

Job No: 1350

Date: Sep 2020

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<u>General</u>

These calculations cover the design of the structural elements for the proposed roof replacement.

The design is based on:

BS 6399-Loading BS 8110-Concrete BS 5950-Steel BS 5628-Masonry BS 5268-Timber

General loading

Roof:

•	Dead load:	$1.0 \frac{\mathrm{kN}}{\mathrm{m}^2}$
•	Imposed load:	$0.75 \frac{\mathrm{kN}}{\mathrm{m}^2}$

Ground, floors:

•	Dead load:	$0.85 \frac{\mathrm{kN}}{\mathrm{m}^2}$
•	Imposed load:	$1.5 \frac{\mathrm{kN}}{\mathrm{m}^2}$

Walls:

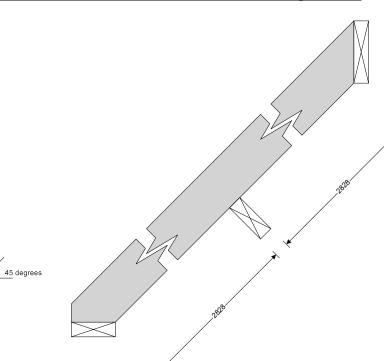
220 mm Brick wall

- $4.4 \frac{kN}{m^2}$ 120 mm Brick wall
- 2.4120 mm Stud wall
- 0.6

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<u>ROOF:</u>

R - TIMBER RAFTER DESIGN (BS5268-2:2002) - 150x50 @400 c/c



TEDDS calculation version 1.0.03

Rafter details

Breadth of timber sections; Depth of timber sections; Rafter spacing; Rafter slope; Clear span of rafter on horizontal; Clear span of rafter on slope; Rafter span; Timber strength class;

Section properties

Cross sectional area of rafter; Section modulus; Second moment of area; Radius of gyration;

Loading details

Rafter self weight; Dead load on slope; Imposed load on plan; Imposed point load;

Modification factors

Section depth factor; Load sharing factor; b = 50 mm h = 150 mm s = 400 mm α = 45.0 deg L_{clh} = 2000 mm L_{cl} = L_{clh} / cos(α) = 2828 mm Continuous C16

 $A = b \times h =$ **7500**mm² $Z = b \times h² / 6 =$ **187500**mm³ $I = b \times h³ / 12 =$ **14062500**mm⁴ $r = \sqrt{(I / A)} =$ **43.3**mm

$$\begin{split} F_{j} &= b \times h \times \rho_{char} \times g_{acc} = \textbf{0.02 kN/m} \\ F_{d} &= \textbf{1.00 kN/m}^{2} \\ F_{u} &= \textbf{0.75 kN/m}^{2} \\ F_{p} &= \textbf{0.90 kN} \end{split}$$

 $K_7 = (300 \text{ mm} / \text{h})^{0.11} = 1.08$ $K_8 = 1.10$

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Consider long term load cor	ndition					
Load duration factor;		K ₃ = 1.0	D			
Total UDL perpendicular to rat	iter;	$F = F_d \times$	$\cos(\alpha) \times s + F_j$	$\times \cos(\alpha) = 0.2$	299 kN/m	
Notional bearing length;		$L_b = F \times$	L _{cl} / [2 \times (b $\times \sigma_{cr}$	_{p1} × K ₈ - F)] =	4 mm	
Effective span;		$L_{eff} = L_{cl}$	+ L _b = 2832 mm	ı		
Check bending stress at pur	lin					
Bending stress parallel to grain	n;	σm = 5.3	00 N/mm²			
Permissible bending stress;		$\sigma_{m_{adm}} =$	$\sigma_m \times K_3 \times K_7 \times I$	K ₈ = 6.292 N	/mm ²	
Applied bending stress;		σm_max =	$F \times L_{\text{eff}}^2$ / (8 \times Z	2) = 1.598 N/r	nm²	
			PASS - Appl	lied bending	stress within per	missible lii
Check compressive stress p	parallel to grain	n at purlin				
Compression stress parallel to	•	-)0 N/mm ²			
Minimum modulus of elasticity	,	E _{min} = 5 8	300 N/mm²			
Compression member factor;		K ₁₂ = 0.6	62			
Permissible compressive stres	s;	—	$\sigma_{c} \times K_{3} \times K_{8} \times K_{8}$			
Applied compressive stress;					$n(\alpha) / 3) / (8 \times A) =$	
		PA	ASS - Applied c	compressive	stress within per	missible li
Check combined bending an	nd compressiv	e stress parall	el to grain at pu	urlin		
Euler stress;		$\sigma_e = \pi^2 \times$	$E_{min} / \lambda^2 = 13.3$	83 N/mm ²		
Euler coefficient;		K _{eu} = 1 -	- (1.5 × σ_{c_max} ×	K ₁₂ / σ _e) = 0.	989	
Combined axial compression a	and bending ch	eck; $\sigma_{m_{max}}$ /	$(\sigma_{m_{adm}} \times K_{eu})$ +	σ_{c_max} / σ_{c_ad}	m = 0.290 ; < 1	
	PASS - Co	ombined comp	ressive and be	ending stres	ses are within per	missible li
Check bending stress in low	ver portion of r	after				
Bending stress parallel to grain	n;	σ _m = 5.3	00 N/mm ²			
Permissible bending stress;		$\sigma_{m_{adm}} =$	$\sigma_m \times K_3 \times K_7 \times I$	K ₈ = 6.292 N	/mm²	
Applied bending stress;		$\sigma_{m_{max}} =$	$9\times F \times L_{eff}^{2}/$ (12	28 × Z) = 0.8	99 N/mm ²	
			PASS - Appl	lied bending	stress within per	missible lii
Check compressive stress p	parallel to grain	n in lower porti	on of rafter			
Compression stress parallel to	grain;	σc = 6.80)0 N/mm²			
Minimum modulus of elasticity	,	E _{min} = 5 8	300 N/mm²			
Compression member factor;		K ₁₂ = 0.6	52			
Permissible compressive stres	s;	$\sigma_{c_{adm}} =$	$\sigma_{c} \times K_{3} \times K_{8} \times K_{8}$	(₁₂ = 4.623 N/	/mm ²	
Applied compressive stress;		σ _{c_max} =	$3 \times F \times L_{eff} \times (cc)$	$ot(\alpha)$ + 13 × ta	an(lpha) / 3) / (8 $ imes$ A) =	= 0.226 N/m
		PA	ASS - Applied c	compressive	stress within per	rmissible lii
Check combined bending ar	nd compressiv	e stress parall	el to grain in lo	wer portion	of rafter	
Euler stress;		$\sigma_e = \pi^2 \times$	ε E _{min} / λ ² = 13.3	83 N/mm ²		
Euler coefficient;		K _{eu} = 1 -	- (1.5 $\times \sigma_{c_{max}} \times$	K ₁₂ / σ _e) = 0.	984	
Combined axial compression a	and bending ch	eck; σ _{m_max} /	$(\sigma_{m_adm} \times K_{eu})$ +	$\sigma_{c_{max}}$ / $\sigma_{c_{ad}}$	m = 0.194 ; < 1	
	PASS - Co	ombined comp	ressive and be	ending stres	ses are within per	rmissible li
Check shear stress						
Shear stress parallel to grain;		τ = 0.67	0 N/mm ²			
		$\tau_{adm} = \tau$	× K ₃ × K ₈ = 0.73	7 N/mm ²		
Permissible shear stress;						
Permissible shear stress; Applied shear stress;		τ _{max} = 15	$5 \times F \times L_{eff}$ / (16	× A) = 0.106	N/mm ²	

