

85 CAMDEN MEWS, LONDON NW1

STRUCTURAL CALCULATIONS

PART B

INTERNAL ALTERATIONS

Notes:

Beams designed to BS and to limit deflection to

min. span/ 300 under service Dead + Imposed loads

Span/ 360 - 500 under Imposed load

Unless noted otherwise for enhancement performance

Existing Brickwork = assumed allowable service bearing pressure of 0.43N/mm2 in padstone designs unless noted otherwise.

Frames to provide stability at the rear are designed to limit sway of H / 300

Job Number: 15005 Date issue: January 2019 Prepared by: KK / AP

RR1 - Timber Roof Rafters					
Span max =	2.95 m				
Slope =	26 degree				
Loading:	20 ucgice				
Luaunig.	0.0111/ 0	1	anthe anti-		
Dead =	0.9 kN/m2	including	self weigth of joists		
Imposed (snow) =	0.6 kN/m2				
From Tedds use =	47x150 C24 at 400 c/c	max. clear	r span =	3.8 m	
	Screw fix and birds mou	ith to beams and	supporting wall		
			supporting wait.		
Uplift to RR1 - Timber Roof Rafters:					
qs =		0.70			
Cp = Cpe + Cpi =		<u>-0.80</u> (-	-0.6-0.2)		
Fw = qs x Cp =		-0.56 kN/m2			
Uplift at edge x $2.95/2 =$	1.475 x	-0.56 x	1.4 =	-1.2 kN/m	
$R_{PS} = SWt R_{PO}f x 2.95/2 =$	1 475 ×	0.25 x	1 –	0.4 kN/m	
Nes - 5W(NOOT X 2.55/2 -	1.475 X	0.23 X		0.4 kN/m	-
			upint =	-0.6 KN/M	
use min. 2x5dia screws x 100mm long	<u>z = 2x14.4N x 35mm</u>		=	1.008 kN	
(standard penetration)	K52 =	1.25		1.26 KN short te	rm
	per m = 1/0.4 =	2.50 joists per	meter	3.15 kN/m	Uplift OK
CJ1 - Ceiling Joists					
Snan =)) m				
Span –	2.2 111				
Loading:			10 1 1 1 1 1		
Dead =	0.30 kN/m2	including	self weigth of joists		
Imposed =	0.3 kN/m2				
From Trada Tables for =	47x120 C24 at 400 c/c	max, clear	r span =	2.65 m	ОК
			, span		
			, span		
					-
TP1 - Timber Purlin:					-
TP1 - Timber Purlin: Loading:					
TP1 - Timber Purlin: Loading:	L DL+IL	kN/m2			
TP1 - Timber Purlin: Loading: New RR x 3.2m/2	L DL+IL 1.6 x	kN/m2 1.5 =	2.4 kN/m		
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2	L DL+IL 1.6 x 1.1 x	kN/m2 1.5 = 0.6 =	2.4 kN/m 0.6 kN/m		
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Sut	L DL+IL 1.6 x 1.1 x	kN/m2 1.5 = 0.6 =	2.4 kN/m 0.6 kN/m		
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt	L DL+IL 1.6 x 1.1 x	kN/m2 1.5 = 0.6 =	2.4 kN/m 0.6 kN/m 0.1 kN/m		
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt	L DL+IL 1.6 x 1.1 x Sum s	kN/m2 1.5 = 0.6 =	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m		
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt	L DL+IL 1.6 x 1.1 x Sum s	kN/m2 1.5 = 0.6 = Is:	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m	_	-
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span	L DL+IL 1.6 x 1.1 x Sum s 2.9 m	kN/m2 1.5 = 0.6 = Is:	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m	_	-
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span w =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m	kN/m2 1.5 = 0.6 = Is:	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m		
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span w = Msls = ql^2/8 =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm	kN/m2 1.5 = 0.6 = Is:	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m		
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span w = Msls = ql^2/8 = Elreg (Lx0.003) =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 326.0 kNm2	kN/m2 1.5 = 0.6 =	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m		
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span w = MsIs = ql^2/8 = EIreq (Lx0.003) = R sls =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 326.0 kNm2 4.5 kN	kN/m2 1.5 = 0.6 = Is:	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m		
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span w = MsIs = qI^2/8 = EIreq (Lx0.003) = R sIs =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 326.0 kNm2 4.5 kN	kN/m2 1.5 = 0.6 = Is:	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m		
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span w = MsIs = ql^2/8 = EIreq (Lx0.003) = R sIs =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 326.0 kNm2 4.5 kN	kN/m2 1.5 = 0.6 = Is:	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m		
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span w = MsIs = ql^2/8 = EIreq (Lx0.003) = R sIs = Design:	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 326.0 kNm2 4.5 kN	kN/m2 1.5 = 0.6 =	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m		
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span w = MsIs = ql^2/8 = EIreq (Lx0.003) = R sIs = Design: Grade C24	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 326.0 kNm2 4.5 kN	kN/m2 1.5 = 0.6 = Is:	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m		
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span w = MsIs = ql^2/8 = Elreq (Lx0.003) = R sIs = Design: Grade C24	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 3.2 kNm 326.0 kNm2 4.5 kN	kN/m2 1.5 = 0.6 = Is: Bending	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m	Shear	
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span w = Msls = ql^2/8 = Elreq (Lx0.003) = R sls = Design: Grade C24 E =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 326.0 kNm2 4.5 kN 7200 N/mm2	kN/m2 1.5 = 0.6 = Is: Bending smll =	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m 7.5 N/mm2	Shear til =	0.71 N/mm2
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span w = MsIs = qI^2/8 = EIreq (Lx0.003) = R sIs = Design: Grade C24 E =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 326.0 kNm2 4.5 kN 7200 N/mm2	kN/m2 1.5 = 0.6 = Is: Bending smll =	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m 7.5 N/mm2	Shear til =	0.71 N/mm2
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span w = Msls = ql^2/8 = Elreq (Lx0.003) = R sls = Design: Grade C24 E = N =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 326.0 kNm2 4.5 kN 7200 N/mm2 1	kN/m2 1.5 = 0.6 = Is: Bending smll = K2 =	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m 7.5 N/mm2	Shear til =	0.71 N/mm2
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span w = Msls = ql^2/8 = Elreq (Lx0.003) = R sls = Design: Grade C24 E = N = b =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 326.0 kNm2 4.5 kN 7200 N/mm2 1 75 mm	kN/m2 1.5 = 0.6 = Is: Bending smll = K2 = K3 =	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m 7.5 N/mm2 1 1.25 medium term	Shear til =	0.71 N/mm2
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span w = Msls = ql^2/8 = Elreq (Lx0.003) = R sls = Design: Grade C24 E = N = b = N x b =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 3.2 kNm 326.0 kNm2 4.5 kN 7200 N/mm2 1 75 mm 75 mm	kN/m2 1.5 = 0.6 = Is: Bending smll = K2 = K3 = K7 =	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m 7.5 N/mm2 1 1.25 medium term 1.03 depth	Shear till =	0.71 N/mm2
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span w = Msls = ql^2/8 = Elreq (Lx0.003) = R sls = Design: Grade C24 E = N = b = N x b = b =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 326.0 kNm2 4.5 kN 7200 N/mm2 1 75 mm 75 mm 75 mm	kN/m2 1.5 = 0.6 = Is: Bending smII = K2 = K3 = K7 = K7 = K9 -	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m 7.5 N/mm2 1 1.25 medium term 1.03 depth 1 1 load-charing	Shear til =	0.71 N/mm2
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span w = Msls = ql^2/8 = Elreq (Lx0.003) = R sls = Design: Grade C24 E = N = b = N x b = h =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 326.0 kNm2 4.5 kN 7200 N/mm2 1 75 mm 75 mm 225 mm	kN/m2 1.5 = 0.6 = Is: Bending smII = K2 = K3 = K7 = K8 = K7 = K8 =	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m 7.5 N/mm2 1 1.25 medium term 1.03 depth 1.1 load-sharing	Shear till =	0.71 N/mm2
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span w = Msls = ql^2/8 = Elreq (Lx0.003) = R sls = Design: Grade C24 E = N = b = N x b = h =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 326.0 kNm2 4.5 kN 7200 N/mm2 1 75 mm 75 mm 75 mm 225 mm	kN/m2 1.5 = 0.6 = Is: Is: K2 = K3 = K7 = K8 = K9 =	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m 7.5 N/mm2 1 1.25 medium term 1.03 depth 1.1 load-sharing 1 number for E	Shear til =	0.71 N/mm2
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span w = Msls = ql^2/8 = Elreq (Lx0.003) = R sls = Design: Grade C24 E = N = b = N x b = h =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 326.0 kNm2 4.5 kN 7200 N/mm2 1 75 mm 75 mm 75 mm 225 mm	kN/m2 1.5 = 0.6 = Is: Bending smll = K2 = K3 = K7 = K8 = K9 =	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m 7.5 N/mm2 1 1.25 medium term 1.03 depth 1.1 load-sharing 1 number for E	Shear til =	0.71 N/mm2
TP1 - Timber Purlin:Loading:New RR x 3.2m/2Ceiling x 2.1/2SwtL = spanw =Msls = ql^2/8 =Elreq (Lx0.003) =R sls =Design:Grade C24E =N =b =N x b =h =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 3.2 kNm 326.0 kNm2 4.5 kN 7200 N/mm2 1 75 mm 75 mm 75 mm 225 mm	kN/m2 1.5 = 0.6 = Is: Bending smll = K2 = K3 = K7 = K8 = K9 =	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m 7.5 N/mm2 1 1.25 medium term 1.03 depth 1.1 load-sharing 1 number for E	Shear til =	0.71 N/mm2
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span w = Msls = ql^2/8 = Elreq (Lx0.003) = R sls = Design: Grade C24 E = N = b = N x b = h = A = Ixx = bh3/12 =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 326.0 kNm2 4.5 kN 7200 N/mm2 1 75 mm 75 mm 225 mm 16875 mm2 71191406 mm4	kN/m2 1.5 = 0.6 = Is: Bending smII = K2 = K3 = K7 = K8 = K9 =	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m 7.5 N/mm2 1 1.25 medium term 1.03 depth 1.1 load-sharing 1 number for E	Shear til =	0.71 N/mm2
TP1 - Timber Purlin: Loading: New RR x 3.2m/2 Ceiling x 2.1/2 Swt L = span w = Msls = ql^2/8 = Elreq (Lx0.003) = R sls = Design: Grade C24 E = N = b = N x b = h = A = Ixx = bh3/12 = Iyy = b3h/12 =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 326.0 kNm2 4.5 kN 7200 N/mm2 1 75 mm 75 mm 225 mm 16875 mm2 71191406 mm4 7910156 mm4	kN/m2 1.5 = 0.6 = Is: Bending smII = K2 = K3 = K7 = K8 = K9 =	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m 7.5 N/mm2 1 1.25 medium term 1.03 depth 1.1 load-sharing 1 number for E	Shear til =	0.71 N/mm2
TP1 - Timber Purlin:Loading:New RR x 3.2m/2Ceiling x 2.1/2Swt $L = span$ $w =$ Msls = ql^2/8 =Elreq (Lx0.003) =R sls =Design:Grade C24E =N =b =N x b =h =A =Ixx = bh3/12 =Iyy = b3h/12 =Zxx = bh2/6 =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 326.0 kNm2 4.5 kN 7200 N/mm2 1 75 mm 75 mm 225 mm 16875 mm2 71191406 mm4 7910156 mm4 632813 mm3	kN/m2 1.5 = 0.6 = Is: Bending smll = K2 = K3 = K7 = K8 = K9 =	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m 7.5 N/mm2 1 1.25 medium term 1.03 depth 1.1 load-sharing 1 number for E	Shear til =	0.71 N/mm2
TP1 - Timber Purlin: Loading:Loading:New RR x 3.2m/2 Ceiling x 2.1/2Swt $L = span$ w = Msls = ql^2/8 = Elreq (Lx0.003) = R sls =Design: Grade C24E = N = b = N x b = h =A = Ixx = bh3/12 = Iyy = b3h/12 = Zxx = bh2/6 =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 326.0 kNm2 4.5 kN 7200 N/mm2 1 75 mm 75 mm 75 mm 225 mm 16875 mm2 71191406 mm4 7910156 mm4 632813 mm3	kN/m2 1.5 = 0.6 = Is: Bending sml1 = K2 = K3 = K7 = K8 = K9 =	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m 7.5 N/mm2 1 1.25 medium term 1.03 depth 1.1 load-sharing 1 number for E m*K2*K3*K7*K8	Shear til =	0.71 N/mm2
TP1 - Timber Purlin:Loading:New RR x 3.2m/2Ceiling x 2.1/2Swt $L = span$ $w =$ Msls = ql^2/8 =Elreq (Lx0.003) =R sls =Design:Grade C24E =N =b =N x b =h =A =Ixx = bh3/12 =Iyy = b3h/12 =Zxx = bh2/6 =K9*FL =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 3.2 kNm 326.0 kNm2 4.5 kN 7200 N/mm2 1 75 mm 75 mm 75 mm 225 mm 16875 mm2 71191406 mm4 7910156 mm4 632813 mm3 512 58 kNm2	kN/m2 1.5 = 0.6 = Is: Bending smll = K2 = K3 = K7 = K8 = K9 = Mrd = Z*S Mrd = Z*S	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m 7.5 N/mm2 1 1.25 medium term 1.03 depth 1.1 load-sharing 1 number for E sm*K2*K3*K7*K8 6.7 kNm	Shear til = Vc,rd = 2/3	0.71 N/mm2 *A*tilisv*K2*K3*K8
TP1 - Timber Purlin:Loading:New RR x 3.2m/2Ceiling x 2.1/2SwtL = spanW =Msls = ql^2/8 =Elreq (Lx0.003) =R sls =Design:Grade C24E =N =b =N x b =h =A =Ixx = bh3/12 =Iyy = b3h/12 =Zxx = bh2/6 =K9*El =	L DL+IL 1.6 x 1.1 x Sum s 2.9 m 3.1 kN/m 3.2 kNm 326.0 kNm2 4.5 kN 7200 N/mm2 1 75 mm 75 mm 225 mm 16875 mm2 71191406 mm4 7910156 mm4 632813 mm3 512.58 kNm2	kN/m2 1.5 = 0.6 = Is: Is: K2 = K3 = K7 = K8 = K9 = Mrd = Z*S Mrd = Z*S	2.4 kN/m 0.6 kN/m 0.1 kN/m 3.1 kN/m 7.5 N/mm2 1 1.25 medium term 1.03 depth 1.1 load-sharing 1 number for E sm*K2*K3*K7*K8 6.7 kNm	Shear til = Vc,rd = 2/3 Vc,rd =	0.71 N/mm2 *A*tilisv*K2*K3*K8 11.0 kN

Use: 1x75x225C24

TP2 - Timber Purlin:						
Loading:						
Now PD v 2 2m /2	L DL+IL kN/m	12 F -	2.4 kN/m			
Swt	1.0 X 1	.5 =	0.1 kN/m			
<u> </u>	Sum sls:		2.5 kN/m			
L = span	2.0 m					
W = Msls - gla 2/8 -	2.5 KN/M 1.2 kNm					
$F_{1} = 4^{1/2} - 8^{-2}$	85.1 kNm2					
R sls =	2.5 kN					
Design						
Grade C24						
		Bending			Shear	
E =	7200 N/mm2	smII =	7.5 N/mm2		tII =	0.71 N/mm2
N =	1	K2 =	1			
b =	75 mm	K3 =	1.25 medium ter	m		
N x b =	75 mm	K7 =	1.03 depth			
h =	225 mm	K8 =	1.1 load-sharing	5		
		K9 =	1 number for	E		
Δ =	16875 mm2					
1xx = bh3/12 =	71191406 mm4					
1 v v = b3h/12 =	7910156 mm4					
Zxx = bh2/6 =	632813 mm3					
		Mrd = Z*s	m*K2*K3*K7*K8		Vc,rd = 2/3*	'A*tllsv*K2*K3*K8
K9*EI =	512.58 kNm2	Mrd =	6.7 kNm		Vc,rd =	11.0 kN
	ОК		ОК			ОК
					lise: 1x75x2	225024
					030. 177 572	-25024
P1 - Timber Post to support Timber Purlin	<u>l</u>					
F sls = TP1 + TP2=	6.9 kN					
M sls = 0.05 x F sls	0.35 kNm					
Try: 2x47x97C16						
Le =	1 m					
Grade C16						
		Bending			Shear	
E =	5800 N/mm2	smll =	5.3 N/mm2		tII =	0.67 N/mm2
		v2 -	1			
h = v	91 mm	K2 -	1 25 medium ter	m		
b = y	97 mm	K7 =	1.13 denth			
	57 1111	K8 =	1 load-sharing	,		
N =	2	К9 =	1.14 number for	Ē		
A =	9118 mm2					
1xx = bh3/12 =	7149272 mm4					
Iyy = D3n/12 =	6/1388/ mm4		0*FI	47.27 1.01-2		
2xx = DH2/0 =	147408 111115	N:	9. EI =	47.27 KINIIIZ		
Compression:						
scII =	6.8 N/mm2	(compress	ion parallel to grain)		
Ley =	1000 mm					
iyy = (lyy/A)^0.5	27.1 mm					
E/scll -	682					
ambda v = e/ =	37					
- / -/	-					
K12 =	0.803					
Pc = sc*A*K2*K3*K12*K8						
Pcv =	62.2 kN					
· - /						
Mc = Z*sm*K2*K3*K7*K8		Vc = 2/3*/	A*t*K2*K3*K8			
		,				

Interaction check compression with be	nding:						
F =	6.9 kN						
M =	0.3 kNm						
Keu = Euler Coeficient = 1-(1.5*S-c,a*K1	2)/s-e						
S-c,a = F/A =	0.76 N/mm2						
K12 =	0.803	K	eu =	0.98			
S-e = pi^2*Emin/(lambda)^2 =		42.2 N/mm2					
	Check: Mmax/Mc'	*Keu =		0.32			
	F/Pcy =			0.11			
	Mmax/Mc [*]	*Keu + F/Pcy =		0.43	<	1	
					ОК	Use: 2x47x9	7C16
LT - Timber Lintel:							
Loading:							
	L DL+IL	kN/m2					
New RR x 2.7m/2	1.4 x	1.5 =	2.0 kN/m				
Swt			0.1 kN/m				
	Sum sl	s:	2.1 kN/m				
L = span	2.8 m						
w =	2.1 kN/m						
Msls = ql^2/8 =	2.0 kNm						
Elreq (Lx0.003) =	197.7 kNm2						
R sls =	2.9 kN						
Design:							
Grade C24							
		Bending				Shear	
E =	7200 N/mm2	smll =	7.5 N/mm2			tII =	0.71 N/mm2
N =	2	K2 =	1				
b =	47 mm	K3 =	1.25 medium term	l			
N x b =	94 mm	K7 =	1.08 depth				
h =	150 mm	K8 =	1.1 load-sharing				
		К9 =	1.14 number for E				
A =	14100 mm2						
lxx = bh3/12 =	26437500 mm4						
100 = 100, 12	10382300 mm4						
$Z_{xx} = bh2/6 =$	352500 mm3						
	5525555	Mrd = 7*s	m*K2*K3*K7*K8			Vc.rd = 2/3*	A*tllsv*K2*K3*K8
K9*EI =	217.00 kNm2	Mrd =	3.9 kNm			Vc.rd =	9.2 kN
	OK	1111.0	OK			,	OK
			.				

