Structural Calculations

Address: 19 Belmont Park, London, SE13 5BJ

Job no: 1666

Subje 101 cf Subje 101 cf Ped Hone, Norden, Suf 45ff stan@vermethructures.co.uk	Address: 19 Belmont Park, London, SE13 5BJ	Job no : 1666		
	Part of structure : Loading on members	Sheet no 2		
	Made by : SW	Checked by		

Structural Calculations for the proposed additional floor to 19 Belmont Park, London, SE13 5BJ. The property is a two storey midterraced house. It is of traditional construction, with cavity masonry elevation walls, suspended timber floors and timber roof with tile finish. The rafters are supported by timber purlin and diagonal struts that are in turn supported on internal loadbearing wall.



	Address: 19 Belmont Parl		Job no: 1666	
Suite 101 C1 Peel House,	Part of structure : Loadin	g on members		Sheet no 3
Morden, SM4 58T stan@exmstructures.co.uk	Made by : SW			Checked by
UNFACTURED LOADING	DEAD LOAD	LIVE LOAD		OAD
Pitched Roof				
Wind			0.70kN/	m2
Imposed		0.75 kN/m2		
Tiles	0.70 kN/m2			
Insulation	0.05 KN/m2			
Rafters, Battens & Lining	0.20 kN/m2			
Total on Slope	0.95 kN/m2			
Load on plan (11 degrees)	0.97 kN/m2			
Existing Flat Ceiling (over First	: Floor)			
Imposed		0.25 kN/m2		
Plasterboard and skim	0.20kN/m2			
Joists	0.05 kN/m2			
Total on plan	1.22 kN/m2	1.00 kN/m2	0.70 kN	/m2
Ceiling				
Imposed		0.25 kN/m2		
loists + insulation	0.10 kN/m2	0.25 Kity in2		
Plasterboard	0.15 kN/m2			
Total	0.25 kN/m2	0.25 kN/m2		
Timber Floor				
Imposed		1.50 kN/m2		
Joists	0.13 kN/m2			
Insulation	0.04 kN/m2			
Floor Boards	0.18 kN/m2			
Ceiling	0.15 kN/m2			
Total	0.50 kN/m2	1.50 kN/m2		
	2.041.04/22			
BLICKWOLK	2.04 KN/m2			
Pidster	0.20 KN/M2			
IOTAI	4.00 kN/m2			

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I SWN	N STRU	CTURES	Address: 2	19 Bel	mor	nt Pa	irk, Loi	ndon,	, SE13 5BJ			Job no : 1666
Suite 101 C1 Peel House, Montes			Part of str	uctur	e : L	oadi	ing on	mem	ibers			Sheet no 4
SM4 5BT stan@swmstructures	.co.uk		Made by	: SW								Checked by
1. Load on Ce	eiling Bea	m R/1										
Steel Beam span		5.80	m	o/p								
									DEAD		LIVE	
** Load from Ce	eiling											
Roof spar	n on plan	=		7.80	m	o/p						
LOAD		LAL /ma 2		SPAN			0.5	_	0.08	LAL /ma		
Live = 0.25		kN/m2	x	5.1	m	×	0.5	_	0.98	KIN/III	0 98	kN/m
LIVE - 0.25		KN/1112	^	5.1		^	0.5	-			0.50	KN/III
				Total	Load	on St	eel Beam	(SLS)				
							De	ead =	1.0	kN/m		
							Li	ve =			1.0	kN/m
Beam self weigh	t	I., 1										
sw=	0.2	Kn/m										
REACTION RA					<u>RE</u>		<u>N RB</u>					
RA _{dead} =	3.41	KN			RB	DEAD=	3.41	KN				
KAuve=	2.83	KN			RE	SLIVE=	2.83	KN				
2. Load on Sta	eel Beam	2/1										
Steel Beam spans	5	5.80	m	o/p								
-									DEAD		LIVE	
* *Load from Sec	ond Floo	r										
			span on plan =	5.00	m	o/p						
Dead = 0.50		kN/m2	x	5PAN 5.0	m	x	0.5	=	1.25	kN/m		
Live = 1.50		, kN/m2	x	5.0	m	x	0.5	=		,	3.75	kN/m
				Total	Load	on St	eel Beam	(SLS)				
							De	ead =	1.3	kN/m	20	kNI /m
Beam self weigh	t						LI	ve –			5.0	KIN/ III
sw=	0.4	kn/m										
					DE							
RADEAD=	4.79	ĸN			REA RB	dead=	<u>чкв</u> 4.79	ĸN				
RAuve=	10.88	KN			RE	BLIVE=	10.88	KN				
3. Load on Ste	eel Beam	2/2										
Steel Beam spans	5	5.80	m	o/p								
* *I and from Sec	and Floo	r							DEAD		LIVE	
	.5114 1100	•	span on plan =	5.50	m	o/p						
LOAD				SPAN		• F						
Dead = 0.50		kN/m2	x	5.5	m	х	0.5	=	1.38	kN/m		
Live = 1.50		kN/m2	x	5.5	m	х	0.5	=			4.13	κN/m
				Total	Load	on St	eel Beam	(SLS)				
							De	ead =	1.4	kN/m		
							Li	ve =		-	4.1	kN/m
Beam self weigh	t											
sw=	0.4	kn/m										
REACTION RA					<u>RE</u> A		N RB					
RA _{DEAD} =	5.15	KN			RB	DEAD=	5.15	KN				
RA _{LIVE} =	11.96	KN			RE	BLIVE=	11.96	KN				
4 1040 041	Fristing F	undation										
LUAD UN	LAISUNG F	<u>unuuu011</u>							ΠΕΔΠ		LIVE	
									DLAD			

2		CTURES	Address: 2	19 Bel	mor	nt Pa	rk, L	ondon, S	E13 5BJ				Job no : 1666
Suite	• V V I V I • 101 C1 House,		Part of str	uctur	e : L	oadi	ng o	n memb	ers				Sheet no 5
Mord SM4	ion, 58T	ļ	Made by	: SW			0						Checked by
stan	www.nSUUCUIPOS.co.UK												
* *Load	from external wal	1											
	Wall Height	9.00	m										
	LOAD			SPAN									
Dead =	4.00	kN/m2	x	9.0	=				36	.00 kM	N/m		
* *Load	from Main roof												
Root	f Rafter span on pl	an =		3.90	m	o/p							
	LOAD			SPAN									
Dead =	1.22	kN/m2	x	3.9	m	x	0.5	=	2.3	8 kN	N/m		
Live =	1.00	kN/m2	x	3.9	m	x	0.5	=				1.95	kN/m
* *Load	from Second Floo	r											
			span on plan =	2.80	m	o/p							
	LOAD			SPAN									
Dead =	0.50	kN/m2	x	2.8	m	x	0.5	=	0.7	'0 kN	N/m	2 10	kN/m
Live =	1.50	KN/M2	x	2.8	m	х	0.5	=				2.10	KN/M
			То	tal Load	on Fo	undati	ion (SL	S) UDL					
								Dead =	3	9.1 kM	N/m		
								Live =				4.1	kN/m
<u>5. Lo</u>	ad on Lintel												
	Lintel span	1.9	m	o/p									
									DE	AD		LIVE	
* *Load	from Main roof												
Root	f Rafter span on pl	an =		3.90	m	o/p							
	LOAD			SPAN									
Dead =	1.22	kN/m2	x	3.9	m	х	0.5	=	2.3	8 kN	N/m		
Live =	1.00	kN/m2	x	3.9	m	x	0.5	=				1.95	kN/m
* *Load	from external wal	I											
	Wall Height	0.50	m										
	LOAD			SPAN									
Dead =	4.00	kN/m2	x	0.5	=				2.0	00 kM	N/m		
					Total	Load o	on Lint	ell (SLS)					
								Dead =	2.4	k ki	N/m		
								Live =				2.0	kN/m
Working	Load on Lintel L1	(ULS)											

