MBP	Michael Barclay Partnership	Job title	Job number	Sheet number	Revision
	consulting engineers	ASTOR COLLEGE	6775	16/1	
	105-109 Strand London WC2R OAA	Calculation/Sketch title	Date	Author	Checked
	т 020 7240 1191 г 020 7240 2241	SOZAR PADELS	JULY	JC	
	E london@mbp-uk.com		17		
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INTRODUCTION

46 NO SOZAR PANEZS ARE PROPOSED ON THE ROOF OF THE EXISTING 1960S AND 19705 BLOCKES.

THE WAD NOG OF THE PANELS IMPOSED ON THE EXISTING ROOF IS CALCULATED BY MICHAEL AUBREN PARTNERSHIP, REFER TO THEIR CALCULATION SMEETS NO'S C3105, 1-13,

THE EXISTING 1970S ROOF IS CONSTRUCTED FROM WOODWOOD SLABS SUPPORTED BY TIMBER JOISTS AND METSEC TRUSSES AS SHOWN IN SECTION 15.

THE WEIGHT OF THE PANELS, ASSOCIATED BALAST AND WIND LOAD EXCEEDS THE GENERAL LOAD CAPACITY OF THE ROOF.

THE FOLLOW, NG CALCULATIONS PROPOSE AN ARRANGE MENT FOR THE PANELS THAT DOES NOT EXCEED THE CAPACITY OF THE ROOF STRUCTURE, THE ARRANGEMENT DIFFERS FROM THAT PROPOSED BY PHOTON ENERGY, THE PANEL SUPPLIERS

BP Michael Barclay Pa	rtnership Job title	Job number	Sheet number	Revision
consulting engineer:	ASTOR COL	LEGE 677	5 1612	
105-109 Strand Lon	don WC2R 0AA Calculation/Sketch title	Date	Author	Checked
т 020 7240 1191 г (	20 7240 2241 SOZAR F	PANELS JULY	JC	
E london@mbp-uk.co	m	71		
www.mbp-uk.com				

ROOF LOADINGS +0 19605 BUILDING WIND LOADS FROM TEDDS 0.43 4N/12, MAX DOWN LOAD 20NED, CASE 4 67 4NIWZ, ZONEA, CASE I MAX SUCTION SNOW LOADS FROM TEODS 0-27 40/2 ACCESS LOADS 0.6 4 NI~. where no ACCESS, ON PANEZS OR GUTTERS (ACLESS TO MAINTAIN ONLY) 115 4N/m2 WHERE ACCESSIBLE SELF WEIGHT EXCLUDING JOISTS 0.5 4N/m (REFER TO SECTIONIS)

MBP	Michael Barclay Partnership	Job title	Job number	Sheet number	Revision
	consulting engineers	UCL ASTOR COLLE	0E 6775	16/3	2
	105-109 Strand London WC2R 0AA	Calculation/Sketch title	Date	Author	Checked
	т 020 7240 1191 г 020 7240 2241	SOLAR PANELS	JULY	SC	1
	E london@mbp-uk.com		17		
	www.mbp-uk.com		• (		

PANER WADS TIMBER CNO 2000 FROM MICHAEZ AUBREN CALCULA : 2005 TOTAL Daw LOAD PER ON DANEZ 2.37401 -INCLUDING WND + BA ST 25 1. LONG Tm 1.1 m WIDE CTRAY SIZE) PANO 2E ONIE JTATED AS FORCEWS ON ROOT 2015 1.7-PANE 2.5-PANER LOIST 3 LOAD 2.13 4N/2 w. E PANEZ 0.640122 ACCESS ONIM Our -GXISTNOG DEFD LOAD ROOF wz Arn GZ 0.9 0.45 401 5 × -21+3 W2+3 war

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МВР	Michael Barclay Partnership	Job title	Job number	Sheet number	Revision
	consulting engineers	COLLEGE	6775	1614	
	105-109 Strand London WC2R OAA	Calculation/Sketch title	Date	Author	Checked
	т 020 7240 1191 г 020 7240 2241	SOZAN	JULY	JC	
	E london@mbp-uk.com	PANELS	17		
	www.mbp-uk.com				

LOADING CONSIDERED CONSERVATIVELY AS MEDIUM TERM. JOISTS OF

NOW CONSIDER LONG TERM COUDITION

PANEL DEAD LOAD AND ROOF DEAD LOAD

PANEL LOAD + BALLAST

11	3	+ 0,	24)/	(1	7 × 1.	$\left( \right)$	-	1.73	4-101-	~ ~
-40	1	-	1.72		0.9		1.51		121	

TERM LOADS, JOISTS OR, SHEET 16/7-8,

MPOSED LOAD OF 1.5 4N(\_\_\_

 $REACTION = (2.512) \times (1.5 + 0.7) \times 0.9$ FOR 1.5KN/m<sup>2</sup> = 2.5 KN

THAT IS COMPARABLE TO THE JOIST REACTION FOR THE PANELS WITH ACCESS ONLY TO THE SIDES = 2.64N

ADOPT 1.5 KN/M2 ON FOR CHECKING THE EDGE TRUSS, WHICH INCLUDES THE GUTTER AND BALLISTRADE.

																	-	-								_
Job Ref. 775	Sheet no /rev. 2	App'd by Date	han = 0.000 kNm	8 kNm m = -2.637 kN	ки 1. <sub>min</sub> = 2.637 kN		в_тт = 2.637 KN																			
		/2017 Crik'd by Date	= 1.918 kNm M	nax(abs(M <sub>max</sub> ),abs(M <sub>min</sub> )) = 1.918 = 2.637 kN F <sub>n</sub>	TaX(abs(rmax),abs(rmin)) = 2.53/1   = 5.274 kN x = 2.637 kN R <sub>3</sub>	ad = 0.610 kN cosed = 2.027 kN	x = 2.53/ KN Ra ad = 0.610 KN cosed = 2.027 KN			7 mm	25 mm	l × b = 47 mm		um term	00 mm		× D × h = 105/5 mm²	$\sim 100 \times 10^{-3} \text{ J/s}^{-3} $	$x (h \times h) / 0 = 0.000 mm^{4}$ $x b \times h^{3} / 12 = 44613281 mm^{4}$	$\times$ (N $\times$ b) <sup>3</sup> / 12 = <b>1946681</b> mm <sup>4</sup>	x / A) = 65.0 mm	y / A) = 13.6 mm	25	0	300 mm / h) <sup>0,11</sup> = 1.03	00
Project	Section	Calc. by Date J 07/07	Mmax	M = r F <sup>mex</sup>	r = rr Wlat = Ra_ma	upportA RA_Da ttsupportA RA_m;	Ipport B Ra_be t support B Ra_fre	X	<b>→</b> 100 • <b>→</b>	b = 4	h=2 N=1	b <sub>b</sub> = ∧ C16		1 Mediu	1 = 1 1		A=N 75		N = N	ly = h	$i_{k} = \sqrt{1}$	$\dot{y} = \sqrt{1}$	K3 = 1	K	$K_7 = (3)$	- -
TEKLA	MBP 105-109 Strand	WC2R0AA	Analysis results Maximum moment	Design moment Maximum shear Design shoor	Total load on beam Reactions at support A	Unfactored dead load reaction at su Unfactored imposed load reaction a Dependent of automed D	Reactions at support B Unfactored dead load reaction at su Unfactored imposed load reaction a	522		Timber section details Breadth of sections	Depth of sections Number of sections in member	Overall breadth of member Timber strength class	Member details	Service class of timber Load duration	Length of bearing	Section properties	Cross sectional area of member Section modulus		Second moment of area		Radius of gyration		Modification factors Dirration of Ioading - Table 17	Bearing stress - Table 18	Total depth of member - cl.2.10.6	Load sharing - d.2.9
												12														
JOD Ref.	Sheet no/rav.	App'd by Date	TECC TEDDS calculation version 1.5.07			4 ¬¤		4	æ		4	2.2	œ.	1					100 mm to 2500 mm		d × 1.00	sed × 1.00	a × 1.00 sed × 1.00	d × 1.00	sed × 1.00	
		Chk'd by Date	MEDIUM	nbination 1		2500	Envelope	619 1940		islape	/	200	-			ight of beam $\times$ 1	L 0.450 kN/m	iai uul 2. 130 kivim irom 4 iai i idi o 540 knim firom 0	ial UDL 0.540 kN/m from 2		Dea	Impo	Imor	Dear	Impo	
Project	Section	Calc. by Date J 07/07/2017	GN TO BS5268-2:2002	Load Envelope - Co			Banding Moment B			Shear Farce Env		2				Dead self we	Dead full UD.	Imposed part	Imposed part		Support A		I upde	Support B		
	105-109 Strand London	wczraza c	TIMBER BEAM ANALYSIS & DESIG	2.618		T T T T T T T T T T T T T T T T T T T	thm	000 000 000 000 000 000 000 000 000 00	¢.	2.637	-00 -00	ل 1637 ع. ا	¢	Applied loading	Beam loads		W3	2	w2 W2	Load combinations	Load combination 1					

