PSC	Project				Job Ref.	
Paul Spane Conculting	Picket P	ost House, Clau	PSC-628			
The Grapary Woodfold Farm	Section		Sheet no./rev.			
Crombleholme Fold, Goosnargh		External Ba	lcony Design		1	
Preston PR3 2ES	Calc. by	Date	Chk'd by	Date	App'd by	Date
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INTRODUCTION

Contractor requested a design for a 3m wide and 2.8m deep external balcony at picket post house. Design set out below for required joists, timber or steel beam to support joists (the joists will be supported on the external wall at one end) and timber or steel columns to support the beam. A baseplate design is provided for the steel column. If timber columns are used then a shoe to houise the timber will need to be welded onto the baseplate. The joists will be supported on the external wall.

Tedds calculation version 1.1.04

Loading is as per domestic floor with dead load of 0.75 kN/m² and imposed lopad of 1.5 kN/m².

TIMBER JOISTS TO BALCONY

Joists span 2.8m

TIMBER JOIST DESIGN (BS5268-2:2002)

Joist details	
Joist breadth;	b = 44 mm
Joist depth;	h = 175 mm
Joist spacing;	s = 400 mm
Timber strength class;	C16
Service class of timber;	1



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Loading details									
Joist self weight;		$F_{swt} = b \times I$	$n \times \rho$ char $\times g_{acc} =$	0.02 kN/m					
Dead load;		Fd_udl = 1.1	0 kN/m ²						
Imposed UDL(Long term);		Fi_udl = 1.5	0 kN/m²						
Imposed point load (Medium term);	Fi_pt = 1.40	kN						
Modification factors									
Service class for bending parallel	to grain	K _{2m} = 1.00							
Service class for compression		K _{2c} = 1.00							
Service class for shear parallel to	grain	K _{2s} = 1.00							
Service class for modulus of elast	icity	K _{2e} = 1.00							
Section depth factor;		K ₇ = 1.06							
Load sharing factor;		K ₈ = 1.10							
Consider long term loads									
Load duration factor;		K ₃ = 1.00							
Maximum bending moment;		M = 1.042	kNm						
Maximum shear force;		V = 1.489	kN						
Maximum support reaction;		R = 1.489	kN						
Maximum deflection; $\delta = 5.217 \text{ mm}$									
Check bending stress									
Bending stress;		σm = 5.300	N/mm ²						
Permissible bending stress;		$\sigma_{\rm m} adm = \sigma$	$_{\rm m} \times K_{\rm 2m} \times K_{\rm 3} \times K_{\rm 3}$	K ₇ × K ₈ = 6.186 N	/mm²				
Applied bending stress;		σ _{m max} = M	/ Z = 4.640 N/n	nm²					
		_	PASS - Applie	d bending stres	s within perr	nissible limits			
Chock shear stress				-					
Shear stress		$\tau = 0.670$ [J/mm ²						
Permissible shear stress		$\tau = 0.070$	۰/۱۱۱۱ (م. × K. × K. = ۲	737 N/mm ²					
Applied shear stress:		$\tau = 3 \times 1$	$(2s \times R_3 \times R_8 - C_{2s}) =$	0 200 N/mm ²					
Applied silear siless,		tmax – 5 ×	PASS - Ann	lind shoar stres	s within nor	missihla limits			
			, A00 - App						
Check bearing stress	• /		0 NH 0						
Compression perpendicular to gra	ain (no wane);	σ _{cp1} = 2.20	v N/mm²		2				
Permissible bearing stress;		$\sigma_{c_{adm}} = \sigma_{c}$	$\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = 2.420 \text{ N/mm}^2$						
Applied bearing stress;		σc_max = R	/ (b × L _b) = 0.33	8 N/mm ²					
			PASS - Applie	ed bearing stres	s within peri	nissible limits			
Check deflection									
Permissible deflection;		δ_{adm} = min	$(L_{s1} \times 0.003, 14)$	mm) = 8.400 mn	n				
Bending deflection (based on E_{me}	an);	$\delta_{\text{bending}} = 4$.922 mm						
Shear deflection;		δ _{shear} = 0.2	95 mm						
Total deflection;		$\delta = \delta_{\text{bending}}$	+ δ _{shear} = 5.217	mm					
			PASS -	Actual deflectio	n within perr	missible limits			
Consider medium term loads									
Load duration factor:		K ₃ = 1.25							
Maximum bending moment:		M = 1.434	kNm						
Maximum shear force;		V = 2.049	kN						
Maximum support reaction;		R = 2.049	kN						
Maximum deflection;		δ = 6.254 ι	nm						
•									