V= 8.2 kN

$\left \right $		Project 17	7 Belmont Park,	Job no. 1665			
	Suite Tot CT Peel House, Morden, Si44 58T stan@sumstructures.co.uk	Calcs for	TIMBER	Start page no./Revision 6			
		Calcs by SW	Calcs date 25/04/2022	Checked by	Checked date	Approved by	Approved date



(SWM STRUCTURES	Project	Relmont Park	London SE13 F	B I	Job no. 1665		
	Suite 101 C1					Start page pp /P		
	Morden, SM4 58T	Calcs IO	TIMBER	RAFTER		Start page 110./h	7	
`	stan ogsumstructures.co.uk	Calcs by SW	Calcs date 25/04/2022	Checked by	Checked date	Approved by	Approved date	
	.							
	Check compressive stress pa	arallel to grain	C 000	1/2				
	Compression stress parallel to	grain	$\sigma_c = 0.800$	N/mm^2				
	Compression member faster		⊏min = 3000	/ IN/IIIII1-				
	Pormissible compressive stress	_	$r_{12} = 0.52$		2 997 N/mm ²			
	Applied compressive stress	5	$\sigma = \nabla c$	$\times 13 \times 18 \times 112 =$	= 3.007 N/IIIII	A) 0 417 N/	mm ²	
	Applied compressive stress		Oc_max = F ×	S - Applied con	pressive stres	s within perm	nissible limits	
	Check combined bending an	d compressive st	ress parallel t	to grain		•		
	Euler stress		$\sigma_{\rm e} = \pi^2 \times E_{\rm r}$	$_{\rm nin} / \lambda^2 = 9.231$ N	l/mm²			
	Fuler coefficient		K _{eu} = 1 – (1		$(\sigma_{\rm e}) = 0.965$			
	Combined axial compression a	nd bending check	σ= ==== / (σ=	$dm \times K_{out} + \sigma_{o}$, εξ) στοστο nov / σο odm = 0.5	24 < 1		
		PASS - Comb	ined compres	sive and bend	ina stresses ar	e within perm	nissible limits	
	Chack chaor stress					<i>p</i>		
	Shear stress parallel to grain		τ – 0 670 Ν	l/mm ²				
	Permissible shear stress		$\tau = 0.070$ K	ν ν × Κο – Ο 737 Ν	l/mm ²			
	Applied shear stress			$3 \times 108 = 0.731$ N	$0.100 \text{ N}/\text{mm}^2$			
	Applied silear siless		$t_{max} = 3 \times 1$	PASS - Appl	ied shear stres	s within perm	nissible limits	
	Check deflection					•		
	Permissible deflection		$\delta_{adm} = 0.003$	3 × L _{eff} = 11.935	mm			
	Bending deflection		$\delta_{\rm b} = 5 \times F \times$	$\times L_{eff}^4$ / (384 $\times E_{rr}$	nean × I) = 5.321 ∣	mm		
	Shear deflection		$\delta_s = 12 \times F$	\times Leff ² / (5 \times Eme	$(an \times A) = 0.158$ r	nm		
	Total deflection		$\delta_{max} = \delta_{b} + \delta_{b}$	δ _s = 5.479 mm				
				PASS -	Total deflectio	n within perm	nissible limits	
	Consider medium term load (condition						
	Load duration factor		K ₃ = 1.25					
	Total UDL perpendicular to raft	er	$F = [F_u \times cc]$	$os(\alpha)^2 + F_d \times cos$	$[\alpha)] \times s + F_j \times c\alpha$	os(α) = 0.610	≺N/m	
	Notional bearing length		$L_b = F \times L_{cl}$	/ $[2 \times (b \times \sigma_{cp1} \times$	K ₈ - F)] = 10 mi	m		
	Effective span		$L_{eff} = L_{cl} + L_{cl}$	_{-b} = 3983 mm	<i>/-</i>			
	Check bending stress							
	Bending stress parallel to grain		σm = 5.300	N/mm ²				
	Permissible bending stress		σ_m adm = σ_m	× K3 × K7 × K8 =	- 7.733 N/mm²			
	Applied bending stress		$\sigma_{m max} = F$	$\times 1_{\text{eff}^2} / (8 \times 7) =$	4.737 N/mm ²			
	. hh		- max	PASS - Applied	l bending stres	s within perm	nissible limits	
	Check compressive stress pa	arallel to grain						
	Compression stress parallel to	grain	σ _c = 6.800	N/mm²				
	Minimum modulus of elasticity		Emin = 5800	N/mm²				
	Compression member factor		K ₁₂ = 0.47					
	Permissible compressive stress	6	$\sigma_{c_{adm}} = \sigma_{c}$	\times K ₃ \times K ₈ \times K ₁₂ =	4.393 N/mm ²			
	Applied compressive stress		$\sigma_{c_{max}} = F \times$	$L_{eff} imes (cot(\alpha) + 3)$	$3 \times \tan(\alpha)) / (2 \times$	(A) = 0.795 N/	mm²	
			PAS	S - Applied con	pressive stres	s within perm	nissible limits	
	Check combined bending an	d compressive st	ress parallel t	to grain				
	Euler stress	·	$\sigma_{e} = \pi^{2} \times E_{r}$	min / $\lambda^2 = 9.209$ N	l/mm²			
	Euler coefficient		K _{eu} = 1 – (1	$.5 imes \sigma_{c_max} imes K_{12}$	/ σ _e) = 0.939			
	Combined axial compression a	nd bending check	σ _{m_max} / (σ _m	_adm $ imes$ Keu) + σ_{c_r}	$max / \sigma_{c_{adm}} = 0.8$	33 < 1		