Use: 2x47x150C24

Hip rafter 1- check simply supported:

Loading - maximum w:	DL+I	L	DL+IL		W	
Roof x (1.1m/2+ 0.8m/2)	0.95 x	1.4 =	1.33			
	Sum	:	1.33 kN/m		\uparrow	\uparrow
L=	1.5 m				Ι	I
from Tedds refer to next page:	use 2x47x150C24					
Ra sis= Rb sis=	0.65 kN 0.33 kN					
Hip rafter 2- check simply supported:						
Loading - maximum w:	DL+I	L	DL+IL		W	
Roof x (2.9m/2+ 1.9m/2)	2.4 x	1.4 =	3.36			
	Sum	:	3.36 kN/m			
L=	3.8 m				·	·
from Tedds refer to next page:	use 2x47x200C24	or altern	atively flitch beam	2x47x150C24 wi	th full depth 10thk MS	Plate
Ra sls=	4.6 kN	Connect	to steel beam via N	/laxi Speedy joists	hangers SWL=5.9kN, he	ence OK
Rb sls=	2.5 kN					
RB1 - Steel beam Loadings:						
	L DL+I	L kN/m2	DL+IL			
RR x 2.9m/2	1.5 x	1.5 =	2.2 kN/m			
Ceiling x 0.4 swt	0.4 x	0.6 =	0.2 kN/m 0.3			
	Sum	sls:	2.7 kN/m			
L = span =	2.2 m					
Muls =	2.5 kNm	Mb = (le = 3 m)	for 152x152x	23 UC S355 45.4 kNm	(from Blue Book)	ОК
Ireq = (def <l 360)<="" td=""><td>66 cm4</td><td></td><td>lxx = def =</td><td>1250 cm4 0.32 mm</td><td>(from Blue Book) L/def = 6807</td><td>ОК</td></l>	66 cm4		lxx = def =	1250 cm4 0.32 mm	(from Blue Book) L/def = 6807	ОК
Rsls A =						
	3.0 kN					

F1

F1

1

F1

t

RB2 - Steel beam				
Loadings:				
	L	DL+IL		DL+IL
RR x 0.7m/2	0.4 x	1.5	=	0.5 kN/m
Ceiling x 0.4	0.4 x	0.6	=	0.2 kN/m
Swt included				
-	0	Sum sls:		0.8 kN/m
Point load:				
F1=TP2 = at 0, 1.1m, 2.2m=	2.5 kN			
L = span =	2.2 m			
Rsls A (from Tedds) =	4.9 kN			
Rsls B (from Tedds) =	4.9 kN			
From Tedds calculation refer to next page	min needed sect	ion is =		152x152x23UC \$355

RB3 - Cranked steel beam						L E	1	F2
Loadings						w.	- 1	I F3
w1:	1	DI +II kN/m2		DI +II		w2	_	w2 w3
$RR \times 4m/2$	20 x	1 5	-	3.0 kN/m	н 1	h1		h2
Colling x 2 $2m/2$	2.0 X	1.5	_	0.7 kN/m				V 112
Celling X 2.211/2	1.1 X	0.0	-	0.7 KN/III		-i		Ī
Swithcluded		Sum eler		2.7 kN/m		-		•
	2.0	Sum sis.		5.7 KIN/III		$\leftarrow \rightarrow \leftarrow$	>	$\leftrightarrow \!$
	2.0			DL . II			12	12.14
w2:	L	DL+IL kN/m2		DL+IL			L2	L3 L4
RR x 0.4m	0.4 x	1.5	=	0.6 kN/m		T		
Swt included						- '		
		Sum sls:		0.6 kN/m				
w2:	L	DL+IL kN/m2		DL+IL				
RR x 2m	1.0 x	1.5	=	1.5 kN/m				
Swt included								
	-	Sum sls:		1.5 kN/m		-		
Point load:								
F1= BB1 + Hin Bafter=	9.6 kN							
$E_2 - B_{R_2} + Hip Patter -$	11.5 kN							
F2-RB2 + Rip Raiter-								
rs= nip kaiter=	0.0 KIN							
And the set of the second								
wind load:								
H= 0.75kN/m2 x 0.5 x 4.1m x 2.2m=	3.4 kN							
L1=	2.1 m							
L2=	2.6 m							
L3=	0.65 m							
L4=	1.4 m							
h1=	1.10 m							
h2=	0.45 m							
L=total= actual length=	7.2 m							
	··= ···							
Refer to Robot use:	203x203x60 UC S	355						
Neter to Robot use.	2032203200 00 3	555						
Mule –	EG KN-	,						
IVIUIS =		1	(frame Div	o Doold			0.22 + 1.0	01/
MD = (10 = 4.7 m) =	108 KINIT	1	(Irom Blu	е воок)			0.33 < 1.0	UK
						111.6	F70 0	01
def (from Robot)	13.8 mm					L/det =	522 > 360	UK
Ra sIs= vertical=	22.7 kN							
Ra sls = horizontal=	5.6 kN							
Rb sls=	23.5 kN							
Rb sls = horizontal=	5.6 kN							

Column below RB3 check:									
Fsls =	22.7 kN								
Fuls =	34.1 kN								
Muls = Robot=	16.8 kNm								
sway def= (IL+WIND)=	4.5 mm			H/def=	444.4	1 > 300	henc	e OK	
Try 100x100x10.0 SHS S355:									
Le =	2.0 m								
Pcy =	807.0 kN		(Blue bo	ook)					
Mcx=	33.0 kNm								
Interaction check compression with be	nding								
Mxx/Mbs + Myy/Mcy + F/Pcy =	Mxx/Mbs =	0.00							
	Myy/Mcy =	0.51							
	F/Pcy =	0.04			0.55	5 <	1.0	ОК	
							Use1	00x100x10.0	SHS S355
Connection 4M20 bolts at 100 crs =									
Mc for 4M20 8.8 grade =	2 x 137 kN x 0.10=			27.4 kNm		>		16.8 kNm	ОК
RB4=CB1=						F1	F2		F3
Loadings:							w1		
w1:	L DL+I	L kN/m2		DL+IL		+		+	ŧ
RR x 0.4m	0.4 x	1.5	=	0.6 kN/m		A T			•
Swt incuded						_			
	Sum	sls:		0.6 kN/m					
Point load:									
F1= RB3+ TP1 + TP2=	30.4 kN								
F2=TP1 + TP2=	6.9 kN								
F3= RB3+ TP1 + TP2=	30.4 kN								
L=span==	2.4 m								
Refer to Robot use:	203x203x46 UC S355								
Rsls A (from Tedds) =	35.1 kN								
Rsls B (from Tedds) =	35.1 kN								
Lpad reqire 100w =	0.25 m		(0.95 N/	mm2 = brick stress	sls)		use n	nin 300mm b	earin either end
From Tedds calculation refer to next p	bage min needed section	is =		203x203x46 UC S3	55				

FJ1 - Timber Floor Joists							
Span =	3.2 m						
Loading:							
Dead =	0.75 kN/m2	including	self weigth of joists				
mposed =	1.5 kN/m2	5					
From Trada Tables for =	47x200 C24 at 400 c/c	max. clea	r span =		3.8 m	ОК	
Trimmer in stair void:							
.oading:							
	L DL+IL						
Stairs x 2m/2	1 x	2.3 =	2.3 kN/m				
st Floor x 0.4m	0.4 x	2.3 =	0.9 kN/m				
tud wall x 2.7m	2.7 x	0.5 =	1.4 kN/m				
wt			1.0 kN/m				
	Sum sl	s:	3.3 kN/m				
= span	3.2 m	(as measu	ured on plan)				
v=	3.3 kn/m						
/Isls = ql^2/8 =	4.2 knm						
ireq (Lx0.0003) =	465.0 knm2						
:=	5.2 kn						
Design:							
Grade C24							
		Bending			Shear		
=	7200 n/mm2	smll =	7.5 N/mm2		tII =	0.71 N/	/mm2
_	04	K2 =	1				
=	94 mm	K3 =					
=	200 mm	K7 =	1.05 depth				
I =	n	K8 = K0 -	1.1 load-shari	ng ar F			
. –	۷.	NJ -					
. =	18800 mm2						
x=bh3/12 =	62666667 mm4						
/y = b3h/12 =	13843067 mm4						
xx =bh2/6 =	626666.67 mm3						
		Mc= Z*sn	n*K2*K3*K7*K8		Vc= 2/3*/	A*tllsv*K2*K	(3*K8
9*EI =	514.37 kNm2	Mc =	5.4 kNm		Vc =	9.8 kN	1
	ОК	C	Ж			ОК	
Jse 2x47x200C24							
B1-1- Steel beam							
uaumgs.		N/m2	DI +II				
st Floor x 4.4m/2	2.2 x	2.3 =	5.1 kN/m				
wt	^		0.3				
	Sum sl	s:	5.4 kN/m				
= span =	2.9 m						
Aule –	9 E LNIM		for 1E3v1E3v	22110 5255			
viuis –	0.3 KINIII	Mb = (le = 3 m)	101 15281528	45.4 kNm	(from Blu	e Book)	ОК
	200		L	4250	16. 51	- De al V	
req = (det <l 360)<="" td=""><td>299 cm4</td><td></td><td>IXX =</td><td>1250 cm4</td><td>(trom Blue</td><td>e Book)</td><td><u> </u></td></l>	299 cm4		IXX =	1250 cm4	(trom Blue	e Book)	<u> </u>
			det =	1.93 mm	L/det =	1505	UK
Pele A -	70 LNI						
nad regire 100w =	0.05 m	(0 95 N/m	m2 = hrick stress ch	5)		150d x 100m	, x 220
200 reque 100W -	0.05 11	(0.33 14/11	SHER 311 C33 51	~1	ujt f j1. j	X 100W	
uls A=	11.7 kN						

<u>1B1-2- Steel beam</u>						
Loadings:		II kN/m2				
1st Floor x 2 1m/2	L DL+	73 =	2.4 kN/m			
Timber wall x 3m	3 x	0.9 =	2.4 kN/m			
swt	5 X	0.5 -	0.3			
	Sun	n sls:	5.4 kN/m			
L = span =	3.0 m					
Muls =	9.1 kNm		for 152x152	23 UC \$355		
		Mb = (le = 3 m	n)	45.4 kNm	(from Blue Book)	ОК
Ireg = (def<1/360)	335 cm4		lxx =	1250 cm4	(from Blue Book)	
			def =	2.23 mm	L/def = 1346	ОК
RSIS A =	8.1 KN	(0.95 N/	mm? - hrick stress s	lc)	uco DS1 · 150d v 100	w v 330l
	0.00 m	(0.33 N)		13)	use FS1. 1500 X 100	W X 3301
Ruls A=	12.2 kN					
1B1-3- Steel beam						
Loadings:		II kN/m2				
1st Floor x 3m/2	1.5 x	2.3 =	3.5 kN/m			
swt	1.5 X	2.5	0.3			
	Sun	n sls:	3.8 kN/m			
L = span =	2.1 m					
Muls =	3.1 kNm		for 152x152>	23 UC \$355		
		Mb = (le = 3 m	n)	45.4 kNm	(from Blue Book)	ОК
lreg = (def<1/360)	80 cm/		lvv =	1250 cm/	(from Blue Book)	
	50 cm4		def =	0.37 mm	L/def = 5667	ОК
RsIs A =	3.9 kN					
Lpad reqire 100w =	0.03 m	(0.95 N/	mm2 = brick stress s	ls)	use PS1: 150d x 100	w x 330l
Ruls A=	5.9 kN					
1B6-1 - Steel beam						
Loadings w1:						
1st Eloor x $0.4m/2$		DL+IL 23 =	0.5 kN/m		F1 .	
Swt included	0.2 X	2.5	0.5 kių m			w1
—	Sun	n sls:	0.5 kN/m		+	
					Ť	1
F1 at 0.45m= non - structural chimney=	10.3 kN	*2.5kN/i	m2 x 1m x 4.1m		I	I
L =span=	2.9 m					
RsIs A (from Tedds)=	9.8 kN					
ksis B (from feads) =	2.7 KN					

From Tedds calculation refer to next page min needed section is =

150x90x24PFC S355

1B6-2 - Steel beam

Loadings w1:				
	L	DL+IL	DL+IL	
1st Floor x 2.9m/2 Swt included	1.5 x	2.3 =	3.3 kN/m	F1 w1
-	Su	ım sls:	3.3 kN/m	 • • •
F1 at 0.2m= non - structural chimney=	6.2 kN	*2.5kl	N/m2 x 0.6m x 4.1m	1 1
L =span=	2.0 m			
Rsls A (from Tedds)=	10.9 kN			
Rsls B (from Tedds) =	5.6 kN			
From Tedds calculation refer to next page	min needed section	on is =	150x90x24PFC \$355	
1B2- Steel beam				F3
Loadings w1 (0-3.2m):				F1 F2 F2
	L	DL+IL	DL+IL	
1st Floor x 5m/2	2.5 x	2.3 =	5.8 kN/m	+ +
Swt included				
	Su	ım sls:	5.8 kN/m	
Loadings w2 (3.2m-6.7m):				
	L	DL+IL	DL+IL	
1st Floor x 2.9m/2	1.5 x	2.3 =	3.3 kN/m	
Swt included	Su	ım sls:	3.3 kN/m	
F1 at 3.2m= 1B1-3=	5.9 kN	0.3DL	+ 0.7IL	
F2 at 6.1m = 1B6-1 + 1B6-2=	20.7 kN	0.8DL	+ 0.2IL	
F3 at 6.7m = RB3=	22.7 kN	0.6DL	+ 0.4IL	
Rsls A (from Tedds)=	24.2 kN			
Rsls B (from Tedds) =	59.8 kN			
From Tedds calculation refer to next page	min needed section	on is =	203x203x60UC S355	
Lpad reqire 100w =	0.93 m	(0.43	N/mm2 = existing brick stre	ss sls) use PS3: 300deep x 100wide x950long

Project: 85 Camden Mews

AXIOM STRUCTURES

<u>Check def for DL+0.1IL :</u>									
Loadings W1 (U-3.2m):		DI +0 1II		DI +II					
1st Floor x 5m/2	2.5 x	0.9	=	2.3 kN/m					
Swt included				,					
		Sum sls:		2.3 kN/m	mn	n			
Loadings w2 (3.2m-6.7m):									
1st Floor x 2.9m/2	1.5 x	0.9	=	1.3 kN/m					
Swt included	210 /	0.5		215 111 / 11					
		Sum sls:		1.3 kN/m					
	DL+0.1IL								
F1 at 3.2m= 1B1-3=	2.2 kN								
F2 at $6.1m = 1B6-1 + 1B6-2=$	17.0 kN								
F3 at 6.7m = KB3=	14.5 KN								
			dof- Tod	de-	8 20 mm	0			
Natural frequency od beam:			uer= reu	us=	8.20 1111				
		1	f=18/(dei	f)^0.5	6.29 Hz		> 5Hz ok		
1B3 - Steel beam									
Loadings:									
w1:	L	DL+IL		DL+IL			F5 F2 F4 F2	F1 F3	
1st Floor x 0.4m	0.4 x	2.3	=	0.9 kN/m					
Swt included		Sum sls:		0.9 kN/m			w1 w2 w		
				010 111 (111			↑		ł
w2:(2.5m - 4.6m)	L	DL+IL		DL+IL			А	В	l
Cavity wall x 2.8 Swt included	2.8 x	4.1	=	11.5 kN/m					
Swimchudeu		Sum sls:		11.5 kN/m					
Point load:									
F1=1B2 at 0.3 m and 3 m=	24.2 kN								
F2= 1B1-1 dt 2.3m= F3= 1B1-2 at 1 6m=	7.8 KN 8.1 kN								
F4 = CB1 at 2.3m =	35.1 kN								
F5=1B1-1 at 0.1m=	7.8 kN								
	0.0								
L (A-B)=	0.8 m								
L (D-C)-	5.6 11								
Rsls A (from Tedds) =	83.5 kN								
Rsls B (from Tedds) =	54.4 kN								
Lpad regire 200w =	0.19 m		(0.95 N/r	nm2 = brick stress sls)					
From Tedds calculation refer to next pa	ge min needed sec	tion is =	7	203UC46 \$355			use min 250mm bear	ng on wall	
	.80								
Column under 1B3 check:									
Fsls =	83.5 kN								
Fuls =	125.3 kN								
M nom= 0.15 x Fuls=	18.8 kNm	ı							
Try 200x100x8.0 RHS S355:									
Le =	2.8 m								
Pcy =	1150.0 kN		(Blue bo	ok)					
Mb =	95.0 kNm	ı							
Mcy=									
1	61.1 kNm	1							
, Interaction check compression with ben	61.1 kNm ding	1							
	61.1 kNm <u>ding</u> Mxx/Mbs =	0.00							
Interaction check compression with ben Mxx/Mbs + Myy/Mcy + F/Pcy =	61.1 kNm ding Mxx/Mbs = Myy/Mcy =	0.00 0.31							

Use 200x100x8.0 RHS S355

Goal Frame:					
Loadings:	I/H	DI +II			
Roof x 3m/2	1.5 x	1.5 =	2.3 kN/m		
1st Floor x 3.1m/2	1.6 x	2.3 =	3.6 kN/m		
9" Wall x 2.3m	2.3 x	5 =	11.5 kN/m		1B3-1 D
Swt included				C2	C3 C3
	Sun	ı sls:	17.3 kN/m	А	в с
Wind load:				•	• •
	Main house - E Ov 8 I	./2 -	21.2 m2		
WL = 0.7 X A	Main house = 5.0x 8.5	5/2 =	21.3 mz		
H =	14.9 kN				
Span1 = L (A-B)=	2.7 m				
Span2= L (B-C)=	1.2 m				
Span2= L (C-D)=	1.8 m				
Height of frame -	2.8 m				
	2.0 111				
Head beam 1B3-1=1B5:					
Refer to Robot use:	203x133x30UB S355				
Muls=	17.5 kNm			M/Mb =	0.25 < 1.0 OK
Mb = (le = 3m) =	69.2 kNm	(from Blu	ue Book)	,	
Vuls=	38.3 kN				
Pv=	282 kN	(from Blu	ie Book)	Vuls/Pv=	0.14 < 1.0 OK
		(ITOTIT DA		v disy i v	0.14 (1.0 0)
def = DL+IL=	2 mm			L/def =	1350 > 360 OK
Rsls A (from Tedds)=	21.2 kN				
Rsls B (from Tedds) =	37.6 kN				
Rsls C(from Tedds)=	26.4 kN				
Rsls D (from Tedds)=	13.5 kN				
Lpad regire 100w =	0.21 m	(0.43 N/I	mm2 = ex. brick stre	ess sls)	
	0.22	(01.10.14)		200 0.07	use 250mm bearing
From Tedds calculation refer to next	page min needed section	is =	203UB30 S355		
Column C3:					
Fsls =	37 6 kN				
Fuls =	56.4 kN				
Muls= from Robot=	22.6 kNm				
sway def= (IL+WIND)=	9.0 mm		H/def=	311.1 > 300	hence OK
Try 200x100x8.0 RHS S355:					
Le =	2.8 m				
Pcy =	1150.0 kN	(Blue bo	ok)		
, Mb =	95.0 kNm	(
Interaction check compression with b	ending				
Mxx/Mbs + Myy/Mcy + F/Pcy =	Mxx/Mbs =	0.24			
	Myy/Mcy =	0.00			
	F/Pcy =	0.05		0.29 <	1.0 OK

Use 200x100x8.0 RHS S355

Connection 4M20 bolts at 300 crs =						
Ps (shear capacity)=	4 x 91.9kN =	(blue book)	367.6 kN	>	20.8 kN	OK
Mc for 4M20 8.8 grade =	2 x 137 kN x 0.30) =	82.2 kNm	>	22.6 kNm	OK

Column C2:									
Fsls =	8 1 kN								
Fuls =	12.2 kN								
1 013 -	12.2 KN								
Muls= from Robot=	12.0 kNm								
Try 203x203x46 \$355									
10-	28 m								
Le –	2.8 III 1470 0 KN		(Plue boo	(k)					
PCy -	1470.0 KN		(Blue DOO	(N)					
MD =	143.0 KNM								
Interaction check compression with be	ending								
Mxx/Mbs + Mvv/Mcv + F/Pcv =	Mxx/Mbs		0.08						
	Mvv/Mcv		0.00	=					
	E/Pov =		0.01	-	0.09 <	1	OK		
	1/FCy -		0.01	-	0.09 <	1	UK		
						Use	: 203UC46 S35	5	
Connection 4M20 bolts at 200 crs =	404.01.01.01.01.01.01						0.0.1.11	01/	
Ps (shear capacity)=	4 x 91.9kN = (b	lue book)		367.6 KN	>		0.0 KN	OK	
Mc for 4M20 8.8 grade =	2 x 137 kN x 0.20 =			54.8 kNm	>		12.0 kNm	ОК	
Steel Box Frame:									
Loadings w1:	I /H	DI +II							
$\frac{1}{2} \frac{1}{2} \frac{1}$	1 5	1 5	_	2.2 kN/m			w1		
$1 \text{ ct} \Gamma \log x 2m/2$	1.5 ×	1.5	_	2.5 kN/m	LI1		W1		
215thly blackwark well w 2m	1.5 X	2.5	-	5.5 KIN/III	11		104		
Swt included	2.0 X	5.9	-	7.0 KIN/III		C 2	104	C2	0
Swillicidded	Su	m sls:		13.5 kN/m		02	w2	F1	C2
	54	111 515.		15.5 kityin	Н2		W2	1.	
							GB5	-	
Loadings w2:	1/н	DI +II					000		
$MD \times \frac{3m}{2}$	15 v	7 2	_	11.0 kN/m					
215thk blockwork wall v 0.3m	1.5 X	3.0	_	1.0 kN/m					
Swt included	0.5 X	5.5		1.2 KN/111		_ →	•	\rightarrow	→
	Su	m sls:		12.1 kN/m					
F1 = blockwork pier=	4.875 kN		0.5m x 2.	5m x 3.9kN/m2					
Wind load:									
WL1 = 0.7 x A	Main house = 3x 8.5	/2 =		12.8 m2					
H1 =	8.9 kN								
WL2 = 0.7 x A	Main house = 1.5x 8	5.5/2 =		6.4 m2					
H1 =	4.5 kN								
Span of frame =	4.2 m								
Height of frame 1=	3.1 m								
Height of frame 2=	2.9 m								
5									