	Project		1.1	Job Ref.	216
MBP 105-109 Strand	Section			Sheet no/rev. 3	
WC2R0AA	Calo. by J	Date Chk/d by 07/07/2017	Date	App'd by	Date
Lateral support - cl.2.10.8 No lateral support		3			
Permissible depth-to-breadth ratio - 7	Table 19	2.00			
Actual depth-to-breadth ratio		h / (N × b) = 4.79			
			FAIL-I	ateral support.	t is inadequate
Compression perpendicular to gra Permissible bearing stress (no wane)	<b>F</b>	0c_adm = 0cp1 × K3 × K4 × K8	= 2.750 N/mm <sup>2</sup>		
Applied bearing stress		$\sigma_{c_a} = R_{a_{max}} / (N \times b \times L_b)$	= 0.561 N/mm <sup>2</sup>		
		σc_a / σc_adm = 0.204			
PASS -	- Applied cc	impressive stress is less the	an permissible co	impressive str	ess at bearing
Bending parallel to grain					
Permissible bending stress		$Gm_adm = Gm \times K_3 \times K_7 \times K_8$ :	= 6.838 N/mm <sup>2</sup>		
Applied bending stress		om_a = M / Z <sub>x</sub> = 4.838 N/mn	2		
		0.707 gm_adm = 0.707			
		PASS - Applied bending s	tress is less than	permissible b	ending stress
Shear parallel to grain					
Permissible shear stress		$t_{adm} = \tau \times K_3 \times K_8 = 0.838$ N	Vmm <sup>2</sup>		
Applied shear stress		$t_a = 3 \times F / (2 \times A) = 0.374$	N/mm <sup>2</sup>		
		Ta / Tadm = 0.447			
		PASS - Applied shear	r stress is less th	an permissible	e shear stress
Deflection					
Modulus of elasticity for deflection		E = E <sup>min</sup> = 5800 Nmm <sup>2</sup>			
Permissible deflection		Sadm = min(14 mm, 0.003 ×	mm <b>7.500</b> mm		
Bending deflection		&s1 = 4.769 mm			
Shear deflection		$\delta_{v_s t} = 0.601 \text{ mm}$			
Total deflection		$\delta_{a} = \delta_{b_{a}s1} + \delta_{v_{a}s1} = 5.370 \text{ mm}$	-		
		Sa / Sedm = 0.716			
		PASS - Total o	feffection is less	than permissil	ble deflection

<b>ABP</b>	Michael Barclay Partnership	Job title	Job number	Sheet number	Revision
	consulting engineers	UCL ASTON COLLEGE	6775	1617	1.000
	105-109 Strand London WC2R 0AA	Calculation/Sketch title	Date	Author	Checked
	т 020 7240 1191 г 020 7240 2241	SOLAR PANELS	Surg	JC	
	E london@mbp-uk.com		17		
	www.mbp-uk.com				

EXISTING TRUSSES

- THERE ARE TOO TRUSS THPES
  - EDGE TRUSS BETDEEN GRIDS G AND F
  - E AND D
- EACH THUSS THE IS CHECKED FOR THE PROPOSED LOAD
- THE MUSSES HAVE BEEN MEASURED ON SITE AND APPEAR TO BE, BASED ON DATA SHEETS 16/13 TO 19 FROM METSEC, AS FORLOWS
- EDGE TRUSS, TYPE IN 14, 356 DEEP
- MOMENT CAPACITY = 294NM
  - PERMISSIBLE LINE LOAD = 54N/m FORA 6.5 m SPAN
- CENTRAL TRUSS, TYPE 16016
  - MOMENT CAPACITY = 34 4N-
  - PERMISSIBLE LINE LOAD = 6.34N/m FOR 4 6.5-

MBP Michael Barclay Partnership Job title Job number Sheet number Revision UCLASTON COLLEGE 6775 1618 consulting engineers Calculation/Sketch title Date Author Checked 105-109 Strand London WC2R OAA SULLY SOLAR 32 17 PANELS E london@mbp-uk.com www.mbp-uk.com

EOGE TRUSS CHECK  
LOAD BASED ON ALL COLF = 1.5 LAN(1-2)  
> PANEZ LOAD WITH ACCESS ONLY TO SIDES  
= 
$$(0.7+1.5) \times (2.5/2+0.8/2)$$
  
=  $3.6 \mu N/m$   
 $M = 3.6 \times 6.1 m^2/8 = 16.74 Nm K294Nm$   
 $W = 3.6 \mu N/m K 5 \mu N/m$   
CENTRAL TRUSS CHECK  
BASED ON PANEL LOAD WITH 1.5 LAN(m)  
ON CANTILEVER SOIST REACTION FROM  
TEDDS, SHEET 9.1-9.2  
=  $5.9 \mu N/$  SOIST OR 6.55  $\mu N/m$   
 $M = 6.55 \times 6.1 m^2/8 = 30.5 \mu N/m K 344Nm$   
 $W = 6.55 \mu N/m Y 6.3 \mu N/m K 344Nm$   
 $W = 6.55 \mu N/m Y 6.3 \mu N/m K 344Nm$   
 $M = 6.55 \mu N/m Y 6.3 \mu N/m K 344Nm$   
 $M = 6.55 \mu N/m Y 6.3 \mu N/m K 344Nm$   
 $M = 6.55 \mu N/m Y 6.3 \mu N/m K 344Nm$   
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 $M = 6.55 \mu N/m Y 6.55 \mu N/m K 344Nm$   
 $M = 6.55 \mu N/m Y 6.55 \mu N/m K 344Nm$   
 $M = 6.55 \mu N/m Y 6.55 \mu N/m K 300 m K 344Nm$   
 $M = 6.55 \mu N/m Y 6.55 \mu N/m K 300 m K 344Nm$   
 $M = 6.55 \mu N/m Y 6.55 \mu N/m K 300 m K$ 

Job Ref.	Sheet no/rev. 2	Date         Crikd by         Date         Appld by         Date           07/07/2017         Crikd by         Date         Appld by         Date	Span 2 Dead x 1.00 Imposed x 1.00 Support C Dead x 1.00	Imposed × 1.00 M <sub>max</sub> = 1.117 kNm M <sub>min</sub> = -1.802 kNm	M = max(abs(M <sub>max</sub> ),abs(M <sub>max</sub> )) = 1.802 kNm F <sub>max</sub> = 2.574 kN F = max(abs(F <sub>max</sub> ),abs(F <sub>man</sub> )) = 3.358 kN	Wise         7.848 kN         RA           RA_main         = 1.916 kN         RA_min           RA_main         = 0.419 kN         RA_min           RA_mined         = 0.419 kN         RA_mined	Ra_max         5.931 kN         Ra_max         5.931 kN           Ra_max         1.486 kN         Ra_max         1.486 kN           Rt B         Ra_max         1.446 kN         Ra_max           Rc_max         0.000 kN         Ra_max         0.000 kN		b = 47 mm h = 225 mm N = 1 b <sub>b</sub> = N × b = 47 mm C16 Medium term	L <sub>b</sub> = 100 mm A = N × b × h = 10575 mm <sup>2</sup>	Z <sub>4</sub> = N × b × h <sup>2</sup> / 6 = 396563 mm <sup>3</sup> Z <sub>4</sub> = h × (N × b) <sup>2</sup> / 6 = 82838 mm <sup>3</sup> k = N × b × h <sup>3</sup> / 12 = 44613281 mm <sup>4</sup>	$l_y = h \times (N \times b)^3 / 12 = 1946681 \text{ mm}^4$ $l_k = \sqrt{(l_k / A)} = 65.0 \text{ mm}$
TEKLA Project	MBP Section 105-109 Strand	London WCZRQAA Calc. by		Analysis results Maximum moment	Design moment Maximum shear Design shear	Total load on beam Reactions at support A Unfactored dead load reaction at support A Unfactored imposed load reaction at suppo	Reactions at support B Unfactored dead load reaction at support B Unfactored imposed load reaction at suppo Reactions at support C Unfactored load load load reaction at support O	Unfactored imposed load reaction at suppo	Timber section details Breadth of sections Depth of sections Number of sections in member Overall breadth of member Timber strength class Member details Service class of timber Load duration	Length of bearing Section properties Cross sectional area of member	Section modulus Second moment of area	Radius of gyration
Project Job Ref.	Section Sheet no/rev.	Calc. by Date Chikd by Date Appd by Date	NALYSIS & DESIGN TO BS5268-2:2002 ・ 0 してつ していしたくし エアム いい TEDDS calculation version 1.5.07	2618 Leas arvespe - Combination 1	0.0 1 <u>6 6 6 6 6 6 6 6 6 6 6 6 6 6 6 6 6 6 </u>	kkm Banding Koment Ervelope -1.8 -1.002		2.574 1.9 Chear Force Envelope 2.6 Chear Force 2.	Dead self weight of beam × 1 Dead full UDL 0.450 kN/m Imposed partial UDL 2.130 kN/m from 400 mm to 2100 mm Imposed partial UDL 0.540 kN/m from 0 mm to 400 mm Imposed partial UDL 0.540 kN/m from 2100 mm to 2500 mm Imposed partial UDL 0.540 kN/m from 2500 mm to 2500 mm	1 Support A Dead × 1.00 Imposed × 1.00	Span 1     Dead x 1.00       Imposed x 1.00       Support B       Dead x 1.00	Imposed x 1.00
	MBP 105-109 Sh	WC2R0	ABER BEAM /						lied loading m loads	d combination i combination		