	Project 17 Belmont Park, London, SE13 5BJ				Job no. 1665				
Suite 101 C1 Peel House,	Calcs for				Start page no./F	levision			
Moraen, SM4 5BT stan@swmstructures.co.uk		TIMBER	RAFTER			8			
	Calcs by SW	Calcs date 25/04/2022	Checked by	Checked date	Approved by	Approved dat			
	PASS - Co	ombined compre	ssive and bei	nding stresses ar	re within pern	nissible lim			
Check shear stress									
Shear stress parallel to grain		τ = 0.670 Ν	J/mm ²						
Permissible shear stress		$\tau_{adm} = \tau \times k$	K ₃ × K ₈ = 0.92 1	I N/mm ²					
Applied shear stress		$\tau_{max} = 3 \times F$	$^{-} \times L_{eff} / (4 \times A)$) = 0.208 N/mm ²					
			PASS - Ap	plied shear stres	s within pern	nissible lim			
Check deflection									
Permissible deflection		$\delta_{adm} = 0.00$	3 × L _{eff} = 11.9	49 mm					
Bending deflection		$\delta_b = 5 \times F >$	imes L _{eff} ⁴ / (384 $ imes$	$E_{mean} \times I) = 10.168$	3 mm				
Shear deflection		$\delta_s = 12 \times F$	\times L _{eff} ² / (5 \times E	mean × A) = 0.301 r	mm				
Total deflection		$\delta_{\text{max}} = \delta_{\text{b}} +$	δ _s = 10.469 m	im					
			n within pern	nissible lim					
Consider chart tarm land	dition								
Load duration factor	lattion	K 1 50							
Total LIDL perpendicular to raft	٥r	$F = F_1 \times co$	$e(\alpha) \vee e + E \vee$	$x \cos(\alpha) = 0.321 \text{ km}$	\/m				
Notional bearing length	GI		$H = E_1 \times \cos(\alpha)$	$(22)^{1} = 0.521$ K	$K_{0} = F(1) = 0$ m	n			
Effective span			s – 3082 mm)] / [Z ~ (b ~ 0cp1 ~	1(8 - 1)] - 3 iiii				
			_D = 3302 mm						
Check bending stress			NU 0						
Bending stress parallel to grain		σ _m = 5.300	N/mm²						
Permissible bending stress		$\sigma_{m_{adm}} = \sigma_{n}$	$_{1} \times K_{3} \times K_{7} \times K_{7}$	^k ⁸ = 9.279 N/mm ²		2			
Applied bending stress		$\sigma_{m_{max}} = F$	<l<sub>eff²/(8×Z)+Fp PASS - Appli</l<sub>	×cos(α)×L _{eff} /(4×Z) ied bending stres	= 5.936 N/mm ss within pern	n ² nissible limi			
Check compressive stress pa	arallel to grai	n							
Compression stress parallel to	grain	$\sigma_c = \textbf{6.800}$	N/mm ²						
Minimum modulus of elasticity		Emin = 5800) N/mm²						
Compression member factor		K ₁₂ = 0.43							
Permissible compressive stress	3	$\sigma_{c_adm} = \sigma_{c}$	$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \textbf{4.780} \text{ N/mm}^2$						
Applied compressive stress		$\sigma_{c_{max}} = F \times$	$\sigma_{c_{max}} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) + F_{p} \times \sin(\alpha) / A = 0.437 \text{ N/mm}^2$						
Ohaale aswebinged banding an					e mani peri				
Fular strass	a compressiv	$\pi = \pi^2 \vee \Box$	10 yrain $/\lambda^2 = 0.01\lambda$	1 N/mm ²					
Fuler coefficient		$\mathbf{U}_{\mathbf{e}} = \mathcal{H}^{-} \times \mathbf{E}$	mm// = 3.∠1 4	$(10 / \sigma_{1}) = 0.070$					
	nd honding ch	$R_{eu} = 1 - ($	I.O X Oc_max X r	$(12 / 0_e) = 0.970$	'51 <i>-</i> 1				
Complined axial complession a		ombined compre	$r_{adm} \times r_{eu} + C$	$D_{c_max} / O_{c_adm} = 0.7$	JI < I rowithin norn	nicciblo limi			
Ohaala ahaa ahaa	1 400 - 00			ianiy sucesces di					
Check shear stress		0 0 7 0 1	1/22/22						
Snear stress parallel to grain		$\tau = 0.670$ N		N //					
Permissible shear stress		$\tau_{adm} = \tau \times K$	$x_3 \times x_8 = 1.106$	או מ/mm² או גערייק או					
Applied shear stress		$\tau_{max} = 3 \times F$	- × L _{eff} / (4 × A PASS - Ap	$(\alpha) + 3 \times F_p \times \cos(\alpha)$) / (2 × A) = 0.2 ss within pern	nissible lim			
Check deflection									
Permissible deflection		$\delta_{adm}=0.00$	3 × L _{eff} = 11.9	46 mm					
Bending deflection		$\delta_{\text{b}} = \ L_{\text{eff}}{}^3 \times$	(5×F×L _{eff} / 38	$4 + F_p \times \cos(\alpha)/48$) / ($E_{mean} \times I$) =	11.254 mm			
		δ. – 12 × I	$L_{eff} \times (F \times L_{eff} + 2 \times F_p \times \cos(\alpha)) / (5 \times E_{mean} \times A) = 0.378 \text{ mm}$						
Shear deflection		$U_S = 1Z \land L$			-mount				

	Project 17	Belmont Park,	Job no. 1665			
Suite 101 C1 Peel House, Morden, SM4 581 stom/Bermstructures.co.uk	Calcs for	TIMBER	RAFTER		Start page no./Re	vision 9
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PASS - Total deflection within permissible limits



	Project Job no.						
Suite 101 C1	19	Beimont Park,	London, SE13	281		000	
Peel House, Morden, SW4 58T	Gaics for	TIMBER CE	ILING JOIST		Start page no./F	11	
stan@swmstructures.co.uk	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date	
	500	03/05/2022					
Reactions at support A		RA_max = 0.4	I34 kN	RA_min =	0.434 kN		
Unfactored dead load reaction a	at support A	$R_{A_Dead} = 0.$	239 kN				
Unfactored imposed load reacti	on at support A	$R_{A_{imposed}} =$	0.195 kN	_			
Reactions at support B	at augment P	$R_{B_{max}} = 0.4$	134 KN	KB_min =	0.434 KN		
Unfactored imposed load reaction a	on at support B	$RB_{B} = \mathbf{U}.$	0.195 kN				
		Ttb_imposed =					
				\leq			
<u>▼</u> <u>v</u> v 4 −50→	$\overline{}$						
Timber section details							
Breadth of sections		b = 50 mm					
Depth of sections		h = 125 mn	ı				
Number of sections in member		N = 1					
Overall breadth of member		$b_b = N \times b =$	= 50 mm				
Timber strength class		C16					
Member details							
Service class of timber		1					
Load duration		Long term					
The beam is part of a load-shar	ing system cons	isting of four or	m more members				
Section properties							
Cross sectional area of membe	r	$A = N \times b \times$	h = 6250 mm ²				
Section modulus		$Z_x = N \times b$	< h ² / 6 = 13020	8 mm³			
		$Z_y = h \times (N$	× b)² / 6 = 5208	3 mm ³			
Second moment of area		$I_x = N \times b \times$	h ³ / 12 = 81380)21 mm⁴			
		$I_y = h \times (N > $	< b) ³ / 12 = 130 2	2 083 mm ⁴			
Radius of gyration		$i_x = \sqrt{(I_x / A)}$	= 36.1 mm				
		$i_y = \sqrt{(I_y / A)}$	= 14.4 mm				
Modification factors							
Duration of loading - Table 17		K3 = 1.00					
Bearing stress - Table 18	•	$K_4 = 1.00$	(1)011 4 44	_			
Total depth of member - cl.2.10	.6	K7 = (300 n	$(m / h)^{0.11} = 1.10$	D			
Load sharing - cl.2.9		$\kappa_8 = 1.10$					
Lateral support - cl.2.10.8	ore hold in line	as by purling or	tio rode at cast	rec not more the	n 30 timos th	o broadth of	
the member		as by putilitis of	ue rous at cent	ies not more tha	in so times (N		
Permissible depth-to-breadth ra	tio - Table 19	4.00					
Actual depth-to-breadth ratio	-	h / (N × b) =	= 2.50				
-		. ,		PASS - L	ateral suppo	rt is adequate	
Compression perpendicular t	o grain						
Permissible bearing stress (no v	wane)	$\sigma_{c_adm} = \sigma_{cp}$	$_1 imes K_3 imes K_4 imes K_8$	= 2.420 N/mm ²			
Applied bearing stress		$\sigma_{c_a} = R_{A_ma}$	$_{\rm ax}$ / (N × b × L _b) =	= 0.087 N/mm ²			

