b/T,d/t,fy,N,M) n Load Cases t Capaci NpLy/ YM0 ent Unife M2, Mo, ψ,μ) Buckling	Member Forces in Torque Moment (kN.m) 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 1 ty Check M.c. 0.002 275 x 15.295 orm Moment 0.0, 0. g Check M.b.F 1 x 6 = 1.127, √ 171. 1.520, 0.419,	Load Case 1 and Ma Shear Force (kN) 10.203 -10.203 -10.203 Area (EN 1993 30.58, 275, 0, 15.3, 0 .y.Rd / 156.396 = 171.3/1 5 / 47.108 = Factor C1 0, 15.3, 0.800, 300.000 Rd = 6.000, 137.6, 4.408, 0 3 × 275 / 20.379 1.575 1.520, 0.942, 1.000	aximum Deflection f Bending Moment (kN.m) 0.000 0.000 3: 2006) 0 0.009848, 210000	Trom Load Case 3 Maximum Moment (kN.m@m) 15.305 @ 3.000 (Axial: Non-Slender) (Axial: Non-Slender) 0 47.108 kN.m 0.325 1.127 6 m 20.379 kN.m 1.520 0.419 0.419	Maximum Deflection (mm @ m) 14.33 @ 3.00 Class 1 Low Shear OK Uniform Curve b 6.3.2.3
$\label{eq:constraints} \begin{array}{c} \mbox{Mem} \\ \mbox{ber} \\ \mbox{No.} \end{array} \\ \label{eq:constraints} \\ \mbox{Rel} \\ $	Node End1 End2 3 7 Cation ar b/T,d/t,f,,N,M h Load Cases t Capaci $N_{pL,y}/\gamma_{M0}$ ent Unife M_2, M_0, ψ, μ) Buckling - Le,Iz,It,Iw,E) /Mcr , λ_{LT950}) En($\chi_{LT},\lambda_{LT},k_c,f$) /pL,y,fy \leq Mc.y,Rd	Member Forces in Torque Moment (kN.m) 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 1 ty Check M.c. 0.002 275 x 15.295 orm Moment 0.0, 0. 0.0 Check M.b.F 1 x 6 = 1.127, √ 171. 1.520, 0.419, 0.419 0.419 15 296	Load Case 1 and Ma Shear Force (KN) 10.203 -10.203 -10.203 Area (EN 1993 30.58, 275, 0, 15.3, 0 Area (EN 1993 30.58, 275, 0, 15.3, 0 y.Rd / 156.396 = 171.3/1 5 / 47.108 = Factor C1 0, 15.3, 0.800, 300.000 Rd = 6.000, 137.6, 4.408, 0 3 x 275 / 20.379 1.575 1.520, 0.942, 1.000 x 171.3 x 275 \leq 47.108 5 / 19.72	Aximum Deflection f Bending Moment (kN.m) 0.000 0.000 3: 2006) 0 0.009848, 210000 8 =	rom Load Case 3 Maximum Moment (kN.m@m)) 15.305 @ 3.000 (Axial: Non-Slender) (Axial: Non-Slender) 0 47.108 kN.m 0.325 1.127 6 m 20.379 kN.m 1.520 0.419 0.419 19.720 kN.m 0.776	Maximum Deflection (mm @ m) 14.33 @ 3.00 Class 1 Low Shear OK Uniform Curve b 6.3.2.3
$\begin{tabular}{lllllllllllllllllllllllllllllllllll$	Node End1 End2 3 7 Cation ar b/T,d/t,f _y ,N,M h Load Cases t Capaci $N_{pL,y}/ \gamma_{M0}$ ent Unife M ₂ , M ₀ , ψ,μ) Buckling - L _e ,I ₂ ,I _t ,I _w ,E) /M _c , λ _L T5950) Fn(χ_{LT} , λ _L T, k _c ,f) /p _L , f _y ≤ M _{c,y,Rd}	Member Forces in Torque Moment (kN.m) 0.000 0.000 nd Effective A hy,M₂) 6.41, 3 1 ty Check M.c. 0.002, 275 x orm Moment 0.002, 275 x orm Moment 0.0, 0. g Check M.b.F 1.127, √ 171. 