Head beam 1B4:

Loadings: as per above	L DL+IL kN/m:	2 C =	13.5 kN/m					
	Sum sls:		13.5 kN/m					
L = span =	4.2 m							
Refer to Robot use:	203x203x46 UC S355							
Muls =	22.1 kNm	1.2(DL+IL+WIN	ID)					
Mb = (le = 3.5m) =	135 kNm	(from Blue Bo	ok)	M,	/Mb =		0.16 < 1.0	ОК
def (from Robot)	2 mm			L/c	def =		2100 > 360	ОК
Check Connection Beam 1B4 with Co	lumn C2							
Muls=	22.1 kNm							
Connection 4M20 bolts at 180 crs =								
Ps (shear capacity)=	4 x 91.9kN = (blue book)	3	67.6 kN		>		12.5 kN	ОК
Mc for 4M20 8.8 grade =	2 x 137 kN x 0.18 =	4	9.32 kNm		>		22.1 kNm	ОК
Provide 15mm thick Steel end plate S	275, 6CFW							
<u>Column C2:</u>								
Fsls =	65.5 kN							
Fuls =	98.3 kN							
Muls= (from Robot)=	27.0 kNm							
def at top=	12.2 mm							
H1+H2/def=	487.7 >300	hence OK						
def at middle=	9.2 mm							
H2/def=	315.2 >300	hence OK						
Try 203x203x46UC S355:								
Le =	3.1 m							
Pcy =	1470.0 kN	(Blue book)						
Mb =	143.0 kNm							
Interaction check compression with b	ending							
Mxx/Mbs + Myy/Mcy + F/Pcy =	Mxx/Mbs =	0.19						
	Myy/Mcy =	0.00						
	F/Pcy =	0.07		0.26	<	1	ОК	

Foundation pad under column C2								
FsIs =	65.5 ki	N						
Pad swt =	4.0 ki	N						
Sum sls:	69.5 ki	N						
Area of foundations = sum / 150GBP	:	= 0.46	5 m2		Bmin =	0.68 m		
Use B = A =	:	= 1	Lm					
Area	-	= 1.00) m2	ОК				
Column C2 - RH side:								
FsIs =	3.0 ki	N						
Fuls =	5.0 ki	l N						
Muls= (from Robot)=	1.5 kl	Nm						
1ry 203X203X46UC 5355:	C D							
Le =	6.0 m							
Pcy =	648.0 KI	N	(Blue book)					
MD =	99.0 ki	Nm						
Interaction check compression with ber	nding							
Mxx/Mbs + Myy/Mcy + F/Pcy =	Mxx/Mbs =		0.02					
	Myy/Mcy =		0.00					
	F/Pcy =		0.01		0.02	< 1	ОК	
Check Connection Beam GB5 with Colu	<u>ımn C2</u>							
Muls=	38.7 ki	Nm						
Vuls =	33.4 ki	Nm						
Connection 2x3 M20 Bolts =								
Ps (shear capacity)=	6 x 91.9N =	(blue book)		551.4 kN		>	33.4 kN	ОК
Mc for 4 M20 8.8 grade (assumed - bot	tom line of bolts n	ot working in t	his condition))=				
Pnom (tension capaity)=	110 ki	N						
Mc= [2xPnom x 0.090x 90/225 + 2xPno	m x 0.225]=			57.42 kNm		>	38.7 kNm	ОК
Fc/t = M/a =	201.56 ki	N						
a = 203mm - 2 x 11mm / 2 =	192 m	m						
Clamping width =	203 m	m						
Steel grade and class =	355 N	lpa						
Flange compresion capacity =	792.7 ki	N >	201.56 kN	ОК				

Check Ground Beam GB5:

Refer to Robot use: 203x203x46 UC S355 Muls = from Robot= 39.0 kNm (from Blue Book) M/Mb = 0.29 < 1.0 OK Mb = (le = 3.5m) = 135 kNm def (from Robot) 1 mm L/def = 4200 > 360 OK

GB1 Steel beam						
Loadings:		ll kN/m2				
GF 150thk MD x 4.6m/2	2.3 x	7.3 =	16.8 kN/m			
Stud wall x 2.8m	2.8 x	0.9 =	2.5 kN/m			
swt			0.4			
	Sur	n sls:	19.7 kN/m			
l – man –	2.9 m					
L – Spail –	2.5 111					
Muls =	31.1 kNm		for 152x152	x30 UC \$355		
		Mb = (le = 3 m)		68.3 kNm	(from Blue Book)	ОК
Irea = (def<1/360)	1101 cm4		lxx =	1750 cm4	(from Blue Book)	
			def =	5.06 mm	L/def = 573	з ок
RsIs A =	28.6 kN	c anta Da		ta ustaininaall		
Kuis A=	42.9 KN	< 0110 Pa		to retaining wair		
GB2 Steel beam:						
Loadings:						
$GE 1E0 + bk MD \times E 4m/2$		FIL KN/M2	10.7 kN/m			
swt	2.7 X	7.5 -	0.9			
	Sur	n sls:	20.6 kN/m			
L = span =	6.7 m					
Muls =	173.5 kNm		for 254x254	x89 UC \$355		
		Mb = (le = 7 m)		285 kNm	(from Blue Book)	ОК
lrag = (dafs)/260)	14105 cm4		lvv –	14200 cm4	(from Plue Pook)	
lied – (dei<)/300)	14195 0114		def =	14300 cm4	L/def = 363	з ок
RsIs A =	69.0 kN					
Ruls A=	103.6 kN	< onto Pa	dstone extended	to retaining wall		
GB4 Steel beam below cavity wall on gL C						
Loadings:						
-	L DL+	HL kN/m2	DL+IL			
New RR x 3m/2	1.5 x	1.5 =	2.3 kN/m			
Internal leaf of blockwork x 5.0	5 x	2.1 =	10.5 kN/m			
GF 150thk MD x 0.4m	0.4 x	7.3 =	2.9 kN/m			
swt			0.3			
	Sur	n sls:	16.0 kN/m			
l = span =	3.0 m					
	510 111					
Muls =	26.9 kNm		for 200x90x	30PFC \$355		
		Mb = (le = 3 m)		52.6 kNm	(from Blue Book)	ОК
lreg = (def<1/360)	987 cm4		lxx =	2520 cm4	(from Blue Book)	
	567 6114		def =	3.26 mm	L/def = 920	о ок
RsIs A =	24.0 kN					
Ruls A=	35.9 kN	< Connec	ted to steel beam	is/ columns with m	in 2M16 8.8 bolts	
		Ps= 2 x 58.	9kN= 117.8kN			

Cavity wall x 3.1

Cavity wall x 3.1

Swt included

Loadings w3 (3.2m-3.9m):

CB-1/3m=

Swt included

GB4 Steel beam below cavity wall on gl. B:										
Loadings:										
	L	DL+IL	kN/m2		DL+IL					
Internal leaf of blockwork x 5.6	5.6	х	2.1	=	11.8 kN/m					
GF 150thk MD x 0.4m	0.4	х	7.3	=	2.9 kN/m					
swt					0.3					
		Sum s	ils:		15.0 kN/m					
L = span =	2.0 ו	m								
Muls =	11.2	kNm			for 200x90x3	30PFC \$355				
			Mb = (le = 2 m)		739 kNm	(from Blue	e Book)	ОК	
lreg = (def <l 360)<="" td=""><td>274 (</td><td>cm4</td><td></td><td></td><td>lxx =</td><td>2520 cm4</td><td>(from Blue</td><td>e Book)</td><td></td><td></td></l>	274 (cm4			lxx =	2520 cm4	(from Blue	e Book)		
					def =	0.60 mm	L/def =	3311	ОК	
Rsls A =	15.0 l	٨N								
Ruls A=	22.5 I	κN	< F	< Conne Ps= 2 x 58	cted to steel beams .9kN= 117.8kN	s/ columns with	n min 2M16 8.8	bolts		
GB3 - Steel beam										
Loadings w1:										
	L		DL+IL		DL+IL		F1	F1 F3		F2
GF 150thk MD x 0.4m	0.4	х	7.3	=	2.9 kN/m		L	w2		
Swt included					2.0.1.1./					
		Sum s	SIS:		2.9 kN/m		w1	• •		
Loadings w2 $(0-1.6m)$:							<u>م</u> 1	B	1	1
Loudings w2 (0-1.011).	L		DL+IL		DL+IL		~ I	U	1	~1

4.1 =

DL+IL

4.1

=

12.7 kN/m

11.7 kN/m

24.4 kN/m

12.7 kN/m

12.7 kN/m

DL+IL

F1= GB1 and GB2 at 3.2m and 0.8m=	97.6 kN	
F2= GB1 and 1B3 at 5.3m=	83.0 kN	
F3= 1B3 at 1.6m=	83.5 kN	
L (A-B)=	1.9 m	
L (B-C)=	3.9 m	
Rsls A (from Tedds)=	47.8 kN	onto padstone extended to basement
Dela D. (france Tandala)	200.2 - 1-14	Ortho C1
RSIS B (from fedus) =	289.3 KN	UNIO CI
Rsls C(from Tedds)=	92.0 kN	onto padstone extended to basement
	52.0 KN	onto publiche extended to basement

3.1 x

L

3.1 x

Sum sls:

Sum sls:

From Tedds calculation refer to next page min needed section is =

203UC46 S355

	port GBS												
s =			289).3 kN									
s =			434	1.0 kN									
10m = (0.1m + 0).15m/2) x GB3	/3	25	5.3 kNm									
152x152x37U	C \$355												
=			2	.8 m									
/ =			920).0 kN		(Blue bo	ok)						
) =			88	3.7 kNm									
eraction check	compression w	ith bendir	<u>ng</u>										
x/Mbs + Myy/N	vicy + F/Pcy =	r P	VIXX/IVIDS			0.29	_						
		F	=/Pcy =			0.00	-		0.76	< 1		OK	
		I	/1 Cy -			0.47			0.70	` I		ÖK	
										U	lse: 152x15	52x37UC S3	355
undation pad u	nder column C	1											
s =			289).3 kN									
d swt =		_	ç).0 kN									
n sls:			298	8.3 kN									
ea of foundation	ns = sum / 150	GBP		=	1.99	9 m2			Bmin =	1.41 m	า		
e B = A =				=	1.	5 m					-		
ea				=	2.25	5 m2	ОК						
Concrete exter	nal lintel												
clear span =			1	9 m									
			L	/H DI	L+IL kN/mi	2	DL+IL						
ernal brick/bloc	ck x 0.4		L, C	/H DI).4 x	L+IL kN/m2 2.0	2 0 =	DL+IL 0.8	kN/m					
ernal brick/bloc w RR x 4.1/2	ck x 0.4	_	L, C 2	/H DI).4 x 2.1 x	L+IL kN/m2 2.0 1.5	2 0 = 5 =	DL+IL 0.8 3.1	kN/m kN/m					
ernal brick/bloc w RR x 4.1/2	ck x 0.4	_	L, () 2	/H DI 0.4 x 2.1 x	L+IL kN/m2 2.(1.! Sum sls	2 0 = 5 =	DL+IL 0.8 3.1 3.9	kN/m kN/m kN/m					
ernal brick/bloc w RR x 4.1/2	ck x 0.4 Section	P100	L, C 2 P150	/H DI).4 x ?.1 x P220	L+IL kN/m2 2.0 1.5 Sum sls P255	2 0 = 5 = :: 510	DL+IL 0.8 3.1 3.9 R15	kN/m kN/m kN/m R15A	R22	R22A	\$15	R21	R21A
ernal brick/bloc w RR x 4.1/2 Manufacture	ck x 0.4 Section Profile	P100 65 x 100	L, C 2 P150 65 x 150	/H DI 0.4 x 2.1 x P220 65 x 220	L+IL kN/m2 2.(1.! Sum sls P255 65 x 255	2 0 = 5 = :: 510 100 x 100	DL+IL 0.8 3.1 3.9 R15 100 x 140	kN/m kN/m kN/m R15A 140 x 100	R22 100 x 215	R22A 215 x 100	515 150 x 140	R21 140 x 215	R21A 215 x 140
ernal brick/bloc w RR x 4.1/2 Manufacture size to order	Section Profile	P100 65 x 100 Service N	L, C 2 P150 65 x 150 Ioment (kN	/H DI 0.4 x 2.1 x P220 65 x 220 m)	L+IL kN/m2 2.(1.) Sum sls P255 65 x 255	2 = 5 = 5 = 5 = 5 = 5 = 5 = 5 = 5 = 5 =	DL+IL 0.8 3.1 3.9 R15 100 x 140	kN/m <u>kN/m</u> R15A 140 x 100	R22 100 x 215	R22A 215 x 100	515 150 x 140	R21 140 x 215	R21A 215 x 140
ernal brick/bloc w RR x 4.1/2 Manufacture size to order	Section Profile CLEAR SPAN	P100 65 x 100 Service N 0.96	L, C 2 P150 65 x 150 foment (kN 1.06	/H DI 0.4 x 2.1 x P220 65 x 220 m) 2.61	L+IL kN/m2 2.(1.) Sum sls P255 65 x 255 3.09	2 = 5 = 5 = 5 = 5 = 5 = 5 = 5 = 5 = 5 =	DL+IL 0.8 3.1 3.9 R15 100 x 140 2.79	kN/m kN/m R15A 140 x 100 4.82	R22 100 x 215 5.88	R22A 215 x 100 12.47	S15 150 x 140 8.61	R21 140 x 215 8.08	R21A 215 x 140 13.33
ernal brick/bloc w RR x 4.1/2 Manufacture size to order 600	Section Profile CLEAR SPAN 300	P100 65 x 100 Service N 0.96 37.75	L, 2 P150 65 x 150 foment (kN 1.06 37.75	/H DI 0.4 x 2.1 x P220 65 x 220 m) 2.61 70.68	L+IL kN/m2 2.(1.3 Sum sls P255 65 x 255 3.09 91.44	2 0 = 5 = :: 510 100 x 100 1.60 50.67	DL+IL 0.8 3.1 3.9 R15 100 x 140 2.79 71.24	kN/m kN/m R15A 140 x 100 4.82 117.11	R22 100 x 215 5.88 152.89	R22A 215 x 100 12.47 210.67	S15 150 x 140 8.61 110.56	R21 140 x 215 8.08 180.00	R21A 215 x 140 13.33 280.00
ernal brick/bloc w RR x 4.1/2 Manufacture size to order 600 750	Section Profile CLEAR SPAN 300 450	P100 65 x 100 Service N 0.96 37.75 21.33	L, C 2 P150 65 x 150 Ioment (kN 1.06 37.75 23.56	/H DI 0.4 x 2.1 x P220 65 x 220 m) 2.61 70.68 53.00	L+IL kN/m2 2.(1.1 Sum sls P255 65 x 255 3.09 91.44 68.58	2 = 5 = 5 = 5 = 5 = 5 = 5 = 5 = 5 = 5 =	DL+IL 0.8 3.1 3.9 R15 100 x 140 2.79 71.24 53.43	kN/m kN/m R15A 140 x 100 4.82 117.11 87.83	R22 100 x 215 5.88 152.89 114.67	R22A 215 x 100 12.47 210.67 158.00	515 150 x 140 8.61 110.56 82.92	R21 140 x 215 8.08 180.00 135.00	R21A 215 x 140 13.33 280.00 210.00
ernal brick/bloc w RR x 4.1/2 Manufacture size to order 600 750 900	Section Profile CLEAR SPAN 300 450 600	P100 65 x 100 Service N 0.96 37.75 21.33 13.65	P150 65 x 150 10ment (kN 1.06 37.75 23.56 15.08	/H DI 0.4 x 2.1 x P220 65 x 220 m) 2.61 70.68 53.00 37.12	L+IL kN/m2 2.(1.1, Sum sls 9255 65 x 255 3.09 91.44 68.58 43.95	2 = 5 = 5 = 5 = 5 = 5 = 5 = 5 = 5 = 5 =	DL+IL 0.8 3.1 3.9 R15 100 x 140 2.79 71.24 53.43 39.68	kN/m kN/m R15A 140 x 100 4.82 117.11 87.83 68.55	R22 100 x 215 5.88 152.89 114.67 83.63	R22A 215 x 100 12.47 210.67 158.00 126.40	515 150 x 140 8.61 110.56 82.92 66.33	R21 140 x 215 8.08 180.00 135.00 108.00	R21A 215 x 140 13.33 280.00 210.00 168.00
ernal brick/bloc w RR x 4.1/2 Manufacture size to order 600 750 900 1050	Section Profile CLEAR SPAN 300 450 600 750	P100 65 x 100 Service N 0,96 37.75 21.33 13.65 9.48	L, C 2 P150 65 x 150 10ment (kN 1.06 37.75 23.56 15.08 10.47	/H DI 0.4 x 2.1 x P220 65 x 220 m) 2.61 70.68 53.00 37.12 25.78	L+IL kN/m2 2.(1.1 Sum sls 9255 65 x 255 3.09 91.44 68.58 43.95 30.52	2 = 5 = 5 = 5 = 5 = 5 = 5 = 5 = 5 = 5 =	DL+IL 0.8 3.1 3.9 R15 100 x 140 2.79 71.24 53.43 39.68 27.56	kN/m kN/m R15A 140 x 100 4.82 117.11 87.83 68.55 47.60	R22 100 x 215 5.88 152.89 114.67 83.63 58.07	R22A 215 x 100 12.47 210.67 158.00 126.40 105.33	515 150 x 140 8.61 110.56 82.92 66.33 55.28	R21 140 x 215 8.08 180.00 135.00 108.00 79.80	R21A 215 x 140 13.33 280.00 210.00 168.00 131.65
ernal brick/bloc w RR x 4.1/2 Manufacture size to order 600 750 900 1050 1200	Section Profile CLEAR SPAN 300 450 600 750 900	P100 65 x 100 Service N 0.96 37.75 21.33 13.65 9.48 6.97	P150 65 x 150 65 x 150 100ment (kN 1.06 37.75 23.56 15.08 10.47 7.69	/H DI 0.4 x 2.1 x P220 65 x 220 m) 2.61 70.68 53.00 37.12 25.78 18.94	L+IL kN/m2 2.(1., Sum sls 9255 65 x 255 3.09 91.44 68.58 43.95 30.52 22.42	2 = 5 = 2 5	DL+IL 0.8 3.1 3.9 R15 100 x 140 2.79 71.24 53.43 39.68 27.56 20.24	kN/m kN/m R15A 140 x 100 4.82 117.11 87.83 68.55 47.60 34.98	R22 100 x 215 5.88 152.89 114.67 83.63 58.07 42.67	R22A 215 x 100 12.47 210.67 158.00 126.40 105.33 90.29	515 150 x 140 8.61 110.56 82.92 66.33 55.28 47.38	R21 140 x 215 8.08 180.00 135.00 108.00 79.80 58.63	R21A 215 x 140 13.33 280.00 210.00 168.00 131.65 96.73
ernal brick/bloc w RR x 4.1/2 Manufacture size to order 600 750 900 1050 1050 1200 1350	Section Profile CLEAR SPAN 300 450 600 750 900 1050	P100 65 x 100 Service N 0,96 37.75 21.33 13.65 9.48 6.97 5.33	L, C 2 P150 65 x 150 65 x 150 65 x 150 10.47 1.06 37.75 23.56 15.08 10.47 7.69 5.89	/H DI 0.4 x 2.1 x P220 65 x 220 m) 2.61 70.68 53.00 37.12 25.78 18.94 14.50	L+IL kN/m2 2.(1.) Sum sls P255 65 x 255 3.09 91.44 68.58 43.95 30.52 22.42 17.17	2 = 5 5 = 2 5 = 2	DL+IL 0.8 3.1 3.9 R15 100 x 140 2.79 71.24 53.43 29.68 27.56 20.24 15.50	kN/m kN/m R15A 140 x 100 4.82 117.11 87.83 68.55 47.60 34.98 26.78	R22 100 x 215 5.88 152.89 114.67 83.63 58.07 42.67 32.67	R22A 215 x 100 712.47 210.67 158.00 126.40 105.33 90.29 69.28	515 150 x 140 8.61 110.56 82.92 66.33 55.28 47.38 41.46	R21 140 x 215 8.08 180.00 135.00 108.00 79.80 58.63 44.89	R21A 215 x 140 13.33 280.00 210.00 168.00 131.65 96.73 74.06
ernal brick/bloc w RR x 4.1/2 Manufacture size to order 600 750 900 1050 1200 1350 1500	Section Profile CLEAR SPAN 300 450 600 750 900 1050 1200	P100 65 x 100 Service N 0.96 37.75 21.33 13.65 9.48 6.97 5.33 4.21	L, C 2 P150 65 x 150 65 x 150 65 x 150 1.06 37.75 2.3.56 15.08 10.47 7.69 5.89 4.65	/H DI 0.4 x 2.1 x P220 65 x 220 m) 2.61 70.68 53.00 37.12 25.78 18.94 14.50 11.46	L+IL kN/m2 2.(1.) Sum sls 9255 65 x 255 3.09 91.44 68.58 43.95 30.52 22.42 17.17 13.56	2 = 5 5 = 2 5 = 2	DL+IL 0.8 3.1 3.9 100 x 140 2.79 71.24 53.43 27.56 20.24 15.50	kN/m kN/m R15A 140 x 100 4.82 117.11 87.83 68.55 47.60 34.98 2.6.78 2.1.16	R22 100 x 215 5.88 152.89 114.67 83.63 58.07 42.67 32.67 25.81	R22A 215 x 100 712.47 210.67 158.00 126.40 105.33 90.29 69.28 54.74	S15 150 x 140 8.61 110.56 82.92 66.33 55.28 47.38 41.46 36.85	R21 140 x 215 8.08 180.00 135.00 108.00 58.63 44.89 35.47	R21A 215 x 140 13.33 280.00 210.00 168.00 131.65 96.73 74.06 58.51
ernal brick/bloc w RR x 4.1/2 Manufacture size to order 600 750 900 1050 1050 1200 1350 1500 1650	Section Profile CLEAR SPAN 300 450 600 750 900 1050 1200 1350	P100 65 x 100 Service N 0.96 37.75 21.33 13.65 9.48 6.97 5.33 4.21 3.41	P150 65 x 150 65 x 150 1.06 37.75 23.56 15.08 10.47 7.69 5.89 4.65 3.77	/H Dl 0.4 x 2.1 x P220 65 x 220 m) 2.61 70.68 53.00 37.12 25.78 18.94 14.50 11.46 9.28	L+IL kN/m2 2.(1.) Sum sls P255 65 x 255 3.09 91.44 68.58 43.95 30.52 22.42 17.17 13.56 10.99	2 = 5 = 5 = 100 x 100 100 x 100 50.67 35.56 22.76 15.80 11.61 8.89 7.02 5.69	DL+IL 3.1 3.3 100 x 140 2.79 71.24 53.43 39.68 20.24 15.50 12.25 9.92	kN/m kN/m 140 x 100 4.82 117.11 87.83 68.55 47.60 34.98 26.78 21.16 17.14	R22 100 x 215 5.88 152.89 114.67 83.63 58.07 42.67 32.67 25.81 20.91	R22A 215 x 100 12.47 210.67 158.00 126.40 105.33 90.29 69.28 54.74 44.34	S15 150 x 140 8.61 110.56 82.92 66.33 55.28 47.38 41.46 36.85 30.61	R21 140 x 215 8.08 180.00 135.00 108.00 58.63 44.89 35.47 28.73	R21A 215 x 140 13.33 280.00 210.00 168.00 131.65 96.73 74.06 58.51 47.40
ernal brick/bloc w RR x 4.1/2 Manufacture size to order 600 750 900 1050 1050 1200 1350 1350 1500 1650 1800	Section Profile CLEAR SPAN 300 450 600 750 900 1050 1200 1350 1500	P100 65 x 100 Service N 0.96 37.75 21.33 13.65 9.48 6.97 5.33 4.21 3.41 2.82	P150 65 x 150 65 x 150 100ment (kN 1.06 37.75 23.56 15.08 10.47 7.69 5.89 4.65 3.77 3.11	 /H DI .4 x .1 x P220 65 x 220 m) 2.61 70.68 53.00 37.12 25.78 18.94 14.50 11.46 9.28 7.67 	+IL kN/m2 2.(1.) 2.5 2.5 65 x 255 3.09 91.44 68.58 43.95 30.52 22.42 17.17 13.56 10.99 9.08	2 = 5 = 5 = 100 x 100 50.67 35.56 22.76 15.80 11.61 8.89 7.02 5.69 4.70	DL+IL 3.1 3.3 8 7 100 x 140 7 2.79 7 1.24 5 3.43 3 9.68 20.24 15.50 12.55 12.55 12.55 9.92 8.20	kN/m kN/m R15A 140 x 100 4.82 117.11 87.83 68.55 47.60 34.98 26.78 21.16 21.16 17.14 14.16	R22 100 x 215 5.88 152.89 114.67 83.63 58.07 42.67 32.67 25.81 20.91 17.28	R22A 215 x 100 12.47 210.67 158.00 126.40 105.33 90.29 69.28 54.74 44.34 36.664	S15 150 x 140 8.61 110.56 82.92 66.33 55.28 47.38 41.46 36.85 30.61 25.30	R21 140 x 215 8.08 180.00 135.00 108.00 58.63 44.89 35.47 28.73 23.74	R21A 215 x 140 13.33 280.00 210.00 168.00 131.65 96.73 74.06 58.51 47.40 39.17
ernal brick/bloc w RR x 4.1/2 Manufacture size to order 600 750 900 1050 1050 1050 1350 1350 1350 1360 1650 1800 2100	Section Profile CLEAR SPAN 300 450 600 750 900 1050 1200 1350 1500 1650	P100 65 x 100 Service N 0,96 37.75 21.33 13.65 9.48 6.97 5.33 4.21 3.41 2.82 2.37	P150 65 x 150 10ment (kN 1.06 37.75 23.56 15.08 10.47 7.69 5.89 4.65 3.77 3.11 2.62	/H DI 0.4 x x.1 x P220 65 x 220 m) 2.61 70.68 53.00 37.12 25.78 18.94 14.50 11.46 9.28 7.67 6.44	+IL kN/m2 2.(1.) 2.5 50m sls 65 x 255 3.09 91.44 68.58 43.95 30.52 22.42 17.17 13.56 10.99 9.08 7.63	2 = 5 = 5 = 100 x 100 50.67 35.56 22.76 15.80 11.61 8.89 7.02 5.69 4.70 3.95	DL+IL 3.1 3.3 8 7 100 x 140 7 2.79 7 1.24 5.34 3.9.68 2.7.56 20.24 15.50 12.55 15.50 12.55 9.92 8.20 6.89	kN/m kN/m 140 x 100 4.82 117.11 87.83 68.55 47.60 34.98 26.78 21.16 17.14 14.16 11.90	R22 100 x 215 5.88 152.89 114.67 83.63 58.07 42.67 32.67 25.81 20.91 17.28 14.52	R22A 215 x 100 215,x 100 12.47 210.67 158.00 126.40 105.33 90.29 69.28 54.74 44.34 36.64 30.79	S15 150 x 140 8.61 110.56 82.92 66.33 55.28 47.38 41.46 36.85 30.61 25.30 21.26	R21 140 x 215 8.08 180.00 135.00 108.00 58.63 44.89 35.47 28.73 23.74 19.95	R21A 215 x 140 13.33 280.00 210.00 168.00 31.65 96.73 74.06 58.51 47.40 39.17 32.91
ernal brick/bloc w RR x 4.1/2 Manufacture size to order 600 750 900 1050 1050 1050 1350 1350 1350 1350 13	Section Profile CLEAR SPAN 300 450 600 750 900 1050 1200 1350 1500 1650 1800	P100 65 x 100 Service N 0.96 37.75 21.33 13.65 9.48 6.97 5.33 4.21 3.41 2.82 2.37 2.82 2.37	P150 65 x 150 60ment (kN 1.06 37.75 23.56 15.08 10.47 7.69 5.89 4.65 3.77 3.11 2.62 2.23	/H DI 0.4 x x.1 x P220 65 x 220 m) 2.61 70.68 53.00 37.12 25.78 18.94 14.50 11.46 9.28 7.67 6.44 5.49	L+IL kN/m2 2.(1.) 2.000 2.000 2.255 3.09 91.44 68.58 43.95 30.52 22.42 17.17 13.56 10.99 9.08 7.63 6.50	2 = 5 = 5 = 5 = 5 = 5 = 5 = 5 =	DL+IL 3.1 3.3 8 7 100 x 140 7 2.79 7 1.24 5.34 3.9,68 2.7,56 2.0,24 15,50 12,25 9.92 8,20 6,89 5.87	kN/m kN/m R15A 140 x 100 4.82 117.11 87.83 68.55 47.60 34.98 26.78 21.16 17.14 14.16 11.90 10.14	R22 100 x 215 5.88 152.89 114.67 83.63 58.07 42.67 32.67 25.81 20.91 17.28 14.52	R22A 215 x 100 215, x 100 12,47 210,67 158,00 126,40 105,33 90,29 69,28 54,74 44,34 36,64 30,79 26,24	S15 150 x 140 8.61 110.56 82.92 66.33 55.28 47.38 41.46 36.85 30.61 25.30 21.26 18.11	R21 140 x 215 8.08 180.00 135.00 108.00 35.47 28.73 23.74 19.95 17.00	R21A 215 x 140 13.33 280.00 210.00 168.00 31.65 96.73 74.06 58.51 47.40 39.17 32.91 28.04