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$ \begin{array}{c cccc} & WB^{C} & & & & & & & & & & & & & & & & & & &$	Tectos	2					
$\label{eq:constraints} \begin{array}{ c c c c c } \hline \begin{tabular}{ c c c c c } \hline \begin{tabular}{ c c c c c c c } \hline \begin{tabular}{ c c c c c c c c c c c c c c c c c c c$	MBP 105-109 Strand	Section				Sheet no /rev. 3	
Modification factors $J_{a} = \langle J_{a} / J_{a} \rangle = 13.6 \text{ mm}$ Modification factors $K_{a} = 1.2$ $K_{a} = 1.2$ Duration of cloarding - Table 17 $K_{a} = 1.03$ $K_{a} = 1.03$ Duration of cloarding - Table 17 $K_{a} = 1.00$ $K_{a} = 1.00$ Total depth of member - d.2 10.5 $K_{a} = 1.00$ $K_{a} = 1.00$ Darmation of cloarding - data depth-or breadth ratio $K_{a} = 1.00$ $K_{a} = 1.00$ Data depth-or breadth ratio $K_{a} = 1.00$ $K_{a} = 1.03$ $K_{a} = 1.03$ Data depth-or breadth ratio $K_{a} = 1.00$ $K_{a} = 1.03$ $K_{a} = 1.03$ Compression perpendicular to grain $K_{a} \times K_{b} \times K_{b} \times K_{b} = 3.033$ Nmm <sup>2</sup> $F_{AL} - Lateral support is inadequate           Compression perpendicular to grain         K_{a} \times K_{b} \times K_{b} \times K_{b} = 8.038 Nmm2 K_{a} K_{a} \times K_{b} \times K_{b} = 8.038 Nmm2           Adding others         C_{a,a,a} = 0.41 K_{a} \times K_{b} \times K_{b} = 8.038 Nmm2 K_{a} = 1.030           Permissible branding stress         C_{a,a,a} = 0.41 K_{a} \times K_{b} \times K_{b} = 8.038 Nmm2 K_{a} \times K_{b} \times K_{b} = 8.038 Nmm2           Permissible branding stress         C_{a,a,a} = 0.41 K_{a} \times K_{b} \times K_{b} = 8.038 Nmm2 K$	London WCZRDAA	Calc. by J	Date 07/07/2017	Chk'd by	Date	App'd by	Date
Modification factors         K= 12b           Duration of identity         K= 12b           Banng serving - 12ble 17         K= 110           Lateral support of member - d.2 (105         K= 130           Lateral support - d.2 (105         K= (300 mm/ly) <sup>111</sup> = 1.05           Lateral support - d.2 (105         K= (300 mm/ly) <sup>111</sup> = 1.05           Lateral support - d.2 (105         K= (300 mm/ly) <sup>111</sup> = 1.05           Lateral support - d.2 (105         K= (300 mm/ly) <sup>111</sup> = 1.05           Primition - fable 17         K= 1.00           Primition - fable 18         N(N × X × L) = 1.32           Primition - fable 19         2.00           Attal depth-to-breadth ratio         N(N × X × L) = 1.32 Nmm <sup>2</sup> Attal depth-to-breadth ratio         0, N × X × L) = 1.32 Nmm <sup>2</sup> Attal depth-to-breadth ratio         0, N × X × L) = 1.32 Nmm <sup>2</sup> Attal depth-to-breadth ratio         0, N × X × L) = 1.32 Nmm <sup>2</sup> Attal depth-to-breadth ratio         0, N × X × L) = 1.32 Nmm <sup>2</sup> Attal depth-to-breadth ratio         0, N × X × L) = 1.32 Nmm <sup>2</sup> Attal depth-to-breadth ratio         0, N × X × L) = 1.32 Nmm <sup>2</sup> Attal depth attact ratios         0, N × X × L) = 1.03 Nmm <sup>2</sup> Attal depticit of strest ratios         0, N × X × L) = 1.03 Nmm <sup>2</sup> Attal			$i_y = \sqrt{(I_y / A)} =$	13.6 mm			
$ \begin{array}{llllllllllllllllllllllllllllllllllll$	Modification factors						8
$ \begin{array}{llllllllllllllllllllllllllllllllllll$	Duration of loading - Table 17		K3 = 1.25				
Total depth of member - d.2 10.6 $K_{\sigma} = (200 \text{ mm}/h)^{0.11} = 1.03$ Lucial depth of member - d.2 10.8 $K_{\sigma} = (200 \text{ mm}/h)^{0.11} = 1.03$ Lucial depth-obmeath ratio - Table 19 2.00 $H/(N \times h) = 4.79$ $FAL$ - Lateral support is inadequate the analysis of opth-obmeath ratio - Table 19 2.00 $H/(N \times h) = 4.79$ $FAL$ - Lateral support is inadequate terms situe depth-obmeath ratio - Table 19 2.00 $H/(N \times h) = 4.79$ $FAL$ - Lateral support is inadequate terms is press (no warre) $\alpha_{\sigma,\sigma} = \alpha_{\sigma,m} \times (N \times K \times K_{\sigma} \times S)$ Nmm <sup>2</sup> Applied bearing stress (no warre) $\alpha_{\sigma,\sigma} = \alpha_{\sigma,m} \times (N \times K \times K_{\sigma} \times S)$ $H/(N \times h) = 1.262 \text{ Nmm}^2$ $Applied bearing stress (no warre) \alpha_{\sigma,\sigma} = \alpha_{\sigma,m} \times (N \times K \times K_{\sigma} \times S) H/(N \times h) = 1.262 \text{ Nmm}^2 Applied bearing stress (no warre) \alpha_{\sigma,\sigma} = \alpha_{\sigma,m} \times (N \times K \times K_{\sigma} \times S) H/(M \times h) = 1.262 \text{ Nmm}^2 Applied bearing stress (no warre) \alpha_{\sigma,\sigma} = \alpha_{\sigma,m} \times (N \times K \times K_{\sigma} \times S) H/(M \times h) = 1.262 \text{ Nmm}^2 Applied bearing stress (no warre) = \alpha_{\sigma,\sigma} / \alpha_{\sigma,m} = 0.6164 M/(M \times h) = 1.262 \text{ Nmm}^2 Applied bearing stress is transitioned stress is transitined stress is transitioned stress i$	Bearing stress - Table 18		Ka = 1.10				
Load sharing - cl.2.9 $K_a = 100$ Lateral support: cl.2.10.8 $H_1(N \times b) = 4.79$ $FAL$ - Lateral support is inadequate Termissible depth-to-breacht ratio. Table 19 $1_1(N \times b) = 4.79$ $FAL$ - Lateral support is inadequate Actual depth-to-breacht ratio. Table 10 $A_{\rm cl}$ and $A_{\rm cl$	Total depth of member - cl.2.10.6		K <sub>2</sub> = (300 mm	$(/h)^{0.11} = 1.03$			
Lateral support - c1.210.8 No lateral support - c1.210.8 No lateral support - c1.210.8 No lateral support - c1.210.8 Actual depth-to-breadth ratio _ Table 19 Actual depth-to-breadth ratio _ Table 19 Compression perpendicular to grain Permissible bearing stress (no varue) _ $\alpha_{ab} = \alpha_{ab} \times k_{b} \times k_{b} = 3.025 \text{ Nmm}^{2}$ Actual depth-to-pressive stress is least than permissible compressive stress at bearing and log parallel to grain _ $\alpha_{ab} = \alpha_{ab} \times k_{b} \times k_{b} \times k_{b} = 1.222 \text{ Nmm}^{2}$ Particular stress _ $\alpha_{ab} = R_{ab} \times k_{b} \times k_{b} = 6.838 \text{ Nmm}^{2}$ Actual depth-to-grain _ $\alpha_{ab} = 0.417$ Particular stress _ $\alpha_{ab} = 0.417$ Particular stress _ $\alpha_{ab} = 0.417$ Particular stress _ $\alpha_{ab} = 0.412$ Particular stress = $R_{ab} \times k_{b} \times k_{b} = 6.838 \text{ Nmm}^{2}$ Actual stress _ $\alpha_{ab} = 0.426 \text{ Nmm}^{2}$ Actual stress = $R_{ab} \times k_{b} \times k_{b} = 0.476 \text{ Nmm}^{2}$ Actual stress = $R_{ab} = 1.222 \text{ Nmm}^{2}$ Actual stress = $R_{ab} = R_{ab} \times R_{b} \times R_{b} = 0.426 \text{ Nmm}^{2}$ Actual stress = $R_{ab} = R_{ab} \times R_{b} = 0.476 \text{ Nmm}^{2}$ Actual stress = $R_{ab} = R_{ab} \times R_{b} = 0.476 \text{ Nmm}^{2}$ Actual stress = $R_{ab} = R_{ab} = 0.476 \text{ Nmm}^{2}$ Actual stress = $R_{ab} = R_{ab} = $	Load sharing - cl.2.9		K <sub>8</sub> = 1.00				
No lateral support       Col         Actual depth-to-breadth ratio       2.0         Actual depth-to-breadth ratio $1/(N \times b) = 4.79$ Actual depth-to-breadth ratio $1/(N \times b) = 4.79$ Compression perpendicular to grain $\alpha_{col} = R_{col} \times (N \times b) = 3.035$ Nmm <sup>2</sup> Permissible bearing stress (no wane) $\alpha_{col} = R_{col} \times (N \times b) = 3.025$ Nmm <sup>2</sup> Applied bearing stress $\alpha_{col} = R_{col} \times (N \times b) = 3.025$ Nmm <sup>2</sup> Applied bearing stress $\alpha_{col} = R_{col} \times (N \times b) = 3.025$ Nmm <sup>2</sup> Applied bearing stress $\alpha_{col} = R_{col} \times (N \times b) = 4.74$ Permissible bearing stress $\alpha_{col} = R_{col} \times (N \times b) = 3.025$ Nmm <sup>2</sup> Applied bearing stress $\alpha_{col} = R_{col} \times (N \times b) = 0.475$ Nmm <sup>2</sup> Applied bearing stress $\alpha_{col} = 0.026$ Nmm <sup>2</sup> Applied bearing stress $\alpha_{col} = 0.026$ Nmm <sup>2</sup> Applied bearing stress $\alpha_{col} = 0.328$ Nmm <sup>2</sup> Applied bearing stress is less than permissible bearing stress       Applied stress         Applied bearing stress $\alpha_{col} = 0.000$ Nmm <sup>2</sup> Applied bearing	Lateral support - cl.2.10.8						
Permissible depth-to-treadth ratio       2.00         Actual depth-to-treadth ratio $h/(N \times b) = 4,79$ $FAL$ . Lateral support is inactequate         Compression perpendicular to grain $m_{(N \times b)} = 4,79$ $FAL$ . Lateral support is inactequate         Permissible bearing stress (no ware) $m_{(2, m)} = (N \times b) = 4,13$ $FAL$ . Lateral support is inactequate         Applied bearing stress $m_{(2, m)} = (N \times b) = 1,220$ Nmm <sup>2</sup> $m_{(2, m)} = 0.417$ $FAL$ . Lateral support is inactequate         Permissible bending stress $m_{(2, m)} = 0.417$ $m_{(2, m)} = 0.417$ $m_{(2, m)} = 0.417$ Pass - Applied compressive stress is less than permissible compressive stress is less than permissible bending stress $m_{(2, m)} = 0.417$ $m_{(2, m)} = 0.417$ Particle $m_{(2, m)} = 0.417$ $m_{(2, m)} = 0.454$ $M m_2^2$ Permissible bending stress $m_{(2, m)} = 0.416$ $m_{(2, m)} = 0.45$ $m_{(2, m)} = 0.464$ Partisciple stress $m_{(2, m)} = 0.416$ $m_{(2, m)} = 0.416$ $m_{(2, m)} = 0.416$ $m_{(2, m)} = 0.416$ Permissible conting stress $m_{(2, m)} = 0.416$ $m_{(2, m)} = 0.416$ $m_{(2, m)} = 0.416$ $m_{(2, m)} = 0.416$ Permissible conting stress $m_{(2, m)} = 0.416$ $m_{(2, m)} = 0.416$ $m_{(2, m)} = 0.416$	No lateral support						