1.520, 0.419, 0.419 0.419, 0.419 0.419, 0.419 0.419, 0.419 0.419, 0.419	Load Case 1 and Ma Shear Force (KN) 10.203 -10.203 -10.203 Area (EN 1993 30.58, 275, 0, 15.3, 0 Area (EN 1993 -10.203	Aximum Deflection f Bending Moment (kN.m) 0.000 0.000 3: 2006) 0 0.009848, 210000 8 =	Trom Load Case 3 Maximum Moment (kN.m@m) 15.305 @ 3.000 (Axial: Non-Slender) (Axial: Non-Slender) 0 47.108 kN.m 0.325 1.127 6 m 20.379 kN.m 1.520 0.419 0.419 0.419 0.776	Maximum Deflection (mm @ m) 14.33 @ 3.00 Class 1 Low Shear OK Uniform Curve b 6.3.2.3 OK
$\begin{tabular}{lllllllllllllllllllllllllllllllllll$	Node End1 End2 3 7 Cation ar b/T,d/t,f,,N,M n Load Cases t Capaci $N_{pL,y}/\gamma_{M0}$ ent Unife M_2, M_0, ψ, μ) Buckling -Le,Iz,It,Iw,E) /Mcr , λ_{LTS950}) En($\chi_{LT}, \lambda_{LT}, k_c, f$) /pLy, $f_y \leq M_{c.y,Rd}$ on Chec 60	Member Forces in Noment (kN.m) 0.000 0.000 0.000 0.000 0.000 0.000 nd Effective A 1 ty Check M.c. 0.002 275 x 15.295 orm Moment 0.0, 0. g Check M.b.F 1 x 6 = 1.127, √ 171. 1.5296 ck - Load Case 14.33	Load Case 1 and Ma Shear Force (KN) 10.203 -10.203 -10.203 Area (EN 1993 30.58, 275, 0, 15.3, 0 Area (EN 1993 30.58, 275, 0, 15.3, 0	Aximum Deflection f Bending Moment (kN.m) 0.000 0.000 3: 2006) 0 0.009848, 210000 8 =	rom Load Case 3 Maximum Moment (kN.m@m)) 15.305 @ 3.000 (Axial: Non-Slender) (Axial: Non-Slender) 0 47.108 kN.m 0.325 1.127 6 m 20.379 kN.m 1.520 0.419 19.720 kN.m 0.776 14.33 mm	Maximum Deflection (mm@m) 14.33 @ 3.00 Class 1 Low Shear OK Uniform Curve b 6.3.2.3 OK
$\label{eq:spin} \begin{matrix} \text{Mem} \\ \text{ber} \\ \text{No.} \end{matrix}$	Node End1 End2 3 7 Cation ar b/T,d/t,fy,N,M h Load Cases t Capaci N_{pLy}/γ_{M0} ent Unife Le,rIz,It,Iw,E) /Mcr , ALT5950) Fn($\chi_{LT}, \lambda_{LT}, k_c, f)$ /pl.y.fy ≤ Mc.y.Rd ion Checc 60	Member Forces in Torque Moment (kN.m) 0.000 0.000 nd Effective A hy,M₂) 6.41, 3 1 ty Check M.c. 0.002, 275 x 1 ty Check M.c. 0.002, 275 x 0.002, 275 x 1 ty Check M.c. 0.002, 275 x 1 1 0.002, 275 x 1 0.002, 275 x 15.296 th - Load Case 14.33	Load Case 1 and Ma Shear Force (KN) 10.203 -10.203 -10.203 Area (EN 1993 30.58, 275, 0, 15.3, 0 Area (EN 1993 -10.203	Aximum Deflection f Bending Moment (kN.m) 0.000 0.000 3: 2006) 3: 2006) 0 0.009848, 210000 8 =	Tom Load Case 3 Maximum Moment (kN.m@m) 15.305 @ 3.000 (Axial: Non-Slender) (Axial: Non-Slender) 0 47.108 kN.m 0.325 1.127 6 m 20.379 kN.m 1.520 0.419 0.419 19.720 kN.m 0.776 14.33 mm	Maximum Deflection (mm @ m) 14.33 @ 3.00 Class 1 Low Shear OK Uniform Curve b 6.3.2.3 OK OK