WP windpost check with EA: H = wind post height

	winita	post	neg

Wind load

p = wind load =	0.7 kN/m2
L = width of wind load = 3.6m/2 =	1.8 m
q = (p x L)/2 =	0.6 kN/m
Vuls = 1.4 x q x H x 0.5	1.3 kN
Muls,1 = 1.4 x q x H^2 / 8 =	1.0 kNm

3.0 m

From Tedds use 120x120x10 EA S275

Tekla Tedds	Project	85 Camo	den Mews		Job no.	5005
	Calcs for	F	R		Start page no./F	Revision 1
	Calcs by KK	Calcs date 04/12/2018	Checked by AP	Checked date 04/12/2018	Approved by	Approved date



Breadth of timber sections	b = 47 mm
Depth of timber sections	h = 150 mm
Rafter spacing	s = 400 mm
Rafter slope	α = 26.0 deg
Clear span of rafter on horizontal	L _{clh} = 2950 mm
Clear span of rafter on slope	$L_{cl} = L_{clh} / \cos(\alpha) = 3282 \text{ mm}$
Rafter span	Single span
Timber strength class	C24
Section properties	
Cross sectional area of rafter	A = b × h = 7050 mm ²
Section modulus	Z = b × h ² / 6 = 176250 mm ³
Second moment of area	I = b × h ³ / 12 = 13218750 mm ⁴
Radius of gyration	r = √(I / A) = 43.3 mm
Loading details	
Rafter self weight	$F_{j} = b \times h \times \rho_{char} \times g_{acc} = 0.02 \text{ kN/m}$
Dead load on slope	F _d = 0.60 kN/m ²
Imposed load on plan	F _u = 0.60 kN/m ²
Imposed point load	F _p = 0.90 kN
Modification factors	
Section depth factor	K ₇ = (300 mm / h) ^{0.11} = 1.08
Load sharing factor	K ₈ = 1.10
Consider long term load condition	
Load duration factor	K ₃ = 1.00
Total UDL perpendicular to rafter	$F = F_{d} \times \cos(\alpha) \times s + F_{j} \times \cos(\alpha) = \textbf{0.237} \text{ kN/m}$
Notional bearing length	$L_{b} = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_{8} - F)] = 3 mm$
Effective span	L _{eff} = L _{cl} + L _b = 3285 mm
Check bending stress	
Bending stress parallel to grain	σ _m = 7.500 N/mm ²

	Project	Project Job no. 85 Camden Mews 15005					
	Calcs for				Start page no./	Revision	
		RR				2	
	Calcs by KK	Calcs date 04/12/2018	Checked by AP	Checked date 04/12/2018	Approved by	Approved date	
Permissible bending stress		$\sigma_{m_{adm}} = \sigma_{m}$	$_{1} \times K_{3} \times K_{7} \times K_{8}$	3 = 8.904 N/mm ²			
Applied bending stress		$\sigma_{m_{max}} = F$:	$\times L_{eff}^2 / (8 \times Z)$	= 1.818 N/mm ²			
			PASS - Applie	ed bending stres	s within peri	missible limits	
Check compressive stress	s parallel to grain						
Compression stress paralle	l to grain	σc = 7.900	N/mm ²				
Minimum modulus of elastic	bity	E _{min} = 7200) N/mm²				
Compression member facto	r	K ₁₂ = 0.55					
Permissible compressive st	ress	$\sigma_{c_{adm}} = \sigma_{c}$	\times K ₃ \times K ₈ \times K ₁₂	2 = 4.805 N/mm ²			
Applied compressive stress		$\sigma_{c_{max}} = F \times$	$< L_{eff} imes (cot(lpha) +$	⊦ 3 × tan(α)) / (2 ×	A) = 0.194 N	l/mm²	
		PAS	S - Applied co	ompressive stres	s within peri	missible limits	
Check combined bending	and compressive s	stress parallel	to grain				
Euler stress		$\sigma_{e} = \pi^{2} \times E_{r}$	min / λ^2 = 12.34	5 N/mm ²			
Euler coefficient		K _{eu} = 1 – (1	$1.5 \times \sigma_{c_{max}} \times K$	₁₂ / σ _e) = 0.987			
Combined axial compressio	n and bending chec	k σ _{m max} /(σ _m	$_{\rm adm} \times K_{\rm eu}$) + $\sigma_{\rm eu}$	c max / oc adm = 0.2	47 < 1		
· ·	PASS - Com	bined compres	ssive and ben	ding stresses ar	e within peri	missible limits	
Check shear stress							
Shear stress parallel to grai	n	τ = 0 710 Ν	l/mm ²				
Permissible shear stress		$\tau = \tau \times \mathbf{k}$	$\tau_{\rm odm} = \tau \times K_0 \times K_0 = 0.781 \text{N/mm}^2$				
			$3 \times 10^{\circ} = 0.701$	-0.093 N/mm ²			
Applied Silear Siless		lmax – 3 × Γ		- 0.003 N/IIIII-	e within nor	missible limits	
			FA33 - Apj	oneu snear stres	s within peri		
Check deflection							
Permissible deflection		$\delta_{adm} = 0.00$	3 × L _{eff} = 9.856	6 mm			
Bending deflection		$\delta_b = 5 \times F >$	$< L_{eff}^4$ / (384 \times E	E _{mean} × I) = 2.523 ∣	mm		
Shear deflection		$\delta_s = 12 \times F$	\times L _{eff} ² / (5 \times E _r	_{mean} × A) = 0.081 r	nm		
Total deflection		$\delta_{max} = \delta_b + \delta_b$	δ _s = 2.604 mm				
			PASS	- Total deflectio	n within peri	missible limits	
Consider medium term loa	ad condition						
Load duration factor		K₃ = 1.25					
Total UDL perpendicular to	rafter	$F = [F_u \times columnwidth{columnwidth}]$	$cos(\alpha)^2 + F_d \times cos(\alpha)^2$	$\mathbf{ps}(\alpha)$] × s + F _j × co	os(α) = 0.431	kN/m	
Notional bearing length		$L_{b} = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_{8} - F)] = 6 \text{ mm}$					
Effective span		$L_{eff} = L_{cl} + L_{cl}$	_b = 3288 mm				
Check bending stress							
Bending stress parallel to g	rain	σm = 7,500	N/mm ²				
Permissible bending stress			× K2 × K7 × K	a = 11 130 N/mm ²			
Applied bending stress		$\sigma_{m,aam} = \sigma_{m} \wedge r_{3} \wedge r_{7} \wedge r_{8} = 11.130 \text{ N/mm}^{2}$					
Applied bending stress		Om_max - F	$\times Leff / (0 \times Z)$	- 3.307 N/IIIII	s within nor	missihla limits	
			, 200 - Abbile	a benuing sues	s wiann peri		
Check compressive stress	s parallel to grain		N1/				
Compression stress paralle	l to grain	σ _c = 7.900	N/mm²				
Minimum modulus of elastic	city	Emin = 7200) N/mm ²				
Compression member facto	r	K ₁₂ = 0.51					
Permissible compressive st	ress	$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = 5.508 \text{ N/mm}^2$					
Applied compressive stress		$\sigma_{c_{max}} = F \times$	$< L_{eff} \times (cot(\alpha) +$	+ 3 × tan(α)) / (2 ×	A) = 0.353 N	l/mm ²	
		PAS	S - Applied co	mpressive stres	s within peri	missible limits	

Tedds	Project 85 Camden Mews				Job no.	5005
С	alcs for		2B		Start page no./I	Revision
с	Calcs by Calcs date Checked by Checked dat			Checked date	Approved by	Approved date
	<u>NN</u>	04/12/2016	AF	04/12/2018		
Check combined bending and c	ompressive s	tress parallel	to grain	E N/mm²		
		$O_e = \pi^- \times E_i$	$\min / \lambda^{-} - 12.32$			
		$\kappa_{eu} = 1 - (1$	$1.5 \times \sigma_{c_{max}} \times K$	₁₂ / σ _e) = 0.978	00 44	
Combined axial compression and	PASS - Com	c σ _{m_max} / (σ _m	₁_ _{adm} × K _{eu}) + σ₀ ssive and ben	c_max / σc_adm = 0.3 ding stresses ar	68 < 1 re within peri	nissible limit.
Check shear stress					/	
Shear stress parallel to grain		τ = 0.710 Ν	l/mm ²			
Permissible shear stress		$\tau_{adm} = \tau \times K$	Հ₃ × Kଃ = 0.976	N/mm ²		
Applied shear stress		$\tau_{max} = 3 \times F$	$ = \times L_{eff} / (4 \times A) $	= 0.151 N/mm ²		
			PASS - App	olied shear stres	s within peri	nissible limit
Check deflection						
Permissible deflection		δ_{adm} = 0.00	3 × L _{eff} = 9.864	mm		
Bending deflection		$\delta_b = 5 \times F >$	imes L _{eff} ⁴ / (384 $ imes$ E	E _{mean} × I) = 4.597	mm	
Shear deflection		$\delta_s = 12 \times F$	imes L _{eff} ² / (5 $ imes$ E _n	nean × A) = 0.147 r	mm	
Total deflection		$\delta_{max} = \delta_b +$	δs = 4.744 mm			
			PASS	- Total deflectio	n within peri	nissible limit
Consider short term load condit	tion					
Load duration factor		K₃ = 1.50				
Total UDL perpendicular to rafter		$F = F_d \times co$	$s(\alpha) \times s + F_j \times$	cos(α) = 0.237 kM	N/m	
Notional bearing length		$L_b = [F \times L_c$	$+ F_{p} \times cos(\alpha)$	/ [2 × (b × σ_{cp1} ×	K ₈ - F)] = 6 m	m
Effective span		L _{eff} = L _{cl} + L	_{-b} = 3289 mm			
Check bending stress						
Bending stress parallel to grain		σ _m = 7.500	N/mm ²			
Permissible bending stress		$\sigma_{m_{adm}} = \sigma_{m}$	$_{1} imes K_{3} imes K_{7} imes K_{8}$	= 13.355 N/mm ²		
Applied bending stress		$\sigma_{m_{max}} = F_{\times}$:L _{eff} ²/(8×Z)+F _p ×	$\cos(\alpha) \times L_{eff}/(4 \times Z)$	= 5.595 N/mr	n²
			PASS - Applie	ed bending stres	s within peri	nissible limit
Check compressive stress para	llel to grain					
Compression stress parallel to gra	ain	σc = 7.900	N/mm²			
Minimum modulus of elasticity		E _{min} = 7200) N/mm²			
Compression member factor		K ₁₂ = 0.46				
Permissible compressive stress		$\sigma_{c_{adm}} = \sigma_{c}$	\times K ₃ \times K ₈ \times K ₁₂	= 6.057 N/mm ²		
Applied compressive stress		o _{c_max} = F× ₽AS	L _{eff} ×(cot(α)+3× S - Applied co	tan(α))/(2×A)+F _p × <i>mpressive stres</i>	sin(α)/A = 0.2 s within per i	251 N/mm² missible limit
Check combined bending and c	ompressive s	tress parallel	to grain			
Euler stress		$\sigma_e = \pi^2 \times E_e$	_{min} / λ ² = 12.32	0 N/mm ²		
Euler coefficient		K _{eu} = 1 – (1	$1.5 \times \sigma_{c_{max}} \times K_{c}$	12 / σe) = 0.986		
Combined axial compression and	bending check	κ σ _{m_max} / (σ _m	$_{\rm adm} imes {\sf K}_{ m eu}$) + $\sigma_{ m eu}$	/ ocadm = 0.4	66 < 1	
	PASS - Com	bined compres	ssive and ben	ding stresses ar	e within peri	nissible limit
Check shear stress						
Shear stress parallel to grain		τ = 0.710 Ν	I/mm ²			
Permissible shear stress		τ_{adm} = $\tau imes k$	K ₃ × K ₈ = 1.172	N/mm ²		
Applied shear stress		τ_{max} = 3 × F	$E \times L_{eff} / (4 \times A)$	+ $3 \times F_p \times \cos(\alpha)$	$/(2 \times A) = 0.$	255 N/mm² nissible limit