	Project 19	Job no. 1666				
Suite 101 C1 Peel House, Morden, SM4 5BT	Calcs for	TIMBER CE	ILING JOIST		Start page no./Re	evision 2
stan@swmstructures.co.uk	Calcs by SW	Calcs date 03/05/2022	Checked by	Checked date	Approved by	Approved date

	$\sigma_{c_a} / \sigma_{c_adm} = 0.036$
PASS - Applied	compressive stress is less than permissible compressive stress at bearing
Bending parallel to grain	
Permissible bending stress	$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \textbf{6.419} \text{ N/mm}^2$
Applied bending stress	$\sigma_{m_a} = M / Z_x = 3.251 \text{ N/mm}^2$
	$\sigma_{m_a} / \sigma_{m_adm} = 0.507$
	PASS - Applied bending stress is less than permissible bending stress
Shear parallel to grain	
Permissible shear stress	$\tau_{adm} = \tau \times K_3 \times K_8 = 0.737 \text{ N/mm}^2$
Applied shear stress	$\tau_a = 3 \times F / (2 \times A) = 0.104 \text{ N/mm}^2$
	$\tau_a / \tau_{adm} = 0.141$
	PASS - Applied shear stress is less than permissible shear stress
Deflection	
Modulus of elasticity for deflection	$E = E_{mean} = 8800 \text{ N/m} \text{ m}^2$
Permissible deflection	δ_{adm} = min(0.551 in, 0.003 \times Ls1) = 11.700 mm
Bending deflection	$\delta_{b_{s1}} = 9.366 \text{ mm}$
Shear deflection	$\delta_{v_s1} = 0.148 \text{ mm}$
Total deflection	$\delta_a = \delta_{b_s1} + \delta_{v_s1} = 9.514 \text{ mm}$
	$\delta_a / \delta_{adm} = 0.813$
	PASS - Total deflection is less than permissible deflection

S		Project 17	' Belmont Park,	Job no. 1665			
Suite Peel Morde SM4 stand	House, en, 581 Semmetructures.co.uk	Calcs for	STEEL E	3EAM 2/1		Start page no./Re 1	vision 6
		Calcs by SW	Calcs date 25/04/2022	Checked by	Checked date	Approved by	Approved date

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support B

Imposed imes 1.60

 $\begin{array}{l} \text{Dead} \times 1.40 \\ \text{Imposed} \times 1.60 \end{array}$

SW/M STRUCTURES	Project	Dolmont Dork	London CE12	EDI	Job no.	CCE
		Beimoni Park,	London, SE13	281		
Norden, SM4 58T	Calcs for	STEEL I	BEAM 2/1		Start page no./F	17
SULINA SAMURATING COLLINES.CO.UK	Calcs by SW	Calcs date 25/04/2022	Checked by	Checked date	Approved by	Approved date
Analysis results						
Maximum moment		M _{max} = 35. 4	l kNm	$M_{min} = 0$) kNm	
Maximum shear		V _{max} = 24.4	kN	V _{min} = -	24.4 kN	
Deflection		δ _{max} = 17.8	mm	$\delta_{\text{min}} = \boldsymbol{0}$	mm	
Maximum reaction at support A		R _{A_max} = 24	. 4 kN	R _{A_min} =	24.4 kN	
Unfactored dead load reaction a	at support A	$R_{A_{Dead}} = 4$. 8 kN			
Unfactored imposed load reacti	on at support A	$R_{A_Imposed} =$	11 kN			
Maximum reaction at support B		R _{B_max} = 24	.4 kN	R _{B_min} =	24.4 kN	
Unfactored dead load reaction a	at support B	$R_{B_{Dead}} = 4$. 8 kN			
Unfactored imposed load reacti	on at support B	$R_{B_{Imposed}} =$	11 kN			
Section details						
Section type		UC 152x15	52x37 (BS4-1)			
Steel grade		S275				
From table 9: Design strength	ı p _y					
Thickness of element		max(T, t) =	11.5 mm			
Design strength		p _y = 275 N/	mm²			
Modulus of elasticity	10	E = 205000) N/mm²			
	1. 1.					
	φ. ∞,					
	▲ ↓ 10 ↓]		
	◀			►		
Lateral restraint						
		Span 1 has	s lateral restrai	nt at supports only	y	
Effective length factors						
Effective length factor in maior	axis	K _x = 1.00				
Effective length factor in minor	axis	K _y = 1.00				
Effective length factor for latera	I-torsional bucklir	ng K _{LT.A} = 1.20) + 2 × D			
J		К _{LT.В} = 1.20) + 2 × D			
Classification of cross sectio	ns - Section 3.5		1/2 / 1	00		
		$\varepsilon = \sqrt{2/5}$ N	$p_{y}[= 1.$	UU		
Internal compression parts -	Table 11					
Depth of section		d = 123.6 r	nm			
		d / t = 15.5	$3 \times 08 \Rightarrow 3 \otimes 08 \Rightarrow 3 \times 08 \Rightarrow 3 \otimes 08 \times 08 \Rightarrow 08 \Rightarrow 08 \Rightarrow 08 \Rightarrow 08 \Rightarrow 08 \Rightarrow 08 \Rightarrow$	Class 1	plastic	