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The Granary, Woodfold Farm Crombleholme Fold, Goosnargh		External Ba		3				
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Check bending stress								
Bending stress;		om = 5.300	N/mm ²					
Permissible bending stress;		$\sigma_{m_{adm}} = \sigma_{m_{adm}}$	$_{m} \times K_{2m} \times K_{3} \times$	K ₇ × K ₈ = 7.733 N	/mm²			
Applied bending stress;		σ _{m_max} = M	/ Z = 6.386 N/	mm ²				
			PASS - Appli	ed bending stres	s within pern	nissible limits		
Check shear stress								
Shear stress;		τ = 0.670 Ν	√mm²					
Permissible shear stress;		$\tau_{adm} = \tau \times F$	$\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = 0.921 \text{ N/mm}^2$					
Applied shear stress;		$\tau_{max} = 3 \times V$	τ_{max} = 3 × V / (2 × b × h) = 0.399 N/mm ²					
			PASS - Ap	plied shear stres	s within pern	nissible limits		
Check bearing stress								
Compression perpendicular to g	grain (no wane)	; $\sigma_{cp1} = 2.20$	0 N/mm ²					
Permissible bearing stress;		$\sigma_{c_{adm}} = \sigma_{c}$	$_{p1} imes K_{2c} imes K_3 imes$	K ₈ = 3.025 N/mm	2			
Applied bearing stress;		$\sigma_{c_{max}} = R$	/ (b × L _b) = 0.4	66 N/mm²				
			PASS - Appl	ied bearing stres	s within pern	nissible limits		
Check deflection								
Permissible deflection;		$\delta_{adm} = min$	(L _{s1} × 0.003, 14	4 mm) = 8.400 mn	n			
Bending deflection (based on E	mean);	$\delta_{\text{bending}} = 5$. 847 mm					
Shear deflection;	·	δ _{shear} = 0.4	06 mm					
Total deflection;		$\delta = \delta_{\text{bending}}$	+ δ _{shear} = 6.254	4 mm				
			PASS -	Actual deflection	n within pern	nissible limits		

TIMBER BEAM TO BALCONY

Beam spans 3m and carries 1.4m of floor load.

Dead load = 1.4x0.75 = 1.05 kN/m Imposed load = 1.4x1.5 = 2.1 kN/m

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TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02



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Crombleholme Fold, Goosnargh		External Ba	lcony Design			4
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The Granany Woodfold Farm	Section	ection Sheet no./rev.					
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		[
Timber section details							
Breadth of sections:		b = 75 mm					
Depth of sections:		h = 250 mr	n				
Number of sections in member;		N = 1					
Overall breadth of member;		$b_b = N \times b$	= 75 mm				
Timber strength class;		C24					
Member details							
Service class of timber		1					
Load duration:		Long term					
Length of span:		L _{s1} = 3000	mm				
Length of bearing;		L _b = 100 m	m				
Section properties							
Cross sectional area of member:		$A = N \times b \times$	h = 18750 mm ²	2			
Section modulus:		$Z_{\rm x} = N \times b$	$\times h^2 / 6 = 78125$	0 mm ³			
		$Z_{\rm x} = h \times (N$	$(x h)^2 / 6 = 2343$	75 mm ³			
Second moment of area		$L_y = N \times h \times h$	$h^3/12 = 97656$	250 mm ⁴			
		$I_{\rm M} = h \times (N)$	$(h^{3})^{12} = 8789$	2062 mm ⁴			
Radius of avration:		$i_{ij} = \sqrt{(l_{ij} / \Delta)}$	= 72.2 mm				
radius of gyration,		$i_{x} = \sqrt{(i_{x} / A)}$	= 21.7 mm				
Madification factors		iy ((iy)) (
Duration of loading Table 17:		Ka = 1 00					
Bearing stress Table 18:		$K_3 = 1.00$					
Total depth of member - cl 2 10 f	3.	$K_4 = 1.00$ $K_7 = (300 \text{ n})$	$K_{7} = (300 \text{ mm} / \text{h})^{0.11} = 1.02$				
Load sharing - cl.2.9:	,	K ₈ = 1.00		-			
Lateral support - cl 2 10 8		-					
Ends held in position and memb	ers held in line	as by purlins or	tie rods at centr	es not more that	n 30 times the	breadth of	
the member							
Permissible depth-to-breadth rat	io - Table 19;	4.00					
Actual depth-to-breadth ratio;		h / (N × b) :	= 3.33				
		, , , , , , , , , , , , , , , , , , ,		PASS - L	ateral suppor	t is adequat	
Compression perpendicular to	arain					-	
Permissible bearing stress (no w	ane):	σ_{c} adm = σ_{c}	1 × K3 × K₄ × K∘	= 2.400 N/mm ²			
Applied bearing stress		$\sigma_{\alpha a} = R_{A} - $	$a_{\rm x}/(N \times h \times l_{\rm h})$	= 0.645 N/mm ²			
		$\sigma_{a} = r_{A}m$	= 0.269				
PASS	- Applied com	npressive stres	s is less than n	ermissible com	pressive stre	ss at bearin	
Panding nerollal to arein							
Demuny parallel to grain				- 7 652 N/?			
remissible bending stress;		$\sigma_{m_{adm}} = \sigma_{n}$	$_1 \times \mathbf{n}_3 \times \mathbf{n}_7 \times \mathbf{n}_8$	- 1.002 IN/IIIII)2			

