CALCULATIONS

FOR

38 FROGNAL LANE

PRELIMINARY LOAD RUNDOWNS AND RETAINING WALL THICKNESSES.

Rev. P2

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Design Philosophy & Superstructure

- New build RC framed house.
- Building is to be 4-storeys.
- Basement is to be formed using a contiguous piled wall.
- Ground floor slab will be a transfer slab.
- RC frame will be clad with masonry.
- Internal partitions will be timber stud.

Foundation

• All internal columns and lift shaft will be supported upon piles with pile caps.

Stability

• Stability in both directions will be provided by the RC lift shaft.

Disproportionate collapse

• Disproportionate collapse has been considered to building consequence class 1.

British Standard	
Imposed Loads	BS6399
Steelwork	BS5950
Timber	BS5268
Masonry	BS5628
Concrete	BS8110



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Loading Data

Concrete slab load excl. concrete		<u>kN/m²</u>	<u>Dead</u>	Live
Dead:	Finishes	0.10		
	75mm screed	1.65		
	Ceiling & services	0.50		
		Total =	2.25	
<u>Normal</u>	l reinforced concrete	<u>kN/m³</u>	<u>Dead</u>	<u>Live</u>
Dead:	Normal RC	25.00		
		Total =	25.00	
1		la dia second		2 5 0
Live:	Allow 2.50kin/m ² for residential	loading case.		2.50
	(Load includes 1.00kN/m ² partit	tions load)		
<u>Sloped</u>	roof w/o attic	<u>kN/m²</u>	Dead	<u>Live</u>
Dead:	Tiles	0.75		
	Battens	0.05		
	Rafter	0.20		
	Insulation	0.05		
		Total / cos30 =	1.25	
Live:	Allow 0.75kN/m²			0.75



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<u>Flat Roof</u>		<u>kN/m²</u>	<u>Dead</u>	<u>Live</u>
Dead:	Finishes	0.70		
	Ceiling & services	0.20		
	Joists	0.20		
		Total =	1.10	
Live:	Allow 0.75kN/m²			0.75
<u>Ext. cav</u>	vity wall w/ 100mm blockwork	<u>kN/m²</u>	Dead	<u>Live</u>
Dead:	103mm brickwork	2.30		
	100mm blockwork	2.00		
	Plaster	0.15		
		Total =	4.45	

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TEKLA MODEL

AREA LOADS UPON SUPPORTS

	Column Loading							
	Wall Load (kN/m)	Dead (kN/m ²)	Live (kN/m²)					
Roof		1.8	0.8					
2nd	0.0	7.3	2.5					
1st	6.4	7.3	2.5					
Grd		12.3	2.5					
Base.	29.4	7.3	2.5					

	Area	Perimeter Wall Length (m)	Perimeter Wall Load (kN)	Dead Load	Live Load	Total (kN)
Roof	183.5	0.0	0.0	330.3	137.6	467.9
2nd	183.5	41.0	328.9	1330.4	458.8	2118.0
1st	183.5	61.9	736.9	1330.4	458.8	2526.0
Grd	247.1	61.9	863.5	3027.0	617.8	4508.2
Base	247.1	70.9	2084.5	1791.5	617.8	4493.7
		4013.7	7809.5	2290.6	14113.8	

Perimeter Wall Load (kN)

0.0

29.4

28.5

Zone

А

В

С

Area (m²)

42.7

6.0

11.0

Perimeter Wall Length (m)

0.0

4.6

4.5

2nd floor wall	
33.3	

D	12.6	4.8	30.9	205.4	72.5		308.7	
E	8.7	5.0	31.9	141.8	50.0	33.3	257.0	
F	15.7	5.6	35.9	255.9	90.3	33.3	415.4	
G	7.4	4.6	29.5	120.6	42.6	33.3	226.0	
Н	7.7	3.8	24.3	125.5	44.3	33.3	227.3	Water
J	9.0	5.7	36.2	146.7	51.8		234.6	Pool Volun
К	10.5	5.1	32.6	171.2	60.4	33.3	297.4	Pool Load
L	11.5	6.3	40.3	187.5	66.1	33.3	327.2	4No. Oute
М	15.5	7.1	45.3	252.7	89.1	33.3	420.4	2No. Inner

Support Load, Dead (kN)

696.0

97.8

179.3

Support Load, Live (kN)

245.5

34.5

63.3

Misc. (kN)

33.3

Total (kN)

941.5

195.0

271.1

Water	10.0 kN/m ³
Pool Volume	86.7 m ³
Pool Load	866.6 kN
4No. Outer Piles	108.3 kN
2No. Inner Piles	216.7 kN

CONTIGUOUS PILED WALL: D=200kN/m, L=30kN/m.

PILE LOADS (NOT TO SCALE)

Tekla Tedds	Project		Job no.			
Train & Kemp		38 Frognal Lane				
10 Kennington Park Place London	Calcs for	14.67	14.67m Prop			Revision 2
SE11 4AS	Calcs by C RH	calcs date 12/05/2021	Checked by	Checked date	Approved by	Approved dat
				Imposed	d × 1.60	
Analysis results						
Maximum moment		M _{max} = 34. 1	l kNm	M _{min} = 0	kNm	
Maximum shear		V _{max} = 9.3	٨N	V _{min} = -9).4 kN	
Deflection		δ _{max} = 23.4	mm	δ _{min} = 0	mm	
Maximum reaction at support	A	R _{A_max} = 9.3	3 kN	R _{A_min} =	9.3 kN	
Unfactored dead load reaction	n at support A	$R_{A_Dead} = 6$. 6 kN			
Maximum reaction at support	В	R _{B_max} = 9.4	4 kN	$R_{B_{min}} =$	9.4 kN	
Unfactored dead load reaction	n at support B	$R_{B_{Dead}} = 6$.7 kN			
Section details						
Section type		UC 254x25	54x73 (BS4-1)			
Steel grade		S275				
From table 9: Design streng	ith p _y	(-))				
Thickness of element		max(1, t) =	14.2 mm			
Medulue of electicity		$p_y = 275 \text{ N/}$	mm^2			
Modulus of elasticity	ý	∟ - 205000	/ N/11111-			
	<u>↓</u> <u>↓</u> <u>−</u>			-		
	-14.2	+	8.6	_		
	± <u>+</u> ∟					
	◀	254.6-		1		
Lateral restraint						
Lateral restraint		Span 1 has	s lateral restrain	at supports only	1	
Lateral restraint Effective length factors		Span 1 has	s lateral restrain	at supports only	,	
Lateral restraint Effective length factors Effective length factor in majo	r axis	Span 1 has K _x = 1.00	alateral restrain	at supports only	,	
Effective length factors Effective length factor in majo Effective length factor in mino	r axis r axis	Span 1 has K _x = 1.00 K _y = 1.00	s lateral restrain	at supports only	/	
Lateral restraint Effective length factors Effective length factor in majo Effective length factor in mino Effective length factor for late	r axis r axis ral-torsional buckling	Span 1 has K _x = 1.00 K _y = 1.00 K _{LT.A} = 1.00	ateral restrain	at supports only	,	
Lateral restraint Effective length factors Effective length factor in majo Effective length factor in mino Effective length factor for later	ır axis ır axis ral-torsional bucklinç	Span 1 has K _x = 1.00 K _y = 1.00 K _{LT.A} = 1.00 K _{LT.B} = 1.00	s lateral restraint	at supports only	,	
Lateral restraint Effective length factors Effective length factor in majo Effective length factor in mino Effective length factor for later Classification of cross section	or axis or axis ral-torsional buckling ions - Section 3.5	Span 1 has $K_x = 1.00$ $K_y = 1.00$ $K_{LT.A} = 1.00$ $K_{LT.B} = 1.00$ $K_{LT.B} = 1.00$	s lateral restraint	at supports only	,	
Lateral restraint Effective length factors Effective length factor in majo Effective length factor in mino Effective length factor for later Classification of cross sect	or axis or axis ral-torsional buckling ions - Section 3.5	Span 1 has $K_x = 1.00$ $K_y = 1.00$ $K_{LT.A} = 1.00$ $K_{LT.B} = 1.00$ $\epsilon = \sqrt{275}$ N	a lateral restraint)) l/mm² / py] = 1.0	at supports only	,	
Lateral restraint Effective length factors Effective length factor in majo Effective length factor in mino Effective length factor for later Classification of cross section Internal compression parts Depth of asstign	or axis or axis ral-torsional buckling ions - Section 3.5 - Table 11	Span 1 has $K_x = 1.00$ $K_y = 1.00$ $K_{LT.A} = 1.00$ $K_{LT.B} = 1.00$ $\epsilon = \sqrt{275}$ N	s lateral restraint)) l/mm² / py] = 1.0	at supports only	,	
Lateral restraint Effective length factors Effective length factor in majo Effective length factor in mino Effective length factor for later Classification of cross section Internal compression parts Depth of section	or axis or axis ral-torsional buckling ions - Section 3.5 - Table 11	Span 1 has $K_x = 1.00$ $K_y = 1.00$ $K_{LT.A} = 1.00$ $K_{LT.B} = 1.00$ $\epsilon = \sqrt{275}$ N d = 200.3 m	s lateral restraint) $I/mm^2 / p_y] = 1.0$ nm	at supports only	nlactic	
Lateral restraint Effective length factors Effective length factor in majo Effective length factor in mino Effective length factor for later Classification of cross secti Internal compression parts Depth of section	or axis or axis ral-torsional buckling ions - Section 3.5 - Table 11	Span 1 has $K_x = 1.00$ $K_y = 1.00$ $K_{LT,A} = 1.00$ $K_{LT,B} = 1.00$ $\epsilon = \sqrt{275}$ M d = 200.3 m d / t = 23.3	s lateral restraint) $l/mm^2 / p_y] = 1.0$ nm $\times \varepsilon \le 80 \times \varepsilon$	at supports only 0 Class 1	, plastic	
Lateral restraint Effective length factors Effective length factor in majo Effective length factor in mino Effective length factor for later Classification of cross section Internal compression parts Depth of section Outstand flanges - Table 11	or axis or axis ral-torsional buckling ions - Section 3.5 - Table 11	Span 1 has $K_x = 1.00$ $K_y = 1.00$ $K_{LT.A} = 1.00$ $K_{LT.B} = 1.00$ $\epsilon = \sqrt{275}$ M d = 200.3 m d / t = 23.3	a lateral restraint $I/mm^2 / p_y] = 1.0$ mm $\times \epsilon <= 80 \times \epsilon$	at supports only 0 Class 1	, plastic	