	Project	7 Belmont Park	London SE13	5B I	Job no.	665
Suite 101 C1		Deimonit Faik,		565		
Norden, SM4 58T stanflewmetructures co.uk	Calcs for	STEEL I	BEAM 2/1		Start page no./F	18
SIGNARAMINO ICCINI COLCOLUM	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	300	23/04/2022				
Outstand flanges Table 11						
Width of section		h – B / 2 –	77 2 mm			
		b = b + 2 = b + 7 = 67	× ε <= 9 × ε	Class 1	plastic	
		571 - 6.7			Section is a	class 1 plastic
Shoor consolity Section 4.2.2						
Design shear force		F⊬ = max(a	hs(Vmax) ahs()	(_{min})) = 24.4 kN		
Design shear lorde		$d/t < 70 \times$	۶ (۱۳۵۲), ۵۵۵(۱	(min)) – 24.4 KK		
			Web does	not need to be c	hecked for s	hear buckling
Shear area		$A_v = t \times D =$	= 1294 mm ²			3
Design shear resistance		$P_v = 0.6 \times r$	$D_{v} \times A_{v} = 213.6$	kN		
		PAS	S - Design sh	ear resistance e	xceeds desig	n shear force
Moment capacity - Section 4 3	0.5		5			
Design bending moment		M = max(al	os(Ms1 max) abs	$S(M_{s1}, min)) = 35.4$	kNm	
Moment capacity low shear - cl.	4.2.5.2	$M_c = min(p)$	$v \times S_{xx}$. 1.2 × p_v	$x \times Z_{xx}$) = 84.9 kNr	n	
Effective length for lateral tor	aional huakling	Section 4.2 F	,, -,	,		
Effective length for lateral torsio	nal buckling	J = 3ection 4.3.c	, +2 ∨ D - 7 2	284 mm		
Slenderness ratio	na bucking	$\lambda = 1 \pi / r_{max}$	_s + 2 ^ D = 72 _ 188 110			
		$\mathcal{R} = \mathbf{L} \mathbf{E} \mathbf{r} \mathbf{y} \mathbf{y}$	- 100.115			
Equivalent sienderness - Sect	lion 4.3.6.7	0.949				
Torsional index		u = 0.040 v - 13 334				
Slenderness factor		x = 10.004 y = 1 / [1 + 10.004]	$0.05 \times (\lambda / x)^{210}$	^{0.25} = 0 -550		
Batio - cl 4 3 6 9		βw = 1.000	0.00 / (// //)]	_ 01000		
Equivalent slenderness - cl.4.3.	6.7	$\lambda_{1T} = U \times V$	×λ×√[ßw] = 8	7.710		
Limiting slenderness - Annex B.	2.2	$\lambda_{10} = 0.4 \times$	$(\pi^2 \times E / p_v)^{0.5} =$	= 34.310		
		$\lambda_{IT} > \lambda_{IO} - $	Allowance sho	ould be made fo	r lateral-torsi	onal buckling
Ponding strongth Costion 4	0 6 5					
Behaving Strength - Section 4.	5.0.5	au 7 0				
Perry factor		$m_{\rm LT} = max/r$	$\gamma = \chi (\lambda = \lambda)$	(1000 0) = 0.37	4	
Fuler stress		$\eta_{L1} = max(0)$	/ λ. - ² - 263 N/	mm^2	-	
		$p_E = \pi \times E$ $\phi_T = (p_v + 1)$	$(n_{1T} + 1) \times n_{E})/$	2 = 318 2 N/mm ²	2	
Bending strength - Annex B 2 1		$\varphi_{E} = 0 = X D_{E}$, / (φ τ + (φ τ ² - 1	$p_{\rm E} \times p_{\rm e})^{0.5} = 148$	2 N/mm ²	
			// (ΨΕΙ Ι (ΨΕΙ	$p \ge \langle p_y \rangle $) = 140.		
Equivalent uniform moment to	actor - Section	4.3.6.6 M. 26.5 L	Nm			
Moment at centre-line of segme	int	$M_2 = 35.4 \text{ k}$	Nm			
Moment at three quarter point o	f seament	M ₄ = 26.5 k	Nm			
Maximum moment in segment		M _{abs} = 35.4	kNm			
Maximum moment governing bu	uckling resistand	e MLT = Mabs	= 35.4 kNm			
Equivalent uniform moment fact	or for lateral-tor	sional buckling				
		$m_{LT} = max(0)$	$0.2 + (0.15 \times M)$	$_{2} + 0.5 \times M_{3} + 0.1$	$5 imes M_4) / M_{abs}$, 0.44) = 0.925
Buckling resistance moment	- Section 4.3.6.	4				
Buckling resistance moment		$M_b = p_b \times S$	5 _{xx} = 45.7 kNm			
		$M_b / m_{LT} = 4$	49.5 kNm			
		PASS - Buckli	ng resistance	moment exceed	ls design ber	nding moment

	Project 1	Project 17 Belmont Park, London, SE13 5BJ			Job no. 1665	
Suite for Cr Peel House, Morden, SM4 587 stan@symstructures.co.uk	Calcs for	STEEL E	BEAM 2/1		Start page no./Re	evision 19
	Calcs by SW	Calcs date 25/04/2022	Checked by	Checked date	Approved by	Approved date

Check vertical deflection - Section 2.	5.2
Consider deflection due to dead and im	posed loads
Limiting deflection	$\delta_{\text{lim}} = L_{\text{s1}} / 250 = \textbf{23.2} \text{ mm}$
Maximum deflection span 1	$\delta = max(abs(\delta_{max}), abs(\delta_{min})) = 17.763 \text{ mm}$
	PASS - Maximum deflection does not exceed deflection limit

	UCTURES Project Job 17 Belmont Park, London, SE13 5BJ					Job no. 1665	
Suite 101 C1 Peel House, Morden, SM4 581 ston@sematructures.co.uk	Calcs for	STEEL E	3EAM 2/2		Start page no./Re	vision 20	
	Calcs by SW	Calcs date 25/04/2022	Checked by	Checked date	Approved by	Approved date	

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support B

Support A

Applied loading Beam loads

Load combinations Load combination 1

Vertically restrained Rotationally free

Dead full UDL 1.4 kN/m Imposed full UDL 4.1 kN/m Dead self weight of beam $\times\,1$

Support A

 $\text{Dead} \times 1.40$ Imposed imes 1.60 $\text{Dead} \times 1.40$ Imposed imes 1.60 $\text{Dead} \times 1.40$ Imposed imes 1.60