True in a comproved mutual of the compression perpendicular to grain       FAIL - Lateral support is inadequate         Compression perpendicular to grain       FAIL - Lateral support is inadequate         Permissible bearing stress (ro wane) $\alpha_{a,b} = \alpha_{a,1} \times (x, x   x, x   x = 3.025 Nmm^2)$ FAIL - Lateral support is inadequate         Permissible bearing stress (ro wane) $\alpha_{a,b} = \alpha_{a,1} \times (x, x   x = 3.025 Nmm^2)$ $\alpha_{a,b} = \alpha_{a,1} \times (x, x   x = 3.025 Nmm^2)$ Applied bearing stress $\alpha_{a,b} = \alpha_{a,1} \times (x, x   x = 3.025 Nmm^2)$ $\alpha_{a,b} = \alpha_{a,1} \times (x, x   x = 3.025 Nmm^2)$ Permissible bending stress $\alpha_{a,b} = \alpha_{a,1} \times (x, x   x = 3.025 Nmm^2)$ $\alpha_{a,b} = \alpha_{a,1} \times (x, x   x = 3.025 Nmm^2)$ Permissible bending stress $\alpha_{a,b} = \alpha_{a,1} \times (x, x   x = 3.035 Nmm^2)$ $\alpha_{a,b} = \alpha_{a,1} \times (x, x   x = 3.035 Nmm^2)$ Permissible bending stress $\alpha_{a,b} = 0.437$ $\alpha_{a,b} = 0.338 Nmm^2$ $\alpha_{a,b} = 0.336 Nmm^2$ Permissible bending stress $\alpha_{a,b} = 0.73$ $\alpha_{a,b} = 0.338 Nmm^2$ $\alpha_{a,b} = 0.336 Nmm^2$ Permissible bending stress $\alpha_{a,b} = 0.73$ $\alpha_{a,b} = 0.338 Nmm^2$ $\alpha_{a,b} = 0.336 Nmm^2$ Permissible bending stress $\alpha_{a,b} = 0.338 Nmm^2$ $\alpha_{a,b} = 0.336 Nmm^2$ $\alpha_{a,b} = 0.336 Nmm^2$ Permissible bending stress $\alpha_{a,b} = 0.368 Nmm^2$ $\alpha_{a,b} = 0.368 Nmm^2$ <td>Permissible depth-to-breadth ratio -</td> <td>Table 19</td> <td>2.00</td> <td>1</td> <td></td> <td></td> <td></td>	Permissible depth-to-breadth ratio -	Table 19	2.00	1			
Compression perpendicular to grain       Compression perpendicular to grain         Permissible bearing stress       Count is a control of the second stress in the stress is less than permissible compressive stress at bearing areas         Applied bearing stress       Count is a control of the second stress is less than permissible compressive stress at bearing areas         Bending parallel to grain       Count is it is less than permissible compressive stress is less than permissible bending stress         Ophied bending stress       Count is if it is it			11 (N × D) = 4	מ	FAIL -	Lateral sunnor	t is inadament
Permissible bearing stress (no wane) $c_{abi} = c_{api} \times (x_{a} \times x_{b} \times z_{b}) = 1.362 Nimm^2$ Applied bearing stress $c_{abi} = 0.417$ PASS - Applied compressive stress is less than permissible compressive stress at bearing         Bending parallel to grain $c_{a,b} = 0.417$ $C_{a,b} = 1.362 Nimm^2$ $c_{a,b} = 1.362 Nimm^2$ Permissible bending stress $c_{a,b} = 0.417$ $C_{a,b} = 0.417$ $C_{a,b} = 0.417$ Permissible bending stress $c_{a,b} = M_{a,b} \times k_{b} \times k_{b} = 6.338 Nimm^2$ $c_{a,b} = 1.362 Nimm^2$ $c_{a,b} = 1.362 Nimm^2$ Opplied bending stress $c_{a,b} = M_{a,b} \times k_{b} \times k_{b} = 0.417$ $c_{a,b} = 1.362 Nimm^2$ $c_{a,b} = 1.362 Nimm^2$ Permissible bending stress $c_{a,b} = M_{a,b} \times k_{b} \times k_{b} = 6.338 Nimm^2$ $c_{a,b} = 1.362 Nimm^2$ $c_{a,b} = 1.362 Nimm^2$ Polytical bending stress $c_{a,b} = 0.338 Nimm^2$ $c_{a,b} = 6.338 Nimm^2$ $c_{a,b} = 1.362 Nimm^2$ Parse stress $c_{a,b} = 0.338 Nimm^2$ $c_{a,b} = 0.338 Nimm^2$ $c_{a,b} = 0.338 Nimm^2$ $c_{a,b} = 0.338 Nimm^2$ Parse stress $c_{a,a} = 0.366$ PASS - Applied stress is less than permissible stress stress $c_{a,b} = 0.00 mm^2$ $c_{a,b} = 0.00 mm^2$ Parse of deflection $\delta_{a,b} = \delta_{a,a} = 0.300 mm^2$ $\delta_{a,b} = 0.00 mm^2$	Compression perpendicular to dr	nia					r io manchnan
Applied bearing stress $\alpha_{a,a} + \alpha_{a,a} = \beta_{a,m} / (N \times b \times L_{a}) = 1.262 Nimm?$ Applied bearing stress $\alpha_{a,a} + \alpha_{a,a} + \alpha_{a,b} \times k_{b} \times k_{b} = 1.262 Nimm?$ Bending parallel to grain $\alpha_{a,a} + \alpha_{a,a} \times k_{b} \times k_{b} = 6.338 Nimm?$ Person of the bending stress $\alpha_{a,a} + \alpha_{a,b} \times k_{b} \times k_{b} = 6.338 Nimm?$ Optied bending stress $\alpha_{a,a} + \alpha_{a,b} = 0.644$ Provement of the stress $\alpha_{a,a} + \alpha_{a,b} = 0.664$ Provement of the stress $\alpha_{a,a} + \alpha_{a,b} = 0.664$ Provement of the stress $\alpha_{a,a} + \alpha_{a,b} = 0.664$ Provement of the stress $\alpha_{a,a} + \alpha_{a,b} + \beta_{a,b} = 0.338 Nimm?$ Provement of the stress $\alpha_{a,a} + \alpha_{a,b} + \beta_{a,b} = 0.476 Nimm?$ Provement of the stress $\alpha_{a,a} + \alpha_{a,b} + \beta_{a,b} = 0.604$ Provement of the stress $\alpha_{a,a} + \alpha_{a,b} + \beta_{a,b} = 0.600$ Provement of the stress $\alpha_{a,a} + \alpha_{a,a} + \beta_{a,a} + \beta_{a,b} = 0.000$ Provement of the stress $\alpha_{a,a} + \delta_{a,a} + \delta_{a,a} = 3.000$ Provement of the stress $\alpha_{a,a} + \delta_{a,a} = 3.000$ Provement of the stress $\alpha_{a,a} + \delta_{a,a} = 3.000$ Provement of the stress $\alpha_{a,a} + \delta_{a,a} = 3.000$ Provement of the stress $\alpha_{a,a} + \delta_{a,a} = 3.000$ P	Permissible bearing stress (no want	(6	Oc adm = Cont ×	Ka × Ka × Ka = 3	3.025 N/mm <sup>2</sup>		
$\alpha_{ab}/\alpha_{ab}$ $\alpha_{ab}/\alpha_{ab}$ $\alpha_{ab}/\alpha_{ab}$ $\alpha_{ab}/\alpha_{ab}$ Bending parallel to grain $\alpha_{ab}/\alpha_{ab}$ $\alpha_{ab}/\alpha_{ab}/\alpha_{ab}$ $\alpha_{ab}/\alpha_{ab}/\alpha_{ab}$ Permissible bending stress $\alpha_{ab}/\alpha_{ab}/\alpha_{ab}/\alpha_{ab}$ $\alpha_{ab}/\alpha_{ab}/\alpha_{ab}/\alpha_{ab}/\alpha_{ab}$ $\alpha_{ab}/\alpha_{$	Applied bearing stress		Ge a = Ra mav /	$(N \times h \times l_h) = 1$	262 Nimm <sup>2</sup>		
PASS - Applied compressive stress is less than permissible compressive stress at bearing         Bending parallel to grain $\operatorname{Comparestive stress is less than permissible bending stress         Permissible bending stress       \operatorname{Comparestive stress is less than permissible bending stress         Opplied bending stress       \operatorname{Comparestive stress is less than permissible bending stress         Shear parallel to grain       \operatorname{Comparestive stress is less than permissible bending stress         Provention       \operatorname{Comparestive stress is less than permissible bending stress         Shear parallel to grain       \operatorname{Comparestive stress is less than permissible shear stress         Opplied shear stress       \operatorname{Lam} = 1 \times K_0 \times K_0 = 0.338 Nimm?         Opplied shear stress       \operatorname{Lam} = 1 \times K_0 \times K_0 = 0.338 Nimm?         Opplied shear stress       \operatorname{Lam} = 0.664         PASS - Applied shear stress is less than permissible shear stress         Doublus of effection       \operatorname{Lam} = 0.569         PASS - Applied shear stress is less than permissible shear stress       \operatorname{Lam} = 0.564         PASS - Applied shear stress is less than permissible shear stress       \operatorname{Lam} = 0.564         PASS - Applied shear stress is less than permissible shear stress       \operatorname{Lam} = 0.564         Pollulus of effection       \operatorname{Lam} = 0.564 \operatorname{Lam} = 0.564         Pollulus of effection       \operatorname{Lam} = 0.000 mm     $				147			
Binding parallel to grain       Om_Jein = Cm × K × K × K = 6.838 Nmm²         Permissible bending stress $\sigma_{m,a} = M/Z = 4.543 Nmm²$ Permissible bending stress $\sigma_{m,a} = M/Z = 4.543 Nmm²$ Permissible bending stress $\sigma_{m,a} = 0.644$ Prissible stress is less than permissible stress         Prissible deflection $\sigma_{m,a} = 0.644$ Prissible deflection $\sigma_{m,a} = 0.380 Nmm²$ Remissible deflection $\sigma_{m,a} = 0.000 mm$ Remissible deflection $\sigma_{m,a} = 0.000 mm$ $\sigma_{m,a} = 0.000 mm$ $\sigma_{m,a} = 0.000 mm$ Remissible deflection $\sigma_{m,a} = 0.000 mm$ $\sigma_{m,a} = 0.000 mm$ $\sigma_{m,a} = 0.000 mm$ Remissible deflection $\sigma_{m,a} = 0.000 mm$ $\sigma_{m,a} = 0.000 mm$ $\sigma_{m,a} = 0.000 mm$ Remissible deflectio	PASS	- Applied co	ormbressive stres	s is less than	nermissihle c	omoracciva <del>ch</del>	the st handler
Permissible bending stress $o_{m_{sin}} = o_{m} \times k_{s} \times k_{s} \times k_{a} = 6.838 Nimm^{2}$ Opfied bending stress $o_{m_{sin}} = M/Z_{a} = 4.543 Nimm^{2}$ Op is a main of the province of the	Bending parallel to grain	•					
Applied bending stress $\sigma_{n_a} = M/Z_a = 4543$ Nimm? $\sigma_{n_a} = M/Z_a$ $BASS - Applied bending stress is less than permissible bending stress         Bithear parallel to grain       \sigma_{n_a} = 0.64         PASS - Applied bending stress is less than permissible bending stress         Permissible shear stress       \tau_{abm} = \tau \times K_a \times K_a = 0.838 Nimm?         Permissible shear stress       \tau_{abm} = \tau \times K_a \times K_a = 0.838 Nimm?         Permissible shear stress       \tau_{ab} = 3 \times F/(Z \times A) = 0.476 Nimm?         Permissible shear stress       \tau_{abm} = 0.563         Pass - Applied shear stress is less than permissible shear stress         Bellection       \tau_{ab} = 0.300 Nimm?         Remissible deflection       \delta_{a,a} = 0.000 mm         Bendection       \delta_{a,a} = 3.080 mm         Bendection       \delta_{a,a} = 3.080 mm         Bendection       \delta_{a,a} = 0.000 mm         Bendection       \delta_{a,a} = 3.080 mm         Bendection       \delta_{a,a} = 0.000 mm         Bendection       \delta_{a,a} = 3.080 mm         Bass than permissible deflection       \delta_{a,b} \in \delta_{a,c} + \delta_{a,c} = 3.080 mm         Bass than permissible deflection       \delta_{a,b} = 0.000 mm         Bass than permissible deflection       \delta_{a,b} = 0.000 mm         Bass than permissible deflection       \delta_{a,$	<sup>D</sup> ermissible bending stress		Gm adm = Gm × F	(a × K7 × Ka = 6	838 N/mm <sup>2</sup>		
The multiple shear parallel to grain $\sigma_{m,\mu}$ = 0.664         PASS - Applied bending stress is less than permissible bending stress         termsible shear stress $\tau_{abm} = 1, K_{a} \times K_{a} = 0.838 \text{ Nmm}^{2}$ termsible shear stress $\tau_{abm} = 1, K_{a} \times K_{a} = 0.838 \text{ Nmm}^{2}$ termsible shear stress $\tau_{abm} = 0.664$ pplied shear stress $\tau_{abm} = 0.664$ termsible shear stress $\tau_{abm} = 0.638$ termsible shear stress $\tau_{abm} = 0.669$ termsible shear stress $\tau_{abm} = 0.639$ termsible shear stress $\tau_{abm} = 0.630$ teffection $\tilde{c}_{ab} > 0.003 \times L_{ab} = 4.200 \text{ mm}$ bed defection $\tilde{c}_{ab} = \tilde{c}_{ab} = \tilde{c}_{ab} = \tilde{c}_{ab} = 3.080 \text{ mm}$ bear deflection $\tilde{c}_{ab} = \tilde{c}_{ab} = \tilde{c}_{ab} = \tilde{c}_{ab} = 3.080 \text{ mm}$ bear deflection $\tilde{c}_{ab} = \tilde{c}_{ab} = \tilde{c}_{ab} = \tilde{c}_{ab} = 3.080 \text{ mm}$ bear deflection $\tilde{c}_{ab} = \tilde{c}_{ab} = \tilde{c}_{ab} = \tilde{c}_{ab} = 3.080 \text{ mm}$ bear deflection $\tilde{c}_{ab} = \tilde{c}_{ab} = \tilde{c}_{ab} = 3.080 \text{ mm}$ bear deflection $\tilde{c}_{ab} = \tilde{c}_{ab} = \tilde{c}_{ab} = 3.080 \text{ mm}$ bear deflection $\tilde{c}_{ab} = \tilde{c}_{ab} = \tilde{c}_{ab} = 3.080 \text{ mm}$ bear deflection $\tilde{c}_{ab} = \tilde{c}_{ab} = \tilde{c}_{ab}$	Applied bending stress			4 543 Nimm2			