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			7	2		
Applied bending stress;		σm_a = M /	Z _x = 4.64 / N/m	1m²		
		σm_a / σm_a				
		PASS - Applie	a benaing stre	ess is less than p	ermissible de	enaing stress
Shear parallel to grain						
Permissible shear stress;		$\tau_{adm} = \tau \times I$	$K_3 \times K_8 = 0.710$	N/mm ²		
Applied shear stress;		$\tau_a = 3 \times F$	$(2 \times A) = 0.38$	7 N/mm ²		
		$\tau_a / \tau_{adm} =$	0.545			
		PASS - A	pplied shear s	stress is less tha	n permissible	shear stress
Deflection						
Modulus of elasticity for deflect	ction;	$E = E_{min} =$	7200 N/mm ²			
Permissible deflection;		δ_{adm} = min	(0.551 in, 0.003	3 × L _{s1}) = 9.000 m	m	
Bending deflection;		δ _{b_s1} = 4.8 4	41 mm			
Shear deflection;		δ _{v_s1} = 0.5	16 mm			
Total deflection;		$\delta_a = \delta_{b_s1} +$	- δ _{v_s1} = 5.357 r	nm		
		δa / δadm =	0.595			
		P	ASS - Total de	eflection is less t	han permissil	ble deflection
;						
STEEL BEAM TO BALCO	<u>ONY</u>					
Loading and span as above for tim	iber.					
Design as unrestrained.						
STEEL BEAM ANALYSIS &		50)				
		<u></u>				
In accordance with BS5950-	1:2000 incorpo	orating Corrigend	ium No.1			tion version 2.0.07
Shear parallel to grain Permissible shear stress; Applied shear stress; Deflection Modulus of elasticity for deflect Permissible deflection; Bending deflection; Shear deflection; Total deflection; Total deflection; Steer deflection; Design as unrestrained. STEEL BEAM ANALYSIS & In accordance with BS5950-	otion; DNY Iber. DESIGN (BS59: 1:2000 incorpc	PASS - Applie $\tau_{adm} = \tau \times I$ $\tau_a = 3 \times F_a$ $\tau_a / \tau_{adm} = I$ PASS - A $E = E_{min} =$ $\delta_{adm} = min$ $\delta_{b_s s1} = 4.8a$ $\delta_{v_s s1} = 0.5i$ $\delta_a = \delta_{b_s s1} +$ $\delta_a / \delta_{adm} =$ PP	d bending stre $K_3 \times K_8 = 0.710$ $(2 \times A) = 0.38$ 0.545 pplied shear s 7200 N/mm ² (0.551 in, 0.003 41 mm 16 mm $\delta_{V_S1} = 5.357$ r 0.595 ASS - Total de	ess is less than p N/mm ² 7 N/mm ² stress is less that $3 \times L_{s1}$) = 9.000 m mm	permissible be n permissible m han permissil	ending str

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LAL		Shear Force Env	elope				
7.548							
0.0-					4		
-7.548					-7.5		
mm LA			3000 1] B		
Support conditions							
Support Conditions		Vertically r	estrained				
Support A		Potational	v froo				
Support B		Vertically r	estrained				
Detationally free							
Rotationally free							
Applied loading							
Beam loads	Dead self	weight of beam	1 × 1				
	Dead full L	JDL 1.05 kN/m					
		Imposed fu	III UDL 2.1 kN/	m			
Load combinations							
Load combination 1		Support A		$\text{Dead} \times$	1.40		
				Impose	d × 1.60		
				Dead ×	1.40		
				Impose	d × 1.60		
		Support B		Dead ×	1.40		
				Impose	d × 1.60		
Analysis results							
Maximum moment;		M _{max} = 5.7	kNm;	$M_{min} = 0$	kNm		
Maximum shear;		V _{max} = 7.5	kN;	$V_{min} = -7$	V _{min} = -7.5 kN		
Deflection;		δ _{max} = 3.9	mm;	δ _{min} = 0	mm		
Maximum reaction at support A		R _{A max} = 7.	5 kN;	R _{A min} =	7.5 kN		
Unfactored dead load reaction	at support A;	R _{A Dead} = 1	.8 kN	_			
Unfactored imposed load reacti	ion at support A;	RA Imposed =	= 3.2 kN				
Maximum reaction at support B	;	R _{B max} = 7.	5 kN;	RB min =	7.5 kN		
Unfactored dead load reaction	at support B;	R _{B_Dead} = 1	$R_{\text{B Dead}} = 1.8 \text{ kN}$				
Unfactored imposed load reaction	ion at support B;	R _{B_Imposed} =	= 3.2 kN				
Section details							
		SHS 100x	100x5.0 (Tata	Steel Celsius (Gr	355 Gr420 G	ir460))	
Section type;		S275					
Section type; Steel grade;							
Section type; Steel grade; From table 9: Design strengt	h p _y						
Section type; Steel grade; From table 9: Design strength Thickness of element;	h p _y	t = 5.0 mm					
Section type; Steel grade; From table 9: Design strength Thickness of element; Design strength;	h p _y	t = 5.0 mm p _y = 275 N	/mm²				