Tekla Tedds	Project Job no.							
Train & Kemp		38 Frog		20080				
10 Kennington Park Place London	Calcs for	14.67	m Prop		Start page no./Revision 3			
SE11 4AS	Calcs by RH	Calcs date 12/05/2021	Checked by	Checked date	Approved by	Approved date		
		b/T-00	· · · · - · · · ·	Close 1	plaatia			
		D/T - 9.0 3	× ε <- 9 × ε	Class	Section is c	lass 1 nlastic		
					Section 13 0	1033 1 piasuc		
Shear capacity - Section 4.2.3				$\rangle\rangle = 0.4$ kM				
Design shear lorce		$F_v = \max(a)$	os(v _{max}), abs(v _m	nin)) – 9.4 KIN				
		u/1 < 70 ×	ε Wob doos n	ot road to be c	hockod for s	hoar huckling		
Shear area		$\Delta_{\rm H} = t \times D =$	2185 mm ²	or need to be c		ilear buckning		
Design shear resistance		$P_{v} = 0.6 \times r$	Δ	N				
Design shear resistance		Π = 0.0 × μ ΡΔς	S - Design she	n ar resistance ex	rceeds desin	in shear force		
		140	e Deelgii enee		loccus acong			
Moment capacity - Section 4.2	2.5	M - max(al		M	Nm			
Moment especity low shear of	1252	M = min(n)	$S(IVI_{s1}_{max}), abs(IVI_{s1}_{max})$	IVIs1_min)) - 34.1 I	Im			
	4.2.3.2		$r \times \mathbf{S}_{xx}, 1.2 \times \mathbf{p}_y \times \mathbf{r}_y$	$(Z_{xx}) - ZIZ.0 KIN$				
Effective length for lateral-tor	sional buckling	- Section 4.3.5						
Effective length for lateral torsio	nal buckling	$L_E = 1.0 \times L_E$	_{-s1} = 14670 mm					
Slenderness ratio		$\lambda = L_E / r_{yy}$ =	= 226.434					
Equivalent slenderness - Sec	tion 4.3.6.7							
Buckling parameter		u = 0.849						
Torsional index		x = 17.259		_				
Slenderness factor		v = 1 / [1 +	$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.568$					
Ratio - cl.4.3.6.9		βw = 1.000	βw = 1.000					
Equivalent slenderness - cl.4.3.	6.7	$\lambda_{LT} = \mathbf{u} \times \mathbf{v}$	$\lambda_{LT} = \mathbf{u} \times \mathbf{v} \times \lambda \times \sqrt{[\beta w]} = 109.159$					
Limiting slenderness - Annex B	.2.2	$\lambda_{L0} = 0.4 \times$	$\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = 34.310$					
		$\lambda_{LT} > \lambda_{L0} - \lambda_{L0}$	Allowance shou	uld be made for	r lateral-torsi	onal buckling		
Bending strength - Section 4.	3.6.5							
Robertson constant		α _{LT} = 7.0						
Perry factor		η∟⊤ = max(α	хlt × (λlt - λlo) /	1000, 0) = 0.52 4	4			
Euler stress		$p_E = \pi^2 \times E$	$p_{E} = \pi^{2} \times E / \lambda_{L} \tau^{2} = 169.8 \text{ N/mm}^{2}$					
		$\phi_{LT} = (p_y + ($	$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 266.9 \text{ N/mm}^2$					
Bending strength - Annex B.2.1		$p_b = p_E \times p_y$	/ (ф _{LT} + (ф _{LT} ² - p _E	≡ × p _y) ^{0.5}) = 110. 5	3 N/mm ²			
Equivalent uniform moment fa	actor - Section	4.3.6.6						
Moment at quarter point of segr	nent	M ₂ = 25.3 k	Nm					
Moment at centre-line of segme	ent	M ₃ = 34.1 k	Nm					
Moment at three quarter point o	f segment	M ₄ = 25.7 k	Nm					
Maximum moment in segment		M _{abs} = 34.1	kNm					
Maximum moment governing bu	uckling resistanc	$He M_{LT} = M_{abs}$	= 34.1 kNm					
Equivalent uniform moment fact	for for lateral-tor	sional buckling						
		m _{L⊺} = max(t	$0.2 + (0.15 \times M_2)$	$+ 0.5 \times M_3 + 0.1$	$5 \times \text{IVI4}$ / IVIabs	, 0.44) = 0.924		
Buckling resistance moment	- Section 4.3.6.4	4						
Buckling resistance moment		$M_b = p_b \times S$	_{xx} = 109.4 kNm					
		$M_b / m_{LT} = 2$	118.4 kNm					
		PASS - Bucklin	ng resistance m	ioment exceed	s aesign ben	aing moment		
Check vertical deflection - Se	ction 2.5.2							
Consider deflection due to dead	I and imposed lo	bads						
Limiting deflection		$\delta_{\text{lim}} = L_{s1} / 3$	60 = 40.75 mm					

Tekla Tedds	Project 38 Frognal Lane				Job no. 20080	
10 Kennington Park Place London	Calcs for 14.67m Prop			Start page no./Revision 4		
SE11 4AS	Calcs by RH	Calcs date 12/05/2021	Checked by	Checked date	Approved by	Approved date

Maximum deflection span 1

 $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = 23.44 \text{ mm}$

PASS - Maximum deflection does not exceed deflection limit

Tekla Tedds	Project	38 Fro	38 Frognal Lane				Job no. 20080		
TAK Structures	Calcs for		0			Start page no /Revision			
10 Kennington Park Place		14.67	14.67m Prop			1			
London	Calcs by	Calcs date	Checked by	Checked date	Approve	ad by	Approved date		
SE11 4AS	RH	13/05/2021	Checked by	Checked date	Appiove	su by	Approved date		
		10/00/2021							
STEEL MEMBER DESIGN (BS	<u>5950)</u>								
In accordance with BS5950-1:	2000 incorporat	ting Corrigen	dum No.1		TERR				
Continu dataila					TEDDS	5 calculatio	on version 3.0.07		
Section details		110 05 4-0	5470 (DO4 4)						
Section type		UC 254x2	54x73 (BS4-1)						
Steel grade		S355							
From table 9: Design strength	ру								
Thickness of element		max(T, t) =	= 14.2 mm						
Design strength		p _y = 355 N	l/mm²						
Modulus of elasticity		E = 20500	0 N/mm ²						
	14.2								
	▲ ↓								
	54.1-		-8.6						
	5								
	5								
	<u>↓</u>		_						
	↓ ↓								
	1	054.0		. 1					
	•	254.6		 ▶					
Lateral restraint									
Distance between maior axis re	straints	L _× = 16500) mm						
Distance between minor axis re	straints	$L_{v} = 16500$) mm						
	otrainto	Ly 10000							
Effective length factors									
Effective length factor in major a	axis	K _x = 1.00							
Effective length factor in minor a	axis	K _y = 1.00							
Effective length factor for lateral	l-torsional bucklir	ng K _{LT} = 1.00							
Classification of cross section	ns - Section 3.5								
		ε = √[275	$N/mm^2 / p_v = 0$.88					
		- -							
Dopth of section	adie 11	d - 200 2	mm						
		u – ∠∪∪.3							
Stress ratios		r1 = min(F	$_{c}$ / (d × t × p _{yw}),	1) = 0.2					
		$r2 = F_{c} / (A)$	A × p _{yw}) = 0.037	7					
		d / t = 26.5	5 × ε <= max(80	0 × ε / (1 + r1),	$40 imes \epsilon$)	Class 1	l plastic		
Outstand flanges - Table 11									
Width of section		b = B / 2 =	127.3 mm						
		b / T = 10	2 x ε <= 15 x e	Cla	ss 3 semi-o	omnact			
		571-10.		. 01a Ca	ction is als		mi_compact		
				Se	cuon is cla	ISS J SE	mi-compact		

Tekla Tedds	Project	38 Frog	nal Lane		Job no. 20	080			
TAK Structures	Calcs for				Start page no./Revision				
10 Kennington Park Place		14.67	m Prop			2			
SE11 4AS	Calcs by RH	Calcs date 13/05/2021	Checked by	Checked date	Approved by	Approved date			
Shear capacity - Section 4.2.3									
Design snear force		$F_{y,v} = 9.4 \text{ K}$	N						
		u/1<70×	ε Web doos	not nood to be c	hockod for sl	hoar huckling			
Shear area		$A_{\nu} = t \times D =$: 2185 mm ²	not need to be c	necked for Si	ieai backiiig			
Design shear resistance		$P_{\rm W} = 0.6 \times$	$n_v \propto \Delta_v = 465$	5 kN					
Design shear resistance		PAS	S - Desian sh	ear resistance ex	xceeds desia	n shear force			
Shoar canacity Soction 4.2.3									
Design shear force		F _{x,v} = 0 kN							
Moment capacity - Section 4.2	5	M = 34 1 kl	Nm						
		IVI – 34. I KI	NIII						
Effective plastic modulus - Se	ction 3.5.6	a 1 0							
Limiting value for class 2 compa	ct flange	$\beta_{2f} = 10 \times \varepsilon$	= 8.801						
Limiting value for class 3 semi-c	Limiting value for class 3 semi-compact flange		$\beta_{3f} = 15 \times \epsilon = 13.202$						
Limiting value for class 2 compact web		$\beta_{2w} = \max(\gamma)$	$\beta_{2w} = \max(100 \times \varepsilon / (1 + 1.5 \times r1), 40 \times \varepsilon) = 67.716$						
Limiting value for class 3 semi-c Effective plastic modulus - cl.3.5	ompact web 5.6.2	β _{3w} = max(1	120 × ε / (1 + 2	2 × r2), 40 × ε) = 9	8.345				
$S_{eff} = min(Z_{xx} + (S_{xx} - Z_{xx}) \times n)$	nin([((β _{3w} / (d / t))² - 1) / ((β _{3w} / β ₂	_{2w})² - 1)], [(β _{3f} /	(b / T) - 1) / (β _{3f} /	β _{2f} - 1)]), S _{xx}) =	= 986918 mm ³			
Moment capacity low shear - cl.4	4.2.5.2	$M_c = min(p)$	$_{V} \times S_{eff}, 1.2 \times p$	_y × Z _{xx}) = 350.4 kN	Im				
Effective length for lateral-tors	sional buckling	g - Section 4.3.5	5						
Effective length for lateral torsion	nal buckling	$L_E = 1.0 \times I$	_{-y} = 16500 mm	1					
Slenderness ratio		$\lambda = L_E / r_{yy}$	= 254.680						
Equivalent slenderness - Sect	ion 4.3.6.7								
Buckling parameter		u = 0.849							
Torsional index		x = 17.259							
Slenderness factor		v = 1 / [1 +	$0.05 \times (\lambda / x)^2]$	^{0.25} = 0.539					
Ratio - cl.4.3.6.9		$\beta_{W} = S_{eff} / S_{eff}$	S _{xx} = 0.995						
Equivalent slenderness - cl.4.3.6	3.7	$\lambda_{\text{LT}} = u \times v$	× λ × √[βw] = 1	16.105					
Limiting slenderness - Annex B.	2.2	λ_{L0} = 0.4 \times	$(\pi^2 \times E / p_y)^{0.5}$	= 30.198					
		$\lambda_{LT} > \lambda_{L0} - \lambda_{L0}$	Allowance sh	ould be made for	r lateral-torsi	onal buckling			
Bending strength - Section 4.3	3.6.5								
Robertson constant		α _{LT} = 7.0							
Perry factor		η∟⊤ = max(e	α _{LT} × (λ _{LT} - λ _{L0})	/ 1000, 0) = 0.60	1				
Euler stress		$p_E = \pi^2 \times E$	/ λ _{LT} ² = 150.1	N/mm²					
		φ _{LT} = (p _y + ((η _{LT} + 1) × p _E) /	2 = 297.7 N/mm ²	2				
Bending strength - Annex B.2.1		$p_b = p_E \times p_b$, / (φ _{LT} + (φ _{LT} ² -	$p_E \times p_y)^{0.5}$) = 109.	7 N/mm ²				
Equivalent uniform moment fate Equivalent uniform moment fact	ictor - Section or for LTB	4.3.6.6 m _{LT} = 1.000)						
Buckling resistance moment -	Section 4.3.6	.4							
Buckling resistance moment		$M_b = p_b \times S$	_{eff} = 108.3 kNr	n					
		$M_b / m_{LT} = r$	108.3 kNm						
		PASS - Buckli	ng resistance	moment exceed	s design ben	ding moment			