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Check deflection									
Permissible deflection;		$\delta_{adm} = 0.1$	003 × L _{eff} = 8.4 9	96 mm					
Bending deflection;			L_{eff}^4 / (185 × Em		0 mm				
Shear deflection;			$F \times L_{eff}^2 / (5 \times E)$						
Total deflection;			+ δ _s = 0.927 mi	,					
· · · · · · · · · · · · · · · · · · ·		•max •b			lection within pe	rmissible lim			
Consider medium term load co	ondition								
Load duration factor;		K ₃ = 1.2	5						
Total UDL perpendicular to rafte	r;	$F = [F_u \times$	$\cos(\alpha)^2 + F_d \times f_d$	$\cos(\alpha)] \times s + I$	$F_i \times \cos(\alpha) = 0.44$	9 kN/m			
Notional bearing length;		-	L _{cl} / [2 × (b × σ _{cr}	() <u>-</u>	, , ,				
Effective span;			+ L _b = 2834 mm	/=					
Check bending stress at purli	n								
Bending stress parallel to grain;		σm = 5.3 0	00 N/mm²						
Permissible bending stress;		$\sigma_{m_{adm}} =$	$\sigma_m imes K_3 imes K_7 imes I$	K ₈ = 7.865 N/I	mm²				
Applied bending stress;		σ _{m_max} =	$\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) = 2.403 \text{ N/mm}^2$						
			PASS - Appl	lied bending	stress within pe	rmissible lim			
Check compressive stress particular	rallel to grain a	t purlin							
Compression stress parallel to g	rain;	σc = 6.80	0 N/mm ²						
Minimum modulus of elasticity;		E _{min} = 58	800 N/mm ²						
Compression member factor;		K ₁₂ = 0.5	8						
Permissible compressive stress;		$\sigma_{c_{adm}} = 0$	$\sigma_{c} \times K_{3} \times K_{8} \times K_{8}$	1 ₁₂ = 5.406 N/r	mm²				
Applied compressive stress;		$\sigma_{c_{max}} = 3$	$3 \times F \times L_{eff} \times (cc)$	$\operatorname{pt}(\alpha)$ + 8 × tan	$(\alpha) / 3) / (8 \times A) =$	0.233 N/mm ²			
		PA	SS - Applied o	compressive	stress within pe	rmissible lim			
Check combined bending and	compressive s	-							
Euler stress;			$E_{min} / \lambda^2 = 13.3$						
Euler coefficient;		K _{eu} = 1 –	\cdot (1.5 × $\sigma_{c_{max}}$ ×	K ₁₂ / σ _e) = 0.9	85				
Combined axial compression an	•		[σ _{m_adm} × K _{eu}) +						
			ressive and be	ending stress	ses are within pe	rmissible lim			
Check bending stress in lower	r portion of raft								
Bending stress parallel to grain;			00 N/mm ²		<u>,</u>				
Permissible bending stress;		-	$\sigma_{\rm m} \times {\rm K}_3 \times {\rm K}_7 \times {\rm I}$						
Applied bending stress;		σ _{m_max} =	$\sigma_{m_max} = 9 \times F \times L_{eff}^2 / (128 \times Z) = 1.352 \text{ N/mm}^2$ PASS - Applied bending stress within permissible lim						
a				nea benaing	stress within per	riiissidie iiii			
Check compressive stress par	-	-							
Compression stress parallel to g	rain;		0 N/mm ²						
Minimum modulus of elasticity; Compression member factor;		Emin = 58 K ₁₂ = 0.5	800 N/mm ²						
Permissible compressive stress;			ν ο σ _c × K3 × K8 × K	12 = 5 406 N/r	mm ²				
Applied compressive stress;		-			$\ln(\alpha) / 3) / (8 \times A)$	= 0.339 N/mn			
Applied compressive sitess,					stress within pe				
Check combined bending and	compressive s			-	-				
Euler stress;		-	$E_{min} / \lambda^2 = 13.3$	-					
					70				
Euler coefficient;		$\kappa_{eu} = 1 -$	\cdot (1.5 $\times \sigma_{c_{max}} \times$	K ₁₂ / σ _e) = U.9	10				