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Maximum shear	Emax = 10.850 kN	Emp = -10 849 kN
Design shear	$F = max(abs(F_{max}) abs(F$	F===)) = 10 850 kN
Total load on beam	W _m = 21.699 kN	
Reactions at support A	R = 10.850 kN	R = 10.850 kN
Unfactored permanent load reaction at support A	RA Bermanent = 2.670 kN	
Unfactored variable load reaction at support A	$R_{A,Variable} = 4.830 \text{ kN}$	
Reactions at support B	$R_{B,max} = 10.850 \text{ kN}$	R _{a min} = 10.850 kN
Unfactored permanent load reaction at support B	R _{B. Permanent} = 2.670 kN	
Unfactored variable load reaction at support B	R _{B. Variable} = 4.830 kN	
		+
Timber exting dataile		
Timber section details	h - 00 mm	
Breadth of timber sections	b = 60 mm	
Depth of timber sections	n = 200 mm	
Timber strength class _ EN 229:2000 Table 1	N = Z C16	
Timber strength class - EN 336.2009 Table T	010	
Steel section details		
Breadth of steel plate	b₅ = 10 mm	
Depth of steel plate	h₅ = 150 mm	
Number of steel plates in member	Ns = 1 f = 275 N/mm ²	
Rolt diameter	$l_y = 273$ N/IIIII	
Boit diameter	φ6 – 12 ΠΠΠ	
Member details	1 4	
Load duration - cl.2.3.1.2	Long-term	
Length of bearing	I L. = 100 mm	
	L8 - 100 mm	
Section properties	A NI I I 24000	2
Cross sectional area of member	$A = N \times D \times n = 24000 m$	nm-
limber section modulus	$W_{yt} = N \times b \times h^2 / 6 = 80$	00000 mm ³
Steel section modulus	$W_{ys} = N_s \times b_s \times h_s^2 / 6 =$	37500 mm ³
Second moment of area of timber	$l_{yt} = N \times b \times h^3 / 12 = 800$	000000 mm ⁴
Second moment of area of steel	$I_{ys} = N_s \times b_s \times h_s^3 / 12 = 2$	2812500 mm ⁴
Load proportions		
Instant deflection under permanent actions	UnstG = 2.795 mm	
Instant deflection under principal variable action	Unstq1 = 5.057 mm	
	k _{def} = 0.6	
	$\psi_2 = 0.3$	
Final fifth percentile value of modulus of elasticity		
E _{0.05.fn} = E _{0.05} ×	$(U_{instG} + U_{instQ1}) / (U_{instG} + U_{instQ1})$	$u_{instQ1} + k_{def} \times (u_{instG} + \psi_2 \times u_{instQ1})) = 4062 \text{ N/mm}^2$