Support B

	Project				Job no.	
Suite 101 C1	17	Belmont Park,	London, SE13	5BJ	1	665
Peel House, Morden, SM4 58T	Calcs for	STEEL I	BEAM 2/2		Start page no./F	Revision 21
stan@swmstructures.co.uk	Calcs by SW	Calcs date 25/04/2022	Checked by	Checked date	Approved by	Approved date
Analysis results						
Maximum moment		Mmax = 38 k	Nm	M _{min} =	0 kNm	
Maximum shear		$V_{max} = 26.2$	kN	$V_{min} = -$	26 2 kN	
Deflection		$\delta_{max} = 10.1$	mm	$\delta_{\min} = 0$	mm	
Maximum reaction at support A		B 26	2 kN	Be use -	- 26 2 kN	
Linfactored dead load reaction	at support A	B_{A} Deed = 5	1 kN	TA_min -	- 20.2 NN	
Linfactored imposed load reaction	ion at support A		11 9 kN			
Maximum reaction at support F		B _R may = 26	2 kN	Bp. min -	- 26 2 kN	
Linfactored dead load reaction	at support B	Br. Deed = 5	1 kN	TtB_min -	- 20.2 NN	
Linfactored imposed load reaction	ion at support B		11 9 kN			
Section dataile		TB_imposed —	11.5 KN			
Section type		UC 152v15	2v37 (BS4-1)			
Steel grade		S275	2,57 (054-1)			
From table 9: Design strengt	h n.	5215				
Thickness of element	i py	max(T_t) -	11 5 mm			
Design strength		$n_{\rm v} = 275 {\rm N}/{\rm s}$	mm ²			
Modulus of elasticity		F = 205000	N/mm ²			
	5.	2 - 200000				
	_ `			-		
	.13.					
	9					
	H 11.5					
	↑					
	4			▶		
Lateral restraint						
		Span 1 has	s lateral restrail	nt at supports on	У	
Effective length factors						
Effective length factor in major	axis	K _x = 1.00				
Effective length factor in minor	axis	K _y = 1.00				
Effective length factor for latera	Il-torsional bucklir	ig K _{LT.A} = 1.20	+ 2 × D			
		K _{LT.B} = 1.20) + 2 × D			
Classification of cross section	ns - Section 3.5					
		ε = √[275 Ν	$1/mm^2 / p_y] = 1.$	00		
Internal compression parts -	Table 11					
Depth of section		d = 123.6 r	nm			
		d / t = 15.5	$3 \times 08 = 3 \times 10^{-3}$	Class 1	plastic	

	Project	7 Belmont Park	London SE13	5B	Job no.	665
	Color for	7 Delifiont Fark,		565	Ctart page pa //	
Peer House, Morden, SM4 5BT stanBermstructures co.uk	Calcs for	STEEL	BEAM 2/2		Start page no./F	22
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	SW	25/04/2022				
Outstand flanges - Table 11			77.0			
width of section		D = B/2 = b/T = 67	//.2 mm	Close 1	plaatia	
		D / T = 0.7	× E <= 9 × E	Class	Soction is	place 1 plactic
					Section is t	ciass i piastic
Shear capacity - Section 4.2.3		– (()) 00 0 I N		
Design shear force		$F_v = max(a$	bs(V _{max}), abs(V	/ _{min})) = 26.2 kN		
		a / t < 70 ×	8 Wah daaa	not need to be a	hookod for a	haarbuakling
Shoor area		∧ + _{>} ⊃	1204 mm ²	not need to be c	necked for s	near buckning
		$A_v = [X D]$		LNI		
Design shear resistance		$P_v = 0.6 \times p$	$O_y \times A_v = 213.0$	KIN	xaaada daaid	an abaar faraa
		FAS	is - Design sin	ear resistance e	kceeus uesig	in shear force
Moment capacity - Section 4.2	2.5					
Design bending moment	4050	M = max(a)	DS(IMs1_max), abs	$S(M_{s1}_{min}) = 38 \text{ KN}$	im	
Moment capacity low snear - cl.	4.2.5.2	$M_c = min(p)$	$y \times S_{xx}$, 1.2 × p_y	$x \times Z_{xx}$) = 84.9 KINF	n	
Effective length for lateral-tors	sional buckling	g - Section 4.3.	5			
Effective length for lateral torsio	nal buckling	$L_E = 1.2 \times I$	$_{s1} + 2 \times D = 72$	2 84 mm		
Slenderness ratio		$\lambda = L_E / r_{yy}$	= 188.119			
Equivalent slenderness - Sect	ion 4.3.6.7					
Buckling parameter		u = 0.848				
Torsional index		x = 13.334				
Slenderness factor		v = 1 / [1 +	$0.05 \times (\lambda / x)^2]^{0}$	^{0.25} = 0.550		
Ratio - cl.4.3.6.9		βw = 1.000				
Equivalent slenderness - cl.4.3.	6.7	$\lambda_{\text{LT}} = u \times v$	$\times \lambda \times \sqrt{[\beta w]} = 8$	7.710		
Limiting slenderness - Annex B.	2.2	$\lambda_{L0} = 0.4 imes$	$(\pi^2 \times E / p_y)^{0.5} =$	= 34.310		
		$\lambda_{LT} > \lambda_{LO} - $	Allowance she	ould be made for	r lateral-torsi	ional buckling
Bending strength - Section 4.3	3.6.5					
Robertson constant		$\alpha_{\text{LT}} = 7.0$				
Perry factor		$\eta_{LT} = max(e)$	$lpha_{LT} imes (\lambda_{LT} - \lambda_{L0})$	/ 1000, 0) = 0.37	4	
Euler stress		$p_E = \pi^2 \times E$	/ $\lambda_{LT}^2 = 263 \text{ N}/$	mm²		
		$\phi_{LT} = (p_y + $	$(\eta_{LT} + 1) \times p_E) /$	2 = 318.2 N/mm ²	2	
Bending strength - Annex B.2.1		$p_b = p_E \times p_b$	$/(\phi_{LT} + (\phi_{LT}^2 - $	$p_E \times p_y)^{0.5}) = 148.5$	2 N/mm²	
Equivalent uniform moment fa	actor - Section	4.3.6.6				
Moment at quarter point of segn	nent	M ₂ = 28.5	٨m			
Moment at centre-line of segme	nt	M ₃ = 38 kN	lm			
Moment at three quarter point o	f segment	M4 = 28.5 k	٨Mm			
Maximum moment in segment		M _{abs} = 38 k	Nm			
Maximum moment governing bu	ickling resistand	$M_{LT} = M_{abs}$	= 38 kNm			
Equivalent uniform moment fact	or for lateral-tor	sional buckling				0.44) 0.005
		$m_{LT} = max(0)$	$0.2 + (0.15 \times M)$	$_{2} + 0.5 \times M_{3} + 0.1$	$5 \times IVI_4$) / IVI _{abs}	, 0.44) = 0.925
Buckling resistance moment -	Section 4.3.6.	4				
Buckling resistance moment		$M_b = p_b \times S$	5 _{xx} = 45.7 kNm			
		$M_b / M_{LT} = C$	49.5 KNM	momenteves	a daaian h	dina momont
		PAJJ - DUCKII	ny resistance	moment exceed	s uesiyn ber	iung noment

	Project	Project 17 Belmont Park, London, SE13 5BJ			Job no. 1665	
Suite 101 C1 Peel House, Morden, SM4 597 ston@symstructure.co.uk	Calcs for	STEEL E	3EAM 2/2		Start page no./Re	evision 23
	Calcs by SW	Calcs date 25/04/2022	Checked by	Checked date	Approved by	Approved date