Tekla Tedds	Project	Job no. 15	005				
	Calcs for RR					Start page no./Revision 4	
	Calcs by KK	Calcs date 04/12/2018	Checked by AP	Checked date 04/12/2018	Approved by	Approved date	

Check deflection

Permissible deflection Bending deflection

Shear deflection

Total deflection

 δ_{adm} = 0.003 \times L_{eff} = **9.866** mm

 $\delta_{\text{b}} = \text{ L}_{\text{eff}}^{3} \times (5 \times F \times L_{\text{eff}} / 384 + F_{\text{p}} \times cos(\alpha) / 48) / (E_{\text{mean}} \times I) = \textbf{6.731} \text{ mm}$

 $\delta_{s} = 12 \times L_{eff} \times (F \times L_{eff} + 2 \times F_{p} \times cos(\alpha))/(5 \times E_{mean} \times A) = 0.249 \text{ mm}$

 δ_{max} = δ_{b} + δ_{s} = 6.980 mm

PASS - Total deflection within permissible limits

AXIOM	Project	85 Cam	Job no.	5005			
STRUCTURES	Calcs for 5				Start page no./Revision		
		H	B2			1	
	Calcs by KL	Calcs date 04/12/2018	Checked by AP	Checked date 04/12/2018	Approved by	Approved date	
					1		
STEEL BEAM ANALYSIS &	DESIGN (BS59	<u>50)</u>					
In accordance with BS5950)-1:2000 incorpo	orating Corrigend	lum No.1		TEDDS calcula	ation version 3 0 05	
		l oad Envelope - Com	bination 1				
3.750		Load Livelope - oon			I		
0.0 J		2	200		<u>*</u>		
A			1		J B		
kNm		Bending Moment E	nvelope				
0.0							
2.993			3.0				
mm [2	1		ј В		
		Shear Force Env	elope				
^{KN} 3.6			·				
0.0-							
-3.566 J mm J		2	200		-3.6		
A			1		B		
Support conditions		Vertically r	estrained				
Support A		Rotationall	v free				
Support B		Vertically r	estrained				
		Rotationall	y free				
Applied loading							
Beam loads		Imposed se	elf weight of be	eam × 1			
		Imposed fu	III UDL 0.8 kN	/m			
		Imposed p	oint load 2.5 k	N at 0 mm			
		Imposed p	oint load 2.5 k	N at 1100 mm			
		imposed p	oint 10ad 2.5 k	in at 2200 mm			
Load combinations		0			4 50		
Load combination 1		Support A		Dead ×	1.50		

ad combination 1	Support A	$Dead \times 1.50$
		Imposed \times 1.50
	Span 1	$\text{Dead}\times 1.50$
		Imposed \times 1.50
	Support B	$\text{Dead}\times 1.50$

AXIOM	Project	85 Cam	den Mews		Job no.	5005	
CTRUCTURES	Color for		Start page pg //				
STRUCTURES	Calcs for	F	RB2		Start page no./F	2	
	Calcs by	Calcs date	Checked by	Checked date	Approved by		
	KL	04/12/2018	AP	04/12/2018	rippioved by	, pproved date	
				Impose	$d \times 1.50$		
Analysis results							
Maximum moment		M _{max} = 3 k	Nm	M _{min} = 0	0 kNm		
Maximum shear		V _{max} = 3.6	kN	V _{min} = -	3.6 kN		
Deflection		δ _{max} = 0.3	mm	$\delta_{min} = 0$	mm		
Maximum reaction at support	A	R _{A_max} = 7.	3 kN	R _{A_min} =	- 7.3 kN		
Unfactored imposed load reac	tion at support A	R _{A_Imposed} =	4.9 kN				
Maximum reaction at support	В	R _{B_max} = 7.	3 kN	R _{B_min} =	= 7.3 kN		
Unfactored imposed load read	tion at support B	R _{B_Imposed} =	4.9 kN				
Section details							
Section type	UC 152x152x2	3 (BS4-1)	Steel grade		S355		
Classification of cross secti	ons - Section 3.	5					
Tensile strain coefficient	ε = 0.88		Section class	ification	Semi-compa	act	
Shear capacity - Section 4.2	.3						
Design shear force	F _v = 3.6 kN		Design shear	resistance	P _v = 188.3 ki	N	
		PAS	SS - Design sh	hear resistance e	xceeds desig	n shear force	
Moment capacity - Section 4	.2.5						
Design bending moment	M = 3 kNm		Moment capa	acity low shear	M _c = 60.5 kN	m	
Buckling resistance momen	t - Section 4.3.6.	4					
Buckling resistance moment	M _b = 48.8 kNm		M _b / m _{LT} = 48	.8 kNm			
		PASS - Buckli	ing resistance	e moment exceed	ls design ber	nding moment	
Check vertical deflection - S	ection 2.5.2						
Consider deflection due to dea	ad and imposed lo	bads					
Limiting deflection	δ _{lim} = 6.111 mm	I	Maximum def	flection	δ = 0.339 mn	n	
		PAS	S - Maximum	deflection does	not exceed d	eflection limit	





🚝 Tekla	Project	85 Cam	den Mews		Job no.	5005
ledds	Color for				Stort page pa //	
	Calcs for	Hip	rafter		Start page no./F	2
	Calcs by KL	Calcs date 17/12/2018	Checked by AP	Checked date	Approved by	Approved date
Unfactored imposed load re	eaction at support B	R _{B_Imposed} =	0.325 kN			
	► 100→j					
Timber section details						
Breadth of sections		b = 47 mm				
Depth of sections		h = 150 mr	n			
Number of sections in merr	ıber	N = 2				
Overall breadth of member		$b_b = N \times b$	= 94 mm			
Timber strength class		C24				
Member details						
Service class of timber		1				
Load duration		Medium te	rm			
Length of bearing		L _b = 100 m	m			
Section properties						
Cross sectional area of me	mber	$A = N \times b \times$: h = 14100 mi	m²		
Section modulus		$Z_x = N \times b$	× h² / 6 = 3525	500 mm ³		
		$Z_y = h \times (N)$	× b) ² / 6 = 220	0900 mm³		
Second moment of area		$I_x = N \times b \times$	h ³ / 12 = 264	37500 mm⁴		
		$I_y = h \times (N > $	× b) ³ / 12 = 10	382300 mm⁴		
Radius of gyration		$i_x = \sqrt{(I_x / A)}$) = 43.3 mm			
		$i_y = \sqrt{(I_y / A)}$	= 27.1 mm			
Modification factors						
Duration of loading - Table	17	K ₃ = 1.25				
Bearing stress - Table 18		K ₄ = 1.00				
Total depth of member - cl.	2.10.6	K7 = (300 n	nm / h) ^{0.11} = 1.	08		
Load sharing - cl.2.10.11		K ₈ = 1.10				
Minimum modulus of elastic	city - Table 20	K₀ = 1.14				
Lateral support - cl.2.10.8 Ends held in position						
Permissible depth-to-bread	th ratio - Table 19	3.00				
Actual depth-to-breadth rat	io	h / (N × b) :	= 1.60			
·		, , , , , , , , , , , , , , , , , , ,		PASS -	Lateral suppo	rt is adequate
Compression perpendicu	lar to grain					
Permissible bearing stress	(no wane)	$\sigma_{c,adm} = \sigma_{cn}$	1 × K3 × K4 × K	K ₈ = 3.300 N/mm ²		
Applied bearing stress	($\sigma_{c,a} = R_{A,m}$	$a_{\rm x}$ / (N × b × L _b) = 0.069 N/mm ²		
· · · · · · · · · · · · · · · · · · ·		$\sigma_{ca} / \sigma_{cadm}$	= 0.021	,		
P	ASS - Applied com	pressive stress	s is less than	permissible cor	npressive stre	ess at bearing
Bonding parallol to grain	, ,					
Permissible bending stress			× K₂ × K- × K	• = 11 130 N/mm	2	
Applied bending stress			□ ^ IN3 ^ IN/ × IN 7 - 0 522 NI/~	$_{\circ} = 11.150$ N/11111		
-philed behaving stress		om_a = IVI / Z	_x - 0.332 IN/II	1111		

Tekla Tedds	Project	85 Camo	Job no. 15005			
	Calcs for	Hip	rafter		Start page no./R	evision 3
	Calcs by KL	Calcs date 17/12/2018	Checked by AP	Checked date	Approved by	Approved date

	σ _{m_a} / σ _{m_adm} = 0.048
	PASS - Applied bending stress is less than permissible bending stress
Shear parallel to grain	
Permissible shear stress	τ_{adm} = $\tau \times K_3 \times K_8$ = 0.976 N/mm ²
Applied shear stress	$\tau_a = 3 \times F / (2 \times A) = 0.069 \text{ N/mm}^2$
	τ _a / τ _{adm} = 0.071
	PASS - Applied shear stress is less than permissible shear stress
Deflection	
Modulus of elasticity for deflection	E = E _{min} × K ₉ = 8208 N/mm ²
Permissible deflection	δ_{adm} = min(14 mm, 0.003 × L _{s1}) = 4.500 mm
Bending deflection	δ _{b_s1} = 0.198 mm
Shear deflection	δ _{v_s1} = 0.031 mm
Total deflection	$\delta_{a} = \delta_{b_{s1}} + \delta_{v_{s1}} = 0.229 \text{ mm}$
	$\delta_a / \delta_{adm} = 0.051$
	PASS - Total deflection is less than permissible deflection





	Project		Job no. 150	005		
redus	Calcs for	Start page no./Re	evision			
	Hipped Beam					2
	Calcs by Calcs of KK 17/	date Che 12/2018	ecked by AP	Checked date	Approved by	Approved date
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	-\$-			- \$-		
← 104 →						
	◀──100──▶					
Timber section details						
Breadth of section	b = 47 mm	Dep	oth of sectio	n	h = 150 mm	
Number of sections	N = 2					
limber strength class	C24					
Steel section details						
Breadth of steel plate	b _s = 10 mm	Dep	oth of steel p	plate	h _s = 150 mm	
Number of steel plates in beam N/mm ²	1	Ns =	= 1		Steel stress	p _y = 165
Bolt diameter	φ _b = 12 mm	Max	ximum bolt s	spacing	S _{max} = 300 mn	n
Member details						
Service class of timber	1	Loa	Load duration		Medium term	
Length of bearing	L _b = 100 mm					
The beam is part of a load-sha	ring system consisting	of four or mor	e members			
Lateral support - cl.2.10.8						
Permiss.depth-to-breadth ratio	3.00	Act	ual depth-to	-breadth ratio	1.44	
				PASS - I	ateral support	t is adequate
Check bearing stress						
Permissible bearing stress	σ _{c_adm} = 3.300 N/mm ²	Арр	plied bearing	g stress	σ _{c_a} = 0.488 N	/mm ²
PASS	S - Applied compressi	ve stress is l	less than p	ermissible con	npressive stre	ss at bearing
Bending parallel to grain						
Permiss. timber bending stress	s σ _{m_adm} = 11.130 N/mm	² App	lied timber	bending stress	σ _{m_a} = 3.212 N	l/mm ²
	PASS - Timb	er bending s	tress is les	s than permiss	sible timber be	nding stress
Permiss. steel bending stress	p _y = 165.000 N/mm ² <i>PASS - S</i>	App teel bending	olied steel be stress is le	ending stress e ss than permi	o _{m_a_s} = 78.36 ssible steel be	1 N/mm ² nding stress
Shear parallel to grain		-				•
Permissible shear stress	τ _{adm} = 0.976 N/mm ²	App	blied shear s	stress	τa = 0.162 N/m	1m²
	Padini Provi Primi	ASS - Applie	ed shear sti	ress is less tha	n permissible	shear stress
Deflection						
Permissible deflection	δ _{adm} = 11 400 mm	Tot	al deflection	n	δ ₂ = 6 277 mm	n
		PASS	- Total defi	lection is less t	than permissib	le deflection
Flitch plate bolting requireme	ents					
Bolts required at beam end	N _{be} = 2.000	Bolt	ts required t	to beam length	N _{bl} = 1.357	
- Provide a minimum of 2 No.1	2 mm diameter bolts at	t each suppor	t			
- Provide 12 mm diameter bolt	s at a maximum of 300	mm centres a	along the le	ngth of the bear	n	
Minimum bolt spacings						
Minimum end spacing	$S_{ord} = 48 \text{ mm}$	Min	imum odao	enacina	Sedge = 48 mm	
Minimum holt spacing			unium euge	spacing	Couge is in	
Minimum bolt spacing	$S_{\text{bolt}} = 48 \text{ mm}$		iiniuni euge	spacing		
Vinimum washer diameter	$S_{\text{bolt}} = 48 \text{ mm}$ $\phi_w = 36 \text{ mm}$	Min	imum wash	er thickness	t _w = 3.0 mm	





		85 Cam	den Mews		1	5005			
	Calcs for			Start page no./	Revision				
		Hip	rafter			2			
	Calcs by KL	Calcs date 17/12/2018	Checked by AP	Checked date	Approved by	Approved da			
Unfactored imposed load read	tion at support I	B R _{B_Imposed} =	2.153 kN	I					
∢ _94 _ ▶	↓ 100 →								
Timber costion details									
Breadth of sections		b = 47 mm							
Depth of sections		h = 200 mr	n						
Number of sections in member	er	N = 2							
Overall breadth of member		$b_b = N \times b$	= 94 mm						
Timber strength class		C24							
Member details									
Service class of timber		1							
Load duration		Medium te	erm						
Length of bearing		L _b = 100 m	m						
Section properties									
Cross sectional area of memb	er	$A = N \times b \times$	<h 18800="" =="" m<="" td=""><td>m²</td><td></td><td></td></h>	m²					
Section modulus		$Z_x = N \times b$	$Z_x = N \times b \times h^2 / 6 = 626667 \text{ mm}^3$						
		$Z_y = h \times (N)$	$(\times b)^2 / 6 = 29$	4533 mm³					
Second moment of area		$I_x = N \times b \times$	I _x = N × b × h ³ / 12 = 62666667 mm ⁴						
		$I_y = h \times (N)$	× b) ³ / 12 = 13	843067 mm⁴					
Radius of gyration		$i_x = \sqrt{(I_x / A)}$	$i_x = \sqrt{(I_x / A)} = 57.7 \text{ mm}$						
		$i_y = \sqrt{(I_y / A)}$	$i_y = \sqrt{(I_y / A)} = 27.1 \text{ mm}$						
Modification factors									
Duration of loading - Table 17		K ₃ = 1.25							
Bearing stress - Table 18		K ₄ = 1.00							
Total depth of member - cl.2.1	0.6	K ₇ = (300 n	nm / h) ^{0.11} = 1 .	.05					
Load sharing - cl.2.10.11	. Table 00	K ₈ = 1.10							
winimum modulus of elasticity	/ - 1 able 20	K9 = 1.14							
Lateral support - cl.2.10.8 Ends held in position									
Permissible depth-to-breadth	ratio - Table 19	3.00							
Actual depth-to-breadth ratio		h / (N × b) =	= 2.13						
				PASS -	Lateral suppo	ort is adequa			
Compression perpendicular	to grain								
Permissible bearing stress (no	o wane)	$\sigma_{c_{adm}} = \sigma_{cp}$	$_{01} imes K_3 imes K_4 imes K_4$	< ₈ = 3.300 N/mm ²	2				
Applied bearing stress		$\sigma_{c_a} = R_{A_m}$	$_{ax}$ / (N × b × L _t	b) = 0.458 N/mm ²					
		σ_{c_a} / σ_{c_adm}	n = 0.139						
DAG	S Annlind co	maraaaiya atraa	s is loss than	nermissihle cou	mpressive str	ess at hear			

 $\label{eq:scalar} \text{Permissible bending stress} \qquad \qquad \sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \textbf{10.783} \ \text{N/mm}^2$

🜉 Tekla'	Project				Job no.			
Tedds		85 Camden Mews				15005		
	Calcs for				Start page no./	Start page no./Revision		
		Hip	rafter			3		
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date		
	KL	17/12/2018	AP					
Applied bending stress		$\sigma_{m_a} = M / Z$	Z _x = 5.026 N/n	nm²				
		σ_{m_a} / σ_{m_a}	_{im} = 0.466					
		PASS - Applied	d bending str	ess is less than	permissible b	ending stress		
Shear parallel to grain								
Permissible shear stress		$ au_{adm} = au imes k$	K ₃ × K ₈ = 0.976	3 N/mm ²				
Applied shear stress		τ_a = 3 \times F /	(2 × A) = 0.34	: A) = 0.344 N/mm ²				
		τ_a / τ_{adm} = ().352					
		PASS - AJ	oplied shear s	stress is less th	an permissibl	e shear stress		
Deflection								
Modulus of elasticity for def	flection	E = E _{min} × I	K9 = 8208 N/m	1m²				
Permissible deflection		$\delta_{adm} = min($	(14 mm, 0.003	s × L _{s1}) = 11.400 i	mm			
Bending deflection		δ _{b_s1} = 8.98	39 mm					
Shear deflection		δ _{v_s1} = 0.39	92 mm					
Total deflection		$\delta_a = \delta_{b_s1} +$	δ _{v_s1} = 9.381	mm				
		$\delta_a / \delta_{adm} = 0$	0.823					
		D	ASS - Total de	oflaction is lass	than normiss	ihla daflaction		

AXIOM	Project	85 Camo	Job no. 15005			
STRUCTURES	Calcs for CB1				Start page no./Revision 1	
	Calcs by KL	Calcs date 04/12/2018	Checked by AP	Checked date 04/12/2018	Approved by	Approved date
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 $\text{Dead}\times 1.50$

	AXIOM	Project	85 Cam	Job no.				
	CT DUICTURE C		05 Call	15005				
	STRUCTURES	Calcs for	Start page no./Revision					
			1			2		
		Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date	
		<u>KL</u>	04/12/2018	AP	04/12/2018			
					Impose	ed × 1.50		
	Analysis results							
	Maximum moment		$M_{max} = 7.3$	$M_{max} = 7.3 \text{ kNm}$		$M_{\min} = 0 \text{ kNm}$		
	Maximum shear Deflection Maximum reaction at support A Unfactored imposed load reaction at support A Maximum reaction at support B		V_{max} = 7.1 kN V_{min} = δ_{max} = 0.3 mm δ_{min} = R_{A_max} = 52.7 kN R_{A_min}			_{′min} = -7.1 kN _{min} = 0 mm _{RA_min} = 52.7 kN		
			R _{A_Imposed} = 35.1 kN					
			R _{B_max} = 5	R _{B_max} = 52.7 kN R _{B_m}				
	Unfactored imposed load react	R _{B_Imposed} = 35.1 kN						
	Section details							
	Section type	UC 203x203x4	6 (BS4-1) Steel grade			S355		
	Classification of cross section	ons - Section 3.5	5					
	Tensile strain coefficient	ε = 0.88		Section classification		Semi-compact		
	Shear capacity - Section 4.2.	3						
	Design shear force	hear force $F_v = 7.1 \text{ kN}$		Design shear resistance		P _v = 311.6 kN		
		PASS - Design shear resistance				exceeds design shear force		
	Moment capacity - Section 4.	2.5						
	Design bending moment	M = 7.3 kNm		Moment capa	city low shear	M _c = 174.1 k	Nm	
	Buckling resistance moment	- Section 4.3.6.4	4					
	Buckling resistance moment	M _b = 155.3 kNm	ı	M _b / m _{LT} = 15	5.3 kNm			
PASS - Buckling resistance moment exceeds design bending								
	Check vertical deflection - Se	ection 2.5.2						
	Consider deflection due to dea	d and imposed lo	bads					
	Limiting deflection	δ _{lim} = 6.667 mm		Maximum def	lection	δ = 0.261 mr	n	
		deflection does	on does not exceed deflection limit					




AXIOM	Project	85 Camo	den Mews		Job no.	5005
STRUCTURES	Calcs for				Start page no./F	Revision
		1	B2			1
	KK	04/12/2018	AP	04/12/2018	Approved by	Approved date
In accordance with BS595	0-1:2000 incorpo	<u>50)</u> orating Corrigend	um No.1			
					TEDDS calcula	ation version 3.0.0
		Load Envelope - Com	bination 1			
34.050					1	
0.0			700			
LA			1		J B	
le blass		Bending Moment E	nvelope			
67.258 –		67.3				
mm [6	700 1		B	
^{kN} 36.349 م	3	Shear Force Env	elope			
0.0 -					4	
		0.2		-2	0 .9	
_55.675 _ mm		6	700		-55.7	
A			I		в	
Support conditions		Vertically r	estrained			
		Rotationally	y free			
Support B		Vertically re	estrained			
		Rotationally	v free			

Applied loading

Beam loads

Load combinations

Load combination 1

Rotationally free Imposed self weight of beam × 1 Imposed partial UDL 5.8 kN/m from 0 mm to 3200 mm Imposed partial UDL 3.5 kN/m from 3200 mm to 6700 r