Tekla Tedds	Project	Project Job						
TAK Structures	Calcs for	001109			Start page no /Revision			
10 Kennington Park Place		14.67	m Prop		otan page no.//	3		
London SE11 4AS	Calcs by RH	Calcs date 13/05/2021	Checked by	Checked date	Approved by	Approved date		
Compression members - Sect Design compression force	ion 4.7	F _c = 122.2	kN	<u> </u>				
Effective length for major (x-x) axis buckling	- Section 4.7.3	5					
Effective length for buckling		$L_{Ex} = L_x \times k$	K _x = 16500 mm					
Slenderness ratio - cl.4.7.2 $\lambda_x = L_{Ex} / r_{xx} = 149.064$								
Compressive strength - Section	on 4.7.5							
Limiting slenderness		$\lambda_0 = 0.2 \times ($	$(\pi^2 \times E / p_y)^{0.5} =$	15.099				
Strut curve - Table 23		b						
Robertson constant		α _x = 3.5						
Perry factor		$\eta_x = \alpha_x \times (\lambda$.x - λ₀) / 1000 =	0.469				
Euler stress		$p_{Ex} = \pi^2 \times E$	Ε / λ _x ² = 91.1 N/	mm ²				
		$\phi_{x} = (p_{y} + (1$	η _x + 1) × p _{Ex}) / 2	2 = 244.4 N/mm ²				
Compressive strength - Annex (C.1	$p_{cx} = p_{Ex} \times$	p _y / (φ _x + (φ _x ² - p	$p_{Ex} \times p_y)^{0.5} = 78.9$	N/mm ²			
Compression resistance - Sec	ction 4.7.4							
Compression resistance - cl.4.7	Compression resistance - cl.4.7.4							
		PASS - Com	pression resis	stance exceeds	design comp	ression force		
Effective length for minor (y-y) axis buckling	- Section 4.7.3	, ,,					
Effective length for buckling		$L_{Ey} = L_y \times k$	K _y = 16500 mm					
Slenderness ratio - cl.4.7.2		$\lambda_y = L_{Ey} / r_y$	y = 254.680					
Compressive strength - Section	on 4.7.5							
Limiting slenderness		$\lambda_0 = 0.2 \times ($	$\lambda_0 = 0.2 \times (\pi^2 \times E / p_y)^{0.5} = 15.099$					
Strut curve - Table 23		C						
Robertson constant		$\alpha_y = 5.5$						
		$\eta_y = \alpha_y \times (\lambda_y - \lambda_0) / 1000 = 1.318$						
		$p_{Ey} = \pi \times L$ $\phi_v = (p_v + t)$	$p_{Ey} = \pi^2 \times E / \lambda_y^2 = 31.2 \text{ N/mm}^2$ $\phi = (n_1 + (n_2 + 1) \times n_2) / 2 = 213.6 \text{ N/mm}^2$					
Compressive strength - Annex (C.1	$\varphi = (Py + $	$\phi_y = (p_y + (\eta_y + 1) \times p_{Ey})/2 = 213.0 \text{ N/mm}^2$ $p_y = p_{Ey} \times p_y / (\phi_y + (\phi_y^2 - p_{Ey} \times p_y^{0.5}) = 27.7 \text{ N/mm}^2$					
Compression resistance. So	ation 474		F)'(Y) (Y) F	Ly ··· Py/ /···				
Compression resistance - cl 4.7	1 A	P= A × n	= 258 kN					
		PASS - Com	pression resis	stance exceeds	design comp	ression force		
Compression members with r	noments - Sect	tion 483			0 /			
Comb.compression & bending of	check - cl.4.8.3.2	$\frac{1011 + 10.0}{2} = F_c / (A \times p_v)$) + M / Mc = 0.1	34				
		PASS -	Combined be	nding and comp	pression chec	k is satisfied		
Member buckling resistance -	Section 4.8.3.3	3						
Max major axis moment govern	ing M₀	M _{LT} = M _x =	34.10 kNm					
Equivalent uniform moment fact	or for major axis	s flexural bucklir	ng					
		m _x = 1.000						
		m _y = 1.000						
Buckling resistance checks - cl.	4.8.3.3.2	F _c / P _{cx} + m	$M_x \times M / M_c \times (1)$	+ 0.5 × F _c / P _{cx}) =	0.272			
		F _c / P _{cy} + m	$M_{LT} \times M_{LT} / M_{b} =$	0.789	tomor - h t			
PASS - Member buckling resistance checks are satis								

Load combination 1

Support B

Support A

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

Tekla: ledds	Project				Job no.	
Train & Kemp		38 Frog	inal Lane		2	0800
10 Kennington Park Place London	Calcs for	Calcs for 16.47m Prop			Start page no./I	Revision 2
SE11 4AS	Calcs by RH	Calcs date 13/05/2021	Checked by	Checked date	Approved by	Approved da
				Imposed	d × 1.60	
Analysis results						
Maximum moment		M _{max} = 61.1	l kNm	M _{min} = 0	kNm	
Maximum shear		V _{max} = 14.6	i kN	V _{min} = -1	4.4 kN	
Deflection		δ _{max} = 34.5	mm	δ_{min} = 0	mm	
Maximum reaction at support A	A	R _{A_max} = 14	. 6 kN	R _{A_min} =	14.6 kN	
Unfactored dead load reaction	at support A	R _{A_Dead} = 1	0.5 kN			
Maximum reaction at support I	В	R _{B_max} = 14	. 4 kN	$R_{B_{min}} =$	14.4 kN	
Unfactored dead load reaction	at support B	$R_{B_{Dead}} = 1$	0.3 kN			
Section details						
Section type		UC 254x25	54x107 (BS4-1)			
Steel grade	-	S275				
From table 9: Design strengt	th p _y					
Thickness of element		max(T, t) =	20.5 mm			
Design strength	$p_y = 265 N/$	'mm²				
Modulus of elasticity	Q	E = 205000	J N/MM ²			
	4-20					
	20.5	-> 4-	-12.8			
	266.7		.12.8			
	266.7	+	.12.8			
	266.7	258.8-	.12.8			
Lateral restraint	266.7	258.8-	.12.8			
Lateral restraint	▲	258.8	.12.8 ▶	at supports only	, ,	
Lateral restraint	→ → 266.7 → 1 → 20.5	258.8Span 1 has	12.8 →	at supports only	,	
Lateral restraint Effective length factors Effective length factor in major	- ^{500.5} ↓ 1 ↓ 1 ↓ 1 ↓ 1 ↓ 1 ↓ 1 ↓ 1 ↓ 1	258.8- Span 1 has Kx = 1.00	.12.8 ► lateral restraint	at supports only	,	
Lateral restraint Effective length factors Effective length factor in major Effective length factor in minor	r axis r axis	258.8- Span 1 has K _x = 1.00 K _y = 1.00	12.8 ▶	at supports only	,	
Lateral restraint Effective length factors Effective length factor in major Effective length factor in minor Effective length factor for later	r axis r axis al-torsional bucklin	258.8- Span 1 has K _x = 1.00 K _y = 1.00 g K _{LT.A} = 1.00	.12.8 →	at supports only	,	
Lateral restraint Effective length factors Effective length factor in major Effective length factor in minor Effective length factor for later	r axis r axis al-torsional buckling	258.8- Span 1 has K _x = 1.00 K _y = 1.00 K _{LT.A} = 1.00 K _{LT.B} = 1.00	12.8 s lateral restraint	at supports only	,	
Lateral restraint Effective length factors Effective length factor in major Effective length factor in minor Effective length factor for later Classification of cross sectio	r axis r axis al-torsional buckling ons - Section 3.5	258.8- Span 1 has K _x = 1.00 K _y = 1.00 K _{LT.B} = 1.00	s lateral restraint	at supports only	,	
Lateral restraint Effective length factors Effective length factor in major Effective length factor in minor Effective length factor for later Classification of cross section	r axis r axis al-torsional buckling ons - Section 3.5	258.8- Span 1 has K _x = 1.00 K _y = 1.00 g K _{LT.A} = 1.00 K _{LT.B} = 1.00 ε = √[275 N	.12.8 s lateral restraint)) l/mm² / py] = 1.0	at supports only	,	
Lateral restraint Effective length factors Effective length factor in major Effective length factor in minor Effective length factor for later Classification of cross section Internal compression parts -	r axis r axis al-torsional buckling ons - Section 3.5	258.8- Span 1 has K _x = 1.00 K _y = 1.00 K _{LT.B} = 1.00 ε = √[275 N	.12.8 s lateral restraint)) l/mm² / py] = 1.0	at supports only	,	
Lateral restraint Effective length factors Effective length factor in major Effective length factor in minor Effective length factor for later Classification of cross section Internal compression parts - Depth of section	r axis al-torsional buckling ons - Section 3.5	258.8- Span 1 has K _x = 1.00 K _y = 1.00 g K _{LT.A} = 1.00 K _{LT.B} = 1.00 ε = √[275 N d = 200.3 r	.12.8 s lateral restraint)) //mm² / py] = 1.0 nm	at supports only	,	
Lateral restraint Effective length factors Effective length factor in major Effective length factor in minor Effective length factor for later Classification of cross section Internal compression parts - Depth of section	r axis r axis al-torsional buckling ons - Section 3.5	258.8- Span 1 has K _x = 1.00 K _y = 1.00 g K _{LT.A} = 1.00 K _{LT.B} = 1.00 ε = √[275 N d = 200.3 r d / t = 15.4	s lateral restraint J J J J J J J J	at supports only 2 Class 1	, plastic	
Lateral restraint Effective length factors Effective length factor in major Effective length factor in minor Effective length factor for later Classification of cross section Internal compression parts - Depth of section Outstand flanges - Table 11	r axis r axis al-torsional buckline ons - Section 3.5	$5pan 1 has$ $K_x = 1.00$ $K_y = 1.00$ $K_{LT.B} = 1.00$ $\varepsilon = \sqrt{275 N}$ $d = 200.3 r$ $d / t = 15.4$	s lateral restraint J J J J J J J J	at supports only 2 Class 1	, plastic	