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	T						
	100			→ 5 ←			
		10					
	4)	── ►			
l ateral restraint							
		Span 1 ha	s lateral restra	int at supports only	/		
Effective length factors		- [
Effective length factor in major	axis.	K. = 1 00					
Effective length factor in minor	axis:	$K_{\rm x} = 1.00$					
Effective length factor for latera	al-torsional bucklin	a: KITA = 1.0	0:				
		K _{LT.B} = 1.0	0;				
Classification of cross section	ons - Section 3.5						
		ε = √[275 Ι	$N/mm^2 / p_{vl} = 1$.00			
Web main ania Table 40		0 12/01					
Web - major axis - Table 12			1 - OF				
Depth of section;		d= D - 3 ×	t = 85 mm				
		d/t = 17.0) × ε <= 64 × ε;	Class 1	plastic		
Flange - major axis - Table 12	2						
Width of section;		b = B - 3 ×	t = 85 mm				
		b / t = 17.0) × ε <= min(28	$B \times \varepsilon$, 80 × ε - d / t);	Clas	s 1 plastic	
					Section is	class 1 plast	
Shear capacity - Section 4.2.	3						
Design shear force;		F _v = max(a	abs(V _{max}), abs((V _{min})) = 7.5 kN			
		(D - 3 × t) /	/ t < 70 × ε				
			Web does	s not need to be o	hecked for s	hear bucklin	
Shear area;		$A_v = A \times D$	/ (D + B) = 93	7 mm ²			
Design shear resistance;		$P_v = 0.6 \times$	$p_y \times A_v = 154.8$	5 kN			
		PAS	SS - Design sl	hear resistance e	xceeds desi	gn shear forc	
	.2.5						
Moment capacity - Section 4.				$hs(M_{c1}, min)) = 5.7 k$	Nm		
Moment capacity - Section 4. Design bending moment;		M = max(a	iDS(IVIs1_max), at				
Moment capacity - Section 4 Design bending moment; Moment capacity low shear - c	1.4.2.5.2;	M = max(a M₀ = min(p	$p_{y} \times S, 1.2 \times p_{y}$	× Z) = 18.2 kNm			
Moment capacity - Section 4 Design bending moment; Moment capacity low shear - c Effective length for lateral-to	I.4.2.5.2; rsional buckling	M = max(a M _c = min(p - Section 4.3 .4	iDS(IM _{s1_max}), at by × S, 1.2 × py 5	× Z) = 18.2 kNm			
Moment capacity - Section 4 Design bending moment; Moment capacity low shear - c Effective length for lateral-to Effective length for lateral torsic	I.4.2.5.2; rsional buckling onal buckling;	M = max(a M₀ = min(p - Section 4.3. L _E = 1.0 ×	abs(Mis1_max), ar by × S, 1.2 × py 5 L _{s1} = 3000 mm	× Z) = 18.2 kNm			

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Equivalent slenderness - Ar	nex B.2.6.1					
Torsion constant;		J = 439410	8 mm ⁴			
		γ _b = (1 - I _{yy}	/ I _{xx}) × (1 - J / ((2.6 × I _{xx})) = 0.000	1	
		$\phi_b = [S_{xx}^2 \times$	γ_b / (A \times J)]^{0.5}	= 0.000		
Ratio - cl.4.3.6.9;		βw = 1.000				
Equivalent slenderness;	ent slenderness; $\lambda_{LT} = 2.25 \times \sqrt{[\phi_b \times \lambda \times \beta_W]} = 0.000$					
Limiting slenderness - Annex	Limiting slenderness - Annex B.2.2; $\lambda_{1.0} = 0.4 \times (\pi^2 \times E / p_v)^{0.5} = 34.310$					
-		λ _{LT} < λ _{L0} - Ι	lo allowance	need be made fo	r lateral-tors	ional buckling
Buckling resistance momen	it - Section 4.3.	6.4				
Bending strength;		p _b = p _y = 2	75 N/mm²			
Buckling resistance moment;		$M_b = p_b \times S$	5 = 18.2 kNm			
-		PA	SS - Moment	capacity exceed	ls design bei	nding moment
Check vertical deflection - S	Section 2.5.2					
Consider deflection due to im	oosed loads					
Limiting deflection:		$\delta_{\text{lim}} = L_{s1} / 3$	360 = 8.333 mi	m		
Maximum deflection span 1:		$\delta = \max(ab)$	$s(\delta_{max})$ abs (δ_{max})	m(n)) = 3866 mm		
			S - Maximum	deflection does	not exceed a	Antipaction limit
			•			
;						
TIMBER COLUMN						
rom calculations above axial load	t is 7.5 kN comr	pression Allow for	2.6m length N	Jominal moment a	nd shear ann	lied
TIMBER MEMBER DESIGN	TO BS5268-2:20	<u>002</u>			TEDDS calcula	ation version 1 7 02
Analysis results						
Design moment in major axis:		M. = 1 000	kNm			
Design memori in major axio,		F = 1 000 l	'N			
Design axial compression:		P = 7 500 /	an Al			
Design axial compression,		F = 7.500 i				
		100				
			\mathbf{i}			
		 ← 10	⊃►			
imber section details		1 (00				
Breadth of sections;		b = 100 mr	n			
Depth of sections;		h = 100 mr	n			
Number of sections in member	<i>:</i> r;	N = 1				
Overall breadth of member;		$b_b = N \times b$	= 100 mm			

Member details Service class of timber;

Timber strength class;