Tonust       Constrained on the constraint of the constraint	0			111111111111111111			
Bits are parallel to grain       taim = t × Ks × Ks = 0.838 Nimm <sup>2</sup> Termissible shear stress       taim = t × Ks × Ks = 0.838 Nimm <sup>2</sup> opplied shear stress       taim = t × Ks × Ks = 0.838 Nimm <sup>2</sup> opplied shear stress       tai = 3 × F / (2 × Å) = 0.476 Nimm <sup>2</sup> topplied shear stress       tai = 3 × F / (2 × Å) = 0.476 Nimm <sup>2</sup> topplied shear stress       tai = 3 × F / (2 × Å) = 0.476 Nimm <sup>2</sup> topplied shear stress is less than permissible shear stress       befaction         Bellection       E = Emi = 5800 Nimm <sup>2</sup> fermissible deflection       So and = min(14 mm, 0.003 × La <sup>2</sup> ) = 4.200 mm         hear deflection       So a = 30.80 mm         dotal deflection       So a = 3.080 mm         dotal deflection       So a = 3.080 mm         dotal deflection       So a = 50.24 + 5.42 = 3.080 mm         for hear deflection       So a = 50.24 + 5.42 = 3.080 mm         for hear deflection       So a = 50.24 + 5.42 = 3.080 mm         for hear deflection       So a = 50.24 + 5.42 = 3.080 mm         for hear deflection       So a = 50.24 + 5.42 = 3.080 mm         for hear deflection       So a = 50.24 + 5.42 = 3.080 mm         for hear deflection       So a + 5.42 = 3.080 mm         for hear deflection       So a + 5.42 = 3.080 mm         for hear deflecti			PASS - Applied	0.664 1 bendina stre	ss is less that	n nermissihla t	anding strace
Termissible shear stress       Team = t × Ko × Ka = 0.838 Nmm <sup>2</sup> oplied shear stress       Team = t × Ko × Ka = 0.838 Nmm <sup>2</sup> train = 0.569       Taam = 0.569         effection       E = Emin = 8800 Nmm <sup>2</sup> following deflection       E = Emin = 8800 Nmm <sup>2</sup> following deflection       E = Emin = 5800 Nmm <sup>2</sup> following deflection       E = Emin = 5800 Nmm <sup>2</sup> following deflection       E = Emin = 5800 Nmm <sup>2</sup> following deflection       E = Emin = 5800 Nmm <sup>2</sup> following deflection       E = Emin = 5800 Nmm <sup>2</sup> following deflection       E = Emin = 5800 Nmm <sup>2</sup> following deflection       E = Emin = 5800 Nmm <sup>2</sup> following deflection       E = Emin = 5800 Nmm <sup>2</sup> following deflection       E = Emin = 5800 Nmm <sup>2</sup> following deflection       E = Emin = 5800 Nmm <sup>2</sup> following deflection       E = Emin = 5800 mm         following deflection       E = 8, 2e = 3.080 mm         following deflection       E = 8, 2e = 3.080 mm         following deflection       E = 8, 2e = 3.080 mm         following deflection       E = 8, 2e = 3.080 mm         following deflection       E = 8, 2e = 3.080 mm         following deflection       E = 8, 2e = 3.080 mm <t< td=""><td>Shear parallel to grain</td><td></td><td>•</td><td></td><td></td><td></td><td>See no Rumuno</td></t<>	Shear parallel to grain		•				See no Rumuno
Train $z \times f_0 \times f_0 = 0.338$ Nmm <sup>2</sup> oplied shear stress $z_a > F / (2 \times k) = 0.476$ Nmm <sup>2</sup> oplied shear stress $z_a > F / (2 \times k) = 0.476$ Nmm <sup>2</sup> $z_a / z_{abin} = 0.569$ PASS - Applied shear stress is less than permissible shear stress         felterion       E = E <sub>min</sub> = 5800 Nmm <sup>2</sup> commissible deflection       Sam = min(14 mm, 0.003 \times La) = 4.200 mm         ending deflection       Sam = min(14 mm, 0.003 \times La) = 4.200 mm         bear deflection       Sa = 3.080 mm         bear deflection       Sa = 8.0.03 mm         bear deflection       Sa = 8.0.25 mm         bear deflection       Sa = 8.0.23 mm         Dial deflection       Sa = 8.0.23 mm         Dial deflection       Sa = 8.0.23 mm         PASS - Total deflection is less than permissible deflection							
opplied shear stress $T_{a} = 3 \times F / (2 \times A) = 0.476 \text{ N/mm}^2$ $T_a / T_{abin} = 0.569$ $T_a / T_{abin} = 0.569$ PASS - Applied shear stress is less than permissible shear stress         helection $E = E_{min} = 5800 \text{ N/mm}^2$ foodulus of elasticity for deflection $\delta_{abin} = \min(14 \text{ mm}, 0.003 \times L_a) = 4.200 \text{ mm}$ for deflection $\delta_{abin} = \min(14 \text{ mm}, 0.003 \times L_a) = 4.200 \text{ mm}$ for deflection $\delta_{abin} = \min(14 \text{ mm}, 0.003 \times L_a) = 4.200 \text{ mm}$ hear deflection $\delta_{abin} = \min(14 \text{ mm}, 0.003 \times L_a) = 4.200 \text{ mm}$ for deflection $\delta_{abin} = \min(14 \text{ mm}, 0.003 \times L_a) = 4.200 \text{ mm}$ hear deflection $\delta_{abin} = 0.000 \text{ mm}$ for deflection $\delta_{abin} = 0.733$ for deflection is less than permissible deflection         for $\Delta_{abin} = 0.733$ PASS - Total deflection is less than permissible deflection	ermissible shear stress		tadm = $\tau \times K_3 \times$	Ka = 0.838 N/m	Ĩ.		
Tay Taken = 0.569       Taken = 0.569         PASS - Applied shear stress is less than permissible shear stress         Rodulus of elasticity for deflection       E = Emin = 5800 Nimm <sup>2</sup> Rodulus of elasticity for deflection       Seam = min(14 mm, 0.003 × Lac) = 4.200 mm         Remissible deflection       Seam = min(14 mm, 0.003 × Lac) = 4.200 mm         Remissible deflection       Seam = min(14 mm, 0.003 × Lac) = 4.200 mm         Remissible deflection       Seam = min(14 mm, 0.003 × Lac) = 4.200 mm         Remissible deflection       Seam = 0.000 mm         Remissible deflection       Seam = 0.733         Refection       Sea = 0.000 mm         Sea = 0.000 mm       Sea = 0.033         PASS - Total deflection is less than permissible deflection	Applied shear stress		τa = 3 × F / (2 ×	c A) = 0.476 N/r	nm²		
PASS - Applied shear stress is less than permissible shear stress         Bellection       fodulus of elasticity for deflection         Rodulus of elasticity for deflection $\delta_{abm} = \min(1, 4 \text{ mm}, 0.003 \times L_a) = 4.200 \text{ mm}$ Remissible deflection $\delta_{abm} = \min(1, 4 \text{ mm}, 0.003 \times L_a) = 4.200 \text{ mm}$ Remissible deflection $\delta_{abm} = \min(1, 4 \text{ mm}, 0.003 \times L_a) = 4.200 \text{ mm}$ Remissible deflection $\delta_{abm} = \min(1, 4 \text{ mm}, 0.003 \times L_a) = 4.200 \text{ mm}$ Remissible deflection $\delta_{abm} = \min(1, 4 \text{ mm}, 0.003 \times L_a) = 4.200 \text{ mm}$ Remissible deflection $\delta_{abm} = 0.000 \text{ mm}$ Read deflection $\delta_{abm} = 0.733$ Relection $\delta_{abm} = 0.733$ PASS - Total deflection is less than permissible deflection			Ta / Tadm = 0.569	•			
effection       E = Emin = 5800 N/mm <sup>2</sup> fodulus of elasticity for deflection $\delta_{ebin}$ = min(14 mm, 0.003 × Liz) = 4.200 mm         fermissible deflection $\delta_{e,z}$ = 3.080 mm         ending deflection $\delta_{e,z}$ = 3.080 mm $\delta_{e,z}$ = 3.080 mm $\delta_{e,z}$ = 3.080 mm         hear deflection $\delta_{e,z}$ = 3.080 mm $\delta_{e,z}$ = 0.000 mm $\delta_{e,z}$ = 3.080 mm         for deflection $\delta_{e,z}$ = 3.080 mm $\delta_{e,z}$ = 0.000 mm $\delta_{e,z}$ = 3.080 mm         for deflection $\delta_{e,z}$ = 0.000 mm $\delta_{e,z}$ = 0.000 mm $\delta_{e,z}$ = 5.020 mm $\delta_{e,z}$ = 0.000 mm $\delta_{e,z}$ = 5.02 mm $\delta_{e,z}$ = 7.33       PASS - Total deflection is less than permissible deflection			PASS - A	oplied shear st	tress is less th	nan permissibl	e shear stress
Image: Second Secon	beflection						
Bernin Emmissible deflection $\delta_{abm} = min(14 mm, 0.003 \times L_{ab}) = 4.200 mm$ ending deflection $\delta_{a,2} = 3.080 mm$ hear deflection $\delta_{a,2} = 3.080 mm$ $\delta_{a,2} = 0.000 mm$ $\delta_{a,2} = 0.000 mm$ for deflection $\delta_{a,2} = 0.000 mm$ $\delta_{a,2} = \delta_{a,2} + \delta_{a,2} = 3.080 mm$ $\delta_{a,3} = 0.733$ PASS - Total deflection is less than permissible deflection	fodulus of elasticity for deflection		$E = E_{min} = 5800$	Nmm <sup>2</sup>			
lending deflection & 22 = 3.080 mm hear deflection & 22 = 0.000 mm ເລື = ວິ2 + 52 = 3.080 mm ເລື = ວິ2 + 52 = 3.080 mm ເລື / ວິ	termissible deflection		Sadm = min(14 n	1m, 0.003 × Ls2	) = 4.200 mm		
hear deflection & &_e^2 = 0.000 mm ເຈົ້ = &2 + δ2 = 3.080 mm ເຈົ້ = %2 + δ2 = 3.080 mm ເຈົ້ = 0.733 PASS - Total deflection is less than permissible deflection	ending deflection		õb_s2 = 3.080 m	F			
otal deflection $\delta_n / \delta_{nam} = 0.733$ PASS - Total deflection is less than permissible deflection	hear deflection		δ <sub>v_s2</sub> = 0.000 m	F			
δ <sub>a</sub> / δ <sub>adm</sub> = 0.733 PASS - Total deflection is less than permissible deflection	otal deflection		õa = õb s2 + õv s2	= 3.080 mm			
PASS - Total deflection is less than permissible deflection			δa / δadm = 0.73:				
			đ	ISS - Total defi	lection is less	than nermissi	hle deflection
							מפופרתחו
<ul> <li>P</li> <li>P</li></ul>							
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BP Michael Barclay Partnership	Job title UCL ASTON COLLEGE	Job number 6775	Sheet number	Revision
consulting engineers		0	16/10	
105-109 Strand London WC2R 0AA	Calculation/Sketch title	Date	Author	Checked
т 020 7240 1191 г 020 7240 2241	SOLAN PANELS	JULY	50	
E london@mbp-uk.com		17		
www.mbp-uk.com		1 7		