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	PASS - Con	nbined compre	essive and ben	ding stresses	are within per	missible l		
Check shear stress								
Shear stress parallel to grain;		τ = 0.670 Ι	N/mm²					
Permissible shear stress;		$\tau_{adm} = \tau \times I$	K ₃ × K ₈ = 0.921	N/mm ²				
Applied shear stress;		τ_{max} = 15 ×	\times F $ imes$ L _{eff} / (16 $ imes$	A) = 0.159 N/m	im ²			
			PASS - App	olied shear stro	ess within per	missible l		
Check deflection								
Permissible deflection;		$\delta_{adm} = 0.00$	03 × L _{eff} = 8.501	mm				
Bending deflection;		$\delta_b = F \times L_e$	$_{\rm ff}^4$ / (185 $ imes$ E _{mean}	n × I) = 1.264 m	m			
Shear deflection;		δ_s = 12 × F	$^{2} \times L_{eff}^{2} / (5 \times E_{n})$	nean × A) = 0.13 ′	1 mm			
Total deflection;		$\delta_{max} = \delta_b +$	δ_s = 1.396 mm					
			PASS	- Total deflect	ion within per	missible l		
Consider short term load co	<u>ndition</u>							
Load duration factor;		K ₃ = 1.50						
Total UDL perpendicular to rat	iter;	$F = F_d \times co$	$ps(\alpha) \times s + F_{j} \times s$	cos(α) = 0.299	kN/m			
Notional bearing length;		$L_{b} = [F \times L]$	$_{cl}$ + $F_{p} \times \cos(\alpha)$]	/ [2 × (b × σ_{cp1}	× K ₈ - F)] = 6 m	nm		
Effective span;		$L_{eff} = L_{cl} +$	L _{eff} = L _{cl} + L _b = 2835 mm					
Check bending stress at pur	lin							
Bending stress parallel to grain		σm = 5.300	N/mm ²					
Permissible bending stress;		$\sigma_{\rm m} adm = \sigma$	$m \times K_3 \times K_7 \times K_8$	a = 9.438 N/mm	2			
Applied bending stress;		σ _{m max} = F	×L _{eff} ²/(8×Z)+3×F	- _p ×cos(α)×L _{eff} /(;	32×Z) = 2.503	V/mm ²		
		_	. ,	ed bending str	,			
Check compressive stress p	arallel to grain a	at purlin						
Compression stress parallel to	grain;	σc = 6.800	N/mm ²					
Minimum modulus of elasticity	;	E _{min} = 580	0 N/mm²					
Compression member factor;		K ₁₂ = 0.54						
Permissible compressive stres	ss;	$\sigma_{c_{adm}} = \sigma_{c}$	$\times K_3 \times K_8 \times K_{12}$	e = 6.047 N/mm	2			
Applied compressive stress;		$\sigma_{c_{max}} = 3$	$F \times L_{eff} \times (cot(\alpha) +$	8×tan(α)/3)/(8×	A)+F _p ×sin(α)/A	= 0.240		
N/mm ²		DAG	C Applied on	marcocius etr	aaa within nar	minnihlal		
Check combined bending ar	nd comprossivo		S - Applied co	-	ess within per	iiiissibie i		
Euler stress;		-	$E_{min} / \lambda^2 = 13.358$					
Euler coefficient;			$1.5 \times \sigma_{c max} \times K$					
Combined axial compression a	and bending chec		$m_{adm} \times K_{eu}$) + σ		0.309: < 1			
	-	nbined compre				missible l		
Check bending stress in low	er portion of raf	ter						
Bending stress parallel to grain	ו;	σ _m = 5.300	N/mm ²					
Permissible bending stress;		$\sigma_{m_{adm}} = \sigma$	$_{ m m} imes m K_3 imes m K_7 imes m K_8$	3 = 9.438 N/mm	2			
Applied bending stress;		$\sigma_{m_{max}} = F$	×L _{eff} ²/(16×Z)+13 PASS - Applie	3×F _p ×cos(α)×L _e ed bending str	. ,			
Check compressive stress p	arallel to grain i	n lower portior		5	•			
Compression stress parallel to	-	σc = 6.800						
Minimum modulus of elasticity	-	E _{min} = 580						
Compression member factor;		K ₁₂ = 0.54						

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Permissible compressive stres	ss;	$\sigma_{c_{adm}} = c$	$\sigma_c imes K_3 imes K_8 imes K$	K ₁₂ = 6.047 N/	mm ²	
Applied compressive stress;		$\sigma_{c_{max}} = 3$	B×F×L _{eff} ×(cot(α))+4×tan(α))/(8	$B \times A$)+ $F_p \times sin(\alpha)/A =$	= 0.297 N/mm
		PA	SS - Applied o	compressive	stress within pe	rmissible lim
Check combined bending an	nd compressi	ve stress paralle	l to grain in lo	ower portion	of rafter	
Euler stress;		σ_{e} = π^{2} ×	$E_{min} / \lambda^2 = 13.3$	358 N/mm ²		
Euler coefficient;		K _{eu} = 1 –	(1.5 $ imes \sigma_{c_{max}} imes$	K ₁₂ / σ _e) = 0.9	82	
Combined axial compression a	and bending cl	neck; σ _{m_max} / ($\sigma_{m_{adm}} imes K_{eu}$) +	σ_{c_max} / σ_{c_adr}	n = 0.346 ; < 1	
	PASS - C	Combined comp	ressive and be	ending stress	ses are within pe	rmissible lim
Check shear stress						
Shear stress parallel to grain;		τ = 0.670	N/mm ²			
Permissible shear stress;		$ au_{adm} = au imes$	K ₃ × K ₈ = 1.10)6 N/mm ²		
Applied shear stress;		τ _{max} = 15	\times F \times L _{eff} / (16	\times A) + 3 \times F _p	$\times \cos(\alpha) / (2 \times A)$	= 0.233 N/mn
			PASS - A	pplied shear	stress within pe	rmissible lim
Check deflection						
Permissible deflection;		$\delta_{adm} = 0.0$	003 × L _{eff} = 8.5	04 mm		
Bending deflection;		$\delta_{b} = L_{eff}^{3}$	× (F×L _{eff} /185 +	- 0.015×F _p ×co	$DS(\alpha)) / (E_{mean} \times I)$	= 2.600 mm
		$\delta_s = 12 \times$	$L_{eff} \times (F \times L_{eff} + 2)$	$2 \times F_{p} \times cos(\alpha))$	$(5 \times E_{mean} \times A) = 0$	0.219 mm
Shear deflection;		-				
Shear deflection; Total deflection;			+ δ _s = 2.818 m	m		

PURLINS PR1 150x150 Timber Joists by inspection

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D1 - TIMBER MEMBER DESIGN TO BS5268-2:2002 - 150x75 Timber Joists

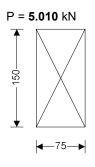
dTEDDS calculation version 1.5.07

Load derivation:

 $P = 4.05 \times 0.5 \times (1.0 + 0.75) \times 1.25 \times 1.6 \times \cos 45 = 5.01 \text{ kN}$

Analysis results

Design axial compression;



	Timber	section	details
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Breadth of sections;	b = 75 mm
Depth of sections;	h = 150 mm
Number of sections in member;	N = 1
Overall breadth of member;	b _b = N × b = 75 mm
Timber strength class;	C16
Member details	
Service class of timber;	1
Load duration;	Long term
Effective length - cl.2.11.3	
Unbraced length in x-axis;	L _x = 2700 mm
Effective length factor in x-axis - Table 21;	K _x = 1
Effective length in x-axis;	L _{ex} = L _x × K _x = 2700 mm
Unbraced length in y-axis;	L _y = 2700 mm
Effective length factor in y-axis - Table 21;	K _y = 1
Effective length in y-axis;	$L_{ey} = L_y \times K_y = 2700 \text{ mm}$
Section properties	
Cross sectional area of member;	A = N \times b \times h = 11250 mm ²
Section modulus;	Z_x = N × b × h ² / 6 = 281250 mm ³
	$Z_y = h \times (N \times b)^2 / 6 = 140625 \text{ mm}^3$
Second moment of area;	$I_x = N \times b \times h^3 / 12 = 21093750 \text{ mm}^4$
	$I_y = h \times (N \times b)^3 / 12 = 5273437 \text{ mm}^4$
Radius of gyration;	i _x = √(I _x / A) = 43.3 mm
	i _y = √(I _y / A) = 21.7 mm
Modification factors	
Duration of loading - Table 17;	K ₃ = 1.00
Total depth of member - cl.2.10.6;	K ₇ = (300 mm / h) ^{0.11} = 1.08
Load sharing - cl.2.9;	K ₈ = 1.00

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Slenderness ratio - cl.2.11.4

Permissible slenderness ratio; Slenderness ratio; λ_{max} = **180**

 $\lambda = \max(L_{ex} / i_x, L_{ey} / i_y) = 124.708$

PASS - Slenderness ratio is less than permissible slenderness ratio

Compression parallel to grain

Permissible compressive stress; Applied compressive stress;
$$\begin{split} \sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \textbf{1.872} \ \text{N/mm}^2 \\ \sigma_{c_a} = \text{P} \ / \ \text{A} = \textbf{0.445} \ \text{N/mm}^2 \end{split}$$

PASS - Applied compressive stress is less than permissible compressive stress

B1 - 152 UC 23 Tie beam Padstone P1 350x100x10mm Th. Steel Plate

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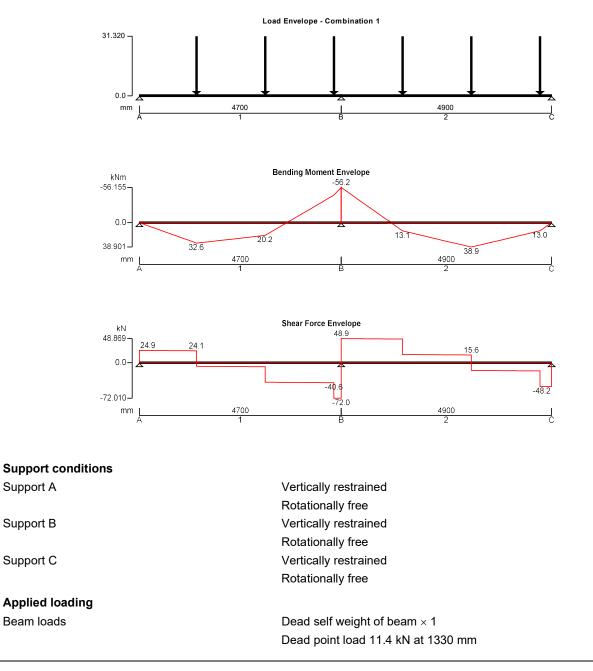
B2 - STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

Dead load: 4.0 x 0.5 x 1.0 x 1.25 x 1.6 x 1.4 = 5.6 kN - point load Nd = 2 x 5.7 = 11.4 kN