Imposed partial UDL 3.5 kN/m from 3200 mm to 6700 mm Imposed point load 5.9 kN at 3200 mm Imposed point load 20.7 kN at 6100 mm Imposed point load 22.7 kN at 6700 mm

Support A	Dead × 1.50
	Imposed \times 1.50
Span 1	Dead × 1.50
	Imposed \times 1.50
Support B	$Dead \times 1.50$

AXIOM	Project	85 Camo	len Mews		Job no. 15	005
STRUCTURES	Calcs for	11	B2		Start page no./R	evision 2
	Calcs by KK	Calcs date 04/12/2018	Checked by AP	Checked date 04/12/2018	Approved by	Approved date
				Impose	d × 1.50	
Analysis results						
Maximum moment		M _{max} = 67.3	kNm	M _{min} = 0) kNm	
Maximum moment span 1 segi	ment 1	M _{s1 seq1 max}	= 67 kNm	M _{s1 seq1}	_{min} = 0 kNm	
Maximum moment span 1 segi	ment 2	Ms1_seg2_max	= 67.3 kNm	 Ms1_seg2	 _{min} = 0 kNm	
Maximum shear		V _{max} = 36.3	kN	V _{min} = -	55.7 kN	
Maximum shear span 1 segme	ent 1	Vs1_seg1_max	= 36.3 kN	Vs1_seg1	_min = 0 kN	
Maximum shear span 1 segme	nt 2	V _{s1_seg2_max}	= 6.2 kN	V _{s1_seg2}	_{_min} = -55.7 kN	
Deflection segment 3		δ _{max} = 16.5	mm	δ _{min} = 0	mm	
Maximum reaction at support A	N	R _{A_max} = 36	.3 kN	R _{A_min} =	36.3 kN	
Unfactored imposed load react	ion at support A	R _{A_Imposed} =	24.2 kN			
Maximum reaction at support E	3	R _{B_max} = 89	.7 kN	R _{B_min} =	89.7 kN	
Unfactored imposed load react	ion at support B	$R_{B_Imposed} =$	59.8 kN			
Section details						
Section type	UKC 203x203x	60 (Tata Steel A	Advance)		Steel grade	S355
Classification of cross section	ons - Section 3.5	;				
Tensile strain coefficient	ε = 0.88		Section classifie	cation	Plastic	
Shear capacity - Section 4.2.	3					
Design shear force	F _v = 55.7 kN		Design shear re	esistance	P _v = 419.7 kN	l
		PAS	S - Design she	ar resistance e	xceeds desig	n shear force
Moment capacity at span 1 s	egment 2 - Secti	on 4.2.5				
Design bending moment	M = 67.3 kNm		Moment capaci	ty low shear	Mc = 232.9 kM	lm
Buckling resistance moment	- Section 4.3.6.4	4				
Buckling resistance moment	M _b = 182 kNm		M _b / m _{LT} = 182	kNm		
		PASS - Bucklin	ng resistance n	noment exceed	ls design ben	ding moment
Check vertical deflection - Se	ection 2.5.2					
Consider deflection due to dea	d and imposed lo	ads				
Limiting deflection	δ _{lim} = 18.611 mr	n	Maximum defle	ction	δ = 16.457 mi	n
-		PAS	S - Maximum d	eflection does	not exceed de	eflection limit

	Project				Job no.	
AXIOM		85 Camo	den Mews		15	005
STRUCTURES	Calcs for				Start page no./R	evision
		1B2 - D	L + 0.1IL			1
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	KK	04/12/2018	AP	04/12/2018		



AXIOM	Project	85 Camo	len Mews		Job no.	5005
ST DILCTUDES	Cales for				Start page po /E	Povision
STRUCTURES		1B2 - DI	L + 0.1IL		Start page no./r	2
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	KK	04/12/2018	AP	04/12/2018		
				Impose	ed × 1.50	
Analysis results						
Maximum moment		M _{max} = 32.3	kNm	M _{min} = 0	0 kNm	
Maximum moment span 1 segr	ment 1	Ms1_seg1_max	= 32.1 kNm	M _{s1_seg1}	_ _{_min} = 0 kNm	
Maximum moment span 1 segr	ment 2	Ms1_seg2_max	= 32.3 kNm	Ms1_seg2	2_min = 0 kNm	
Maximum shear		V _{max} = 17 k	N	V _{min} = -	35 kN	
Maximum shear span 1 segme	ent 1	Vs1_seg1_max	= 17 kN	Vs1_seg1	_min = 0 kN	
Maximum shear span 1 segme	ent 2	V _{s1_seg2_max} :	= 3.4 kN	Vs1_seg2	_ _{min} = -35 kN	
Deflection segment 3		δ _{max} = 8.2 n	าm	$\delta_{min} = 0$	mm	
Maximum reaction at support A	A Contraction of the second seco	RA_max = 17	kN	R _{A_min} =	= 17 kN	
Unfactored imposed load react	ion at support A	R _{A_Imposed} =	11.3 kN			
Maximum reaction at support E	3	R _{B_max} = 56	. 8 kN	R _{B_min} =	= 56.8 kN	
Unfactored imposed load react	ion at support B	$R_{B_Imposed} =$	37.9 kN			
Section details						
Section type	UKC 203x203x	60 (Tata Steel A	Advance)		Steel grade	S355
Classification of cross section	ons - Section 3.5	5				
Tensile strain coefficient	ε = 0.88		Section classifi	ication	Plastic	
Shear capacity - Section 4.2.3	3					
Design shear force	F _v = 35 kN		Design shear r	esistance	P _v = 419.7 kM	١
		PAS	S - Design she	ear resistance e	xceeds desig	n shear force
Moment capacity at span 1 s	egment 2 - Secti	ion 4.2.5				
Design bending moment	M = 32.3 kNm		Moment capac	ity low shear	M _c = 232.9 kl	Nm
Buckling resistance moment	- Section 4.3.6.4	4				
Buckling resistance moment	M _b = 182 kNm		M _b / m _{LT} = 182	kNm		
		PASS - Bucklir	ng resistance i	moment exceed	ls design ben	ding momen
Check vertical deflection - Se	ection 2.5.2					
Consider deflection due to dea	d and imposed lo	bads				
Limiting deflection	δ _{lim} = 18.611 mr	n	Maximum defle	ection	δ = 8.159 mn	า
		PAS	S - Maximum o	leflection does	not exceed d	eflection limi

AXIOM	Project	85 Camo	len Mews		Job no. 150	005
STRUCTURES	Calcs for	16	33		Start page no./Re	vision 1
	Calcs by KL	Calcs date 04/12/2018	Checked by AP	Checked date 12/04/2017	Approved by	Approved date
		•	•		•	•



Support conditions Support A

Support B

Support C

Applied loading Beam loads

Vertically free Rotationally free Vertically restrained Rotationally free Vertically restrained Rotationally free

Imposed self weight of beam × 1 Imposed full UDL 0.9 kN/m Imposed point load 24.4 kN at 300 mm Imposed point load 24.4 kN at 3000 mm Imposed point load 35.1 kN at 2300 mm Imposed point load 7.8 kN at 2300 mm Imposed point load 7.8 kN at 2300 mm Imposed point load 8.1 kN at 4600 mm Imposed partial UDL 11.5 kN/m from 2500 mm to 4600 mm

AXIOM	Project	05 Com			Job no.	005
		85 Came			0	005
STRUCTURES	Calcs for	1	B3		Start page no./R	evision 2
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	KL	04/12/2018	AP	12/04/2017		
Load combinations						
Load combination 1		Support A		Dead ×	1.50	
				Impose	d × 1.50	
		Span 1		Dead ×	1.50	
				Impose	d × 1.50	
		Support B		Dead ×	1.50	
				Impose	d × 1.50	
		Span 2		Dead ×	1.50	
		·		Impose	d × 1.50	
		Support C		Dead ×	1.50	
				Impose	d × 1.50	
Analysis results						
Maximum moment		Mmax = 86.7	' kNm	Mmin = -	27.1 kNm	
Maximum moment span 1		M_{s1} max = 0	kNm	Ms1 min =	= -27.1 kNm	
Maximum moment span 2		M _{s2_max} = 80	6.7 kNm	Ms2 min =	= -27.1 kNm	
Maximum shear		- V _{max} = 75.4	kN	V _{min} = -(69.5 kN	
Maximum shear span 1		V _{s1_max} = 0	kN	V _{s1_min} =	= -49.9 kN	
Maximum shear span 2		V _{s2_max} = 75	5 .4 kN	V _{s2_min} =	= -69.5 kN	
Deflection		δ _{max} = 8.4 n	nm	$\delta_{\min} = 4.$. 3 mm	
Deflection span 1		δ _{s1_max} = 0 r	nm	$\delta_{s1_min} =$	4.3 mm	
Deflection span 2		δs2_max = 8.4	1 mm	$\delta_{s2_{min}} =$	0 mm	
Maximum reaction at support A		RA_max = 0 k	(N	R _{A_min} =	0 kN	
Maximum reaction at support B		R _{B_max} = 12	5.3 kN	R _{B_min} =	125.3 kN	
Unfactored imposed load reacti	on at support B	$R_{B_{Imposed}} =$	83.5 kN			
Maximum reaction at support C		R _{C_max} = 81	.7 kN	$R_{C_{min}} =$	81.7 kN	
Unfactored imposed load reacti	on at support C	$R_{C_{Imposed}} =$	54.4 kN			
Section details						
Section type	UC 203x203x46	(BS4-1)	Steel grade		S355	
Classification of cross sectio	ns - Section 3.5		Continue al contra	ation	Com!	-1
i ensile strain coefficient	ε = υ.88		Section classific	ation	Semi-compa	CI
Shear capacity - Section 4.2.3	.					
Design shear force	F _v = 75.4 kN	PAG	Design shear re	sistance	P _v = 311.6 kN	n shear force
Moment canacity at ones 2 6	Saction 4.2 5	170	e zoorgii onec			
Moment capacity at span 2 - 3 Design bending moment	M = 86.7 kNm		Moment canacit	v low shear	Ma = 174 1 kM	lm
	W - 00.7 NINII	PA	SS - Moment ca	apacity exceed	s design ben	ding moment
	ation 2 E 2				-	
Check vertical detiection - Se						
Consider deflection due to dead	d and imposed loa	ads				
Consider deflection due to deac Limiting deflection	d and imposed loa δ _{lim} = 10.556 mm	ads	Maximum deflec	ction	δ = 8.387 mm	

Tekla Tedds	Project	85 Camo	den Mews		Job no. 15	005
	Calcs for	1B5 - tors	sion check		Start page no./Re	evision 1
	Calcs by KL	Calcs date 05/12/2018	Checked by AP	Checked date	Approved by	Approved date

STEEL MASONRY SUPPORT

In accordance with BS5950-1:2000 incorporating Corrigendum No.1



Steel member details

Torsion beam Steel grade of support angle Modulus of elasticity Length of plate beyond beam Thickness of plate Area of plate

Supported materials detail

Density mas. main beam Height masonry main beam Ecc. of main beam material Add dead force main beam Density mas. support beam Height masonry support beam $h_{msb} = 2300 \text{ mm}$ Add dead force support beam P_{Gaddsb} = 0.0 kN/m

Geometry Cavity width

c = 1 mm

UKB 203x133x30

E = 205000 N/mm²

A_{sbu} = 2330.0 mm²

 $\rho_{m,mb} = 20.0 \text{ kN/m}^3$

PGaddmb = 5.8 kN/m

ρ_{m,sb} = 20.0 kN/m³

h_{mmb} = **2300** mm

e_{mb} = **0** mm

l_h = **100** mm

t_{sb} = 10 mm

User

Biaxial stress effects in the plate (SCI-P-110) Max overall bending moment M_x = 21.2 kNm Second moment of area I_{xx,all} = 4596 cm⁴ Section modulus of plate Z_{xx,plate} = 16.67 cm³/m P₁ = **5.9** kN/m Force on support plate Moment capacity of plate M_c = **7.1** kNm/m

Masonry support angle	plate
Design strength support angle	p _{ysb} = 355 N/mm ²
Constant	ε = 0.880
Total length of plate	I _{plate} = 233 mm
Width of main beam	B _{mb} = 134 mm
Dist weld position to CoG	c _{yysb} = -17 mm
Width masonry main beam	b _{mmb} = 133 mm
Add live force main beam	P _{Qaddmb} = 0.0 kN/m
Width masonry support beam	b _{msb} = 92 mm
Add live force support beam	P _{Qaddsb} = 0.0 kN/m
Supported width of masonry	d _m = 99 mm
Dist to NA combined section Elastic section modulus Eccentricity on support beam Bending at heel	$y_{e,all}$ = 41 mm $Z_{xx,all}$ = 635.39 cm ³ e_1 = 46 mm $M_{x,plate}$ = 0.3 kNm/m

Tedds calculation version 1.0.04

Tokio	Droject				loh no	
Tedds	Project	85 Carr	iden Mews		15	005
	Calcs for	1B5 - to	rsion check		Start page no./R	evision 2
	Calco by	Colos data	Chaokad by	Chaokad data	Approved by	Approved date
	KL	05/12/2018	AP	Checked date	Approved by	Approved date
			PASS	6 - Design strei	ngth exceeds	stress at heel
Long stress overall bending	σ1 = 33.4 N/mm	1 ²	Von Mises curv	/e constant	c _{fp} = 707.6 N/	mm²
Trans bending stress ratio limit	αts = 0.994		Trans bending	stress ratio	αls = 0.038	
		PASS -	Transverse ben	nding stress ra	tio less than a	llowable limit
Deflection at toe						
Unfact force on plate	P _{1SLS} = 4.2 kN/r	m	Distance from	weld to load	a _m = 46 mm	
Load resultant to edge of plate	b _m = 54 mm		Weld to load po	os as ratio	a _i = 0.460	
Effect second mnt of inertia	l _{eff_def} = 83333 r	nm⁴/m	Deflection at to	е	δ = 0.02 mm	
Deflection limit	δ _{lim} = 2.00 mm					
			PA	SS - Deflection	n is within spe	cified criteria
Weld details - assume a full l	ength weld and	that the plate	acts as a propp	bed cantilever v	with the prop a	at the weld
position and the fixed end at	the centre of tr	le torsion bear	Threat size of a	wald	10	-
Leg length of weld	$S_{weld} = 6 \text{ mm}$		I nroat size of V		a _{weld} = 4.2 mn	n
Shear force at weld position	$R_A = 12.0 \text{ km/m}$	-	Max possible to	orce in plate	$R_p = 830.3 \text{ km}$	N
Long snear beam/plate	R = 615.1 KN/n	n 	Horizontal snea	ar beam/plate	$R_h = 34.1 \text{ kN}/$	M
Resultant weld force	R _{weld} = 0.616 K	N/mm	Strength of wel	d (Table 37)	p _{weld} = 220.0	N/mm²
Capacity of full length weld	$p_{c,weld} = 0.933 \text{ K}$.N/MM				$1/\sqrt{2} \times S_{weld}$
Torsional loading ULS						.,
Loading support beam	w _{1ULS} = 5.92 kN	l/m	Loading of mai	n beam	W _{2ULS} = 16.69	kN/m
Self weight of support beam	w _{3ULS} = 0.26 kN	l/m	C C			
Torsional loading SLS						
	$W_{1010} = 4.23 \text{ kN}$	l/m	Loading of mai	n heam	Wast c = 11 92	kN/m
Self weight of support beam	$w_{13LS} = 4.20 \text{ kN}$	l/m	Louding of mar	ii beaiii	W23L3 - 11.92	
				h		
Distance of shear centre	$e_{0mb} = 0 mm$		Ecc of support	beam masonry	e _{1mb} = 114 mi	n
Ecc of main beam masonry	e _{2mb} = 0 mm		Ecc of support	beam	e _{3mb} = 50 mm	
Torsional effects						
Applied torque	T _{qULS} = 0.70 kN	lm/m	Torsional mom	ent (ULS)	T _q = 1.88 kNn	n
Applied torque (SLS)	T _{qSLS} = 0.50 kN	m/m	Torsional mom	ent (SLS)	T _{qu} = 1.34 kN	m
STEEL BEAM TORSION DES	IGN					
In accordance with BS5950-1	2000 incorpora	ating Corrigen	dum No.1		Tedds calcula	tion version 2 0 02
Section details						
Section type	UKB 203x133x	30	Steel grade		S355	
Design stength	p _{yw} = p _y = 355 N	N/mm ²	Constant		ε = 0.880	
Geometry - Beam unrestrain	ed against later	al-torsional bu	ckling between	supports.		
Effective span	L = 2700 mm		3			
Length of segment LTB	L _{LT} = 2700 mm		Effective length	n for LTB	L _{E_LT} = 3114 r	nm
Loading - Torsional loading	comprises only	full-length uni	formly distribut	ed load(s)		
Internal forces & moments o	n member unde	er factored load	ding for uls desi	ign		
Applied shear force	F _{vy} = 31.4 kN		Maximum bend	- ling moment	M _{LT} = M _x = 21	.22 kNm
Applied torque	T _q = 1.88 kNm		Minor axis ben	ding moment	M _y = 0 kNm	
Compression force	F _c = 0 kN			-		

	Project	85 Cam	den Mews		Job no. 15005		
	Calcs for		nion charle		Start page no./F	Revision	
	Calar hu	1B5 - tor		Chesteratelate		3	
	KL	05/12/2018	AP		Αμριονέα by	Approved	
Equivalent uniform moment	factors	-	•		•		
EUM factor (CI.4.3.6.6 & T18)	m _{LT} = 1.000						
Torsional deflection paramet	ters						
Beam is torsion fixed and warp	ing free at each	end. (as defined	d in SCI-P-057	section 2.1.6) - A	Appendix B cas	e 4	
Dist for first deriv of twist	z ₁ = 0 mm		Dist for secor	nd deriv of twist	z ₂ = L / 2 = 1	350 mm	
First deriv of angle of twist	φ' ₁ = 4.21 ×10 ⁻²	rads/m	Third deriv of	angle of twist	φ''' ₁ = -7.78×1	0-2 rads/n	
Angle of twist	φ ₂ = 0.035 rads	;	Second deriv	of angle of twist	φ"₂ = -4.55×1	0 ⁻² rads/m	
Design parameters							
Total angle of twist	ϕ = 0.035 rads		First derivativ	e of φ	φ' = 4.21 ×10 ⁻	² rads/m	
Second derivative of $\boldsymbol{\phi}$	φ" = 4.55 ×10⁻² r	rads/m ²	Third derivative of $\boldsymbol{\phi}$		φ''' = 7.78×10	-2 rads/m ³	
Section classification							
	b / T = 7.0				d / t = 26.9		
	r _{1s} = 0.000				r _{2s} = 0.000		
				Sec	tion classifica	tion is pla	
Shear capacity (parallel to y-	axis)				_		
Design shear force	F _{vy} = 31.4 kN		Design shear	resist (cl. 4.2.3)	P _{vy} = 281.9 k	N	
						Pass - S	
Moment capacity (x-axis)							
Design bending moment	M _x = 21.2 kNm	-	Mnt cap low s	shear (cl. 4.2.5.1)	M _{cx} = 111.6 k	Nm	
		P	ass - Moment	capacity excee	as aesign ber	aing moi	
Lateral torsional buckling							
Effective length for LIB	$L_{E_{LT}} = 3114 \text{ m}$	m			0.004		
Sienderness ratio	λ = 98		Buckling para	ameter	u = 0.881		
Flange ratio	η = 0.5		Torsional inde	ex	x = 21.5		
Slenderness factor	∨ = 0.84		Ratio - cl 4.3.	6.9	β _w = 1.000		
Equiv slenderness – cl 4.3.6.7	λ _{LT} = 72	0	Limit slenderr	nes – Ann B2.2	λ _{L0} = 30		
Euler stress	p _E = 386 N/mm	1 ²	Perry factor		η∟⊤ = 0.295	0	
	φ _{LT} = 42773181	3.312	Bending strer	ngth	p _b = 214 N/m	m²	
Buckling resistance moment	M _b = 67.2 kNm		Max mnt gov	buckling resist	M _{LT} = 21.2 kľ	Nm	
Equiv uniform mnt factor LTB	m _{LT} = 1.00				$M_{\rm b} / m_{\rm LT} = 67$.2 kNm	
					Pass - lat.	ors. Duci	
Buckling under combined be	ending & torsion	n -SCI-P-057 se	ction 2.3	we to each of the	a a na rata la a d	offente e	
For simplicity, a conservative of	neck is applied t	using the maxim	ium stresses d	ue to each of the	separate load	effects, e	
Span factor		same section a			4 - 0 035 roc	C	
	L / a = 2.70	$rada/m^2$		vr avia moment	$\psi = 0.035$ rac	5	
Normal stress flance M	$\psi = 43.3 \times 10^{10}$	au5/111 ⁻ 2	Normal stress		$r_{\rm viyt} = 0.75 {\rm Ki}$	n11 n2	
Interaction index	obyt = 13 N/MM		NUTHAI STRESS	s nange warping	o _w = o∠ iN/mr	11-	
	10 – U.JO		Pass - Con	nbined bendina	and torsion c	heck sati	
l ocal canacity under combin	ad handing 8 +	orsion		sale soluting			
For simplicity, a conservative of	heck is applied in	using the maxim	um stresses d	ue to each of the	separate load	effects e	
though these do not necessari	ly all occur at the	e same section a	along the mem	ber.		5	
Max. direct stress due to M _v	$\sigma_{bx} = M_x / Z_x = 7$	76 N/mm ²					
Combined stress - ean 2.22	$\sigma_{bx} + \sigma_{bvt} + \sigma_w =$	= 150 N/mm ²	Desian strend	ath	p _v = 355 N/m	m ²	