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				:			
	Email enquiries@beam-designs.co.uk						
	Final mean value of modulus of elasticity						
	E0.mean.fin = E0.mean ×	(UinstG + UinstQ1) / (UinstG +	UinstQ1 + Kdet	\times (UinstG + ψ_2 × UinstQ1)) = 6017 N/mm ²			
	Proportion of applied load in timper to check bendi	ng, snear and instantar	eous defiec	uon			
	Proportion of applied load in steel to check strengt	Kt = E0.mean × lyt / (E0.mea h of holts and steel plat	n ×lyt ∓ ⊑SEC o	3 × lys) = 0.520			
	Troportion of applied load in steel to check strengt	k = 1.1 x Farma x la //	e Faare vlut	Earon x (m) = 0.710			
	Proportion of applied load in timber to check final of	deflection	Eulosian o types				
		$k_{t,def} = E_{0,mean,fin} \times _{vt} / (E_{t,def})$		$+ E_{\text{REC3}} \times _{\text{vs}} = 0.449$			
	Dartial factor for material properties and resist:						
	Partial factor for material properties - Table 2.3	m = 1 300					
	Madification for that and properties - Table 2.5	m = 1.500					
	Modification factors	contant Table 2.1					
	Modification factor for load duration and moisture o	keed = 0.700					
	Deformation factor for service classes - Table 3.2	ker = 0.600					
	Depth factor for bending - exp.3.1	kh.m = 1.000					
	Depth factor for tension - exp.3.1	k _{h.t} = 1.000					
	Bending stress re-distribution factor - cl.6.1.6(2)	km = 1.000					
	Crack factor for shear resistance - cl.6.1.7(2)	ker = 0.670					
	Load configuration factor - exp.6.4	kc.so = 1.500					
	System strength factor - cl.6.6	$k_{sys} = 1.000$					
	Effective length - Table 6.1	$L_{ef} = 1.0 \times L_{s1} + 2 \times h =$	= 4300 mm				
	Critical bending stress - exp.6.32	$G_{m,ent} = 0.78 \times (N \times D)^{-1} \times E_{0.05} / (N \times L_{ef}) = 70.327 N/IIIIIP$					
	Relative signaturess for bending - exp.6.30	$\lambda_{\text{relm}} = \sqrt{[\text{Im},k]} \sigma_{\text{m,crt}} = 0.476$					
	Lateral buckling factor - exp.o.34	Kerit = 1.000					
	Compression perpendicular to the grain - cl.6.1	.5	0	20 mm))) 0 005 N/mm2			
	Design compressive stress	$G_{c.90.d} = R_{B_{max}} / (N \times D)$	× (Lb + min(l	_, 30 mm))) = 0.695 N/mm-			
	PASS Design compressive strength	$I_{c.90,d} = K_{mod} \times K_{sys} \times K_{c.9}$	0 × Ic.90.k / γμ	= 1.777 N/IIIII-			
	PASS - Design C	ompressive surengure	exceeds des	sign compressive stress at bearing			
	Bending - cl 6.1.6		0.07				
	Design timber bending stress	$\sigma_{m.t.d} = K_t \times M / W_{yt} = 4.$	937 N/mm-	0.045 Marcal			
	Design timber bending strength	Im.d = Kh.m × Kmod × Ksys >	< Kcrit × Tm.k / γ	M = 8.615 N/mm ²			
	PASS • De: Design steel hending stress	sign umber bending s σ .−k ×M/W −1	u engui exc 1/3 713 N/m	m ²			
	Design steel bending strength	$f_{\rm res} = f_{\rm r} / m_{\rm res} = 275000$	N/mm ²				
	PASS -	Design steel bending	strenath ex	ceeds desian steel bending stress			
	Members subjected to either bending or combi	nod bonding and com	proceion o				
	Lateral torsional stability check - eq.6.33	$\sigma_{\rm rest}$ / (ker x feet) = 0.5	pression - 0 73	4.0.5.5			
	Lateral torsional stability theth - eq.0.55	PASS - Member	desian mee	ts lateral torsional stability criteria			
	Sheer of 6.4.7	TASS - Member	uesign mee	as lateral torsional stability enteria			
	Shear - Cl.0.1.7			N/mm2			
	Applieu sileal suess	$td = 3 \times Rt \times F / (2 \times Rcr$	× A) = 0.520	(mm ²			
	1 CITIISSING SITCAL SUCSS	PASS - Desin	m = 1.123 N In shear str	enath exceeds design shear stress			
	Deflection of 7.2	i Abb - Dealy	n oncar our	angar okoocus uosigii siicai su 635			
	Deflection limit	$r_{\rm m} = \min(44 \text{ mm} - 0.00)$	2	700 mm			
	Denection innit	oim - min(14 mm, 0.00	5 × Ls1) = 11				
I							