Imposed load: 4.0 x 0.5 x 0.75 x 1.25 x 1.6 x 16 = 4.8 kN NI = 2 x 4.8 = 9.6 kN

TEDDS calculation version 3.0.05



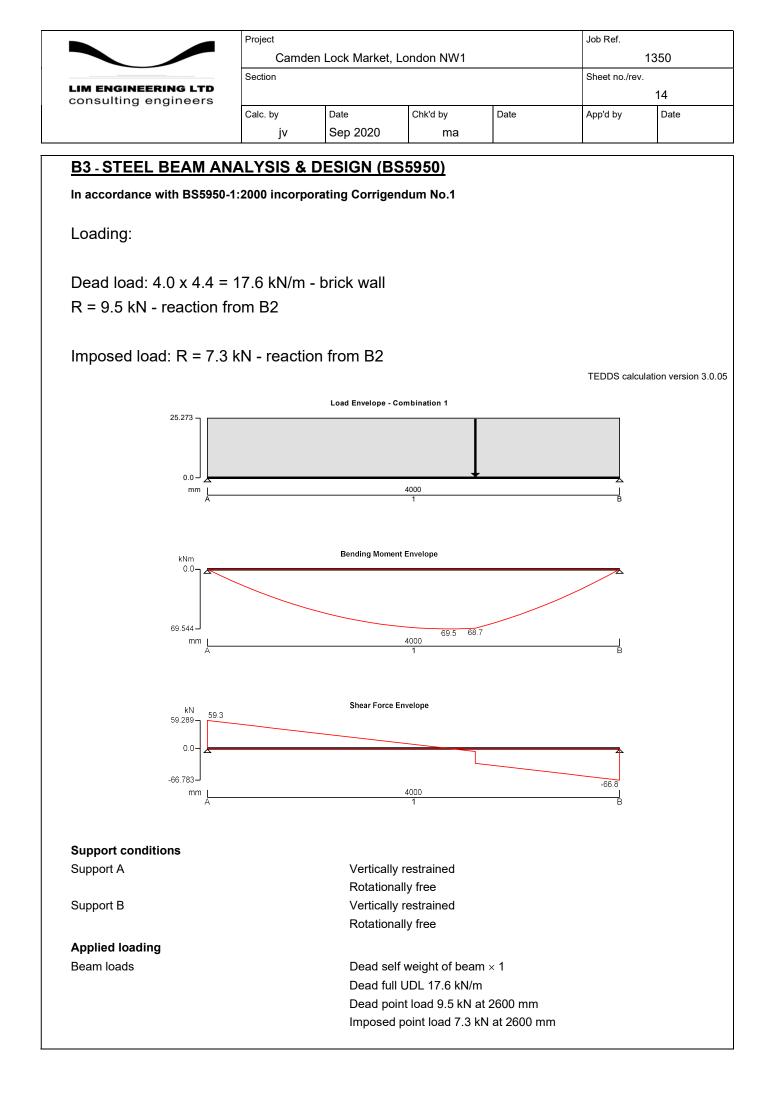
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				N at 1330 mm		
		•	t load 11.4 kN			
				N at 2930 mm		
		•	t load 11.4 kN			
				N at 4530 mm		
		•	t load 11.4 kN	N at 6130 mm		
			t load 11.4 kN			
		-		N at 7730 mm		
			t load 11.4 kN			
		•		N at 9330 mm		
Load combinations		1 · P				
Load combination 1		Support A		Dead >	< 1.40	
		11,			ed × 1.60	
		Span 1		Dead⇒		
				Impose	ed × 1.60	
		Support B		Dead >		
				Impose	ed × 1.60	
		Span 2		Dead >		
				Impose	ed × 1.60	
		Support C		Dead >	< 1.40	
				Impose	ed × 1.60	
Analysis results						
Maximum moment;		M _{max} = 38.	9 kNm;	M _{min} =	-56.2 kNm	
Maximum moment span 1;		Ms1_max = 3	32.6 kNm;	Ms1_min	= -56.2 kNm	ı
Maximum moment span 2;		M _{s2_max} = 3		-	= -56.2 kNm	ı
Maximum shear;		V _{max} = 48.	9 kN;	V _{min} = -	-72 kN	
Maximum shear span 1;		V _{s1_max} = 2		-	= -72 kN	
Maximum shear span 2;		V _{s2_max} = 4			= -48.2 kN	
Deflection;		δ _{max} = 2.1		δ _{min} = (
Deflection span 1;		δ _{s1_max} = 1			= 0 mm	
Deflection span 2;	•	δ _{s2_max} = 2			= 0 mm	
Maximum reaction at support		$R_{A_{max}} = 2$		R _{A_min} :	= 24.9 kN	
Unfactored dead load reaction		$R_{A_Dead} = 9$				
Unfactored imposed load reac Maximum reaction at support		RA_Imposed = RB_max = 12		D - · ·	= 120.9 kN	
Unfactored dead load reaction		$R_{B \text{ Dead}} = 4$		rs_min ·	- 120.3 KIN	
Unfactored imposed load reaction		-				
Maximum reaction at support		$R_{C max} = 4$		Rc min	= 48.2 kN	
Unfactored dead load reaction		$R_{C_{Dead}} = 1$				
Unfactored imposed load read						
Section details						
Section type;		UKC 203x	203x46 (Tata	Steel Advance)		
Steel grade;		S275				
From table 9: Design streng	th p _y					
Thickness of element;		max(T, t) =	= 11.0 mm			
Design strength;		py = 275 N	/mm ²			

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Modulus of elasticity;		E = 20500	00 N/mm ²			
	_ ↓					
	-203.2	-	→ ←7.2			
	▼ ~ 					
	◀		-203.6			
Lateral restraint						
		Span 1 ha	is lateral restrair	t at supports or	nly	
		-	is lateral restrair		-	
Effective length factors						
Effective length factor in major a		K _x = 1.00				
Effective length factor in minor a Effective length factor for lateral		K _y = 1.00				
		, КША – 1.0 К _{LT.В} = 1.0				
		K _{LT.C} = 1.0				
Classification of cross section	ns - Section 3.5					
		ε = √[275	N/mm² / p _y] = 1 .0	00		
Internal compression parts -	Table 11					
Depth of section;		d = 160.8				
		d / t = 22.3	3 × ε <= 80 × ε;	Class	1 plastic	
Outstand flanges - Table 11			404 0			
Width of section;			= 101.8 mm 5 × ε <= 10 × ε;	Class	2 compact	
		671 - 5.6	· · · · · · · · · · · · · · · · · · ·	01033	Section is cl	ass 2 com
					-	
Shear capacity - Section 4.2.3	}					
Shear capacity - Section 4.2.3 Design shear force;	}	F _v = max(a	abs(V _{max}), abs(V	′ _{min})) = 72 kN		
	}	F _v = max(a d / t < 70 >	3 ×			
Design shear force;	\$	d / t < 70 >	×ε Web does	′′ _{min})) = 72 kN not need to be	checked for s	shear buck
Design shear force; Shear area;	8	d / t < 70	κε <i>Web does</i> = 1463 mm ²	not need to be	checked for s	shear buck
Design shear force;	3	d / t < 70 $A_v = t \times D$ $P_v = 0.6 \times$	× ε <i>Web does</i> = 1463 mm ² p _y × A _ν = 241.4	not need to be		
Design shear force; Shear area; Design shear resistance;		d / t < 70 $A_v = t \times D$ $P_v = 0.6 \times$	κε <i>Web does</i> = 1463 mm ²	not need to be		
Design shear force; Shear area;		$d / t < 70 \Rightarrow$ $A_v = t \times D$ $P_v = 0.6 \times$ PA	× ε <i>Web does</i> = 1463 mm ² p _y × A _ν = 241.4	not need to be kN ear resistance	exceeds desi	