Tekla Tedds	Project				Job no.			
Train & Kemp		38 Frog	nal Lane		20	080		
10 Kennington Park Place London	Calcs for	16.47	m Prop		Start page no./F	Revision 3		
SE11 4AS	Calcs by RH	Calcs date 13/05/2021	Checked by	Checked date	Approved by	Approved date		
			-		·			
		b / T = 6.2 :	×ε <= 9 ×ε	Class 1	plastic			
					Section is o	class 1 plastic		
Shear capacity - Section 4.2.3								
Design shear force		F _v = max(a	bs(V _{max}), abs(V _m	_{in})) = 14.6 kN				
		d / t < 70 ×	3					
			Web does n	ot need to be c	hecked for s	hear buckling		
Shear area		$A_v = t \times D =$	3414 mm ²					
Design shear resistance		$P_v = 0.6 \times p$	o _y × A _v = 542.8 k	N				
		PAS	S - Design shea	ar resistance ex	xceeds desig	n shear force		
Moment capacity - Section 4.2	2.5							
Design bending moment		M = max(at	os(M _{s1 max}), abs(M _{s1 min})) = 61.1 ∣	kNm			
Moment capacity low shear - cl.	4.2.5.2	$M_c = min(p_1)$	$_{1} \times S_{xx}$, 1.2 × p _v ×	: Z _{xx}) = 393.4 kN	lm			
Effective length for lateral tor	sional buckling	Section 4.3 E		,				
Effective length for lateral torsio		$J = -10 \times 1$, - 16170 mm					
	nai buckiing	$L_E = 1.0 \times L$	_s1 - 16470 mm					
Sienderness ratio		$\lambda = L_E / r_{yy}$	= 249.824					
Equivalent slenderness - Sect	tion 4.3.6.7							
Buckling parameter		u = 0.848						
Torsional index		x = 12.393						
Slenderness factor		v = 1 / [1 +	$0.05 \times (\lambda / x)^2]^{0.2}$	⁵ = 0.465				
Ratio - cl.4.3.6.9		βw = 1.000						
Equivalent slenderness - cl.4.3.	6.7	$\lambda_{LT} = \mathbf{u} \times \mathbf{v}$	$\times \lambda \times \sqrt{[\beta w]} = 98.$	593				
Limiting slenderness - Annex B.	2.2	λ_{L0} = 0.4 \times	$(\pi^2 \times E / p_y)^{0.5} = 3$	34.951				
		$\lambda_{LT} > \lambda_{L0} - \lambda_{L0}$	Allowance shou	ıld be made foi	r lateral-torsi	onal buckling		
Bending strength - Section 4.	3.6.5							
Robertson constant		αιτ = 7.0						
Perry factor		$m_{\rm T} = max(a$	$m_{LT} = max(m_{LT} \times (\lambda_{LT} - \lambda_{LO}) / 1000 \ 0) = 0.445$					
Fuler stress		$p_{\rm E} = \pi^2 \times {\rm F}$	$n_{\rm E} = \pi^2 \times E / \lambda_{\rm F}^2 = 208.1 \text{N/mm}^2$					
		$\phi_{\perp} = (p_{\perp} + (p_{\perp}))$	$p_E = \pi^- \times E / \pi_E + 200.1 \text{ N/mm}^2$					
Bending strength - Anney B 2 1		φ _{⊑1} = (p _y : ($\varphi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 202.3 \text{ N/IIIII^-}$ $p_y = p_{T} \times p_y / (d_{y,T} + (d_{y,T}^2 + p_{T} \times p_{T})^{0.5}) = 125.2 \text{ N/mm}^2$					
		$P_{D} = P_{F} \times P_{A}$, (ψι · (ψι - Ρε	-^ Py/ / - 1 20.	≖ IX/IIII			
Equivalent uniform moment fa	actor - Section	4.3.6.6	N 1					
Moment at quarter point of segr	nent	IVI₂ = 46.6 k	inm Ning					
Moment at three quester point a	ni f sogmost	IVI3 = 61.1 K	inm Nm					
Maximum moment in accord	rsegment	IVI4 = 43.9 K	kNm					
Maximum moment governing b	ickling resistanc	$M_{\rm T} = M_{\odot}$	= 61 1 kNm					
Fauivalent uniform moment fact	or for lateral-tor	sional huckling						
		mi⊤ = max/().2 + (0 15 × M₂ ·	+ 0.5 × M₃ + 0 1	$5 \times M_4$) / Maha	().44) = 0 927		
	0			0.0 A 100 ' 0.1	• · · · · · · · · · · · · · · · · abs	,, 		
Buckling resistance moment	- Section 4.3.6.	4	405 0 1 1 1					
Buckling resistance moment		$M_b = p_b \times S$	xx = 185.8 kNm					
		$IVI_b / M_{LT} = 2$		omort associ	o docine har	ding manand		
		FAJJ - BUCKIII	ig resistance m	ioment exceed	s aesign ber	iung moment		
Check vertical deflection - Se	ction 2.5.2	_						
Consider deflection due to dead	and imposed lo	bads						
Limiting deflection		$\delta_{\text{lim}} = L_{s1} / 3$	60 = 45.75 mm					

Tekla , Tedds Train & Kemp	Project	Job no. 20080				
10 Kennington Park Place London	Calcs for 16.47m Prop				Start page no./Revision 4	
SE11 4AS	Calcs by RH	Calcs date 13/05/2021	Checked by	Checked date	Approved by	Approved date

Maximum deflection span 1

 $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = 34.548 \text{ mm}$

PASS - Maximum deflection does not exceed deflection limit

Tekla Tedds	Project	38 Fro		Job no. 20080		
TAK Structures	Calcs for		_		Start page no.	/Revision
10 Kennington Park Place	1	16.4	7m Prop			1
SE11 4AS	Calcs by RH	Calcs date 13/05/2021	Checked by	Checked date	Approved by	Approved date
STEEL MEMBER DESIGN (BS5 In accordance with BS5950-1:2	<u>950)</u> 000 incorporat	ing Corrigen	dum No.1			
Section details					TEDDS calcu	lation version 3.0.0
Section type		110 25422	54×107 (BSA 4	0		
Steel grade		S355	34X107 (B34-1	")		
From table 9: Design strength r	0v	0000				
Thickness of element	Sy .	max(T, t) =	= 20.5 mm			
Design strength		p _v = 345 N	l/mm ²			
Modulus of elasticity		E = 20500	0 N/mm ²			
-	20.5					
-	★ [¥]					
	▲					
1		-	-12.8			
5						
	-20.5					
	↓ <u>+</u>					
-	▲					
		258.8		→		
Lateral restraint						
Distance between major axis rest	raints	L _x = 1650) mm			
Distance between minor axis rest	raints	L _y = 1650) mm			
Effective length factors						
Effective length factor in major ax	kis	K _x = 1.00				
Effective length factor in minor ax	(is	K _y = 1.00				
Effective length factor for lateral-t	orsional bucklin	ig K _{LT} = 1.00				
Classification of cross sections	s - Section 3.5					
		ε = √[275	N/mm² / p _y] = 0	.89		
Internal compression parts - Ta	able 11					
Internal compression parts - Ta Depth of section	able 11	d = 200.3	mm			
Internal compression parts - Ta Depth of section Stress ratios	able 11	d = 200.3 r1 = min(F	mm $f_c / (d \times t \times p_{yw}),$	1) = 0.245		
Internal compression parts - Ta Depth of section Stress ratios	able 11	d = 200.3 r1 = min(F r2 = F _c / (A	mm c / (d × t × pyw), A × pyw) = 0.046	1) = 0.245		
Internal compression parts - Ta Depth of section Stress ratios	able 11	d = 200.3 r1 = min(F r2 = F _c / (A d / t = 17.{	mm c / (d × t × p _{yw}), A × p _{yw}) = 0.046 5 × ε <= max(80	1) = 0.245 δ Ο × ε / (1 + r1), 40	×ε) Clas	ss 1 plastic
Internal compression parts - Ta Depth of section Stress ratios	able 11	d = 200.3 r1 = min(F r2 = F _c / (<i>F</i> d / t = 17.5	mm c / (d × t × p _{yw}), A × p _{yw}) = 0.046 5 × ε <= max(80	1) = 0.245 5 Ο × ε / (1 + r1), 40	×ε) Clas	ss 1 plastic
Internal compression parts - Ta Depth of section Stress ratios Outstand flanges - Table 11 Width of section	able 11	d = 200.3 r1 = min(F r2 = F _c / (A d / t = 17.5 b = B / 2 =	mm c / (d × t × pyw), A × pyw) = 0.046 5 × ε <= max(80 : 129.4 mm	1) = 0.245 δ Ο × ε / (1 + r1), 40	×ε) Clas	ss 1 plastic
Internal compression parts - Ta Depth of section Stress ratios Outstand flanges - Table 11 Width of section	able 11	d = 200.3 r1 = min(F r2 = F _c / (A d / t = 17.5 b = B / 2 = b / T = 7 1	mm $f_{c} / (d \times t \times p_{yw}),$ $A \times p_{yw}) = 0.046$ $5 \times \varepsilon \le max(80)$ 129.4 mm $\times \varepsilon \le 9 \times \varepsilon$	1) = 0.245 δ 0 × ε / (1 + r1), 40 Class	l×ε) Clas 1 plastic	ss 1 plastic

Tekla Tedds	Project Job no.					180		
TAK Structures	O al a a fair	30 FI0g			20			
10 Kennington Park Place	Calcs for	16.47	n Prop		Start page no./Re	2		
SE11 4AS	Calcs by RH	Calcs date 13/05/2021	Checked by	Checked date	Approved by	Approved date		
Shear capacity - Section 4.2.3								
Design shear force		F _{y,v} = 12.4	٨N					
		d / t < 70 ×	3					
			Web does n	ot need to be cl	hecked for sh	ear buckling		
Shear area		$A_v = t \times D =$	3414 mm ²					
Design shear resistance		$P_{y,v}$ = 0.6 \times	$p_y \times A_v = \textbf{706.6}$	kN				
		PAS	S - Design she	ar resistance ex	ceeds desigr	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 = 61.1 kM	٨m					
Moment capacity low shear - cl.	4.2.5.2	$M_c = min(p_s)$	× S _{xx} . 1.2 × p _v >	< Z _{xx}) = 512.1 kN	m			
Effective length for lateral ter	nional huakling	Section 4.2 E	,	,				
Effective length for lateral torsion	nal buckling	1 - 3ection 4.3.3	- 16500 mm					
Slondornoop ratio	Effective length for lateral torsional buckling							
Siendemess Tallo		$\lambda - LE / Tyy -$	- 250.279					
Equivalent slenderness - Sect	ion 4.3.6.7							
Buckling parameter		u = 0.848						
		x = 12.393						
		V = 1 / [1 + 2000]	$0.05 \times (\lambda / X)^2]^{0.2}$	= 0.465				
Ratio - cl.4.3.6.9	o -	βw = 1.000						
Equivalent slenderness - cl.4.3.6	5.7	$\lambda_{LT} = \mathbf{U} \times \mathbf{V}$	× λ × √[βw] = 98	.687				
Limiting slenderness - Annex B.	2.2	$\lambda_{L0} = 0.4 \times 10^{-10}$	$(\pi^2 \times E / p_y)^{0.5} =$	30.632				
		$\lambda_{LT} > \lambda_{L0} - \lambda_{L0}$	Allowance sho	uld be made for	lateral-torsio	nal buckling		
Bending strength - Section 4.3	3.6.5							
Robertson constant		α _{LT} = 7.0						
Perry factor		η _{LT} = max(α _{LT} × (λ _{LT} - λ _{L0}) / 1000, 0) = 0.476						
Euler stress		$p_{E} = \pi^{2} \times E / \lambda_{LT}^{2} = 207.7 \text{ N/mm}^{2}$						
		$\phi_{LT} = (p_y + ($	$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 325.9 \text{ N/mm}^2$					
Bending strength - Annex B.2.1		$p_{b} = p_{E} \times p_{y}$	$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 140.1 \text{ N/mm}^2$					
Equivalent uniform moment fa	actor - Section	4.3.6.6						
Equivalent uniform moment fact	or for LTB	m _{LT} = 1.000)					
Buckling resistance moment -	Section 4.3.6.4	4						
Buckling resistance moment		$M_b = p_b \times S$	_{xx} = 208 kNm					
		M₀ / m∟⊤ = 2	2 08 kNm					
		PASS - Bucklin	ng resistance n	noment exceeds	s design bend	ling moment		
Compression members - Sect	ion 4.7							
Design compression force		Fc = 222.8	٨N					
Effective length for major (x-x) axis buckling	- Section 4.7.3						
Effective length for buckling	, J	$L_{Ex} = L_x \times K$	x = 16500 mm					
Slenderness ratio - cl.4.7.2		$\lambda_x = L_{Fx} / r_x$	= 145.618					
Comprossive strength Santis	on 175							
Limiting clondorness	/11 4.7.0	$\lambda_{-} = 0.2 \times 0$		15 316				
Strut curve - Table 22		$h_0 = 0.2 \times (2)$	n ×⊏/Py)°°°=1	13.310				
		U						