Check vertical deflection - Sec	ction 2.5.2					
Consider deflection due to dead	and imposed lo	ads				
Limiting deflection		$\delta_{lim} = L_{s1} \; / \; 2$	50 = 23.2 mm			
Maximum deflection span 1		$\delta = \max(ab)$	$s(\delta_{max}), abs(\delta_{min})$)) = 19.064 mm		
		PAS	S - Maximum de	eflection does r	not exceed de	flection limit

	Project 17	' Belmont Park,	London, SE13 5	īBJ	Job no. 16	65
Suite for Cr Peel House, Morden, SM4 587 ston@symstructures.co.uk	Calcs for	PADST	ONE P1		Start page no./Re	evision 24
	Calcs by SW	Calcs date 25/04/2022	Checked by	Checked date	Approved by	Approved date

GN TO BS5628-1:2005		
		TEDDS calculation version 1.0.0
Clay or calcium silicate	e bricks	i.,
$p_{unit} = 5.0 \text{ N/mm}^2$		IV Normal
	Construction control	normai
γm = 3.5		$I_k = 2.2 N/11111^2$
l = 112 (1)(1) h = 2600 mm	Effective beight of wall	$l_{ef} = 223$ [[[[]]
Beam to span of the span of th	but of plane of wall	
of wall B = 152 mm _{Xedge} = 2000 mm	Length of bearing	l _b = 100 mm
G _k = 5 kN	Concentrated imposed load	Q _k = 12 kN
F = 26.5 kN		
g _k = 0.0 kN/m	Distributed imposed load	q _k = 0.0 kN/m
f = 0.0 kN/m		
Туре 2	Bearing safety factor	$\gamma_{\text{bear}} = 1.50$
out a spreader		
$f_{ca} = 1.742 \text{ N/mm}^2$	Allowable bearing stress	f _{cp} = 0.943 N/mm ²
FAIL - Design b	earing stress exceeds allowable be	aring stress. use a spread
		J
l. – 450 mm	Depth of spreader	h. – 150 mm
s = 1851 mm	Depth of spreader	
	GN TO BS5628-1:2005 Clay or calcium silicate punit = 5.0 N/mm ² Category II $\gamma_m = 3.5$ t = 112 mm h = 2600 mm Beam to span of ader bege being being being	GN TO BS5628-1:2005 Clay or calcium silicate bricks Point = 5.0 N/mm ² Mortar designation Category II Construction control $\gamma_m = 3.5$ Characteristic strength t = 112 mm Effective wall thickness h = 2600 mm Effective height of wall Description Of wall Beam to span out of plane of wall Of wall Of wall Mortar designation Of wall Beam Length of bearing Adop = 2000 mm Can e 5 kN Concentrated imposed load Tege 2 Bearing safety factor Out kN/m Distributed imposed load Fall to Design bearing stress FAIL - Design bearing stress exceeds allowable bearing

s_{edge} = **1851** mm

Edge distance

	Project				Job no	
		7 Belmont Park	London, SE13	5BJ	1	665
Norden, SM4 5BT	Calcs for	PADS ⁻	TONE P1		Start page no./F	Revision 25
stan@swmstructures.co.uk	Calcs by	Calcs date 25/04/2022	Checked by	Checked date	Approved by	Approved date
Spreader bearing type		- I	_			-
Bearing type	Type 1		Bearing safet	/ factor	$\gamma_{\text{bear}} = 1.25$	
Check design bearing with	h a spreader					
Design bearing stress	spreader f _{ca} = 0.525 N/r	nm²	Allowable bea	ring stress	f _{cp} = 0.786 N/	/mm²
		PASS -	Allowable bea	aring stress exc	eeds design b	earing stres
Check design bearing at 0	$1.4 \times h$ below the l	bearing level			6 0.440 N	/ 2
Design bearing stress	$f_{ca} = 0.106 \text{ N/r}$	nm ²	Allowable bea	iring stress	$f_{cp} = 0.412 \text{ N/}$	mm ²
PAS	S - Allowable bea	ring stress at 0	.4 x h below b	earing level exc	eeds design b	earing stres



(SWM STRUCTURES	Project	7 Belmont Park,	Job no. 1665				
	Suite 101 C1 Peel House,	Calcs for				Start page no./Revision		
	Morden, SM4 5BT stanØswmstructures.co.uk		TIMBER FL	OOR JOISTS			27	
		Calcs by SW	Calcs date 25/04/2022	Checked by	Checked date	Approved by	Approved date	
	Load sharing factor		K ₈ = 1.10					
	Consider long term loads							
	Load duration factor		K ₃ = 1.00					
	Maximum bending moment		M = 0.805	kNm				
	Maximum shear force		V = 1.150 k	ΚN				
	Maximum support reaction		R = 1.150	kN				
	Maximum deflection		δ = 5.901 n	nm				
	Check bending stress							
	Bending stress		σ _m = 5.300	N/mm ²				
	Permissible bending stress		$\sigma_{m_{adm}} = \sigma_{m}$	$1 \times K_{2m} \times K_3 \times k_3$	$K_7 \times K_8 = 6.292 \text{ N/}$	/mm²		
	Applied bending stress		$\sigma_{m_{max}} = M$	/ Z = 4.567 N/r	mm ²			
				PASS - Applie	ed bending stres	s within pern	nissible limits	
	Check shear stress							
	Shear stress		τ = 0.670 Ν	l/mm²				
	Permissible shear stress		$\tau_{adm} = \tau \times k$	$K_{2s} \times K_3 \times K_8 = 0$	0.737 N/mm ²			
	Applied shear stress		$\tau_{max} = 3 \times N$	$I / (2 \times b \times h) =$	■ 0.245 N/mm ²			
				PASS - Apj	plied shear stres	s within pern	nissible limits	
	Check bearing stress							
	Compression perpendicular to g	grain (no wane)	$\sigma_{cp1} = 2.20$	0 N/mm²				
	Permissible bearing stress		$\sigma_{c_adm} = \sigma_{cp}$	$_{51} imes K_{2c} imes K_{3} imes K_{3}$	K ₈ = 2.420 N/mm ²	2		
	Applied bearing stress		$\sigma_{c_{max}} = R /$	(b × L _b) = 0.2 4	45 N/mm²			
				PASS - Appli	ied bearing stres	s within pern	nissible limits	
	Check deflection							
	Permissible deflection		$\delta_{adm} = min($	L _{s1} × 0.003, 14	↓ mm) = 8.400 mn	n		
	Bending deflection (based on E	mean)	$\delta_{ ext{bending}}=$ 5.	652 mm				
	Shear deflection		$\delta_{\text{shear}} = 0.24$	49 mm				
	Total deflection		$\delta = \delta_{\text{bending}}$.	+ $\delta_{shear} = 5.901$	mm			
				PASS -	Actual deflectio	n within pern	nissible limits	
	Consider medium term loads							
	Load duration factor		K ₃ = 1.25					
	Maximum bending moment		M = 1.197	kNm				
	Maximum shear force		V = 1.710 k	٨N				
	Maximum support reaction		R = 1.710	kN				
	Maximum deflection		δ = 7.398 n	nm				
	Check bending stress							
	Bending stress		$\sigma_{\text{m}}=\textbf{5.300}$	N/mm ²				
	Permissible bending stress		$\sigma_{m_{adm}} = \sigma_{m}$	$_{1} imes K_{2m} imes K_{3} imes k$	K ₇ × K ₈ = 7.865 N/	/mm²		
	Applied bending stress		$\sigma_{m_max} = M$	/ Z = 6.792 N/r	mm²			
				PASS - Applie	ed bending stres	s within pern	nissible limits	
	Check shear stress							
	Shear stress		τ = 0.670 Ν	l/mm²				
	Permissible shear stress		$\tau_{adm} = \tau \times k$	$K_{2s} \times K_3 \times K_8 = 0$	0.921 N/mm ²			
	Applied shear stress		$\tau_{max} = 3 \times N$	$I / (2 \times b \times h) =$	= 0.364 N/mm ²			
				PASS - Apj	plied shear stres	s within pern	nissible limits	