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Effective length for lateral-to	rsional buckli	ing - Section 4.	3.5			
Effective length for lateral torsid	onal buckling;	L _E = 1.0	× L _{s2} = 4900 m	ım		
Slenderness ratio;		$\lambda = L_E / I$	yy = 95.437			
Equivalent slenderness - Sec	ction 4.3.6.7					
Buckling parameter;		u = 0.84	7			
Torsional index;		x = 17.7	13			
Slenderness factor;		v = 1 / [1	+ 0.05 \times (λ / x	() ²] ^{0.25} = 0.799		
Ratio - cl.4.3.6.9;		βw = 1.0	00			
Equivalent slenderness - cl.4.3	.6.7;	λ _{LT} = u ×	$\mathbf{v} \times \mathbf{\lambda} \times \sqrt{[\beta w]}$	= 64.564		
Limiting slenderness - Annex E	3.2.2;	$\lambda_{L0} = 0.4$	\times ($\pi^2 \times E / p_y$)	^{0.5} = 34.310		
		λιτ > λι	o - Allowance	should be mad	de for lateral-tor	sional buck
Bending strength - Section 4	.3.6.5					
Robertson constant;		αLT = 7.0				
Perry factor;		η∟⊤ = ma	$\mathbf{x}(\alpha_{LT} \times (\lambda_{LT} - \lambda_{T}))$	LLO) / 1000, 0) =	0.212	
Euler stress;		$p_E = \pi^2 \times$	E / λ _{LT} ² = 485	.4 N/mm ²		
		φ _L τ = (p _y	+ (η _{LT} + 1) × p	E) / 2 = 431.6 N	l/mm ²	
Bending strength - Annex B.2.7	1;	p _b = p _E ×	а р _у / (фіт + (фіт	⁻² - p _E × p _y) ^{0.5}) =	201.8 N/mm ²	
Equivalent uniform moment	factor - Sectio	on 4.3.6.6				
Moment at quarter point of seg	ment;	M ₂ = 3.2	kNm			
Moment at centre-line of segme	ent;	M ₃ = 29 .	7 kNm			
Moment at three quarter point of	of segment;	M ₄ = 28 .	7 kNm			
Maximum moment in segment;		M _{abs} = 5	6.2 kNm			
Maximum moment governing b	ouckling resista	ance; $M_{LT} = M_{a}$	_{abs} = 56.2 kNm			
Equivalent uniform moment fac	tor for lateral-	torsional bucklin	g;			
		m∟⊤ = ma	x(0.2 + (0.15 ×	M_2 + 0.5 × M_3	+ 0.15 × M₄) / Ma	ubs, 0.44) = 0 .
Buckling resistance moment	- Section 4.3					
Buckling resistance moment;		•	< S _{xx} = 100.4 k	Nm		
			= 182.6 kNm			
			PASS - Mome	nt capacity ex	ceeds design b	ending mon
Check vertical deflection - Se						
Consider deflection due to imp	osed loads			_		
Limiting deflection;			/ 360 = 13.61 1			
Maximum deflection span 2;				(δ _{min})) = 2.087 r		
		P	ASS - Maximu	m deflection d	loes not exceed	deflection l

 $\frac{(9.5+7.3)\cdot 10^3}{0.42\cdot 100} = 400$ mm

Provide 450x100x12mm Th. Steel Plate



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Load combinations						
Load combination 1		Support	A	De	ead \times 1.40	
				Im	posed \times 1.60	
		Span 1		De	ead \times 1.40	
				Im	posed \times 1.60	
		Support	В	De	ead \times 1.40	
				Im	posed \times 1.60	
Analysis results						
Maximum moment;		M _{max} = 6	9.5 kNm;	M	_{min} = 0 kNm	
Maximum shear;		V _{max} = 59	9.3 kN;	Vr	_{nin} = -66.8 kN	
Deflection;		$\delta_{max} = 0.5$	9 mm;	δπ	_{nin} = 0 mm	
Maximum reaction at support			59.3 kN;	R	4_min = 59.3 kN	
Unfactored dead load reaction	at support A;	$R_{A_{Dead}} =$	= 39.4 kN			
Unfactored imposed load reac			_d = 2.6 kN			
Maximum reaction at support			66.8 kN;	R	_{3_min} = 66.8 kN	
Unfactored dead load reaction		-	= 42.3 kN			
Unfactored imposed load reac	tion at support B;	RB_Impose	_d = 4.7 kN			
Section details						
Section type;		UKC 203	3x203x46 (Tata	ı Steel Advan	ce)	
		0075	•		,	
Steel grade;	4h	S275			,	
From table 9: Design streng	th p _y				,	
From table 9: Design streng Thickness of element;	th p _y	max(T, t) = 11.0 mm			
From table 9: Design streng Thickness of element; Design strength;	th p _y	max(T, t p _y = 275) = 11.0 mm N/mm²			
From table 9: Design streng Thickness of element;	th py ↓	max(T, t p _y = 275) = 11.0 mm			
From table 9: Design streng Thickness of element; Design strength;	th py ↑ ↓ ↓	max(T, t p _y = 275) = 11.0 mm N/mm²			
From table 9: Design streng Thickness of element; Design strength;	th py ↑ ↓ ↓ ↓ ↓ ↓ ↓	max(T, t p _y = 275) = 11.0 mm N/mm²			
From table 9: Design streng Thickness of element; Design strength;	th py ↑ $\stackrel{↓}{\stackrel{\leftarrow}{\stackrel{\leftarrow}{\stackrel{\leftarrow}{\stackrel{\leftarrow}{\stackrel{\leftarrow}{\stackrel{\leftarrow}{\stackrel{\leftarrow}{$	max(T, t p _y = 275) = 11.0 mm N/mm²			
From table 9: Design streng Thickness of element; Design strength;	th py ↑ ↓ ↓ ↓ ↓ ↓ ↓ ↓	max(T, t p _y = 275) = 11.0 mm N/mm²			
From table 9: Design streng Thickness of element; Design strength;		max(T, t p _y = 275) = 11.0 mm N/mm²			
From table 9: Design streng Thickness of element; Design strength;	th py	max(T, t p _y = 275) = 11.0 mm N/mm²			
From table 9: Design streng Thickness of element; Design strength;		max(T, t p _y = 275) = 11.0 mm N/mm ² 000 N/mm ²			
From table 9: Design streng Thickness of element; Design strength;		max(T, t p _y = 275) = 11.0 mm N/mm ² 000 N/mm ²			
From table 9: Design streng Thickness of element; Design strength;		max(T, t p _y = 275) = 11.0 mm N/mm ² 000 N/mm ²			
From table 9: Design streng Thickness of element; Design strength;		max(T, t p _y = 275) = 11.0 mm N/mm ² 000 N/mm ²			
From table 9: Design streng Thickness of element; Design strength;		max(T, t p _y = 275) = 11.0 mm N/mm ² 000 N/mm ²			
From table 9: Design streng Thickness of element; Design strength;		max(T, t p _y = 275) = 11.0 mm N/mm ²)00 N/mm ²			
From table 9: Design streng Thickness of element; Design strength;		max(T, t p _y = 275) = 11.0 mm N/mm ² 000 N/mm ²			
From table 9: Design streng Thickness of element; Design strength;		max(T, t p _y = 275) = 11.0 mm N/mm ²)00 N/mm ²			
From table 9: Design streng Thickness of element; Design strength; Modulus of elasticity;		max(T, t) py = 275 E = 2050) = 11.0 mm N/mm ² D00 N/mm ²			
From table 9: Design streng Thickness of element; Design strength; Modulus of elasticity; Lateral restraint		max(T, t) py = 275 E = 2050) = 11.0 mm N/mm ²)00 N/mm ²			
From table 9: Design streng Thickness of element; Design strength; Modulus of elasticity; Lateral restraint Effective length factors		max(T, t) py = 275 E = 2050) = 11.0 mm N/mm ² 000 N/mm ²			
From table 9: Design streng Thickness of element; Design strength; Modulus of elasticity; Lateral restraint Effective length factors Effective length factor in major	Taxis;	max(T, t) py = 275 E = 2050 Span 1 h K _x = 1.00) = 11.0 mm N/mm ² 000 N/mm ² -7.2 -203.6 			
From table 9: Design streng Thickness of element; Design strength; Modulus of elasticity;	Taxis; Taxis;	max(T, t) py = 275 E = 2050 Span 1 h K _x = 1.00 K _y = 1.00) = 11.0 mm N/mm ² D00 N/mm ²			