Load duration; Effective length - cl.2.11.3 C24 1 Long term

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I Inbraced length in x-axis:		L = 2600 r	mm					
Effective length factor in x-axis,	able 21 [.]	K _x = 1						
Effective length in x-axis:		$ _{ex} = _{x} \times K$	(_x = 2600 mm					
Unbraced length in v-axis:		$L_v = 2600 \text{ r}$	nm					
Effective length factor in v-axis - T	able 21:	K _v = 1						
Effective length in y-axis;		$L_{ev} = L_v \times K$	K _v = 2600 mm					
Soction properties								
Cross sectional area of member:		$A = N \times b \times$	/ h = 10000 mm ²	2				
Section modulus:		$\overline{A} = N \times b \times b$	$\times h^2 / 6 = 16666$	7 mm ³				
Section modulus,		$Z_x = N \times D$	$\times h^2 / 6 - 1666$	67 mm ³				
Second moment of area:		$Z_y = \Pi \times (N)$	$(h^3 / 12 - 93333)$	33 mm^4				
Second moment of area,		$I_X = I_X \times D \times D$	$(11^{\circ} / 12 - 03333)$	933 mm ⁴				
Dedius of gyration		$i_y = 11 \times (10)$	$(\times D)^{-7} 2 - 0333$	5555 11111				
Radius of gyration,		$I_X = \sqrt{(I_X / A)}$) - 20.9 mm					
		$I_y = V(I_y / A_y)$) = 28.9 mm					
Modification factors								
Duration of loading - Table 17;		K ₃ = 1.00						
I otal depth of member - cl.2.10.6;	K ₇ = (300 n	$K_{0} = 1.00$						
Load sharing - cl.2.9;	aian Tabla O	$K_8 = 1.00$						
Members subject to axial compres	sion - Table Z	$2; \mathbf{n}_{12} = 0.46$						
Lateral support - cl.2.10.8								
No lateral support	T 11 40							
Permissible depth-to-breadth ratio	- Table 19;	2.00	4.00					
Actual deptn-to-breadth ratio;		n / (N × b) =	= 1.00			tia adamuata		
				PA33 - La	ateral support	is adequate		
Slenderness ratio - cl.2.11.4								
Permissible slenderness ratio;		λ _{max} = 180	$\lambda_{\text{max}} = 180$					
Slenderness ratio;		λ = max(L _e	$x / i_x, L_{ey} / i_y) = 90$	0.067				
		PASS - Sler	iderness ratio i	s less than pern	nissible slend	lerness ratio		
Bending parallel to grain								
Permissible bending stress;		$\sigma_{m_{adm}} = \sigma_{m}$	$n \times K_3 \times K_7 \times K_8$	= 8.463 N/mm ²				
Applied bending stress;		$\sigma_{m_a} = M_x /$	Z _x = 6.000 N/m	m²				
		σ_{m_a} / σ_{m_a}	_{im} = 0.709					
	I	PASS - Applied	d bending stres	s is less than p	ermissible be	nding stress		
Compression parallel to grain								
Permissible compressive stress;		$\sigma_{c_{adm}}$ = σ_{c}	\times K ₃ \times K ₈ \times K ₁₂ :	= 3.618 N/mm ²				
Applied compressive stress;		$\sigma_{c_a} = P / A$	σ _{c_a} = P / A = 0.750 N/mm ²					
		$\sigma_{c_a} / \sigma_{c_adn}$	n = 0.207					
	PASS - Ap	oplied compres	ssive stress is l	less than permis	ssible compre	essive stress		
Members subject to axial comp	ression and b	ending - cl.2.1	1.6					
Euler critical stress;		σ_{e} = (π^{2} × E	E_{min}) / λ^2 = 8.760	N/mm ²				
Euler coefficient;		K _{eu} = 1 – (1	$1.5 \times \sigma_{c_a} \times K_{12}$ /	σe) = 0.941				
Combined axial compression and	bending check	k; σ _{m_a} / (σ _{m_a}	adm × Keu) + σc_a	/ σc_adm = 0.961 ;	< 1			
	PASS - Com	bined compre	ssive and bend	ing stresses are	e within perm	issible limits		

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			•	•	•	
Shear parallel to grain						
Dermissible chear stress:			(a.u. Ka = 0.710	N/mm ²		
Permissible shear stress;		τ adm = $\tau \times \mathbf{r}$	$X_3 \times K_8 = 0.710$	N/mm-		
Applied shear stress;		$\tau_a = 3 \times F /$	$(2 \times A) = 0.150$	0 N/mm ²		
		τ_a / τ_{adm} = ().211			
		PASS - A	pplied shear s	tress is less tha	n permissible	shear stress
		-	-		•	
•						
<u>STEEL COLUMN</u>						
Details as above for timber.						
STEEL MEMBER DESIGN (BS	5 <u>950)</u>					
In accordance with BS5950-1:2	2000 incorporat	ing Corrigend	um No.1			
					TEDDS calcula	tion version 3.0.07
Section details						
Section type:		SHS 100x ²	100x5.0 (Tata \$	Steel Celsius (G	355 Gr420 Gi	460))
Steel grade:		S275				//
Erem toble 0: Design strength		0210				
From table 9: Design strength	ру					
Thickness of element;		t = 5.0 mm				
Design strength;		py = 275 N/	/mm²			
Modulus of elasticity;		E = 205000	0 N/mm²			
		100		→ 5 ←		
Lateral restraint						
Distance between major axis res	straints.	l = 2600 r	mm			
	anno,					
Distance between minor axis res	straints;	L _y = 0 mm				
Effective length factors						
Effective length factor in major a	xis:	K _* = 1 00				
Effortivo longth factor in migur	xio;	K = 4 00				
	AIS,	$\mathbf{x}_{y} = 1.00$				
Effective length factor for lateral-	torsional buckling	g; K _{LT} = 1.00 ;				
Classification of cross section	s - Section 3.5					
		ε = √[275 Ν	J/mm² / p _y] = 1 .	00		