	Project	85 Came	den Mews		Job no. 15	005
	Calcs for				Start page no./R	evision
		1B5 - tors	sion check			4
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	KL	05/12/2018	AP			
even though these do not neces Shear stress bending web Shear stresses torsion web	ssarily all occu τ_{bw} = 27 N/mm τ_{tw} = 21 N/mm	r at the same sec ¹²	tion along the Shear stress Shear stresse	member. bending flange es torsion flange	τ _{bf} = 7 N/mm² τ _{tf} = 32 N/mm	2
Shear stresses warping flange Shear str tors & warp flange	τ _{wf} = 4 N/mm ² τ _{vtf} = 41 N/mm	2	Shear stress	tors & warp web	τ _{vtw} = 25 N/m	- 112
Shear stresses warping flange Shear str tors & warp flange Combined shear stresses due	τ _{wf} = 4 N/mm ² τ _{vtf} = 41 N/mm e to bending, 1	² torsion & warpin	Shear stress	tors & warp web	τ _{vtw} = 25 N/mr	- n²
Shear stresses warping flange Shear str tors & warp flange Combined shear stresses due Comb shear stresses in web	$\tau_{wf} = 4 \text{ N/mm}^2$ $\tau_{vtf} = 41 \text{ N/mm}^2$ e to bending, t $\tau_w = 51 \text{ N/mm}^2$	² torsion & warpin	Shear stress g: Comb shear s	tors & warp web stresses in flange	τ _{vtw} = 25 N/mr τ _f = 48 N/mm ²	- m² 2
Shear stresses warping flange Shear str tors & warp flange Combined shear stresses due Comb shear stresses in web Shear strength	$\tau_{wf} = 4 \text{ N/mm}^2$ $\tau_{vtf} = 41 \text{ N/mm}^2$ e to bending, t $\tau_w = 51 \text{ N/mm}^2$ $p_v = 213 \text{ N/mm}^2$	1 ² torsion & warpin 2 n ²	Shear stress g: Comb shear s	tors & warp web stresses in flange	τ _{vtw} = 25 N/mr τ _f = 48 N/mm ²	- m² 2

Total applied torque Max twist under sls loading	T _{qu} = 1.34 kNm φ _{sis} = 1.44 deg	Twist limit	φ _{lim} = 2.00 deg <i>Pass - Twist</i>
Deflection			
Maximum y-axis deflection	δ _{y_max} = 1.9 mm	Deflection limit - cl. 2.5.2	δ _{lim} = 7.5 mm

Pass - Deflection within specified limit

AXIOM	Project	85 Camo	Job no. 15005			
STRUCTURES	Calcs for	1B	6-1		Start page no./F	Revision 1
	Calcs by KK	Calcs date 04/12/2018	Checked by AP	Checked date 04/12/2018	Approved by	Approved date
STEEL BEAM ANALYSIS &	DESIGN (BS59	50)				
In accordance with BS5950	-1:2000 incorp	orating Corrigend	um No.1		TEDDS calcula	ation version 3.0.0
		Load Envelope - Com	bination 1			
15.450 –						
0.0 -						
mmA		29	100 1		J B	
kNm 0.0		Bending Moment E	velope			
6.481 – mm j	6.5	29	000		1	
Ă			1		В	
kN		Shear Force Enve	lope			
14.649						
0.0 -						
-3.994 J mm L A		29	100		-4.0 B	
Support conditions		Vertically re	strained			
		Rotationally	/ free			
Support B		Vertically re	estrained			
		Rotationally	/ free			
Applied loading			16			
Beamiloads		Imposed se	II UDI 0.5 kN/	m		
		Imposed po	pint load 10.3 l	kN at 450 mm		
Load combinations						
Load combination 1		Support A		Dead ×	1.50	
				Impose	ed × 1.50	
		Span 1		Dead ×	1.50	
		• • •		Impose	ed × 1.50	
		Support B		Dead ×	: 1.50	
					4 50	

	Project				loh no	
AXIOM	Filleci	85 Camo	den Mews		15	005
STRUCTURES	Calcs for			Start page no./Revision		
STROCTORES			2			
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	KK	04/12/2018	AP	04/12/2018		
• • • •						
Analysis results		M - 6 5	k N Im	M () kNm	
Maximum moment		$V_{max} = 6.5$	KINITI E KNI	$N_{min} = 0$	J KINITI A KNI	
		v max - 14.0		v min — -		
Maximum reaction at support A		D. – 14				
Linfactored imposed load reacti	on at support A	RA_max - 14		ra_min -	14.0 KIN	
Maximum reaction at support B	Maximum reaction at support R			Ro min =	= 4 kN	
Unfactored imposed load reaction	on at support B		2.7 kN	TCB_min =		
Section details		· · · · · · · · · · · · · · · · · · ·				
Section type	DEC 150-00-2	1 (BS1-1)	Steel grade		\$355	
Section type		+ (634-1)	Sieer grade		3333	
Classification of cross section	ns - Section 3.5	5				
l'ensile strain coefficient	ε = 0.88		Section classi	fication	Plastic	
Shear capacity - Section 4.2.3						
Design shear force	F _v = 14.6 kN		Design shear	resistance	P _v = 207.7 kN	
		PAS	S - Design sh	ear resistance e	xceeds desig	n shear force
Moment capacity - Section 4.2	2.5					
Design bending moment	M = 6.5 kNm		Moment capa	city low shear	Mc = 63.4 kNr	n
Buckling resistance moment	- Section 4.3.6.	4				
Buckling resistance moment	M _b = 40.3 kNm		M _b / m _{LT} = 40 .	3 kNm		
		PASS - Bucklin	ng resistance	moment exceed	ls design ben	ding moment
Check vertical deflection - Se	ction 2.5.2					
Consider deflection due to deac	and imposed lo	bads				
Limiting deflection	δ _{lim} = 8.056 mm		Maximum def	lection	δ = 1.291 mm	
		PAS	S - Maximum	deflection does	not exceed de	eflection limit

AXIOM	Project 85 Camden Mews					Job no. 15005	
STRUCTURES	Calcs for	18	Start page no./Revision 1				
	Calcs by KK	Calcs date 04/12/2018	Checked by AP	Checked date 04/12/2018	Approved by	Approved date	
			-				



AXIOM	Project	85 Cam	den Mews		Job no. 15	005	
STRUCTURES	Calcs for	Start page no./R	Start page no./Revision 2				
	Calcs by KK	Calcs date 04/12/2018	Checked by AP	Checked date 04/12/2018	Approved by	Approved date	
Analysis results							
Maximum moment		M _{max} = 6.5	kNm	M _{min} = 0) kNm		
Maximum shear		V _{max} = 16.3	kN	V _{min} = -	8.3 kN		
Deflection		δ _{max} = 1.6 r	nm	δ _{min} = 0	mm		
Maximum reaction at support A		R _{A_max} = 16	. 3 kN	R _{A_min} =	16.3 kN		
Unfactored imposed load react	ion at support A	$R_{A_Imposed} =$	10.9 kN				
Maximum reaction at support B	Maximum reaction at support B		R _{B_max} = 8.3 kN R _{B_min} =			= 8.3 kN	
Unfactored imposed load react	ion at support B	$R_{B_{Imposed}} =$	5.6 kN				
Section details							
Section type	PFC 150x90x2	4 (BS4-1)	Steel grade		S355		
Classification of cross section	ons - Section 3.5	5					
Tensile strain coefficient	ε = 0.88		Section classi	ification	Plastic		
Shear capacity - Section 4.2.3	3						
Design shear force	- F _v = 16.3 kN		Design shear	resistance	P _v = 207.7 kN	1	
		PAS	S - Design sh	ear resistance e	xceeds desig	n shear force	
Moment canacity - Section 4	25		·		· ·		
Design bending moment	M = 6.5 kNm		Moment capa	city low shear	Mc = 63.4 kNr	n	
Buckling resistance moment	Section 4.2.6	4					
Buckling resistance moment	- Section 4.3.6.	4	$M_{\rm b} / m_{\rm rr} = 40$	3 kNm			
Bucking resistance moment	Wb - 40.3 KINIII	PASS - Buckli	na resistance	moment exceed	ls desian hen	dina moment	
		2000	ig recictance		e accigit son	ang nomon	
Consider deflection due to dea	ction 2.5.2	ade					
Limiting deflection	s = 9.056 mm	Jaus	Movimum dof	laation	s = 1 620 mm		
Limiting denection	Olim – 0.050 IIIII					fla atia m limit	
		PAS	s - waximum	denection does	not exceed de	enection limit	

AXIOM	Project	85 Cam	den Mews		Job no.	5005
STRUCTURES	Calcs for GB3 Start page no				Start page no./F	Revision
	Calcs by KL	Calcs date 04/12/2018	Checked by AP	Checked date 04/12/2018	Approved by	Approved date
STEEL BEAM ANALYSIS & I	DESIGN (BS59	<u>50)</u>				

In accordance with BS5950-1:2000 incorporating Corrigendum No.1







Support conditions						
Support A	Vertically restrained					
	Rotationally free					
Support B	Vertically restrained					
	Rotationally free					
Support C	Vertically restrained					
	Rotationally free					
Applied loading						
Beam loads	Imposed self weight of beam × 1					
	Imposed full UDL 2.9 kN/m					
	Imposed partial UDL 24.4 kN/m fr	om 0 mm to 1600 mm				
	Imposed point load 97.6 kN at 80	0 mm				
	Imposed point load 97.6 kN at 32	00 mm				
	Imposed point load 83 kN at 5300) mm				
	Imposed point load 83.5 kN at 16	00 mm				
	Imposed partial UDL 12.7 kN/m fr	rom 3200 mm to 3900 mm				
Load combinations						
Load combination 1	Support A	$\text{Dead}\times 1.50$				

TEDDS calculation version 3.0.05

AXIOM	Project	85 Came	ten Mews		Job no.	5005
CTRUCTUREC	Color for				Start page po //	
STRUCTURES	Calcs for	G	B3		Start page no./F	2
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	KL	04/12/2018	AP	04/12/2018		
				Impose	d × 1.50	
		Span 1		$Dead \times$	1.50	
				Impose	d × 1.50	
		Support B		Dead ×	1.50	
				Impose	d × 1.50	
		Span 2		Dead ×	1.50	
				Impose	d × 1.50	
		Support C		Dead ×	1.50	
				Impose	d × 1.50	
Analysis results						
Maximum moment		M _{max} = 79.1	kNm	M _{min} = -	135.8 kNm	
Maximum moment span 1		M _{s1_max} = 4 4	4.1 kNm	M _{s1_min} =	= -135.8 kNm	
Maximum moment span 2		M _{s2_max} = 79	9.1 kNm	Ms2_min =	= -135.8 kNm	
Maximum shear		V _{max} = 165.	9 kN	$V_{min} = -2$	268 kN	
Maximum shear span 1		V _{s1_max} = 71	1.7 kN	V _{s1_min} =	= -268 kN	
Maximum shear span 2		V _{s2_max} = 16	6 5.9 kN	V _{s2_min} =	= -138 kN	
Deflection		δ _{max} = 8.2 n	nm	$\delta_{\min} = 0$.2 mm	
Deflection span 1		δ _{s1_max} = 0.4	4 mm	δs1_min =	• 0.2 mm	
Deflection span 2		δ _{s2_max} = 8.2	2 mm	δ_{s2} min =	• 0 mm	
Maximum reaction at support A	Ą	R _{A_max} = 71	.7 kN	R _{A_min} =	71.7 kN	
Unfactored imposed load react	tion at support A	R _{A_Imposed} =	47.8 kN	_		
Maximum reaction at support E	3	R _{B_max} = 43	3.9 kN	R _{B_min} =	433.9 kN	
Unfactored imposed load react	tion at support B	RB_Imposed =	289.3 KN	D –	400 601	
Infactored imposed load react	- tion at support C	$R_{C_{max}} = 13$	00 KIN 02 KN	RC_min =	138 KIN	
Section details	tion at support C	NC_Imposed -	JZ KIN			
Section type	UC 203x203x46	(BS4-1)	Steel grade		S355	
Classification of cross section	ons - Section 3.5	. ,	·			
Tensile strain coefficient	ε = 0.88		Section classi	fication	Semi-compa	ict
Shear capacity - Section 4.2.	3					
Design shear force	F _v = 268 kN		Design shear	resistance	P _v = 311.6 kM	N
		PAS	S - Design sh	ear resistance ex	xceeds desig	n shear force
Moment capacity at span 1 -	Section 4.2.5					
Design bending moment	M = 135.8 kNm		Moment capao	city high shear	M _c = 160.4 kl	Nm
Buckling resistance moment	t - Section 4.3.6.4					
Buckling resistance moment	M _b = 167.2 kNm		M _b / m _{LT} = 167	7.2 kNm		
		PA	SS - Moment	capacity exceed	s design ben	nding momen
Check vertical deflection - Se	ection 2.5.2					
Consider deflection due to dea	id and imposed loa	ads				
Limiting deflection	δ _{lim} = 10.833 mm		Maximum defl	ection	δ = 8.181 mn	n

AXIOM	Project 85 Camden Mews					Job no. 15005	
STRUCTURES	Calcs for	Win	dpost		Start page no./Revision 1		
	Calcs by KK	Calcs date 05/12/2018	Checked by AP	Checked date 05/12/2018	Approved by	Approved date	

STEEL ANGLE DESIGN (BS5950-1:2000) TEDDS calculation version 1.0.04 F +ve Mx +ve U **Element definition** Element being designed 1 Section RSA 120x120x10 S275 Steel grade Design strength (Table 9) py = 275 N/mm² **Design forces and moments** Shear force parallel to y axis F_{vy} = **0.00** kN Shear force parallel to x axis F_{vx} = 1.30 kN F = 0.0 kN Axial force M_x = **1.00** kNm Max moment about x axis Max moment about y axis My = 0.00 kNm Section classification (Table 11) The section is Class 3 (semi-compact) for bending **Design for shear** For shear force parallel to x axis (cl. 4.2.3) P_{vx} = **178.20** kN Shear capacity Shear cap. for 'low shear' Pvx low = 106.92 kN PASS - The angle is in low shear parallel to x axis **Design for bending** The angle is not restrained against lateral torsional buckling Moment capacities M_{cx_min} = **10.02** kNm Min mt cap about x-x axis Max mt cap about x-x axis M_{cx_max} = **26.15** kNm PASS - The moment capacity about the x-x axis exceeds the applied moment Design moments about principal axes

	Project				loh no		
AXIOM	Toject	85 Cam	iden Mews		15005		
STRUCTURES	Calcs for				Start page no /Revision		
STRUCTURES		Wir	ndpost			2	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date	
	KK	05/12/2018	AP	05/12/2018			
 ELIM factor m							
	IIILT = 1.000						
Min u axis section modulus	Z _u = 59.2 cm ³		Min v axis sec	ction modulus	$Z_v = 27.7 \text{ cm}^3$		
Buckling resistance moment							
Buckling resistance moment	$M_b = p_b \times Z_u =$	14.16 kNm	Eff buckling re	esistance mt	$M_{beff} = M_b/m_{LT}$	= 14.16 kNm	
	PASS - Ef	fective buckling	resistance m	oment exceeds a	pplied major	axis moment	
Minor axis bending resistan	ce						
Bending resistance	M _{cv} = 7.63 kN	m					
	PASS - Min	or axis bending	resistance m	oment exceeds a	pplied minor	axis moment	
Equivalent uniform moment	factors						
EUM factor m _u	m _u = 1.000		EUM factor m	v	m _v = 1.000		
Member buckling resistance	(cl 4.8.3.3)						
Equation 1	UF1 = 0.136		Equation 2		UF ₂ = 0.143		
			PASS	- Member buck	ing resistance	e is adequate	



	Project	85 Cam		Job no. 1	5005	
	Calcs for				Start page no./I	Revision
		215thk Blo	ckwork Wall			2
	Calcs by KK	Calcs date 05/12/2018	Checked by	Checked date	Approved by	Approved date
	·		·			·
Masonry details		Aggrogot	oonoroto bla	oko with no voi	da	
Compressive strength of up	i+		/mm ²		us	
Mortar strength Class/Desig	nation	M4 / (iii)	N/11111			
Height of masonry units		h _b = 440 m	ım			
Density of masonry		γ = 18.0 kM	√m³			
Least horizontal dimension	of masonry units	t _{min} = 215 r	nm			
Ratio of height to least horizontal dimension		$h_b / t_{min} = 2$.05			
From BS5628-1 Table 2d -	Characteristic c	ompressive stre	ngth of maso	nry		
Characteristic compressive	strength	f _k = 6.40 N	/mm²			
From BS5628-1 Table 3 - C	Characteristic fle	xural strength of	masonry			
Plane of failure parallel to be	ed joints	$f_{kx_para} = 0.$	17 N/mm ²			
Plane of failure perpendicula	ar to bed joints	$f_{kx_perp} = 0.$	41 N/mm ²			
Lateral loading details						
Characteristic wind load on	panel	W _k = 0.70) kN/m²			
Partial safety factors for m	naterial strength	_				
Category of manufacturing of	control	Category				
Category of construction co	ntrol	Normal				
Partial safety factor for mase	onry in compress	ion $\gamma_{mc} = 3.50$				
Partial safety factor for mase	onry in flexure	$\gamma_{mf} = 3.00$				
Partial safety factor for mase	onry in shear	$\gamma_{mv} = 2.50$				
Horizontal loading (cl 32)						
Limiting dimensions (cl 32	2.3)	b _ 40 \	(+ 9600 mg	2		
Limiting wail height		$\Pi_{max} = 40$ >		n PASS - Limitina	wall height is	not exceede
Partial cafaty factors for d	ocian loodo					
Partial safety factor for desig	esign loads	VEW - 1 40				
Partial safety factor for desig	an dead load	$\gamma_{\rm fG} = 0.90$				
Decise memorie of an lat		,				
Self weight of well	ance in paneis (S 0 27	syhytyy=1	2 76 kN/m		
Design vertical compressive	stress	$S_{wt} = 0.373$, ληλιλγ=4 Οι τ Ο Δ/+= 4	D 01 N/mm ²		
Enhanced flexural strength	f maconny	$y_d = \gamma_{fG} \times ($	$G_K = O_{Wt} / I = I$	$-0.21 \text{ N/IIIII}^{-2}$		
Section modulus of wall	or masority	$Ika_para = Ikx$ $7 - t^2 / C =$	_para + $\gamma mf \times gd$	$= 0.21 \text{ in/iiiii}^{-3/m}$		
Elastic design moment of re	sistance	$\Sigma = t^{-} / t^{0} =$ Md = fka par	$X = \frac{1}{2} \times \frac{1}{2} / \gamma_{mf} = 0.5$	534 kNm/m		
Design moment in panels	(cl 32.4.2)	····d ··ka_pai	, <u>, , , , , , , , , , , , , , , , , , </u>			
Using elastic analysis to d	 letermine bendir	ng moment coeff	cients for a v	ertically spanni	ng panel	
Bending moment coefficient		α = 0.125		, -pairin	5	
Design moment in wall		$M = \alpha \times W$	$_{\rm k} \times \gamma_{\rm fW} \times {\rm h}^2 = 0$).442 kNm/m		
U			•			



	Tekla Tedds Project				Job no. 15005	
i cuus	Calcs for				Start page no./R	evision
		215thk Blo		Chooked data	Approved by	Z Approved det-
	KK	05/12/2018	Checked by	Checked date	Approved by	Approved date
•• • • •						
Masonry details Masonry type		Aggregate	concrete blo	cks with no void	s	
Compressive strength of uni	t	p _{unit} = 7.3 N	l/mm ²			
Mortar strength Class/Desig	nation	M4 / (iii)				
Height of masonry units		h _b = 440 m	m			
Density of masonry		γ = 18.0 kN	/m³			
Least horizontal dimension of	of masonry units	t _{min} = 215 m	nm			
Ratio of height to least horiz	ontal dimension	$h_{b} / t_{min} = 2.$	05			
From BS5628-1 Table 2d -	Characteristic co	mpressive strer	igth of mason	iry		
Characteristic compressive	strength	f _k = 6.40 N/	mm²			
From BS5628-1 Table 3 - C	haracteristic flex	ural strength of	masonry			
Plane of failure parallel to bed joints		f _{kx_para} = 0.1	7 N/mm ²			
Plane of failure perpendicula	ar to bed joints	f _{kx_perp} = 0.4	1 N/mm ²			
Lateral loading details						
Characteristic wind load on	banel	W _k = 0.700	kN/m ²			
Partial safety factors for m	aterial strength					
Category of manufacturing c	ontrol	Category I	l			
Category of construction cor	ntrol	Normal				
Partial safety factor for maso	onry in compression	$\gamma_{mc} = 3.50$				
Partial safety factor for mase	onry in flexure	$\gamma_{mf} = 3.00$				
Partial safety factor for mase	onry in shear	$\gamma_{mv} = 2.50$				
Horizontal loading (cl 32)						
Limiting dimensions (cl 32	.3)		0			
Area of panel		$A_p = h \times L =$	= 2.3 m ²			
Limiting area of panel		$A_{max} = 1350$) × t _{ef} ² = 62.4 n	n²		
		PAS	S - Area of pa	anel does not ex	ceed limiting	area of pane
Limiting panel dimension		$L_{max} = 50 \times$	t _{ef} = 10750 mr	n <i></i> .		
			PASS -	Limiting panel of	dimension is i	not exceede
Partial safety factors for de	esign loads					
Partial satety factor for desig	in wind load	γ _{fw} = 1.40				
Partial safety factor for desig	in dead load	γ _{fG} = 0.90				
Design moments of resista	ance in panels (cl	32.4.2 <u>)</u>				
Design vertical compressive	stress	$g_d = \gamma_{fG} imes G$	_k / t = 0.00 N/m	nm²		
Enhanced flexural strength of	of masonry	$f_{ka_para} = f_{kx_p}$	$_{para}$ + $\gamma_{mf} \times g_{d}$ =	• 0.17 N/mm ²		
Section modulus of wall		$Z = t^2 / 6 =$	7704167 mm ³ /	/m		
Elastic design moment of re-	sistance	$M_d = f_{ka_para}$	\times Z / γ_{mf} = 0.44	45 kNm/m		
Design moment in panels	(cl 32.4.2)					
Orthogonal strength ratio		$\mu = f_{ka_{para}}$ /	f _{kx_perp} = 0.42			
Using yield line analysis to	calculate bendin	g moment coef	ficient			
Bending moment coefficient		α = 0.232				
Design moment in wall		$M = \mu \times \alpha \times$	$W_k \times \gamma_{fW} \times L^2$	= 0.139 kNm/m		
-		•	, DAOO D-		nt avaaada da	- .