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Classification of cross section	ons - Section 3	-					
		ε = √[275	5 N/mm² / p _y] = '	1.00			
Internal compression parts -	Table 11						
Depth of section;		d = 160.					
		d / t = 22	.3 × ε <= 80 × ε	; Cl	ass 1 plastic		
Outstand flanges - Table 11							
Width of section;			= 101.8 mm				
		b / T = 9	$3 \times \varepsilon \le 10 \times \varepsilon;$	Cl	ass 2 compact		
					Section is c	lass 2 comp	
Shear capacity - Section 4.2	.3						
Design shear force;			(abs(V _{max}), abs	(V _{min})) = 66.8	kN		
		d / t < 70					
		· · ·		s not need to	be checked for	shear buckl	
Shear area;			D = 1463 mm ²				
Design shear resistance;			$\times p_y \times A_v = 241.$		nce exceeds des	ian choor fo	
		F.	433 - Design s	iledi lesisidi	ice exceeds des	iyii shear io	
Moment capacity - Section 4	.2.5				00 E I NI		
Design bending moment;			(abs(M _{s1_max}), a	,			
Moment capacity low shear - c			$(p_y \times S_{xx}, 1.2 \times $	ρy × Ζ _{xx}) – 130	D.O KINIII		
Effective length for lateral-to		-					
Effective length for lateral tors	ional buckling;		× L _{s1} = 4000 mn	n			
Slenderness ratio;		$\lambda = L_E / r$	_{yy} = 77.908				
Equivalent slenderness - Se	ction 4.3.6.7		_				
Buckling parameter;		u = 0.84					
Torsional index;		x = 17.7		210 25 - 0 0 4 4			
Slenderness factor; Ratio - cl.4.3.6.9;		ν = 17[1 β _W = 1.0	+ 0.05 × $(\lambda / x)^2$	⁻] ^{0.20} = 0.844			
Equivalent slenderness - cl.4.3	367		$\mathbf{v} \times \mathbf{\lambda} \times \sqrt{[\beta_W]} =$	55 686			
Limiting slenderness - Annex I			× $(\pi^2 \times E / p_v)^{0.5}$				
Limiting sich concess - Annex I	J.Z.Z,		(, , , , , , , , , , , , , , , , , , ,		de for lateral-tor	sional huckl	
Donding strength Oration		70L1 F 70L(
Bending strength - Section 4 Robertson constant;	t.J.D.J	α _{LT} = 7.0					
Perry factor;				a) / 1000 0) –	0 150		
Euler stress;		-	$η_{LT} = max(α_{LT} \times (λ_{LT} - λ_{L0}) / 1000, 0) = 0.150$ $p_E = π^2 \times E / λ_{LT}^2 = 652.5 \text{ N/mm}^2$				
		•	+ $(\eta_{LT} + 1) \times p_E$		J/mm ²		
Bending strength - Annex B.2.	1:		$p_{y} / (\phi_{LT} + (\phi_{LT}^{2})^{2})$				
			ראָי (אָבו י (אָבו	r⊢∩ry/ /-			
Equivalent uniform moment Moment at quarter point of seg		n 4.3.6.6 M ₂ = 46.	7 kNm				
	-	$M_2 = 40.$ $M_3 = 68$					
		M ₃ = 54 .					
Moment at centre-line of segment							
	-	M _{abs} = 69	9.5 kNm				
Moment at centre-line of segment at three quarter point	·,						
Moment at centre-line of segment Moment at three quarter point Maximum moment in segment	;; buckling resistal	nce; M _{LT} = M _a	_{bs} = 69.5 kNm				

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Buckling resistance momer	nt - Section 4.3	.6.4				
Buckling resistance moment;		$M_b = p_b >$	< S _{xx} = 111.4 kN	١m		
		M_b / m_{LT}	= 122.9 kNm			

Check vertical deflection - Section 2.5.2 Consider deflection due to imposed loads Limiting deflection; Maximum deflection span 1;

 $\delta_{\text{lim}} = L_{s1} / 360 = 11.111 \text{ mm}$

 $\boldsymbol{\delta} = \text{max}(\text{abs}(\boldsymbol{\delta}_{\text{max}}), \text{ abs}(\boldsymbol{\delta}_{\text{min}})) = \textbf{0.921} \text{ mm}$