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Web - major axis - Table 12									
Depth of section;		d= D - 3 ×	t = 85 mm						
Stress ratios;	r1 = min(F _c / (2 × d × t × p _{yw}), 1) = 0.032								
		$r2 = F_c / (A \times p_{yw}) = 0.015$							
		d / t = 17.0	$\times \varepsilon \le \max(64)$	× ɛ / (1 + r1), 40 ×	ε); Class	1 plastic			
Flange - major axis - Table 12									
Width of section;		b = B - 3 ×	t = 85 mm						
		b / t = 17.0	× ε <= 40 × ε;	Class 3	semi-compac	t			
				Sectio	n is class 3 s	emi-compact			
Shear capacity - Section 4.2.3									
Design shear force;		F _{y,v} = 1 kN							
		(D - 3 × t) /	t < 70 × ε						
			Web does r	not need to be cl	hecked for sl	near buckling			
Shear area;		$A_v = A \times D$	A _v = A × D / (D + B) = 937 mm ²						
Design shear resistance;		$P_{y,v} = 0.6 \times$	$P_{y,v} = 0.6 \times p_y \times A_v = 154.5 \text{ kN}$						
		PAS	SS - Design she	ar resistance ex	ceeds desig	n shear force			
Shear capacity - Section 4.2.3									
Design shear force;		F _{x,v} = 0 kN							
Moment capacity - Section 4.2	2.5								
Design bending moment;		M = 1 kNm	I						
Effective plastic modulus - Se	ction 3.5.6								
Limiting value for class 2 compa	act flange;	$\beta_{2f} = min(3)$	$2 \times \varepsilon$, $62 \times \varepsilon$ - 0.	5 × d / t) = 32					
Limiting value for class 3 semi-c	compact flange;	$\beta_{3f} = 40 \times \epsilon$; = 40						
Limiting value for class 2 compact web;		$\beta_{2w} = max($	β_{2w} = max(80 × ϵ / (1 + r1), 40 × ϵ) = 77.513						
Limiting value for class 3 semi-c	Limiting value for class 3 semi-compact web;		β_{3w} = max(120 × ϵ / (1 + 2 × r2), 40 × ϵ) = 116.605						
Effective plastic modulus - cl.3.5	5.6.3								
S _{eff} = min(Z + (S	6 - Z) × min([(β ₃ ,	w / (d / t) - 1) / (β	3w / β2w - 1)], [(β3	af / (b / t) - 1) / (βзi	r / β _{2f} - 1)]), S)	= 66358 mm ³			
Moment capacity low shear - cl.	4.2.5.2;	M _c = min(p	$v_y imes S_{eff}, \ 1.2 imes p_y$	× Z) = 18.2 kNm					
		PA	ASS - Moment o	apacity exceeds	s design ben	ding moment			
Compression members - Sect	ion 4.7								
Design compression force;		Fc = 7.5 kN	1						
Effective length for major (x-x) axis buckling	- Section 4.7.3							
Effective length for buckling;		$L_{Ex} = L_x \times H$	≺ _x = 2600 mm						
Slenderness ratio - cl.4.7.2;		$\lambda_x = L_{Ex} / r_x$	∝ = 67.317						
Compressive strength - Section	on 4.7.5								
Limiting slenderness:		$\lambda_0 = 0.2 \times 0$	$(\pi^2 \times E / p_v)^{0.5} = 0$	17.155					
Strut curve - Table 23;		a							
Robertson constant;		α _x = 2.0							
Perry factor;		$\eta_x = \alpha_x \times (\lambda)$	(x - λ ₀) / 1000 = (0.100					
Euler stress;	$p_{Ex} = \pi^2 \times E / \lambda_x^2 = 446.5 \text{ N/mm}^2$								
		$\phi_x = (p_y + ($	η _x + 1) × p _{Ex}) / 2	= 383.1 N/mm ²					
Compressive strength - Annex (C.1;	p _{cx} = p _{Ex} ×	p _y / (φ _x + (φ _x ² - p	_{Ex} × p _y) ^{0.5}) = 228.	2 N/mm ²				
Compression resistance - Sec	tion 4.7.4	-	•						
Compression resistance - cl 4 7	.4:	$P_{cx} = A \times n$	_{cx} = 427.4 kN						
	,								