culations Prepared by JMS Consulting Engineers Ltd			BD110624.doc
www.BEAM-DESIGNS	.co.uk	Job ref Sheet Made By Date	: BD110624 : Structure / 6 - : Beam-Designs - SR : June 2011 / :
Email enquiries@beam-designs.co.uk			
Instantaneous deflection due to permanent load	δinstG = 2.982 mm		
Final deflection due to permanent load	$\delta_{\text{fing}} = \delta_{\text{instg}} \times (1 + k_{\text{def}})$	= 4.772 mm	1
Instantaneous deflection due to variable load	δinsta = 5.395 mm		
Factor for quasi-permanent variable action	$\psi_2 = 0.3$		
Final deflection due to variable load	$\delta_{nnQ} = \delta_{instQ} \times (1 + \psi_2 \times$	kdet) = 6.360	6 mm
Total final deflection	δrin = δring + δring = 11.13	38 mm	
	PASS - Total	final deflec	ction is less than the deflection limit
Steel-to-timber connections - cl.8.2.3			
Characteristic yield moment - exp.8.30	$M_{y,R,k}=0.3\ mm^{0.4}\times f_{u,k}$	× 4,0 ^{2.6} = 76	745 Nmm
Char.embed.strength par.to grain - exp.8.32	fn.o.k = 0.082e9 m/sec2	× (1 mm - 0	.01 × φ _b) × ρ _k = 22.370 N/mm ²
	k _{so} = 1.35 + 0.015 × φ _b	/ 1 mm = 1	.530
Char.embed.strength perp.to grain - exp.8.31	fn.so.x = fn.o.x / Kso = 14.6	21 N/mm ²	
Thickness limit for thin steel plates	bs.thn = φ ₆ / 2 = 6 mm		
Thickness limit for thick steel plates	bs.thk = φ _b = 12 mm		
Characteristic load-carrying capacity for a plate of	any thickness as the ce	ntral memb	er in double shear - exp.8.11
	$F_{v,Rk,f} = f_{h,k} \times b \times \varphi_b = 10$).527 kN	
	$F_{V,Rk,g} = f_{h,k} \times b \times \phi_{b} \times (\mathbf{v})$	[2 + 4 × M ₂	
	Fv.rk.h = 2.3 × √[My.rk ×	fn.к × фь] = 8	.440 kN
	FV.RK = Min(FV.RK.T, FV.RK.g	, Fv.Rk.h) = 6	5.071 kN
Elitch plate bolting requirements			
Total load on member	W _{tot} = 21.699 kN		
Total load taken by steel	$W_s = k_s \times W_{tot} = 15.398$	kN	
Number of interfaces	$N_{int} = (N + N_s) - 1 = 2$		
Number of bolts required at supports	$N_{be} = max(k_s \times R_{B_{max}})$	(Net × Even), 2) = 2
Limiting bolt spacing	$S_{imit} = min(2.5 \times h.600)$	mm) = 500) mm
Maximum bolt spacing	S _{max} = 500 mm	,	
Minimum number of bolts along length of member	$N_{bl} = W_s / (N_{int} \times F_{v,Rk})$	= 1.268	
- Provide a minimum of 2 No.12 mm diameter bol	ts at each support		
- Provide 12 mm diameter bolts at maximum 500	mm centres staggered 5	50 mm alter	nately above and below the centre
line			-
Minimum bolt spacings - cl.8.5			
Minimum end spacing - Table 8 4	$S_{end} = 7 \times \phi_b = 84 \text{ mm}$		
Minimum edge spacing - Table 8 4	$S_{edge} = 4 \times d_b = 48 \text{ mm}$		
Minimum bolt spacing - Table 8.4	Shot = $4 \times \phi_0 = 48 \text{ mm}$		
Minimum washer diameter - cl 10 4 3	$\phi_w = 3 \times \phi_s = 36 \text{ mm}$		
Minimum washer thickness - cl 10.4.3	$\psi = 3 \times \psi = 30$ mm		
Winimum Washer Unickness - cl. 10.4.5	w = 0.5 × φ ₀ = 5.0 mm		
Use 200x130 C16 Flitch Beam \	With A 10mm 15	0 Depth	Steel Plate
	Lintels		
Load to Lintel	_		