Tekla Tedds	Project	38 Frog	nal I ane		Job no. 20080				
TAK Structures	Color for	001109		Start nage no /Revision					
10 Kennington Park Place	Calcs for	16.47	m Prop		Start page no./Re	3			
London	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date			
SE11 4AS	RH	13/05/2021			, pprovod by				
Robertson constant		a _× = 3.5							
Perry factor		$\alpha_x = 3.3$							
Fuler stress		$n_{\rm Ex} = \pi^2 \times {\rm E}$	$(\lambda_{2}^{2} = 95.4 \text{ N/m})$	nm ²					
$\phi_x = (p_y + (n_x + 1) \times p_{Fx})/2 = 242 \text{ N/mm}^2$									
Compressive strength - Annex (1	$\varphi_{x} = p_{F_{x}} \times r$	ייין איין איין איין איין איין איין איין	$x = \frac{1}{2} + $	N/mm ²				
	4			(, , , , , , , , , , , , , , , , , , ,					
Compression resistance - Sec	10n 4.7.4		- 4446 6 KN						
Compression resistance - ci.4.7	Compression resistance - cl.4.7.4 $P_{cx} = A \times p_{cx} = 1116.6 \text{ kN}$					raccion forca			
		PASS - Com	pression resis		uesigii compi	ession force			
Effective length for minor (y-y) axis buckling	- Section 4.7.3							
Effective length for buckling	$L_{Ey} = L_y \times K$	_y = 16500 mm							
Slenderness ratio - cl.4.7.2	$\lambda_y = L_{Ey} / r_{yy}$	$\lambda_y = L_{Ey} / r_{yy} = 250.279$							
Compressive strength - Section	on 4.7.5								
Limiting slenderness		$\lambda_0 = 0.2 \times (2)$	$\pi^2 \times E / p_y)^{0.5} = 1$	15.316					
Strut curve - Table 23		С							
Robertson constant		α _y = 5.5							
Perry factor		$\eta_y = \alpha_y \times (\lambda_y - \lambda_0) / 1000 = 1.292$							
Euler stress		$p_{Ey} = \pi^2 \times E$	$1/\lambda_y^2 = 32.3 \text{ N/m}$	nm²					
		$\phi_y = (p_y + (r$	η _y + 1) × p _{Ey}) / 2	= 209.5 N/mm ²					
Compressive strength - Annex (2.1	$p_{cy} = p_{Ey} \times p$	$p_y / (\phi_y + (\phi_y^2 - p_E))$	$(y \times p_y)^{0.5}$) = 28.5	N/mm ²				
Compression resistance - Sec	tion 4.7.4								
Compression resistance - cl.4.7	.4	$P_{cy} = A \times p_{c}$	_y = 389.2 kN						
		PASS - Com	pression resis	tance exceeds (design compr	ression force			
Compression members with n	noments - Sect	ion 4.8.3							
Comb.compression & bending of	heck - cl.4.8.3.2	$F_c / (A \times p_y)$	+ M / M _c = 0.16	57					
		PASS -	Combined ber	nding and comp	ression chec	k is satisfied			
Member buckling resistance -	Section 4.8.3.3	5							
Max major axis moment governi	ng M₀	$M_{LT} = M_x =$	61.10 kNm						
Equivalent uniform moment fact	or for major axis	flexural bucklin	g						
		m _x = 1.000							
		m _y = 1.000							
Buckling resistance checks - cl.	4.8.3.3.2	F _c / P _{cx} + m	$_{\rm x} imes M$ / M _c $ imes$ (1 +	$-0.5 \times F_c / P_{cx}) =$	0.331				
		F _c / P _{cy} + m	$_{LT} \times M_{LT} / M_{b} = 0$.866					
		F	PASS - Member	buckling resis	tance checks	are satisfied			

Tekla. Tedds	Project 38 Frognal Lane				Job no. 20080	
TAK Structures 10 Kennington Park Place	Calcs for 5m high retaining wall				Start page no./Revision 1	
SE11 4AS	Calcs by RH	Calcs date 12/05/2021	Checked by	Checked date	Approved by	Approved date

Tekla Tedds	Project	38 Frod	inal Lane	Job no.		
TAK Structures	Calcs for		, _		Start page no./Re	evision
10 Kennington Park Place		5m high re	etaining wall			2
SE11 4AS	Calcs by RH	Calcs date 12/05/2021	Checked by	Checked date	Approved by	Approved date
Saturated density of retained ma	aterial	γs = 23.0 kl	N/m ³			
Design shear strength		φ' = 24.2 de	eg			
Angle of wall friction		δ = 18.6 de	èg			
Base material details						
Moist density		γ _{mb} = 18.0	kN/m³			
Design shear strength		φ' _b = 24.2 c	leg			
Design base friction		δ _b = 18.6 d	eg			
Allowable bearing pressure		P _{bearing} = 10	JU KN/M ²			
Using Coulomb theory		- 1				
Active pressure coefficient for re	$\pm 4^{1}$ $(cin(\alpha)^2)$	∃I ∖y cip(a _ S) y [1 J	$\sqrt{(cin(4'+8))}$	$\sin(4' - \beta) / (\sin(\alpha))$	$(\alpha + 1)$	R)))12) - 0 360
$R_a - SIII(\alpha)$	$+ \varphi$)- / (SIII(α)-	$\times \sin(\alpha - \delta) \times [1 - \delta]$	$+ v(\sin(\phi + o) \times$	sin(φ - p) / (sin(α	$- o) \times \sin(\alpha + \beta)$	5)))] ⁻) – 0.369
	$K_{\rm p} = \sin^2$	(90 - Ⴛ' _Ⴆ)² / (sin(9() - δ⊳) × [1 - √(si	n(Ⴛ' _Ⴆ + ჽ _Ⴆ) × sin(Ⴛ	' _ʰ) / (sin(90 + δ	(h)))] ²) = 4.187
At root process						<i>s</i> ///] /e.
At-rest pressure At-rest pressure for retained ma	iterial	K₀ = 1 – si	n(di') = 0 590			
	tonar					
Surcharge load on plan		Surcharge	= 10 0 kN/m ²			
Applied vertical dead load on wa	all	W _{dead} = 0.0) kN/m			
Applied vertical live load on wall	1	W _{live} = 0.0	kN/m			
Position of applied vertical load	on wall	I _{load} = 0 mn	n			
Applied horizontal dead load on	wall	F _{dead} = 0.0	kN/m			
Applied horizontal live load on w	vall	F _{live} = 0.0 k	:N/m			
Height of applied horizontal load	1 ON Wall	Moad - U III	TI			
				10		
21.4	F		op	3.5 38.9		
				Loads shown	n in kN/m. pressure	es shown in kN/m²
				200001000		

Tekla Tedds	Project	38 Fro	Job no.	Job no. 20080				
TAK Structures	Calcs for		5	Start page no./Revision				
10 Kennington Park Place		5m high i	retaining wall		otart page no./	3		
London SE11 4AS	Calcs by RH	Calcs date 12/05/2021	Approved by	Approved by Approved date				
Vertical forces on wall								
Wall stem		$w_{wall} = h_{ste}$	$_{m} \times t_{wall} \times \gamma_{wall} =$	47.2 kN/m				
Wall base	Wbase = Ibas	$s_{e} imes t_{base} imes \gamma_{base}$	= 24.1 kN/m					
Total vertical load	$W_{total} = W_{v}$	_{/all} + w _{base} = 71.	3 kN/m					
Horizontal forces on wall								
Surcharge		F _{sur} = K _a ×	$\cos(90 - \alpha + \delta)$) × Surcharge × ł	n _{eff} = 18.5 kN/m	ı		
Moist backfill above water table	F _{m_a} = 0.5	\times K _a \times cos(90 \cdot	- α + δ) × γ_m × (he	eff - h _{water}) ² = 10	3.2 kN/m			
Total horizontal load	F _{total} = F _{su}	+ F _{m_a} = 121.7	′ kN/m					
Calculate total propping force)							
Passive resistance of soil in fro	$F_p = 0.5 \times$	$K_p imes cos(\delta_b) imes v$	(d _{cover} + t _{base} + d _d	s - d _{exc}) ² × γ _{mb} =	3.2 kN/m			
Propping force	F _{prop} = ma	x(F _{total} - F _p - (W	V_{total} × tan(δ_{b}), 0 k	(N/m)				
		F _{prop} = 94 .	5 kN/m					
Overturning moments								
Surcharge		M_{sur} = $F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2 = 49.1 \text{ kNm/m}$						
Moist backfill above water table		$M_{m_a} = F_{m_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3 = 182.3 \text{ kNm/m}$						
Total overturning moment		M _{ot} = M _{sur}	M _{ot} = M _{sur} + M _{m_a} = 231.4 kNm/m					
Restoring moments								
Wall stem		$M_{wall} = W_{wall}$	$_{\rm all} imes (I_{\rm toe} + t_{\rm wall} / 2$	2) = 151 kNm/m				
Wall base		$M_{base} = W_{b}$	$M_{\text{base}} = W_{\text{base}} \times I_{\text{base}} / 2 = 40.9 \text{ kNm/m}$					
Total restoring moment		M _{rest} = M _w	M _{rest} = M _{wall} + M _{base} = 192 kNm/m					
Check bearing pressure								
Total vertical reaction		R = W _{total}	= 71.3 kN/m					
Distance to reaction		$\mathbf{x}_{\text{bar}} = \mathbf{I}_{\text{base}}$	x _{bar} = I _{base} / 2 = 1700 mm					
Eccentricity of reaction		$e = abs((I_{base} / 2) - x_{bar}) = 0 mm$						
				Reaction acts	within middle	e third of ba		
Bearing pressure at toe		p _{toe} = (R /	I_{base}) - (6 × R ×	e / I _{base} ²) = 21 kN	l/m²			
Bearing pressure at heel		$p_{heel} = (R)$	$p_{heel} = (R / I_{base}) + (6 \times R \times e / I_{base}^2) = 21 \text{ kN/m}^2$					
	P	ASS - Maximum	bearing press	ure is less than	allowable bea	nring pressu		

Propping force to top of wall

 $F_{prop_top} = (M_{ot} - M_{rest} + R \times I_{base} / 2 - F_{prop} \times t_{base} / 2) / (h_{stem} + t_{base} / 2) = 28.437 \text{ kN/m}$