CONCLUSION

THE PROPOSED PANEZ ARRANGEMENT IS SMOON ON THE FOLLOWING SMEETS, 11-12

THE ARRANGEMENT IS TO BE FINALISED, BUT AIMS TO LIMIT THE LOADS ON THE 2007 TRUSSES BY SPACING THEM AT 1.5 CIC AS OPPOSED TO 1.1 M CIC ASSUMED IN THE DESIGN

PANELS ARE ALSO PERMITTED ON THE CENTRAL PART OF THE 1960'S ROOF WHICH IS AN R.C. SLATS. THE OUTER PART BETWEEN GRIDS 18 TO 19 AND 20 TO 21 IS NOT TO BE LOADED AS THESE SLABS ARE CANTILEVER STRUCTURES OR SUPPORTED BY CANTILEVERS

THE PANELS TO STRENGTHEN THE WOOD WOOD SCABS AND PROVIDE LATERAL RESTRAINT TO THE WOISTS



	NOTES: 1. THIS DRAWING TO BE READ IN CONJUNCTION WITH ALL RELEVANT ARCHITECTS AND ENGINEERS DRAWINGS AND	CHARLOTTE STREET	C2	09/1	/12/16	AREA BETWEEN GRIDLINES "B" TO "G" AND "6" TO "14" ISSUED FOR CONSTRUCTION AT 1st FLOOR AND ABOVE	JL	Job Astor College, UCL Charlette Street, W1T 40
	SPECIFICATIONS. 2. FOR SETTING OUT REFER TO ARCHITECT'S DRAWINGS	ZONE 1 ZONE 2	C1	25/1	/11/16	AREA BETWEEN GRIDLINES "B" TO "G" AND "6" TO "14" ISSUED FOR	JL	
49						AND ABOVE		Drawing Status
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016 1	4. DO NOT SCALE FROM THE DRAWING OR THE COMPUTER	EXTENSION	T2	14/0	/09/16	ISSUED FOR TENDER	JL	CONSTRUCTION
09/12/2	DIGITAL DATA. ONLY FIGURED DIMENSIONS TO BE USED.	KEY PLAN	Rev	Da	ate	Description	By	
	Subministry (200), 6700(6775), Anter Building, University, College Lander (11 Descriment) 1.2 MDD/Devis(MDD 6775) Anter Building, University, College Lander, Lander and		Michael Barclay	Partners	rshin IIP i	is a Limited Liability Partnership registered in Engla	and and V	Wales - Reg No OC 325164 - Registered address 105-109 Strand

			MBP	Michael Barclay Partnership				
	EIGHTH FLOOR PLAN	DOGED		consulting engineers				
)B	EXISTING BUILDING - PRO	PUSED		105-109 Strand				
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	Drawing Number	Revision		E london@mbp-uk.com				
N	6775 / 121	C2		www.mbp-uk.com				