Roof	3.9/2*0.6=1.17	3.9/2*0.75=1.463
(flat)		

Total 1.17 kN/m Dead 1.463 kN/m Live



: BD110624 Job ref Sheet : Structure / 8 www.BEAM-DESIGNS.co.uk : Beam-Designs - SR Made By : June 2011/ Date Email enquiries@beam-designs.co.uk Fmax = 7.518 kN Fmin = -7.483 kN Maximum shear F = max(abs(Fmax),abs(Fmin)) = 7.518 kN Design shear Total load on beam Wtot = 15.001 kN Reactions at support A RA max = 7.518 kN RA min = 7.518 kN Unfactored permanent load reaction at support A RA_Permanent = 2.039 kN Unfactored variable load reaction at support A RA_variable = 3.177 kN Reactions at support B RB_max = 7.483 kN RB_min = 7.483 kN Unfactored permanent load reaction at support B RB_Permanent = 1.918 kN Unfactored variable load reaction at support B RB_variable = 3.262 kN <−76→ -100-+ Timber section details Breadth of timber sections b = 35 mm h = 200 mm Depth of timber sections Number of timber sections in member N = 2 C16 Timber strength class - EN 338:2009 Table 1 Steel section details bs = 6 mm Breadth of steel plate Depth of steel plate h₅ = 150 mm Number of steel plates in member Ns = 1 Nominal yield stress fy = 275 N/mm² Bolt diameter φ₀ = 12 mm Member details Load duration - cl.2.3.1.2 Long-term Service class of timber - cl.2.3.1.3 1 Length of bearing $L_{b} = 100 \text{ mm}$ Section properties $A = N \times b \times h = 14000 \text{ mm}^2$ Cross sectional area of member $W_{yt} = N \times b \times h^2 / 6 = 466667 \text{ mm}^3$ Timber section modulus Steel section modulus $W_{ys} = N_s \times b_s \times h_s^2 / 6 = 22500 \text{ mm}^3$ lyt = N × b × h³ / 12 = 466666667 mm⁴ Second moment of area of timber Second moment of area of steel lys = Ns × bs × hs3 / 12 = 1687500 mm4 Load proportions Instant deflection under permanent actions UinstG = 0.685 mm UinstQ1 = 1.142 mm Instant deflection under principal variable action k_{def} = 0.6 $\psi_2 = 0.3$ Final fifth percentile value of modulus of elasticity

 $E_{0.05,fin} = E_{0.05} \times (U_{instG} + U_{instQ1}) / (U_{instG} + U_{instQ1} + k_{def} \times (U_{instG} + \psi_2 \times U_{instQ1})) = 4038 \text{ N/mm}^2$