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		PASS - Con	npression resi	stance exceeds	design comp	ression force		
Compression members with	moments - Sect	ion 4.8.3						
Comb.compression & bending	check - cl.4.8.3.2	P_{c} ; $F_{c} / (A \times p_{y})$	/) + M / M _c = 0.0	69				
		PASS	- Combined be	nding and com	pression chec	k is satisfied		
Member buckling resistance	- Section 4.8.3.3	3						
Max major axis moment governing M_b ; $M_{LT} = M_x = 1.00$ kNm								
Equiv uniform mnt factor - majo	Equiv uniform mnt factor - major axis flex buckling; $m_x = 1.000$							
Buckling resistance check - cl.4.8.3.3.3; $F_c / P_{cx} + m_x \times M / M_c \times (1 + 0.5 \times F_c / P_{cx}) = 0.073$								
PASS - Member buckling resistance checks are satisfied								
;								
COLUMN BASE PLATE DESIGN (BS5950)								
Load as per column above – 7.5 kN								
COLUMN BASE PLATE DESIGN (BS5950-1:2000)								
					I EDDS calculati	on version 1.0.09;		





6 mm thick base plate

Base plate reference;

Design forces and moments

Axial force; Bending moment; Shear force;

Column details

Column section; Depth; Breadth; Flange thickness; Web thickness; Design strength; Column flange to base plate weld; Column web to base plate weld;

Column;

$$\label{eq:Fc} \begin{split} F_c &= (\textbf{7.5}; \, kN \ (\text{Compression}) \\ M &= \textbf{1.0} \ kNm; \ (about major axis) \\ F_v &= \textbf{1.0} \ kN \end{split}$$

SHS 100x100x5.0 (Grade S275)

D = 100.0 mm B = 100.0 mm T = 5.0 mm t = 5.0 mm p_{yc} = 275 N/mm² 6 mm FW; 8 mm FW;

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Baseplate details									
Steel grade;		S275							
	$D_p = 200 \text{ mm}$								
Breadth;	B _p = 200 mm								
Design strength:	ckness; $t_p = 6 \text{ mm}$								
Design strength; $p_{yp} = 275 \text{ N/mm}^2$									
Holding down bolt and anchor plate details									
l otal number of bolts;		4 NO. M12	Grade 4.6						
Bolt spacing;		Sbolt = ;145	; mm						
Edge distance;		e ₁ = 25 mn	1						
Anchor plate steel grade;	`	S275							
Anchor plate dimension (square),	Dap = 60 m	ITI						
Anchor plate thickness;		$t_{ap} = 5 \text{ mm}$	N1/2						
Design strengtn;	-4	p _{yap} = 275	p _{yap} = 275 N/mm ²						
Embeddment to top of anchor pi	ate;	E = 100 mi	n 2						
Characteristic strength of concrete; $f_{cu} = 25 \text{ N/mm}^2$									
Concrete compression force a	ind bolt tensio	n force							
Plate overhang beyond face of f	L1 = (Dp - [$L_1 = (D_p - D)/2) = ;$ 50.0 ; mm							
Effective width of plate;	$B_{pc} = min(B)$	$B_{pc} = min(B_p, B + 2 \times L_1) = ;200.0; mm$							
Distance from bolts to compression edge; $h = D_p - e_1 = 175$									
Assuming a rectangular compre-	Assuming a rectangular compression block of width b_{pc} , length x and intensity 0.6f _{cu} then:-								
From static equilibrium;		$M = 0.6 f_{cu} E$	3 _{pc} x(h-x/2) - F _c (h	-D _p /2)					
Rearranging the quadratic equal	0.3f _{cu} B _{pc} x ²	- 0.6f _{cu} B _{pc} hx + F	$F_{c}(h-D_{p}/2) + M = 0$	0					
Factor a; a = 0.3 × f _{cu} × B _{pc} = 1500.0 N/mm									
Factor b;	b = -0.6 × f	$b = -0.6 \times f_{cu} \times B_{pc} \times h = -525000.0 \text{ N}$							
Constant c;	$c = F_c \times (h \cdot$	$c = F_c \times (h-D_p/2) + M = 1562500.0 \text{ Nmm}$							
Depth of compression block;	x = [-1.0×b	x = [-1.0×b - √(b² - 4×a×c)]/(2×a) = 3.0 mm							
Compression force in concrete;	$C_f = 0.6 \times 1$	$C_f = 0.6 \times f_{cu} \times B_{pc} \times x = 9.0 \text{ kN}$							
Tension force in bolts; $T_f = C_f - F_c = 1.5 \text{ kN}$									
				Therefo	ore the bolts a	are in tension			
Compression side bending									
Moment in plate;		m_c = 0.6 \times	$f_{cu} \times x \times (L_1 - 0.8)$	8×s _{wf} - x/2) = ; 196	S8 Nmm/mm;				
Plate thickness required;		$t_{pc} = \sqrt{4 \times 10^{-5}}$	m _c /p _{yp}) = 5.3 mn	า					
Tension side bending									
Lever arm;		m = L1 - e1	- 0.8×s _{wf} = 20.2	mm					
Moment in plate;		$m_t = T_f \times m$	= 30417 Nmm						
Distance from bolt cl. to face of o	column;	lumn; $L_f = L_1 - e_1 = 25.0 \text{ mm}$							
Effective plate width:	B _{pt} = min(B _p , s _{bolt} ×(N _{bolt} /2-1) + 2×L _f) = 195.0 mm								
Plate thickness required; $t_{pt} = \sqrt{4 \times mt/(p_{vp} \times B_{pt})} = 1.5 \text{ mm}$									
Plate thickness		. X							
Plate thickness required		to rea = may	((t _{no} t _{ot}) = · 5 3 m	ım [.]					
Plate thickness provided:	$t_{p} = 6 \text{ mm}$								
	$p = 0$ mm $D\Delta SS = Diata thickness provided is adorwate (0.80)$					quate (0.892)			
Tension conscitu of florade			vn - 197 -	N					
$F_{tt} = D \times I \times P_{yc} = (T \cdot J \cdot J \cdot S) \times I \times P_{tt}$									
Force in tension hange;		rtt − IVI/(D ·	· ,) - Fc × (D×1)/	π – , 0.3 , κιν					