Propping force to base of wall

 $F_{prop_base} = F_{prop} - F_{prop_top} = 66.091 \text{ kN/m}$

Tokla Todda	Project				Job no.				
		38 Frog	nal Lane		20	080			
TAK Structures	Calcs for				Start page no./Re	evision			
10 Kennington Park Place		5m high re	taining wall			4			
SE11 4AS	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date			
	RH	12/05/2021							
RETAINING WALL DESIGN (B	S 8002:1994)								
				1	EDDS calculation	version 1.2.01.08			
Ultimate limit state load factor	rs								
		γ _{f_d} = 1.4							
		γ _{f_l} = 1.6							
Earth and water pressure factor		γ _{f_e} = 1.4							
Factored vertical forces on wa	all								
Wall stem		$W_{wall_f} = \gamma_{f_d}$	$\times \mathbf{h}_{stem} \times \mathbf{t}_{wall} \times \gamma_{w}$	all = 66.1 kN/m					
Wall base		$W_{base_f} = \gamma_{f_d}$	$\times \ I_{\text{base}} \times t_{\text{base}} \times \gamma$	_{base} = 33.7 kN/n	n				
Total vertical load		$W_{total_f} = w_w$	all_f + W _{base_f} = 99	.8 kN/m					
Factored horizontal at-rest for	rces on wall								
Surcharge		$F_{sur_f} = \gamma_{f_l} \times$	$K_0 \times \text{Surcharge}$	\times h _{eff} = 50 kN/m	ı				
Moist backfill above water table	$F_{m_a_f} = \gamma_{f_e}$	\times 0.5 \times K ₀ \times γ_m \times	$(h_{eff} - h_{water})^2 = 2$	243.7 kN/m					
Total horizontal load		$F_{total_f} = F_{sur}$	$F_{total_f} = F_{sur_f} + F_{m_a_f} = 293.7 \text{ kN/m}$						
Calculate total propping force)								
Passive resistance of soil in fror	nt of wall	$F_{p_f} = \gamma_{f_e} \times$	$0.5 \times K_{p} \times \cos(\delta)$	b) × (d _{cover} + t _{base}	+ d _{ds} - d _{exc}) ² ×	γ _{mb} = 4.5			
kN/m									
Propping force		F _{prop_f} = ma	x(F _{total_f} - F _{p_f} - (V	$V_{total_f}) imes tan(\delta_b),$	0 kN/m)				
		F _{prop_f} = 255	5.6 kN/m						
Factored overturning moment	ts								
Surcharge		$M_{sur_f} = F_{sur_f}$	$_{_{\rm f}} \times ({\rm h_{eff}} - 2 \times d_{\rm ds})$) / 2 = 132.6 kN	m/m				
Moist backfill above water table		$M_{m_a_f} = F_{m_a}$	M _{m_a_f} = F _{m_a_f} × (h _{eff} + 2 × h _{water} - 3 × d _{ds}) / 3 = 430.5 kNm/m						
Total overturning moment		$M_{ot_f} = M_{sur_f} + M_{m_a_f} = 563.1 \text{ kNm/m}$							
Restoring moments									
Wall stem		M _{wall f} = w _{wa}	$_{\rm II f} \times (I_{\rm toe} + t_{\rm wall} / 2$?) = 211.5 kNm/r	n				
Wall base		$M_{\text{base f}} = W_{\text{b}}$	_{ase f} × I _{base} / 2 = 5	5 7.3 kNm/m					
Total restoring moment		M _{rest_f} = M _{wa}	$M_{\text{rest }f} = M_{\text{wall} f} + M_{\text{base }f} = 268.7 \text{ kNm/m}$						
Factored bearing pressure									
Total vertical reaction		R _f = W _{total f}	= 99.8 kN/m						
Distance to reaction		$\mathbf{x}_{\text{bar f}} = \mathbf{I}_{\text{base}}$	/ 2 = 1700 mm						
Eccentricity of reaction		e _f = abs((I _{ba}	_{se} / 2) - x _{bar_f}) = () mm					
				Reaction acts v	vithin middle	third of base			
Bearing pressure at toe		$p_{toe_f} = (R_f / $	I_{base}) - (6 × R _f × 6	e _f / I _{base} ²) = 29.3	kN/m²				
Bearing pressure at heel		$p_{heel_f} = (R_f)$	/ I _{base}) + (6 \times R _f \times	e _f / I _{base} ²) = 29.	3 kN/m²				
Rate of change of base reaction	ı	rate = (p _{toe} _	_f - p _{heel_f}) / I _{base} =	0.00 kN/m²/m					
Bearing pressure at stem / toe		$p_{stem_toe_f} = 1$	max(p _{toe_f} - (rate	\times I _{toe}), 0 kN/m ²)	= 29.3 kN/m ²				
Bearing pressure at mid stem		$p_{stem_mid_f} =$	max(p _{toe_f} - (rate	\times (I _{toe} + t _{wall} / 2))	, 0 kN/m²) = 2	9.3 kN/m²			
Bearing pressure at stem / heel		p _{stem_heel_f} =	max(p _{toe_f} - (rate	$e \times (I_{toe} + t_{wall})), 0$	kN/m ²) = 29.3	kN/m ²			
Calculate propping forces to t	top and base o	of wall							
Propping force to top of wall	-								
	F _{prop_top_f} =	(M _{ot_f} - M _{rest_f} + R _f	× I _{base} / 2 - F _{prop} _	_f × t _{base} / 2) / (h _{st}	_{em} + t _{base} / 2) =	82.641 kN/m			

Propping force to base of wall

 $top_f = (Mot_f - Mrest_f + R_f \times Ibase / 2 - F_{prop_f} \times Ibase / 2) / (Nstem + Ibase / 2) = 82.641 \text{ KN/m}$ $F_{prop_base_f} = F_{prop_f} - F_{prop_top_f} = 172.975 \text{ kN/m}$

Design of reinforced concrete retaining wall toe (BS 8002:1994)

Tokla Todds	Project		Job no.					
		38 Frog		20	080			
TAK Structures 10 Kennington Park Place	Calcs for	5m high re	etaining wall		Start page no./Re	evision 6		
London SE11 4AS	Calcs by RH	Calcs date 12/05/2021	Checked by	Checked date	Approved by	Approved date		
Design of reinforced concrete Material properties	e retaining wal	ll stem (BS 8002	:1994 <u>)</u>					
Characteristic strength of concr	ete	f _{cu} = 40 N/n	nm²					
Characteristic strength of reinfo	rcement	f _v = 500 N/r	mm²					
Wall details		,						
Minimum area of reinforcement		k = 0 13 %						
Cover to reinforcement in stem		R = 0.15 %	nm					
Cover to reinforcement in wall		Curr = 50 m	m					
			111					
Factored horizontal at-rest fo	rces on stem	_						
Surcharge		$F_{s_sur_f} = \gamma_{f_i}$	$\times K_0 \times Surchargenergy$	$ge imes (h_{eff} - t_{base} - c)$	d _{ds}) = 47.2 kN/	m		
Moist backfill above water table		$F_{s_m_a_f} = 0$	$.5 imes \gamma_{f_e} imes K_0 imes \gamma_r$	$m \times (h_{eff} - t_{base} - d_{d})$	s - h _{sat}) ² = 216	.9 kN/m		
Calculate shear for stem desi	gn							
Surcharge		$V_{s_sur_f} = 5$	× F _{s_sur_f} / 8 = 29	0.5 kN/m				
Moist backfill above water table		V _{smaf} =F	s m a f × b∣ × ((5 :	\times L ²) - b ²) / (5 \times L	_ ³) = 170.8 kN	/m		
Total shear for stem design		V _{stem} = V _{s s}	ur f + Vs m a f = 2	200.4 kN/m	,			
Calculate moment for stem d	osian	-						
Surcharge	esign	M. – F.		4 kNm/m				
Majet beekfill eheve weter teble			$sur_f \times L / O = JU$		15 2) - 457			
Tetel memory for stem design		$\frac{1}{100} = \frac{1}{100} = \frac{1}$						
Total moment for stem design		IVIstem – IVIs_s	sur + IVIs_m_a - 10	7.4 KINIII/III				
Calculate moment for wall de	sign							
Surcharge		$M_{w_{sur}} = 9 \times$	<pre>Fs_sur_f × L / 128</pre>	8 = 17.1 kNm/m				
Moist backfill above water table		$M_{w_m_a} = F_s$	$a_m_a_f \times 0.577 \times b$	ı×[(bi³+5×ai×L²)/(5	5×L ³)-0.577 ² /3] = 63.3		
kNm/m		NA — NA	±M − 80	4 kNm/m				
Total moment for wall design		IVIwali — IVIw_s	ur + Iviw_m_a - 60	.4 KINIII/III				
	← 100 → >	•	•	•				
	 ⊲ 200–							
Check wall stem in bending		, , , , , , , , , , , , , , , , , , , ,	1					
Width of wall stem		b = 1000 m	im/m					
Depth of reinforcement		d _{stem} = t _{wall} -	- C _{stem} - (φ _{stem} / 2	2) = 340.0 mm				
Constant		K _{stem} = M _{ste}	m / (b × d _{stem} ² × f	t _{cu}) = 0.041		, <u> </u>		
			Co	ompression rein	ntorcement is	not required		
Lever arm		z _{stem} = min(z _{stem} = 323	0.5 + √(0.25 - (r mm	min(K _{stem} , 0.225)	/ 0.9)),0.95) ×	d _{stem}		
Area of tension reinforcement r	equired	As_stem_des =	M_{stem} / (0.87 \times 1	$f_y \times z_{stem}$) = 1334	mm²/m			

Tekla Tedds	Project	38 Fro	Job no. 20080					
TAK Structures	Calcs for				Start page no./F	Revision		
10 Kennington Park Place		5m high r	etaining wall			7		
SE11 4AS	Calcs by RH	Calcs date 12/05/2021	Approved by	Approved date				
Minimum area of tension reinfor	rcement	As_stem_min =	= k × b × t _{wall} =	520 mm²/m				
Area of tension reinforcement re	equired	A _{s stem req} =	= Max(A _{s stem de}	es, A _{s stem min}) = 1 3	334 mm²/m			
Reinforcement provided		20 mm dia	a.bars @ 200 ı	mm centres				
Area of reinforcement provided		As stem prov	= 1571 mm²/m	ı				
		PASS - Reinfo	orcement prov	vided at the reta	ining wall ste	m is adequate		
Check shear resistance at wa	ll stem							
Design shear stress	in Stern	Votor = Voto		0 589 N/mm ²				
Allowable shear stress		vstem – vste	$v_{\text{stem}} = v_{\text{stem}} / (D \times Q_{\text{stem}}) = 0.559 \text{ N/Mm}^2$					
Allowable silear siless				(1,1,1,1,1,1,1,1,1,1,1,1,1,1,1,1,1,1,1,	////// 5.000 /			
Erom BS9110-Dort 1:1007 T	abla 2 0	PASS ·	Design shea	r stress is less i	nan maximun	n shear stress		
Profile BS0110.Fait 1.1997 – 16	58110:Part 1:1997 – Table 3.8							
Design concrete shear stress		vc_stem – U.	JJJJ N/IIIII	< v No st	oar roinforco	mont required		
			v stem			inent required		
Check mid height of wall in be	ending							
Depth of reinforcement		$d_{wall} = t_{wall}$ -	– C _{wall} – (φ _{wall} / 2	?) = 344.0 mm				
Constant		$K_{wall} = M_{wall}$	$I / (b \times d_{wall}^2 \times f_{r})$	_{cu}) = 0.017				
				Compression re	inforcement i	s not required		
Lever arm		z _{wall} = Min(0.5 + √(0.25 -	(min(K _{wall} , 0.225)	/ 0.9)),0.95) ×	d _{wall}		
		Z _{wall} = 327	mm					
Area of tension reinforcement re	equired	As_wall_des =	$A_{s_wall_des} = M_{wall} / (0.87 \times f_y \times z_{wall}) = 566 \text{ mm}^2/\text{m}$					
Minimum area of tension reinfor	rcement	$A_{s_wall_min} =$	$A_{s_wall_min} = k \times b \times t_{wall} = 520 \text{ mm}^2/\text{m}$					
Area of tension reinforcement re	equired	A _{s_wall_req} =	Max(A _{s_wall_des} ,	, A _{s_wall_min}) = 566	mm²/m			
Reinforcement provided		12 mm dia	a.bars @ 100 ı	mm centres				
Area of reinforcement provided		As_wall_prov =	= 1131 mm²/m					
	PASS	S - Reinforcemen	t provided to	the retaining wa	ll at mid heig	ht is adequate		
Check retaining wall deflection	on							
Basic span/effective depth ratio	1	ratio _{bas} = 2	0					
Design service stress		$f_s = 2 \times f_v >$	As stem rea / (3	$\times A_{s \text{ stem prov}}$ = 2	83.0 N/mm²			
Modification factor	factor _{tens} = mi	in(0.55 + (477 N/n	$100^{2} - f_{s})/(120 \times 10^{2})$	< (0.9 N/mm ² + (N	$l_{\text{stem}}/(b \times d_{\text{stem}}^2)$)))).2) = 1.19		
Maximum span/effective depth	ratio	$\operatorname{ictor_{tens}} = \min(0.55 + (477 \text{ N/mm}^2 - T_s)/(120 \times (0.9 \text{ N/mm}^2 + (M_{\text{stem}}/(b \times d_{\text{stem}}^2)))$ tio ratio _{max} = ratio _{bas} × factor _{tens} = 23.82						