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Final mean value of modulus of elasticity	·					
En mean π = En mean x	(Uinsta + Uinstat) / (Uinsta +	Unston + Kris	$r \times (\text{Uinster} + W_2 \times \text{Uinster})) = 5982 \text{ N/mm}^2$			
Proportion of applied load in timber to check bendi	ing, shear and instantan	eous deflec	tion			
	kt = E0.mean × lvt / (E0.mea	n × lvt + Esec	c3 × lvs) = 0.513			
Proportion of applied load in steel to check strengt	h of bolts and steel plate	e				
	ks = 1.1 × Esecs × lys / (l	Eo.os.nin × lyt +	+ Esecs × lys) = 0.718			
Proportion of applied load in timber to check final of	deflection					
	Kt.der = E0.mean.fin × lyt / (E	0.mean.fin × lyt	+ Esec3 × Iys) = 0.441			
Partial factor for material properties and resista	ances					
Partial factor for material properties - Table 2.3	γ _M = 1.300					
Modification factors						
Modification factor for load duration and moisture of	content - Table 3.1					
	k _{mod} = 0.700					
Deformation factor for service classes - Table 3.2	kdef = 0.600					
Depth factor for bending - exp.3.1	kn.m = 1.000					
Depth factor for tension - exp.3.1	kh.t = 1.000					
Bending stress re-distribution factor - cl.6.1.6(2)	km = 1.000					
Crack factor for shear resistance - cl.6.1.7(2)	k _{cr} = 0.670					
Load configuration factor - exp.6.4	kc.90 = 1.500					
System strength factor - cl.6.6	Ksys = 1.000	- 1350 mm				
Effective length - Table 6.1	Lef = $0.5 \times L_{s1} + 2 \times N =$	1350 mm	$1 \rightarrow -76$ 440 Mmm ²			
Chucal bending stress - exp.o.32	$\sigma_{m,orit} = 0.78 \times (N \times D)^{-1}$	$m_{crit} = 0.16 \times (N \times D)^2 \times E0.057 (II \times Ler) = 70.440 (N/IIIII)^2$				
Relative signaturess for bending - exp.6.30	$\lambda_{rel.m} = \sqrt{[Im.k/Gm.ort]} = 0$	- 0.430				
Lateral buckling factor - exp.o.34	Kont = 1.000					
Compression perpendicular to the grain - cl.6.1	.5					
Design compressive stress	$\sigma_{c.90.d} = RA_max / (N \times b)$	× (L _b + min(Lb, 30 mm))) = 0.826 N/mm ²			
Design compressive strength	Tc.90.d = Kmod × Ksys × Kc.90	0 × Tc.90.k / γΜ	= 1./// N/mm²			
PASS - Design c	ompressive strength e	exceeds de	sign compressive stress at bearing			
Bending - cl 6.1.6						
Design timber bending stress	$\sigma_{m.t.d} = Kt \times M / Wyt = 6.1$	581 N/mm ²				
Design timber bending strength	fm.d = Kh.m × Kmod × Ksys ×	Kont × 1m.k /	γ _M = 8.615 N/mm ²			
PASS - De	sign timber bending si	trength exc	ceeds design timber bending stress			
Design steel bending stress	$Gm.s.d = Ks \times IVI / VVys = 1$	91.071 N/M	im-			
Design steel bending strength	Ty.d = Ty / γ_{MD} = 275.000	N/mm-	veoade dasign stool bonding strass			
FA33 -	Design steer bending	suengui e.	Looo			
Members subjected to either bending or combi	ned bending and com	pression - (cl.6.3.3			
Lateral torsional stability check - eq.6.33	om.td / (Kerit × Im.d) = 0.70)4 				
	PASS - Member	aesign me	ets lateral torsional stability criteria			
Shear - cl.6.1.7			2			
Applied shear stress	$\tau d = 3 \times kt \times F / (2 \times kcr)$	× A) = 0.61	/ N/mm ⁺			
Permissible shear stress	$Iv.d = Kmod \times Ksys \times fv.k / \gamma$	M = 1.723 N	//mm ⁺			
	PASS - Desig	n snear sti	engin exceeds design shear stress			
Deflection - cl.7.2						
Deflection limit	διιm = min(14 mm, 0.003	3 × Ls1) = 5.	700 mm			