PASS - Maximum deflection does not exceed deflection limit

Padstone P3

$$\frac{(42.3+4.7)\cdot10^3}{1.0\cdot100} = 470$$
mm

Provide 500x100x20mm Th. Steel Plate

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FIRST FLOOR:

C1 - STEEL MEMBER DESIGN (BS5950)

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

N = 120.9 + 0.6 = 121.5 kN

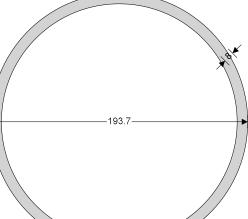
M = 0.1 x 121.5 = 12.2 kNm

TEDDS calculation version 3.0.05

Section details Section type; Steel grade; From table 9: Design strength py Thickness of element; Design strength; Modulus of elasticity;

CHS 193.7x8.0 (Tata Steel Celsius) S275

t = **8.0** mm p_y = **275** N/mm² E = **205000** N/mm²



Lateral restraint Distance between major axis restraints; Distance between minor axis restraints;	L _x = 2700 mm L _y = 0 mm	
Effective length factors Effective length factor in major axis; Effective length factor in minor axis; Effective length factor for lateral-torsional buckling;	K _x = 1.00 K _y = 1.00 K _{LT} = 1.00;	
Classification of cross sections - Section 3.5 Tubular sections - Table 12	ε = √[275 N/mm² / p _y] = 1.00	
	D / t = 24.2 × ε <= 80 × ε ² ;	Class 3 semi-compact Section is class 3 semi-compact

Moment capacity - Section 4.2.5 Design bending moment;

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Effective plastic modulus - S	ection 3.5.6					
Limiting value for class 2 comp		$\beta_{2f} = 10 \times s$	e = 10			
Limiting value for class 3 semi-	-	β _{3f} = 15 × a				
Limiting value for class 2 comp		$\beta_{2w} = \max($	(100 × ε / (1 + 1	l.5 × r1), 40 × ε)) = 100	
Limiting value for class 3 semi-		-		2 × r2), 40 × ε) =		
Effective plastic modulus - cl.3	-	1	X X	,, ,		
,		1.485 × (S - Z) × [√[(140 / (D	/ t)) × (275 N/m	m² / p _v)] - 1], S	s) = 276047 n
Moment capacity low shear - c				y × Z) = 68.7 kN		,
	- ,		•	capacity exce		ending mom
Compression members - Sec	ction 4 7				g	J
Design compression force;		F₀ = 121.5	kN			
	w) ewie buekline					
Effective length for major (x-	x) axis buckling					
Effective length for buckling;			K _x = 2700 mm			
Slenderness ratio - cl.4.7.2;		$\lambda_x = L_{Ex} / r_x$	_{xx} = 41.086			
Compressive strength - Sect	ion 4.7.5					
Limiting slenderness;		$\lambda_0 = 0.2 \times$	$(\pi^2 \times E / p_y)^{0.5} =$	= 17.155		
Strut curve - Table 23;		а				
Robertson constant;		αx = 2.0				
Perry factor;		$\eta_{x} = \alpha_{x} \times (2)$	λx - λο) / 1000 =	= 0.048		
Euler stress;		$p_{Ex} = \pi^2 \times$	Ε / λ _x ² = 1198.0	3 N/mm ²		
		$\phi_x = (p_y + ($	η_x + 1) × p _{Ex}) /	2 = 765.5 N/mn	n ²	
Compressive strength - Annex	C.1;	$p_{cx} = p_{Ex} \times$	$p_y / (\phi_x + (\phi_x^2 -$	$p_{Ex} \times p_y)^{0.5}$) = 2	59.2 N/mm ²	
Compression resistance - Se	ection 4.7.4					
Compression resistance - cl.4.	7.4;	$P_{cx} = A \times p$	o _{cx} = 1209.6 kN			
		PASS - Cor	npression res	istance exceed	ls design con	npression fo
Compression members with	moments - Secti	on 4.8.3				
Comb.compression & bending	check - cl.4.8.3.2	$F_c / (A \times p)$	/) + M / M _c = 0.	272		
		PASS	- Combined b	ending and co	mpression ch	eck is satis
Member buckling resistance	- Section 4.8.3.3					
-		$M_{LT} = M_x =$	12.15 kNm			
Max major axis moment gover	-	g; m _x = 1.000)			
Max major axis moment gover Equiv uniform mnt factor - majo					\ 0.000	
		Fc / Pcx + r	$fi_X \times IVI / IVI_c \times (1)$	$+0.5 \times F_c / P_{cx}$	() = 0.286	

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GENERAL NOTES:

- 1: All dimension to be verified on site.
- 2: All drawings to be read in conjunction with Architect Drawings.
- 3: All steelwork design and fabrication in accordance with BS 5950.
- 4: Apply 2 coats of red oxide primer to all steel prior to erection.

5. All structural steelwork to be Mild Steel Grade S275, designed, fabricated & erected in accordance with B5950 Part 1. Details of main connections are shown on drawings. All other connections to have a minimum of 2No. M20 8.8 grade bolts, sherardized or zinc plated (generally use 12mm end plate and 4No M20 8.8 grade bolts)

- 6: All welding to be min. 6 mm fillet welds.
- 6: All bolts to be grade 8.8. For Splice connection use HSFG Bolts.
- 7. Timber joists to be min. grade C16.
- 8: Double joist to be bolted together with M10 bolts + 63 dia. TP connectors and washers plate @ 400 mm c/c.
- 9: Connections:
 - Timber/Masonry: BAT SPH HANGERS
 - Timber/Timber: BAT JIFFY HANGERS or Framing Anchors.
- 10: Allow for Bat M305 Straps @ 1200 mm c/c for restraints to all structural levels.
- 11: Concrete Padstones to be grade C25 (1:2:4).
- 12: Temporary propping by Contractor.
- 13: All works to be approved by Building Control Officer.
- 14: Mass concrete foundation to be grade C20(SR). General RC to be grade C35.
- 15: All Waterproofing and Drainage to Architect specification.
- 16: New brickwork to be 21 N/mmsq. New blockwork to be min. 7 N/mmsq setin 1:1:6 mortar.

17. Underside of the foundation to be found on undisturbed ground and to be approved by Building Control Surveyor on site.