Maximum span/effective depth ratio Actual span/effective depth ratio

 $\label{eq:ratio} ratio_{act} = h_{stem} \ / \ d_{stem} = \textbf{14.71}$ PASS - Span to depth ratio is acceptable

Tekla Tedds	Project Job no.						no.	
		38 Frog	Inal Lane			20	080	
10 Kennington Park Place	Calcs for	3.3m high r	etaining wall			Start page no./Re	evision 2	
London SE11 4AS	Calcs by RH	Calcs date 12/05/2021	Checked by	Checked da	ite	Approved by	Approved date	
Saturated density of retained m	aterial	γs = 23.0 kl	N/m ³	•				
Design shear strength		φ' = 24.2 de	eg					
Angle of wall friction		δ = 18.6 de	eg					
Base material details								
Moist density		γ _{mb} = 18.0 I	kN/m ³					
Design shear strength		φ' _b = 24.2 d	leg					
Design base friction		δ _b = 18.6 d	eg					
Allowable bearing pressure		P _{bearing} = 10	00 kN/m²					
Using Coulomb theory	atained material							
$K_{a} = \sin(\alpha)$	+ $d'^2 / (\sin(\alpha)^2)$	$(\sin(\alpha - \delta) \times [1 + $	$-\sqrt{(\sin(\phi' + \delta))}$	sin(d' - B) / ((sin(a -	$(\delta) \times \sin(\alpha + 1)$	R()()]2) = 0 369	
Passive pressure coefficient for	base material	$\sqrt{3}$	ν(Siii(ψ + 0) × 3	σπι(φ - μ) / ((511(0	· 0) × 3ii (u · j	b)))]) – 0.303	
	$K_{\rm p} = \sin(9)$	90 - φ' _b)² / (sin(90) - δ⊳) × [1 - √(sir	ე(ძ' ь +	sin(d'b) / (sin(90 + δ	(h)))] ²) = 4.187	
		+ 2) · (((+ 2 - 02)	(+ 2	,, ((<i>s</i> ///1 /	
At-rest pressure for retained ma	atorial	K_ = 1 _ sir	o(4') = 0 590					
			ι(ψ) = 0.000					
Loading details		Surabarga	$-100 \text{kN}/\text{m}^2$					
Applied vertical dead lead on w			= 10.0 KN/m ²					
Applied vertical live load on wal	an 	$W_{\text{live}} = 0.0$	kN/m					
Position of applied vertical load	on wall	$I_{load} = 0 \text{ mm}$	1					
Applied horizontal dead load on	wall	F _{dead} = 0.0	kN/m					
Applied horizontal live load on v	vall	F _{live} = 0.0 k	N/m					
Height of applied horizontal load	d on wall	h _{load} = 0 mr	n					
				10				
Z1.4	52.3		0.0	3.5	26.4			
				Loads	s shown i	in kN/m, pressure	es shown in kN/m²	

Tekla, Tedds	Project	38 Frog	Job no. 20080					
TAK Structures	Calcs for	-	Start page no./Revision					
10 Kennington Park Place		3.3m high i		3				
London SE11 4AS	Calcs by RH	Calcs date 12/05/2021	Approved by	Approved by Approved date				
Vertical forces on wall								
Wall stem		w _{wall} = h _{stem}	$\times t_{wall} \times \gamma_{wall} =$	31.2 kN/m				
Wall base		w _{base} = I _{base}	$1 imes \mathbf{t}_{base} imes \gamma_{base}$	= 20.5 kN/m				
Total vertical load		$W_{total} = W_{wa}$	+ w _{base} = 51.	7 kN/m				
Horizontal forces on wall								
Surcharge		$F_{sur} = K_a \times$	cos(90 - α + δ) × Surcharge × ł	n _{eff} = 12.6 kN/m	ı		
Moist backfill above water table	e	F _{m a} = 0.5	$\times K_a \times \cos(90)$	$-\alpha + \delta$) × γ_m × (he	_{eff} - h _{water}) ² = 47	.6 kN/m		
Total horizontal load		$F_{\text{total}} = F_{\text{sur}} + F_{\text{m}} = 60.2 \text{ kN/m}$						
Calculate propping force								
Passive resistance of soil in fro	$F_{p} = 0.5 \times K_{p} \times cos(\delta_{b}) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^{2} \times \gamma_{mb} = 3.2 \text{ kN/m}$							
Propping force	$F_{prop} = max(F_{total} - F_p - (W_{total}) \times tan(\delta_b), 0 \text{ kN/m})$							
		F _{prop} = 39.6 kN/m						
Overturning moments								
Surcharge		M _{sur} = F _{sur} :	× (h _{eff} - 2 × d _{ds}	s) / 2 = 22.7 kNm	/m			
Moist backfill above water table	e	M_{m_a} = $F_{m_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = 57.1 kNm/m						
Total overturning moment		M _{ot} = M _{sur} + M _{m_a} = 79.8 kNm/m						
Restoring moments								
Wall stem		$M_{wall} = W_{wall}$	\times (I _{toe} + t _{wall} / 2	2) = 84.1 kNm/m				
Wall base		M _{base} = w _{base} × I _{base} / 2 = 29.8 kNm/m						
Total restoring moment		M _{rest} = M _{wal}	+ M _{base} = 113	. 9 kNm/m				
Check bearing pressure								
Total moment for bearing		M _{total} = M _{res}	.t - M _{ot} = 34.1 k	Nm/m				
Total vertical reaction	R = W _{total} = 51.7 kN/m							
Distance to reaction	$x_{bar} = M_{total} / R = 659 mm$							
Eccentricity of reaction		e = abs((I _{ba}	_{ase} / 2) - x _{bar}) =	791 mm				
				Reaction acts of	outside middle	e third of ba		
Bearing pressure at toe		p _{toe} = R / (1	l.5 × x _{bar}) = 52	.3 kN/m ²				
Bearing pressure at heel		p _{heel} = 0 kN	l/m² = 0 kN/m²	2				

Tokla Todda	, Project Job no.					
		38 Frog	20	080		
TAK Structures	Calcs for				Start page no./Revision	
10 Kennington Park Place London		3.3m high r	etaining wall			4
SE11 4AS	Calcs by RH	Calcs date 12/05/2021	Checked by	Checked date	Approved by	Approved date
RETAINING WALL DESIGN (B	S 8002:1994)					
					TEDDS calculation	version 1.2.01.08
Ultimate limit state load factor	rs					
Dead load factor		$\gamma_{f_d} = 1.4$				
Live load factor		γ _{f_l} = 1.6				
Earth and water pressure factor		γ _{f_e} = 1.4				
Factored vertical forces on wa	all					
Wall stem		W _{wall f} = γ _{f d}	\times h _{stem} \times t _{wall} \times γ_w	_{all} = 43.6 kN/m		
Wall base		$W_{base f} = \gamma_{f c}$	\times I _{base} \times t _{base} \times γ	_{base} = 28.7 kN/r	n	
Total vertical load		$W_{total_f} = W_w$	all_f + Wbase_f = 72	. 4 kN/m		
Factored horizontal at-rest for	rces on wall	_	_			
Surcharge		$F_{surf} = \gamma_{f} + X$	K₀ × Surcharge	× h _{eff} = 34 kN/n	n	
Moist backfill above water table		$F_{m,a,f} = \gamma_{f,e}$	$\times 0.5 \times K_0 \times v_m \times$	$(h_{eff} - h_{water})^2 =$	112.4 kN/m	
Total horizontal load		$F_{total_f} = F_{sur}$	_f + F _{m_a_f} = 146.	4 kN/m		
Calculate propping force		_				
Passive resistance of soil in from	nt of wall	$F_{p,f} = \gamma_{f,e} \times$	$0.5 \times K_{P} \times \cos(\delta)$	$(d_{cover} + t_{base})$	$+ d_{ds} - d_{exc})^2 >$	γ _{mb} = 4.5
kN/m		. <u>p_</u> . <u>1.</u>	(e	2) ((20070) (2007		1
Propping force		F _{prop_f} = ma	x(F _{total_f} - F _{p_f} - (V	$V_{total_f} \times tan(\delta_b)$, 0 kN/m)	
		F _{prop_f} = 11 7	7.6 kN/m			
Factored overturning moment	ts					
Surcharge		M _{sur f} = F _{sur}	$_{f} \times (h_{eff} - 2 \times d_{ds})$) / 2 = 61.2 kNn	n/m	
Moist backfill above water table		M _{m a f} = F _m	 a_f × (h _{eff} + 2 × h	, _{water} - 3 × d _{ds}) / 3	3 = 134.9 kNm	/m
Total overturning moment		M _{ot_f} = M _{sur_}	_{_f} + M _{m_a_f} = 196.	<i>.</i> 1 kNm/m		
Restoring moments						
Wall stem		M _{wall f} = w _{wa}	$_{\rm II} f \times (I_{\rm toe} + t_{\rm wall} / 2$	2) = 117.8 kNm/	m	
Wall base		$M_{\text{base f}} = W_{\text{b}}$	$_{ase f} \times I_{base} / 2 = 4$, 1.7 kNm/m		
Total restoring moment		M _{rest_f} = M _{wa}		9.4 kNm/m		
Factored bearing pressure						
Total moment for bearing		M _{total f} = M _{re}	est f - M _{ot f} = -36.6	kNm/m		
Total vertical reaction		$R_f = W_{total_f}$	= 72.4 kN/m			
Distance to reaction		$\mathbf{x}_{bar_f} = \mathbf{M}_{tota}$	_f / R _f = -506 mm	า		
Eccentricity of reaction		e _f = abs((I _{ba}	_{ise} / 2) - x _{bar_f}) = 1	1 956 mm		
				WARNING - Be	eyond scope o	of calculation
Bearing pressure at toe		$p_{toe_f} = R_f /$	$(1.5 \times x_{bar_f}) = -9$	5.3 kN/m²		
Bearing pressure at heel		p _{heel_f} = 0 kl	$N/m^2 = 0 kN/m^2$			
Rate of change of base reaction	1	rate = p _{toe_f}	/ (3 × X _{bar_f}) = 62	.69 kN/m²/m		
Bearing pressure at stem / toe		p _{stem_toe_f} =	max(p _{toe_f} - (rate	\times Itoe), 0 kN/m ²)	$= 0 \text{ kN/m}^2$	
Bearing pressure at mid stem		p _{stem_mid_f} =	max(p _{toe_f} - (rate	\times (I _{toe} + t _{wall} / 2)), 0 kN/m ²) = 0	kN/m ²
Bearing pressure at stem / heel		$p_{stem_heel_f} =$	max(p _{toe_f} - (rate	$e \times (I_{toe} + t_{wall})), C$) kN/m²) = 0 kN	N/m ²
Design of reinforced concrete	e retaining wal	l toe (BS 8002:1	994 <u>)</u>			
Material properties						
Characteristic strength of concre	ete	f _{cu} = 40 N/n	nm²			