		Job ref Sheet	: BD110624 : Structure / 10 -				
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		Date	: June 2011/				
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Instantaneous deflection due to permanent load	δ _{instG} = 0.948 mm						
Final deflection due to permanent load	δfing = δinstg × (1 + kder)	= 1.516 mn	n				
Instantaneous deflection due to variable load	δinsta = 1.590 mm						
Factor for quasi-permanent variable action	$\psi_2 = 0.3$						
Final deflection due to variable load	$\delta nq = \delta x + \psi 2 \times$	kder) = 1.87	'6 mm				
Total final deflection	$\delta \sin = \delta \sin \phi + \delta \sin \phi = 3.393$	2 mm					
	PASS - Total	final defle	ction is less than the deflection limit				
Steel-to-timber connections - cl.8.2.3							
Characteristic yield moment - exp.8.30	$M_{y,R,k} = 0.3 \text{ mm}^{0.4} \times f_{u,k}$	$\times \phi_0^{2.6} = 76$	0745 Nmm				
Char.embed.strength par.to grain - exp.8.32	$f_{h.0.k} = 0.082e9 \text{ m/sec}^2$	× (1 mm - (0.01 × φ _b) × ρκ = 22.370 N/mm ²				
	keo = 1.35 + 0.015 × φe	/ 1 mm = 1	1.530				
Char.embed.strength perp.to grain - exp.8.31	f _{h.90.k} = f _{h.0.k} / k ₉₀ = 14.6	21 N/mm ²					
Thickness limit for thin steel plates	bs.thn = φb / 2 = 6 mm						
Thickness limit for thick steel plates	bs.tnk = φb = 12 mm						
Characteristic load-carrying capacity for a plate of	of any thickness as the central member in double shear - exp.8.11						
	$F_{V,Rk,f}=f_{h,k}\timesb\times\phi_{b}=6,$.141 kN					
	$F_{v.Rk.g} = f_{h.k} \times b \times \phi_b \times (\gamma)$	√[2 + 4 × M	y.rk / (fn.k × φb × b²)] - 1) = 5.229 kN				
	$F_{v,Rk,h} = 2.3 \times \sqrt{[M_{y,Rk} \times f_{h,k} \times \phi_b]} = 8.440 \text{ kN}$						
	Fv.Rk = Min(Fv.Rk.f, Fv.Rk.g, Fv.Rk.h) = 5.229 kN						
Flitch plate bolting requirements							
Total load on member	Wtot = 15.001 kN						
Total load taken by steel	Ws = ks × Wtot = 10.773	3 kN					
Number of interfaces	$N_{Int} = (N + N_{S}) - 1 = 2$						
Number of bolts required at supports	$N_{be} = max(k_s \times R_{A_max})$	(Nint × Fv.R	(k), 2) = 2				
Limiting bolt spacing	Silmit = min(2.5 × h, 600	0 mm) = 50	0 mm				
Maximum bolt spacing	S _{max} = 500 mm						
Minimum number of bolts along length of member	Γ Nbi = Ws / (Nint × Fv.Rk) =	= 1.03					
- Provide a minimum of 2 No.12 mm diameter bo	lts at each support						
- Provide 12 mm diameter bolts at maximum 500	mm centres staggered	50 mm alte	rnately above and below the centre				
line							
Minimum bolt spacings - cl.8.5							
Minimum end spacing - Table 8.4	$S_{\text{end}} = 7 \times \phi_{\text{b}} = 84 \text{ mm}$						
Minimum edge spacing - Table 8.4	Sedge = $4 \times \phi_b$ = 48 mm	1					
Minimum bolt spacing - Table 8.4	Sbot = $4 \times \phi_b$ = 48 mm						
Minimum washer diameter - cl.10.4.3	$\phi_w = 3 \times \phi_b = 36 \text{ mm}$						
Minimum washer thickness - cl 10 4 3	tw = 0.3 × do = 3.6 mm						

Use 200x76 C16 Flitch Beam With A 6mm 150 Depth Steel Plate



Job ref : Sheet : Made By : Date :

: BD110624 : **Structure / 11 -**: Beam-Designs - SR **: June 2011**/

Connections

Connection between flitch beam and lintel

JHA450 SPECIFICATIONS

	Supported	Madal	Dimensions (mm) ¹⁹			Number of Fasteners ^[2]			Safe Working Loads (kN) ^[4]			Characteristic Loads (kN)		
Installation Method	Member Width	No.	w	н	нв	C	Hea	Header		Download		Short	Download	Unlift
				n			Face	Тор	00131	Long Term	Short Term	Uplitt	Download	opint
	1 ply 38	JHA450/38	38	481	50	191				4.2	4.8	1.6	10.0	3.13
	1 ply 44	JHA450/44	44	478	50	188	8	4		4.8	5.5	1.6	11.6	3.13
	1 ply 45	JHA450/47	47	477	50	187				5.2	5.9	1.6	12.4	3.13
	1 ply 50	JHA450/50	50	475	50	185			6	5.5	6.3	1.6	13.2	3.13
	1 ply 63	JHA450/63	63	469	50	179				5.5	6.3	1.6	13.2	3.13
Wrap Over	1 ply 75	JHA450/75	75	463	50	173				5.5	6.3	1.6	13.2	3.13
	2 pły 45	JHA450/91	91	455	50	165				5.5	6.3	1.6	13.2	3.13
	2 pły 50	JHA450/100	100	450	50	160				5.5	6.3	1.6	13.2	3.13
	2 ply 63	JHA450/125	125	453	63	163				5.7	6.5	1.6	13.6	3.13
	3 ply 45	JHA450/137	137	447	63	157				5.7	6.5	1.6	13.6	3.13
	3 pły 50	JHA450/150	150	440	63	150				5.7	6.5	1.6	13.6	3.13

Use JHA450/137 Simpson Joist Hanger