	Project 17	' Belmont Park,	Job no. 1665			
Suite 101 C1 Peel House, Morden, SM4 581 ston@samstructures.co.uk	Calcs for	TIMBER FLO	OOR JOISTS		Start page no./Re	evision 28
	Calcs by SW	Calcs date 25/04/2022	Checked by	Checked date	Approved by	Approved date

Check bearing stress

Compression perpendicular to grain (no wane)	$\sigma_{cp1} = 2.200 \text{ N/mm}^2$
Permissible bearing stress	$\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = \textbf{3.025} \ N/mm^2$
Applied bearing stress	$\sigma_{c_max} = R / (b \times L_b) = \textbf{0.364} N/mm^2$
	PASS - Applied bearing stress within permissible limits
Check deflection	
Permissible deflection	$\delta_{adm}=min(L_{s1}\times0.003,~14~mm)=\textbf{8.400}~mm$
Bending deflection (based on E _{mean})	$\delta_{\text{bending}} = 7.028 \text{ mm}$

Shear deflection

Total deflection

 $\delta_{\text{shear}} = 0.370 \text{ mm}$

 $\delta = \delta_{\text{bending}} + \delta_{\text{shear}} = \textbf{7.398} \text{ mm}$

PASS - Actual deflection within permissible limits

	Project	7 Belmont Park,	London, SE13	5BJ	Job no. 1665		
Suite 101 C1 Peel House, Morden, SM4 587 ston@symmetructures.co.uk	Calcs for	CONCRETE STR		N	Start page no./Re	evision 29	
	Calcs by SW	Calcs date 25/04/2022	Checked by	Checked date	Approved by	Approved date	

STRIP FOOTING ANALYSIS	AND DESIGN (BS8110-1:1997)	1	Toddo colouistico version 2.0.07
	→ 150 ← 30 150 ↓ 1000 ↓ ↓ ↓ ↓ ↓ ↓ ↓ ↓ ↓ ↓ ↓ ↓ ↓	00→ 150 150 00 98.6 kN/m ²	
Strip footing details Width of strip footing Depth of soil over strip footing	B = 600 mm h _{soil} = 150 mm	Depth of strip footing Density of concrete	h = 1000 mm ρ _{conc} = 23.6 kN/m ³
Load details Load width	b = 300 mm	Load eccentricity	e _P = 0 mm
Soil details Depth of soil over pad footing Allowable bearing pressure	h _{soil} = 150 mm P _{bearing} = 100 kN/m²	Density of soil	ρ_{soil} = 20.0 kN/m ³
Axial loading on strip footing Dead axial load Wind axial load	9 P _G = 39.1 kN/m P _w = 0.0 kN/m	Imposed axial load Total axial load	P _Q = 4.1 kN/m P = 43.2 kN/m
Foundation loads Dead surcharge load Strip footing self weight Total foundation load	F _{Gsur} = 0.000 kN/m ² F _{swt} = 23.600 kN/m ² F = 16.0 kN/m	Imposed surcharge load Soil self weight	$F_{Qsur} = 0.000 \text{ kN/m}^2$ $F_{soil} = 3.000 \text{ kN/m}^2$
Calculate base reaction Total base reaction	T = 59.2 kN/m	Eccentricity of base reaction Base reaction acts	e⊤ = 0 mm within middle third of base
Calculate pad base pressure	S		
Base pressures	q ₁ = 98.600 kN/m ²	q ₂ = 98.600 kN/m ²	
Minimum base pressure	q _{min} = 98.600 kN/m ²	Maximum base pressure	q _{max} = 98.600 kN/m ²
	PASS - Maximu	Im base pressure is less than	allowable bearing pressure

	Project 17	Belmont Park,	London, SE13 5	BJ	Job no. 1665		
Suite 101 C1 Peel House, Morden, SM4 587 stan@exematu.ctures.co.uk	Calcs for	ONCRETE STR	IP FOUNDATIO	N	Start page no./Re	vision 30	
	Calcs by SW	Calcs date 25/04/2022	Checked by	Checked date	Approved by	Approved date	





NEW INPROVED PROFILE

STEEL LINTELS



CAVITY WALL LINTELS

95-110 mm

* A continuous bottom plate added Note: Maximum block dimensions 125mm (95mm cavity)

L1/S 100

Manufactured Length 150mm increments	0600 1200	1350 1500	1650 1800	1950 2100	2250 2400	2550 2700	2850 3000	3150 3600	3750 4050	*4200 4800
Height 'h'	88	88	107	125	150	162	171	200	200	200
Thickness 't'	1.6	2	2	2	2	2.6	2.6	3.2	3.2	3.2
Total UDL(kN) Load ratio (1)	12	16	19	21	23	27	27	27	26	27
Total UDL(kN) Load ratio (2)	10	13	16	17	18	22	20	20	19	22

For 95-110mm cavity wall construction. Standard duty loading condition.



*	А	continuous	bottom	plate	added
		00110110000	0000000	piaco	aaaoo

Note: Maximum block dimensions 125mm (95mm cavity)

L1/HD 100

For 95-110mm cavity wall construction. Heavy duty loading condition.



Manufactured Length 150mm increments	0600 1200	*1350 1500	*1650 2100	*2250 2550	*2700 3000	*3150 3600	*3750 4200		
Height `h'	110	135	163	203	203	203	203		
Thickness 't'	3.2	3.2	3.2	3.2	3.2	3.2	3.2		
Total UDL(kN) Load ratio (1)	30	30	40	40	40	35	33		
Total UDL(kN) Load ratio (2)	22	22	35	35	35	32	28		

* A continuous bottom plate added Note: Maximum block dimensions 125mm (95mm cavity)

Manufactured Length 150mm increments	*0600 1500	*1650 1800	*1950 2100				
Height 'h'	163	163	203				
Thickness 't'	3.2	3.2	3.2				
Total UDL(kN) Load ratio (1)	50	50	55				
Total UDL(kN) Load ratio (2)	45	45	45				

L1/XHD 100

For 95-110mm cavity wall construction. Extra heavy duty loading condition.