Tekla Tedds	Project 85 Camden Mews				Job no. 15005	
	Calcs for Timber Frame				Start page no./Revision 1	
	Calcs by KL	Calcs date 05/12/2018	Checked by AP	Checked date 14/04/2017	Approved by	Approved date

TIMBER PANEL RACKING RESISTANCE - BS5268:SECTION 6.1:1996

TEDDS calculation version 1.0.05

Dwellings not exceeding seven storeys

Timber panel

Variation in nail spacing

Panel height	Secondary sheathing Primary sheathing				
Wall panel details	L 0 000				
Length of panel	L = 2.600 m				
Total area of wall panel	$\Delta_{\rm r} = 1 \times H_{\rm r} = 7 \ 020 \ {\rm m}^2$				
Aggregate area of framed panel openings	$A_{t} = 1 200 \text{ m}^{2}$				
Timber members	A _a = 1.200 m ² 38 mm x 72 mm or larger				
Uniformly distributed load on timber frame wall	$F_{\rm url} = 0.000 \rm kN/m$				
For calculation equivalent uniformly distributed load kN/m	d $F = min(F_{udl}, 10.5 \text{ kN/m}) = 0.000$				
Primary sheathing details					
Primary board type	OSB				
Standard board thickness	t _p = 9.00 mm				
Proposed board thickness	T _p = 9.00 mm				
Ratio of proposed to standard board thickness	$B_p = min(max(T_p / t_p, 0.75), 1.25) = 1.00$				
Nail diameter	$D_{p} = 3.50 \text{ mm}$				
Standard perimeter nail spacing	$s_p = 150 \text{ mm}$				
Proposed perimeter nall spacing	$o_p = i \sigma \min$				
From Table 2 – Basic racking resistance for a ra Basic racking resistance	ange of materials and combinations of materials $R_{bp} = 1.680 \text{ kN/m}$				
Modification factors for variation in fixing and t Variation in nail diameter	hickness of primary sheathing $K_{101p} = D_p / 3 \text{ mm} = 1.167$				

 $K_{102p} = 1 \ / \ (0.6 \times (S_p \ / \ s_p) + 0.4) = \textbf{1.429}$

Tekla Tedds	Project	Project 85 Camden Mews				Job no. 15005				
	Calcs for	Calcs for Timber Frame			Start page no./Revision 2					
	Calcs by KL	Calcs date 05/12/2018	Checked by AP	Checked date 14/04/2017	Approved by	Approved date				
Variation in board thickness		K _{103p} = 2.8	$K_{103p} = 2.8 \times B_p - B_p^2 - 0.8 = 1.000$							
Material modification factors		$K_{mp} = K_{101p}$	$K_{mp} = K_{101p} \times K_{102p} \times K_{103p} = \textbf{1.667}$							
Secondary sheathing detail	S									
Secondary board type	Secondary board type			Plywood						
Standard board thickness	Standard board thickness			t _s = 9.50 mm						
Proposed board thickness		T _s = 9.00 n	T _s = 9.00 mm							
Ratio of proposed to standard	Ratio of proposed to standard board thickness		$B_s = min(max(T_s / t_s, 0.75), 1.25) = 0.95$							
Nail diameter	Nail diameter		D _s = 3.50 mm							
Standard perimeter nail spac	Standard perimeter nail spacing		s _s = 150 mm							
Proposed perimeter nail space	ing	S _s = 150 m	m							
From Table 2 – Basic racki	ng resistance fo	or a range of ma	terials and co	ombinations of m	naterials					
Basic racking resistance		R _{bs} = 0.840) kN/m							
Modification factors for var	iation in fixing	and thickness o	f secondary s	sheathing						
Variation in nail diameter	Variation in nail diameter			K _{101s} = D _s / 3 mm = 1.167						
Variation in nail spacing	Variation in nail spacing		$K_{102s} = 1 / (0.6 \times (S_s / s_s) + 0.4) = 1.000$							
Variation in board thickness	Variation in board thickness		$K_{103s} = 2.8 \times B_s - B_s^2 - 0.8 = 0.955$							
Material modification factors	Material modification factors			$K_{ms} = K_{101s} \times K_{102s} \times K_{103s} = 1.114$						
Modification factors for wa	ll height, length	, openings, vert	ical load and	interaction						
Height of wall panels	Height of wall panels		$K_{104} = 2.4 \text{ m} / H_{wp} = 0.889$							
Length of walls	Length of walls		K ₁₀₅ = (L / 2.4 m) ^{0.4} = 1.033							
Fully framed openings in wall	Fully framed openings in walls		$K_{106} = (1 - 1.3 \times A_a / A_t)^2 = 0.605$							
Vertical load on timber frame wall		K ₁₀₇ = 1 + [$K_{107} = 1 + [(0.09 \times (F / 1 \text{ kN/m}) - 0.0015 \times (F / 1 \text{ kN/m})^2) \times (2.4 \text{ m} / \text{L})^{0.4}]$							
K ₁₀₇ = 1.000										
Interaction		K ₁₀₈ = 1.10	K ₁₀₈ = 1.100							
Wall modification factors		$K_w = K_{104} \times$	$K_w = K_{104} \times K_{105} \times K_{106} \times K_{107} \times K_{108} = \textbf{0.611}$							
Racking resistance of wall	panel									
Racking resistance of wall pa	Racking resistance of wall panel		$R_{R} = L \times K_{w} \times (R_{bp} \times K_{mp} + R_{bs} \times K_{ms}) = 5.933 \text{ kN}$							
				/	7					
	Wind Load=	=								
$\frac{2.7 \text{m x } 6.2 \text{m}/2 \text{ x } 0.7 \text{kN/m2} = 5.86 \text{kN}}{-1}$										
	< Rr hence	OK								


























SCI JOB REFERENCE: DECK REFERENCE: COMPANY NAME: CLIENT NAME: FILENAME: 150MD.pmd	Calcs by:	Cor 85 Camder ComFlor 6 Axiom Stru KK	nFlor n Mews 0/1.2/G500 ctures Limited	Date: Time: Job No.: Checked by:	v9.0.33.0 12/17/2018 16:50:34 15005
		SUMMAR	Y OUTPUT		
NOTE:	SECTION	DESIGNE	D TO BRITISH	STANDARDS	
Construction Stage: Normal Stage: Fire Condition: Serviceability:		PASS PASS PASS SATISFA	CTORY	Max Unit Max Unit Max Unit Max Unit	ty Factor = 0.49 ty Factor = 0.27 ty Factor = 0.00 ty Factor = 0.70
	*** (SECTION	ADEQUATE **	*	
FLOOR PLAN DATA : (unpro Beam centres Beam or wall width	pped comp 3.10 m 150 mm	oosite cons	truction with Co Span type Propping	omFlor 60/1.2/0	G500 decking) SINGLE NONE
PROFILE DATA : (ComFlor 6 Depth Trough width Nominal sheet thickness Deck weight	0/1.2/G500 60 mm 120 mm 1.20 mm 0.14 kN/m ²) decking)	Pitch of deck ribs Crest width Design sheet thin Yield strength	s ckness	300 mm 130.7 mm 1.16 mm 500 N/mm²
CONCRETE SLAB : [Normal Overall slab depth Concrete characteristic strength Modular ratio	Weight Co 150 mm 30 N/mm² 10	ncrete ; M	esh : 662/265] Concrete wet der Concrete dry der	nsity nsity	2400 kg/m³ 2350 kg/m³
Bar reinforcement : Diameter Distance from slab soffit Mesh reinforcement :	8 mm 30 mm		Yield strength		300 N/mm²
Mesh Cover to Mesh Account for End Anchorage Diameter of Shear Connectors	662/265 30 mm NO N/A 75 mm		Yield strength Mesh Layers Shear connector	s per rib	460 N/mm² Single N/A
SECTION PROPERTIES : ***NOTE - 1: All values of inertia ***NOTE - 2: Average inertia is u ***NOTE - 3: Cracked dynamic ir	are express sed for defle	sed in steel ection calcu d for natural	units lations for the cor	nposite stage ations	2000 Ng/11
DECK PROFILE: Sagging Inertia, Ixx Hogging Inertia, Iyy	132.910 cr 121.600 cr	n4/m n4/m	Area of profile (N	let), Ap	1721 mm²/m

v9.0.33.0

COMPOSITE:

nertia, Ixx - Uncracked 2472 cm4/m		Cracked	1385 cm4/m	
Average inertia	1928 cm4/m	Cracked inertia (dynamic)	1592 cm4/m	
Shear bond coefficients - Mr	208.86	Kr	0.014700	
Concrete volume	0.118 m³/m/m			
LOADS ACTING ON SLAB :				
*** NOTE: Slab subjected to unifer	mly distributed loads (LIDL)			

NOTE: Slab subjected to uniformly distributed loads (UDL) ONLY

Imposed (occupancy)	1.50 kN/m²	Partitions	0.50 kN/m²
Ceilings and services	0.50 kN/m ²	Finishes	0.50 kN/m ²
Self weight of concrete slab (wet)	2.79 kN/m ²	Self weight of decking	0.14 kN/m ²
Self weight of concrete slab (dry)	2.73 kN/m ²	Self weight of screeds	1.47 kN/m²
Construction load	1.50 kN/m ²	(applied over middle 3m length,	
		0.75kN/m ² elsewhere)	

LINE LOADS PERPENDICULAR TO DECK SPAN :

None

LINE LOADS PARALLEL TO DECK SPAN :

None

FIRE DATA :

Design method	FIRE ENGINEERING	Fire resistance period	30 mins
Non-permanent imposed loads	0.0 kN/m ²		
PARTIAL SAFETY FACTOR	S :		
Dead (self weight)	1.40	Imposed	1.60
Super imposed dead	1.40	Fire (occupancy loads)	0.80

Project	85 Ca	amden Mews	;	G		The	Concrete Ce	ntre
Client	Private)			pa The Concrete Centre	Made by	Date	Page
Location	150thk	Slab	1 to :	5: A to B		KK	17-Dec-2018	1
	2-WAY Originated	SPANNING INSI I from RCC94.xls v4.	TU CONCRETE SLABS 1 0 © 2006 TCC	o BS 8110:2005	(Table 3.14)	Checked AP	Revision -	Job No 15005
DIMENSIONS	;		MATERIALS			STATUS	VALID DESIG	N
short span, lx	(m	<u>0.80</u>	fcu N/mm	² <u>40</u>	γc = <u>1.50</u>	<u>1</u>		<u>5</u>
long span, ly	/ m	<u>2.70</u>	fy N/mm	² <u>500</u>	γs = <u>1.15</u>		Edge 1	
h Top cover	i mm	<u>150</u> 50	steel clas	s <u>A</u> 3 0	Plan	A	E E	
Btm cover	· mm	<u>50</u>	(Lightweight o	concrete)	i lait	4	0.8 г	2
LOADING ch	aracteris	tic	EDGE		S	dge	/ = ×	dge
Extra dead	kN/m² kN/m²	0.00 2.00	Edge	1 <u>d</u> C 2 d D	2 = Continuous) = Discontinuou	us	لے Lv = 2.7 m	
Total Dead, gk	kN/m²	2.00 γ	f= <u>1.40</u> Edge	3 <u>d</u>		<		
Imposed, qk	kN/m²	<u>1.50</u> γ	f= <u>1.60</u> Edge	4 <u>d</u>		<u> </u>	↓ 	
Design ioad, n	i kN/m²	5.20	See Figure 3.8 and	clauses 3.5.3.5-6		I	Edge 3	Ι
MAIN STEEL		SHORT SPAN X	LONG SPAN v	EDGE 1 Free	EDGE 2 Free	EDGE 3 Free	EDGE 4 Free	BS8110 Reference
ßs	;	0.134	0.056	0.000	0.000	0.000	0.000	Table 3.14
M N	kNm/m	0.4	0.2	0.0	0.0 88.0	0.0	0.0	
k'	, ,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,	0.156	0.156	0.156	0.156	0.156	0.156	
k	ί.	0.001	0.001	0.000	0.000	0.000	0.000	
	. mm mm²/m	91.2 11	83.6	91.2	83.6 0	91.2	83.6	3.4.4.4
As min	1 mm²/m	195	195	195	195	195	195	Table 3.25
As deflection	mm²/m	11	5	~	~	~	~	
Ø	mm	<u>8</u>	<u>8</u>	<u>8</u>	<u>8</u> T 2	<u>8</u> T 4	<u>8</u> T 2	
ayer @	mm	250	В 2 250	250	250	250	250	
As prov	′ mm²/m	201	201	201	201	201	201	
=	%	0.209	0.228	0.209	0.228	0.209	0.228	%
S max	(mm	296	272	296 (a)	272 (2)	296	272	Clause
DEFLECTION	, I	(d)	(a)	(d)	(a)	(a)	(a)	3.12.11.2.7
fs		19	9	0	0	0	0	Eqn 8
Perm I /d		2.000	Actual	′d 8.33	Asx enhan	ced 0.0% for deflec	ction control	Eqn 7 Table 3 10
		0.000						
TORSION ST	EEL	10	BOTH EDGES DIS	CONTINUOUS	ONE EDGE	E DISCONTINUOUS		
0	mm As reo	10 1 mm²/m	X 195	Y	X	Y 195	-	3535
As	s prov T	mm²/m	201	201	5000	5000		0.0.0.0
Additional A	s T req	mm²	0	0 201	0	0 201		
	- 2107 0		Bottom	steel not curtailed	d in edge strips at f	iree edges	J	
SUPPORT RE	EACTIC	NS (kN/m cha	r uno) (See	Figure 3.10)		Sum	ßvx = 1.136	Table 3.15
		EDGE 1	EDGE 2	EDGE 3	EDGE 4	Sum	ßvy = 0.667	
		A, 1-5	5, B-A	B, 1-5	1, B-A		0751	equations
ßv	/ 	0.568	0.333	0.568	0.333		0.75 L	19 & 20
Imposed	kN/m	0.91	0.33	0.91	0.53			
Vs	kN/m	2.4	1.4	2.4	1.4	0	<u> </u>	<u> </u>
OUTPUT/SUN	MARY	SHORT	LONG	FDGE 1	EDGE 2	EDGE 3	FDGF 4	
PROVID	E	SPAN	SPAN	A, 1-5	5, B-A	B, 1-5	1, B-A	
MAIN STE	EL	H8 @ 250 B1	H8 @ 250 B2	H8 @ 250 T1	H8 @ 250 T	F2 H8 @ 250	T1 H8 @ 250	Τ2
ADDITION	IAL	CORNER 1	CORNER 2	CORNER 3	CORNER 4			
TORSION S	TEEL	1A	5A	5B	1B	1		
Y directio	n					placed in edge strips		
CHECKS		BAR Ø	SINGLY	MIN	ΜΔΧ		CI (OBAL
Lx > Ly		< COVER	REINFORCED	SPACING	SPACING	DEFLECTION	STA	TUS
I OK		OK	OK	OK	OK	OK	VALID	DESIGN



AXIOM STRUCTURES

200PFC to 254UC connection Connection Design Beam to Beam					
	Vuls =	50 kN			
Check 1:	Essential detailing requi Use 10thk end plate, 2 D = min. pitch = 2.5 x D = gauge = min. edge distance =	rements M16 8.8 bolts 18 mn 45 mn 90 mn 26 mn	ו ו ו		
Check 2: \$	Shear capacity of bolt gr Bolts: M16 8.8	roup connecting end =	plate to supporting be	eam	
	Ps = no bolts =	58.9 kN 2			
	Vb = V/bolts =	25 kN	< Ps =	58.9 kN	ОК
	Bearing strength: pb = Bearing capacity per b	430 N/mm2 olt:			
	Pbs = dbolt x pb x tmir	n = 16mm x 430N/m	m x 10mm =	68.8 kN	
	Vb = V/bolts =	25 kN	< Pbs =	68.8 kN	ОК
Check 3: (Capacity of endplate Shear capacity of 10th Pv = 0.6 x 10mm x (45 Pv =	k endplate: mm + 26 mm) x 27 117.2 kN	5 N/mm2 > 0.5 x Vuls =	25 kN	ОК
Check 4: S	Shear capacity of the su py = notch deep = d notched = $Py = 0.6 \times t \times d$ notched	pported beam web a 355 N/mm2 30 mm 170 mm	at the endplate		
	$Pv = 0.0 \times 1 \times 0$ holicite Pv =	195.5 kN	> Vuls =	50 kN	ОК
Check 5: (Capacity of fillet welds b	etween supported b	eam and end plate		
	E web weld = $2 \times d$ not	ched =	340 mm		
	Shear = V / Eweb	0.15 kN	/mm < pL =	0.924 kN/mm	ОК
Check 6: I	Bending capacity of redu py = L notch = Znotched = modulus o * Refer to next page for	uced supported bear 355 N/mm2 128 mm f notched section * = r Tedds calculations	n section at the notch	29 cm3	
	Mc = moment capacity Mapplied = Vuls x (Lno	ot notched section otch + tendplate)	= py x Znotched	10.3 kNm 6.9 kNm	
	Mc =	10.3 kN	m > Mapplied =	6.9 kNm	ОК

AXIOM STRUCTURES	Project 85 Camden Mews				Job no. 15005		
Axiom Structures Ltd	Calcs for	Calcs for Notched PFC - section properties				Start page no./Revision 1	
	Calcs by KK	Calcs date 02/01/2019	Checked by	Checked date	Approved by	Approved date	
CALCULATION OF SECTION		90.0	14:0		Tedds calcula	tion version 2.0.05	
	▲		> ↑				
	<u>▼</u> →7.0	←					
Area A = 23.52 cm ²							
2 nd moment of area							

I _{uu} = 725. cm ⁴	I _{vv} = 107. cm ⁴	I _{xx} = 646. cm ⁴	I _{yy} = 186. cm ⁴
Radius of gyration			
r _{uu} = 55.5 mm	r _{vv} = 21.4 mm	r _{xx} = 5.2 cm	r _{yy} = 2.8 cm
Plastic section module	us (only shapes with all recta	ingles at 90 degs)	
S _{xx} = 93.9 cm ³	S _{yy} = 52.4 cm ³		

Distance to combined centroid

X_e = **0.0** mm Y_e = **0.0** mm

Distance to equal axis area (only shapes with all rectangles at 90 degs) X_p = -18.8 mm Y_p = 33.4 mm

Elastic section modulus

Z_{xx} = **52.3** cm³ Z_{yy} = **29.0** cm³