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Flange weld design force;		$F_f = min(P_t)$	f, max(F _{tf} , 0 kN))) = 8.5 kN					
Weld force per mm;		$f_{wf} = F_f/B =$; 0.085 ; kN/mm						
Transverse capacity of 6 mm fillet weld; p _{wf} = 1.155 kN/mm; (Cl. 6.8.7.3)									
			PASS	- Flange weld ca	pacity is ade	quate (0.074)			
Longitudinal capacity of web	weld								
Weld force per mm;		$f_{wwl} = F_v/(2)$	f _{wwi} = F _v /(2×(D-2×t)) = ; 0.006 ; kN/mm						
Longitudinal capacity of 8 mm fil	let weld;	p _{wwl} = 1.23	2 kN/mm; (Cl.	6.8.7.3)					
		PASS	S - Longitudina	I capacity of we	b weld is ade	quate (0.005)			
Holding down bolts									
Force per bolt;		F _{bolt} = (2×T	f)/ N _{bolt} = 0.8 kN						
Tensile area per bolt;		At_b = 84.3	$A_{t b} = 84.3 \text{ mm}^2$						
Tensile strength;		pt_b = 240 I	pt b = 240 N/mm ²						
Tension capacity (cl. 6.6);	$P_{t_b} = 0.8 \times$	$P_{t_{b}} = 0.8 \times p_{t_{b}} \times A_{t_{b}} = ;16.2; kN;$							
				PASS - Bolt ca	pacity is ade	quate (0.047)			
Anchor plates									
Force per anchor plate;		F _{ap} = F _{bolt} =	= 0.8 kN						
Bolt hole diameter in anchor plat	d _h = 13 mn	ı							
Anchor plate bearing area;	$A_{ap} = b_{ap}^2$ -	π×d _h ²/4 = 3467	mm ²						
Bearing capacity;	$P_{ap} = 0.6 \times$	f _{cu} × A _{ap} = 52.0	kN						
		F	PASS - Anchor	plate bearing ca	pacity is ade	quate (0.014)			
Bearing pressure on anchor plat	$f_{ap} = F_{ap} / A$	$f_{ap} = F_{ap} / A_{ap} = 0.2 \text{ N/mm}^2$							
Width of bolt head (across flats);	d _{bh} = 19.0	d _{bh} = 19.0 mm							
Maximum cantilever length;	l _{ap} = b _{ap} /2⇒	$I_{ap} = b_{ap}/2 \times \sqrt{(2)} - d_{bh}/2 = 32.9 \text{ mm}$							
Bending moment in plate;	$m_{ap} = f_{ap} \times$	$m_{ap} = f_{ap} \times I_{ap}^2/2 = 0.1 \text{ Nm/mm}$							
Bending capacity;	m _{cap} = p _{yap}	$m_{cap} = p_{yap} \times t_{ap}^2/4 = 1.7 \text{ Nm/mm}$							
		P	ASS - Anchor µ	olate bending ca	pacity is ade	quate (0.068)			
Holding down bolt anchorage									
Note - the following calculation to	o check the hole	ding down bolt a	nchorage into th	ne foundation ass	umes that the	edges of the			
foundation are sufficiently far fro	m the anchor p	lates to not affeo	t the punching s	shear perimeter.		-			
Tension force to be resisted;	-	$F_{t} = T_{f} = 1.$	5 kN	-					
Nominal cover to top reinforcem	c _{nom} = 30 mm								
Effective depth of HD bolts;	L _{HD} = E - c	L _{HD} = E - c _{nom} = 70 mm							
Shear strength of concrete;	vc = 0.34 N	v _c = 0.34 N/mm ²							
Effective shear perimeter;	P _{HD} = 2 × [$P_{HD} = 2 \times [(b_{ap} + 2 \times 1.5 \times L_{HD}) + (s_{bolt} \times (N_{bolt}/2 - 1) + b_{ap} + 2 \times 1.5 \times L_{HD})]$							
		P _{HD} = 1370	mm						
Pull-out capacity of tension bolts	$P_t = v_c \times P_{HD} \times L_{HD} = 32.6 \text{ kN}$								
		I	PASS - Holding	down bolt anch	norage is ade	quate (0.046)			
Shear transfer to concrete									
Assumed coefficient of friction;		μ = 0.30							

PASS - Frictional shear capacity is adequate (0.370)

Available shear resistance;