	( SOLAR			5)				D		(21)	16
	PANEL ZONE 1 ZONE 2 ZONE 2 ZONE 2 ZONE 2 ZONE 2 ZONE 2 TOTAL TOTAL TOTAL	S TO BE IN SING S AS FOLLOWS 1. 4 AT 1.5M C/C 2. 4 AT 1.5M C/C 3. 8 AT 1.5M C/C 4. 2 AT 1.5M C/C 5 4 EQUALLY SF PERMITTED = 3 REQUIRED = 46 YOUT SHOWN I FORMATION ON	SLE ROWS, MA PACED SAY AT 2+28+8 = 68 S NOT A CONS	XIMUM NO OF F	AWING AND IS	1. ADDITION INSTALLED SUPPORT T THREE ROV TIMBER JOI A35 FRAMIN WITH THE N	AL TIMBER BL BELOW EDGE HE EXISTING VS PER PANEL STS. BLOCKIN G ANGLES TO IANUFACTURE	LOCKING 150 S AND CENT WOODWOOL FIXED TIGH NG TO BE FIX O'THE EXISTI ER'S INSTRU	DEEP X 50 WID RE OF SOLAR P SLAB. BLOCKI IT BETWEEN TH (ED WITH PAIRS NG JOISTS IN AG CTIONS.	E TO BE ANELS TO NG TO BE IN E EXISTING OF SIMPSO CCORDANCE	
BB 128	CONTR LAYOU ASSOC	ACTOR TO SUB T OF ALL PLAN IATED LOADS.	MIT CO-ORDIN F, WALKWAYS,							· ·	— — — (H)
130				STEELWORK TO ENABLE REMOVAL OF EXISTING WALL							G G G G G G G G G G G G G G
3G. No. 6775/121		SOLAR PANEL ZONE 3	SOLAR PAN	IEL ZONE 1	SOLAR PAN	EL ZONE 2		SOLAR			(E)
ON REFER TO DF								PANEL ZONE 5	SOLAR - PANEL		
FOR CONTINUAT		SOLAR PANEL ZONE 3	SOLAR PAN		SOLAR PAN	ELZONE 2					— - —(c)
				2000 (0000)							(B)
	     133)							<b>REF.</b> BR1 BR2	BRACING SCHEDULE SIZE COMM 8x80 STEEL PLATE 6x50 STEEL PLATE	A REF. P1 17/ P2 17/ P3 21/ P4 43/ P5 20/	PADSTONE SIZE JWJX450(D)x225(HJmm MAS CONCRETE PADSTONE JWJX350(D)x150(HJmm MAS CONCRETE PADSTONE SWJX100(D)x150(HJmm MAS CONCRETE PADSTONE OWJX215(D)X50(HJmm MAS CONCRETE PADSTONE
NOTES: 1. THIS DRAWI RELEVANT ARC SPECIFICATION 2. FOR SETTING 3. ALL DIMENS NOTED OTHERS	NG TO BE READ IN O HITECTS AND ENGI S. : OUT REFER TO AR ONS ARE IN MILLIM VISE.	CONJUNCTION WITH ALL NEERS DRAWINGS AND CHITECT'S DRAWINGS. IETRES (mm) UNLESS		CHARLOTTE EXISTING BUILDING ZONE 1 NEW REAR	STREET EXISTING BUILDING ZONE 2				C4         0900617         BEAM & COLUMN ADDED           C3         2112/10         ISSUED FOR CONSTRUCTION           AREA BETWEEN GRIDLINES         "G" AND 16" TO 1'14" ISSUED FOR CONSTRUCTION AT 1st FL           AND ABOVE         AND ABOVE GRIDLINES           C1         2511101         "G" AND 76" TO 1'14" ISSUED	JL Job Astor Collec Charlotte S Charlotte S JL Drawing Status	:ge, UCL Street, W1T 4QB
DO NOT SCA DIGITAL DATA	LE FROM THE DRAV ONLY FIGURED DIM 775 - Astor Building, Unive	VING OR THE COMPUTER MENSIONS TO BE USED. I rrsity College London\11 Drawings11.3 MBP/Revit\MBP-6775-A	stor Building, University College London_Jose.Letras.rvt	EXTENSIO KEY PI	LAN		1	Mich	Rev Date Description ael Barclay Partnership LLP is a Limited Liability Partnership regist	By B	NSTRUCTION

140	LEGEND:		
117		EXISTING RC 5	STRUCTURE TO BE RETAINED
		EXISTING BRIC	KWORK TO BE RETAINED
		NEW REINFOR	CED CONCRETE STRUCTURE
		NEW BRICKWO	DRK
	e777773	NEW LOADBEA	ARING BLOCKWORK
	<u></u>	NEW LOADBEA	ARING PARTITIONS
	B1	NEW STEEL BE REFER TO SCH	AM AND REFERENCE. IEDULE FOR TYPE AND SIZE
	EX-B1	EXISTING STER	EL BEAM AND REFERENCE. IEDULE FOR TYPE AND SIZE
	L1	NEW LINTEL A REFER TO SCH	ND REFERENCE IEDULE FOR TYPE AND SIZE.
		NEW TIMBER M	MEMBER; JOIST/RAFTER ETC.
		NEW PADSTO REFER TO SCH	NE AND REFERENCE. IEDULE FOR TYPE AND SIZE.
	C1	COLUMN REFE REFER TO SCH	RENCE. IEDULE FOR TYPE AND SIZE
	ి	COLUMN BELC	W SYMBOL
		TWO-WAY SPA REFER TO LAY	AN OF RC SLAB. OUT FOR DEPTH.
	<del></del>	ONE-WAY SPA REFER TO LAY	N OF RC SLAB. OUT FOR DEPTH.
	<b>~</b>	SPAN OF PREC	CAST CONCRETE FLOOR
	$\longleftrightarrow$	SPAN OF COM	POSITE DECKING
	<del>~,</del>	SPAN OF 50x2 JOISTS AT 35 OF 18mm PLY	200 GRADE C24 TIMBER Omm c/c WITH 1No. LAYER SHEATHING TOP
	<u> </u>	SPAN OF 50x2 JOISTS AT 40 OF 18mm PLY	200 GRADE C24 TIMBER Omm c/c WITH 1No. LAYER SHEATHING TOP
		SPAN OF 50x1 JOISTS AT 40 OF 18mm PLY	50 GRADE C24 TIMBER 0mm c/c WITH 1No. LAYER SHEATHING TOP
	ההו	STEP IN LEVEL	SYMBOL
	ABBREVIATION	S:	
	FL - SSL - FFL - TOB - TOB - TOS - TOS - TOW - L/L - H/L - CR - J -	FORMATION STRUCTURA FINISHED FI TOP OF FOL TOP OF PELE TOP OF PLE TOP OF STE TOP OF WA LOW LEVEL HIGH LEVEL CRANKED S JOINT	I LEVEL LI SLAB LEVEL LOOR LEVEL INDATION M ECAP EL LL TEEL BEAM
		BEAM SCI	HEDULE
	REF. SIZ B1 UKC152x	E 152x30	COMMENTS
	B2 UKC203x	203x46	
	I B3 I UB457x1	91x74	

D

<u>\_\_\_\_\_</u>C4 {

	B3	UB457x191x74	
	B4	PFC150x90x24	
	B5	UKC254x254x89	
	B6	UKC203x203x60	
	B7	UC152x152x23	
	B8	RHS300x200x8	
	B9	UKC152x152x37	
	B10	PFC200x90x30	
	B11	UKB203x133x30	
	B12	RHS150x100x6.3	
	B13	PFC430x100x64	
	B14	UKB152x89x16	
	B15	UKB254x102x28	
	B16	PFC230x90x32	
$\sim$	817	UB305x165x54	$\sim$
	B18	Y UKC254¥254x132 Y	Y Y
	B19	UB152x89x16	
	RCB1	1 250x1000mm DEEP RC	
~		SPANDREDBEAM	
	RCB2	250x350mm DEEP RC	
		DOWNSTAND BEAM	
	RCB3	250x780mm DEEP RC	
		SFANDREL BEAM	

				COLUMN SCHEDULE								
			REF.	SIZE	COMMENTS							
			C1	250x500mm RC COLUMN								
			C2	UKC203x203x46								
IE SCH	EDULE		C3	PFC230x90x32								
		_	C4	PFC200x90x30								
	COMMENTS		C5	UKC254x254x89								
MASS			C6	PFC150x90x24								
			C7	UKC203x203x60								
MASS			C8	UKC152x152x23								
4466			C9	SHS60x60x5								
:			C10	UKC152x152x30								
MASS			C11	RHS150x100x6.3								
			Q12	UB254x102x28								
MASS			C13	Y RHS300x200x8 Y	Y Y							
			C14	UB152x89x16								
		_ \	$\supset$									

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# Physical Properties

16/13

### Imperial

Joist Typa	Overall Depth ins	Distance between Neutral Axes of Booms ins	Moment of Inertia Ixx In*	Moment of Inertia Iyy in*	Radius of Gyration ryy ins	Maximum Permissible Moment Mxx Ib in
		And the second s			and the second	
1W 14	14	12.162	75.5	2.95	1.21	252,118
2W 14	14	12.250	103.1	5.00	1.36	340,207
1W 16	16	14.162	102.0	2.95	1.21	293,578
2W 16	16	14.250	139.1	5.00	1.36	395,751
3W 16	16	14.378	154.2	6.97	1.54	436,027
1W 18	18	16.162	132.5	2.95	1.21	335,032
2W 18	18	16.250	180.6	5.00	1.36	451,295
3W 18	18	16.378	199.6	6.97	1.54	496,679
4W 18	18	16.478	229.3	10.89	1.81	565,475
2W 20	20	18.250	227.4	5.00	1.36	506,839
3W 20	20	18.378	251.0	6.97	1.54	557,331
4W 20	20	18.478	288.0	10.89	1.81	634,109
5W 20	20	18.526	349.6	15.36	1.95	765,457
3W 22	22	20.378	308.3	6.97	1.54	617,983
4W 22	22	20.478	353.3	10.89	1.81	702,743
5W 22	22	20.526	428.8	15.36	1.95	848,093
4W 24	24	22.478	425.4	10.89	1.81	771,377
6W 24	24	22.526	516.0	15.36	1.95	930,729
4W 26	26	24.478	504.2	10.89	1.81	840,011
5W 26	26	24.526	611.4	15.36	1.95	1,013,365
5W 28	28	26.526	714.9	15.36	1.95	1,096,001

### Metric

Joist Type	Overall Depth mm	Distance between Neutral Axes of Booms mm	Moment of Inertia Ixx cm*	Moment of Inertia Iyy cm*	Radius of Gyration ryy cm	Maximum Permissible Moment Mxx kg cm
1W 14	356	309	3141	123	3.07	290.471
2W 14	356	311	4,293	208	3.45	391,960
1W 16	406	360	4,244	123	3.07	338,238
2W 16	406	362	5,791	208	3.45	455,954
3W 16	406	365	6,417	290	3.90	502,357
1W 18	457	411	5,515	123	3.07	385,998
2W 18	457	413	7,515	208	3.45	519,948
3W 18	457	416	8,309	290	3.90	572,236
4W 18	457	419	9,545	453	4.60	651,497
2W 20	508	464	9,465	208	3.45	583,941
3W 20	508	467	10,447	290	3.90	642,114
4W 20	508	469	11,986	453	4.60	730,572
5W 20	508	471	14,552	639	4.94	881,901
3W 22	559	518	12,831	290	3.90	711,993
4W 22	559	520	14.707	453	4.60	809,647
5W 22	559	521	17,847	639	4.94	977,108
4W 24	610	571	17,708	453	4.60	888,722
5W 24	610	572	21,479	639	4.94	1,072,315
4W 26	660	622	20,987	453	4.60	967,797
5W 26	660	623	25,449	639	4.94	1,167,522
5W 28	711	674	29,756	639	4.94	1,262,730

# Load Tables Metric

### Maximum applied loads in kilogrammes per linear metre (ignoring deflection)

N = Standard Bracing System

	and the second	
	por linear mate to cive a detlection o	1
I DAGS IN KHOREAUTHIES	Der undar mene to give a democration o	

H = Hea	vy Bracing System					المرجعة المرجعة				10.00			-	10.0	10.5	120	125	14.0	14.5	15.0	15.5	16.0
SPAN IN	METRES	6.0	6.5	7.0	7.5	8.0	8.5	9.0	9.5	10.0	10.5	11.0	11.5	12.0	12.0	15.0	15.5	14.0				
DEFLECT	ON = SPAN IN mm 360	17	18	19	21	22	24	25	26	28	29	31	32	33	35	36	38	39	40	42	43	44
Joist Type	Wt per metre Steel only		- Andrewson					-		010	100	170	161	147	134	123	_					
1W 14 N	13.5	631	536	460	399	349	307	272	243	218	190	170	101							-		
H	14.0	631								400	440	3.0.2	20	79	70	62						
1W 14			497	398	323	266	222	187	169	130	266	241	219	199	182	167	154	142	131	121		
2W 14 N	17.0	674	621	575	536	472	416	369	3.29	230	200	241	210				The second second					-
	17.5	853	724	622	539	472				100	4.54	140	100	108	95	85	75	68	61	55		
2W 14			680	544	442	364	304	256	217	180	221	200	190	173	159	146	134	124	114	106		
1W 16 N	13.5	737	626	538	467	408	360	320	285	200	201	203	100	- Longer								
H	14.0	737		a							470	4.00	404	106	9.4	84	75	67	60	54		
TW/ 16			100	538	437	360	300	253	215	104	158	1.30	257	235	215	197	182	168	155	144	133	124
2W 16 M	17.0	768	707	656	611	552	486	432	386	340	312	203	201	200	- 10							
	18.0	995	845	726	630	552				0.04	114.12	4.00	105	1.85	129	114	102	91	82	74	67	61
2W 16					597	491	410	345	293	201	- 21/ baa	212	282	250	237	217	200	185	171	158	147	137
3W 16 M	18.5	772	711	658	613	574	536	476	425	381	344	312	200	200						A		man aliver
	19.5	1096	931	800	694	608	536				0.44	200	100	161	142	127	113	101	91	82	75	66
214 16				100 - C	661	545	454	382	325	279	241	208	210	100	182	167	154	142	131	122	113	105
1W 18 1	15.0	842	716	615	534	467	412	366	327	293	200	240	210	100	102		Contraction of the					
	15.0	842			-//								125	1.00	122	109	97	87	78	71	64	61
11/1 1.9					- 7 U.		390	329	279	239	207	189	107	260	247	227	209	193	178	165	154	143
2W 18 1	18.5	854	786	729	679	631	556	494	441	397	358	324	200	200								
	1 18.5	1136	965	829	720	631			N.		000	DAE	216	189	167	148	132	119	107	96	87	70
21/ 18			See Lor			the local sector	532	448	381	326	282	240	210	20.9	273	250	231	213	197	183	170	158
3W 18	19.5	858	790	732	682	638	599	545	487	437	395	300	320	200	210			-				1000
	20.0	1251	1063	914	793	695	613	545			010	0.14	202	200	195	164	146	131	118	107	97	8
3W 18							588	495	421	361	312	2/1	274	220	310	285	263	243	224	208	194	180
AW 18	21.5	862	794	735	685	640	601	567	536	498	449	407	371	333	510	200	-					
	1 22.5	1345	1210	1040	903	791	698	620	554	498	010	010	0.22	240	212	189	168	151	136	123	111	10
AW 18	the second s				A martines		676	569	484	415	358	312	213	240	270	256	236	218	202	187	174	16;
2W 20	19.0	937	863	800	745	697	626	556	497	447	403	366	333	504	213	200	200					
	20.0	1277	1085	933	810	710	626				-		0.70	200	210	197	167	150	135	122	110	10
21M 20				-		V. Surger	and the second second		480	411	365	905	270	238	206	282	259	240	222	206	191	17
3W 20	N 20.0	935	861	798	743	696	653	612	547	491	444	402	300	334	300	202	205					
	4 21.5	1405	1193	1026	891	780	689	612	-		_		240	0.00	202	206	194	165	149	134	122	11
2111 20	Contractor of this care to a		and the second second						530	454	392	341	298	203	232	200	104	100	110		a stars	

Each timber fillet weighs approximately 1.3 kg per metre



### **Boom Section**



Imperial					Metric				
Boom	A	B ins	C ins	Thickness	Boom	A mm	B mm	C mm	Thickness mm
1W	4	1	<del>9</del> 16	0.124	1W	102	25	14	3
2W	41/2	11	34	0.157	2W	114	32	19	4
ЗW	51	1릏	34	0.157	ЗW	133	41	19	4
4W	6	2	1	0.157	4W	152	51	25	4
5W	61/2	21	1	0.187	БW	165	57	25	5
		1							

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This brochure contains information regarding the Metsec range of short and intermediate span open web joists, together with information on Metsec structural steel frames. Each main section is clearly tabulated, but the following index may be of assistance in locating detailed items of information.

No prices are given in this brochure but quotations will be quickly made against specific enquiries sent either direct to the Company or through its regional sales offices.

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## Introduction 16/17

#### General

In conformity with Metal Sections' policy of reviewing its products in the light of modern requirements and manufacturing techniques, the Company has rationalised its short and intermediate span open web beam ranges in order to produce (at competitive prices) a fully co-ordinated modular range of components for the building industry.

Metal Sections' intermediate span joists are now brought into line with the short span range in that they incorporate solid bar bracing. The use of this form of bracing has facilitated the introduction of modern manufacturing techniques and has increased the load carrying capacity. It has also led the way to the more efficient use of the short and intermediate span joists in complete structural frames.

#### Metric

In keeping with national plans for the change over to the metric system, load tables are given in both imperial and metric units.

### Service

Metal Sections' beam range continues to be backed by a free and comprehensive technical design service which can be made use of through technical representatives from the regional sales offices or through specialised design engineers based at the Company's head office. In conjunction with the supply of components and of complete structural frames, Metal Sections offer an erection service with fully qualified supervision. The improved production techniques enable quicker deliveries to be effected and facilitate the maintenance of continuous quality control by technical supervision at all stages in production.

### Applications

Metal Sections' joists can be used for floor and roof construction in conjunction with steelwork, brickwork and concrete, or any combination of these materials. The short span range is intended for use in spans up to 40 feet with beam depths of 7, 8, 10, 12, 14 inches, and the intermediate range, for spans of between 20 feet and 60 feet with beam depths ranging between 14 inches and 28 inches, in 2 inch increments.

### Advantages

Economies in the design of structural supports and foundations are made possible because :

(a) The light weight of Metal Sections' joists reduces the dead load.

(*b*) The open web design facilitates the installation of electrical, plumbing and heating services.

(c) The steel construction ensures that the joists will not shrink or warp, thus distortion to ceilings and floors is avoided.

(*d*) Erection costs are reduced because all joists are tailored to the job in question and are simple and quick to erect.

(e) Ease of handling on site, combined with simple end fixings and well designed accessories, reduce site labour to a minimum.

(*f*) The stoved primer finish ensures greatly reduced maintenance costs.

(g) All necessary connecting and holding down bolts are included in the supply.

## Lateral Restraint

16/19

TRUSSES HAVE RESTRAINT EVERY 4TH JOIST BY CLEATS, 2.7M C/C

(i) The figures given in the load tables assume that the top chord is fully restrained laterally. This restraint must be obtained from a positive connection between the top chord and the roof or floor slab.

(ii) During the erection period bridging is required to enable the joists to support construction loads and to assist in lining up. The recommended spacing of this bridging is given in table A. The spacing of lateral restraints shown have been calculated for an I/r (unsupported length divided by the radius of gyration about the vertical axis of the chord) of 120. Joists up to 26 in (660 mm) deep: Roof joistslateral support to be provided by straight angle bridging to top chord only. Floor joists-lateral support to be provided by straight angle bridging connecting to top and bottom chords. Joists over 26 in (660 mm) deep: Lateral stability to be provided by angles cross braced from top to bottom chords. When the centres of joists exceed 3 ft 6 in the crossed angles should be connected together by a single bolt at their intersection.

Recommended sizes of bridging angles for various centres of joists are shown in tables B and C.

A	Maximum Spacing of Lateral Restraint	
Chord Size	ft ins	mm
1W	12′ 0″	3680 C
2W	13′ 6″	- 4110 J
ЗW	15′ 0″	4670 E
4W	18' 0"	5510 F
5W	19' 6"	5920 B

B	Centres of Joists ft	Size of Bridging Angle ins
	0 - 5	1¼ x 1¼ x 0.100
	over 5 - 8	2 x 2 x 0.124
	over 8 - 10	2½ x 2½ x 0.157
	over 10 - 12	3 x 3 x 0.25

Centre of Joists mm	Size of Bridging Angle mm
0 - 1550	32 x 32 x 2.5
1550 - 2450	50 x 50 x 3
2450 - 3110	64 x 64 x 4
3110 - 3740	76 x 76 x 6