 $\begin{array}{ll} \mbox{Characteristic strength of concrete} & f_{cu} = {\bf 40} \ \mbox{N/mm}^2 \\ \mbox{Characteristic strength of reinforcement} & f_y = {\bf 500} \ \mbox{N/mm}^2 \end{array}$

Tekla Tedds	Project	38 Frog	Job no. 20080			
TAK Structures	Calcs for				Start page no./R	evision
10 Kennington Park Place		3.3m high r	etaining wall		1.0	5
London SE11 4AS	Calcs by RH	Calcs date 12/05/2021	Checked by	Checked date	Approved by	Approved date
Base details		k - 0 42 %				
Minimum area of reinforcement		K = 0.13 %	~			
Calculate shear for toe design	1			1 01 0 LA	1/	
Shear from weight of base		V toe_wt_base =	- γf_d × γbase × Itoe	e × Ibase = 24.8 KN	v/m	
$V_{toe} = V_{toe_wt_base} = 24.8 \text{ KN/m}$						
Calculate moment for toe des	ign					
Moment from weight of base		M _{toe_wt_base} =	= $(\gamma_{f_d} \times \gamma_{base} \times t_t)$	$base \times (I_{toe} + t_{wall} / 2)$	2) ² / 2) = 36.1	<nm m<="" td=""></nm>
I otal moment for toe design		M _{toe} = M _{toe} _	wt_base = 36.1 KN	lm/m		
300	>					
	•	•	•	•	•	
★						
	┥ ──200──	→				
Check toe in bending						
Width of toe		b = 1000 m	ım/m			
Depth of reinforcement		$d_{toe} = t_{base}$ –	$-c_{toe} - (\phi_{toe}/2) =$	= 242.0 mm		
Constant		$K_{toe} = M_{toe} /$	$(b \times d_{toe}^2 \times f_{cu})$	= 0.015		
			C	ompression reii	nforcement is	not required
Lever arm		z _{toe} = min(0).5 + √(0.25 - (m	nin(K _{toe} , 0.225) / ($(0.9), 0.95) \times d_{t}$	oe
		z _{toe} = 230 n	nm			
Area of tension reinforcement re	equired	$A_{s_toe_des} = 1$	M_{toe} / (0.87 × f _y :	× z _{toe}) = 361 mm ²	²/m	
Minimum area of tension reinfor	cement	$A_{s_toe_min} = I$	$\mathbf{K} \times \mathbf{b} \times \mathbf{t}_{base} = 3$	90 mm²/m	24	
Area of tension reinforcement re	equired	$A_{s_{toe_{req}}} = 1$	Vlax(A _{s_toe_des} , A	us_toe_min) = 390 mi	m²/m	
Area of reinforcement provided		16 mm dia	.bars @ 200 m	m centres		
Area of reinforcement provided		As_toe_prov -	forcement pro	vided at the retain	aining wall to	o is adoquato
Chook observasioterses at to-		, AUG - Nelli				
Design shear resistance at toe		M = M = I	(b v d) - 0 40	2 N/mm ²		
Allowable shear stross		$v_{toe} - v_{toe} /$	$(D \land U_{\text{toe}}) = U.IU$	$/\text{mm}^2$ 5) \sim 1 N/m	nm ² = 5 000 N	/mm ²
Allowable Shear Siless		vadm - mm()	Design shoer	stross is lose th	nn – 5.000 N Ian mavimum	shoar stress
From BS8110:Part 1:1997 – Ta	able 3.8	, 400 -	- congin shieur			511001 301633
Design concrete shear stress		Vc toe = 0.62	25 N/mm²			
			Vtoe	< v _{c_toe} - No she	ear reinforcen	nent required
Design of reinforced concrete	retaining wall	stem (BS 8002	:1994)			-
Material properties	y					
Characteristic strength of concre	ete	f _{eu} = 40 N/n	nm²			
Characteristic strength of reinfor	rcement	f _v = 500 N/r	mm²			
Wall dotaile		,				
Minimum area of reinforcement		k = 0 13 %				

	Project 38 Frognal Lane				Job no. 20080	
TAK Structures	Calcs for				Start page no /Pc	vision
10 Kennington Park Place		3.3m high r	etaining wall		Start page no./rte	6
SE11 4AS	Calcs by RH	Calcs date 12/05/2021	Checked by	Checked date	Approved by	Approved date
Cover to reinforcement in stem Cover to reinforcement in wall		c _{stem} = 50 n c _{wall} = 50 m	nm Im			
Factored horizontal at-rest for	ces on stem					
Surcharge		$F_{s,sur,f} = \gamma_{f,i}$	⊥× K₀ × Surcharc	ie × (h _{eff} - t _{hase} - c	dds) = 31.2 kN/	m
Moist backfill above water table		$F_{smat} = 0.$	$.5 \times \gamma_{\rm fe} \times K_0 \times \gamma_{\rm m}$	\times (h _{eff} - t _{base} - d _d	$(s - h_{sat})^2 = 94.5$	kN/m
Calculate aboar for stom desir				. (5	
Shear at base of stem	Ju	V _{stem} = F _{s_st}	_{ur_f} + F _{s_m_a_f} - F _{pl}	_{rop_f} = 8.1 kN/m		
Calculate moment for stem de	sign					
Surcharge	5	Ms sur = Fs s	sur f × (hstem + tbas	_e) / 2 = 56.1 kNn	n/m	
Moist backfill above water table		 Ms_m_a = Fs	$m_{a f} \times (2 \times h_{sat} +$, ⊦ h _{eff} - d _{ds} + t _{base} /	/ 2) / 3 = 118.1	kNm/m
Total moment for stem design	$M_{stem} = M_{s sur} + M_{s m a} = 174.2 \text{ kNm/m}$			4.2 kNm/m	,	
L L	● • • • • • • • • • • • • • • • • •	••	•	•	•	
Check wall stem in bending	● • • • • • • • • • • • • • • • •	• •	•	•	•	
Check wall stem in bending Width of wall stem	● ←175→	• • •	• ım/m	•	•	
Check wall stem in bending Width of wall stem Depth of reinforcement	● • • • • • • • • • • • • • • • •	b = 1000 m d _{stem} = t _{wall} -	• nm/m – c _{stem} – (φ _{stem} / 2	•) = 340.0 mm	•	
Check wall stem in bending Width of wall stem Depth of reinforcement Constant	● ↓ ↓ 175 →	b = 1000 m d _{stem} = t _{wall} - K _{stem} = M _{ste}	• nm/m – c _{stem} – (φ _{stem} / 2 _m / (b × d _{stem} ² × f _c	•) = 340.0 mm _{5u}) = 0.038	•	
Check wall stem in bending Width of wall stem Depth of reinforcement Constant	● 175 →	b = 1000 m d _{stem} = t _{wall} - K _{stem} = M _{ste}	• nm/m - c _{stem} - (φ _{stem} / 2 m / (b × d _{stem} ² × f ₀ Co	•) = 340.0 mm cu) = 0.038 compression rein	• forcement is	not required
Check wall stem in bending Width of wall stem Depth of reinforcement Constant Lever arm	● ↓ ↓ 175 →	b = 1000 m d _{stem} = t _{wall} - K _{stem} = M _{ste} z _{stem} = min($m/m = -c_{stem} - (\phi_{stem} / 2)$ $m / (b \times d_{stem}^2 \times f_0)$ $Canonic Canonic Cano$	•) = 340.0 mm cu) = 0.038 cmpression rein hin(K _{stem} , 0.225)	• forcement is / 0.9)),0.95) ×	<i>not required</i>
Check wall stem in bending Width of wall stem Depth of reinforcement Constant Lever arm	• 175 • •	b = 1000 m d _{stem} = t _{wall} - K _{stem} = M _{ste} z _{stem} = min(z _{stem} = 323	find find find find find find find find	•) = 340.0 mm cu) = 0.038 pmpression rein hin(K _{stem} , 0.225)	• • • • • • • • • • • • • •	<i>not required</i> d _{stem}
Check wall stem in bending Width of wall stem Depth of reinforcement Constant Lever arm Area of tension reinforcement re Minimum area of tension roinforcement	• 175 • equired	b = 1000 m d _{stem} = t _{wall} - K _{stem} = M _{ste} z _{stem} = min(z _{stem} = 323 A _{s_stem_des} =	frightarrow for the second s	•) = 340.0 mm bu) = 0.038 pmpression rein hin(K _{stem} , 0.225) y × z _{stem}) = 1240 0 mm ² /m	• forcement is / 0.9)),0.95) × mm²/m	<i>not required</i> d _{stem}
Check wall stem in bending Width of wall stem Depth of reinforcement Constant Lever arm Area of tension reinforcement re Minimum area of tension reinforcement re	• 175 • equired	$b = 1000 \text{ m}$ $d_{stem} = t_{wall} - K_{stem} = M_{stel}$ $Z_{stem} = min($ $Z_{stem} = 323$ $A_{s_stem_des} = $ $A_{s_stem_min} = $ $A_{c_stem_stem_stem_stem_stem_stem_stem_stem$	find find find find find find find find	•) = 340.0 mm bu) = 0.038 compression rein hin(K _{stem} , 0.225) y × z _{stem}) = 1240 0 mm ² /m As stem rein) = 124	• forcement is / 0.9)),0.95) × mm²/m 0 mm²/m	<i>not required</i> d _{stem}
Check wall stem in bending Width of wall stem Depth of reinforcement Constant Lever arm Area of tension reinforcement re Minimum area of tension reinforcement re Area of tension reinforcement re Reinforcement provided	• 175 • equired cement equired	b = 1000 m d _{stem} = t _{wall} - K _{stem} = M _{ste} Z _{stem} = min(Z _{stem} = 323 A _{s_stem_des} = A _{s_stem_min} = A _{s_stem_req} = 20 mm dia	m/m $- C_{stem} - (\phi_{stem} / 2)$ $m / (b \times d_{stem}^2 \times f_0)$ Ca $(0.5 + (0.25 - (n m m m m m m m m m m m m m m m m m m $	•) = 340.0 mm bu) = 0.038 pmpression reim hin(K _{stem} , 0.225) y × z _{stem}) = 1240 0 mm ² /m A _{s_stem_min}) = 124 n centres	• forcement is / 0.9)),0.95) × mm²/m 0 mm²/m	<i>not required</i> d _{stem}
Check wall stem in bending Width of wall stem Depth of reinforcement Constant Lever arm Area of tension reinforcement re Minimum area of tension reinforcement re Reinforcement provided Area of reinforcement provided	• 175 • equired cement equired	b = 1000 m d _{stem} = t _{wall} - K _{stem} = M _{ste} Z _{stem} = min(Z _{stem} = 323 A _{s_stem_des} = A _{s_stem_req} = 20 mm dia A _{s_stem_prov} =	• • • • • • • • • • • • • •	•) = 340.0 mm au) = 0.038 pmpression reim hin(K _{stem} , 0.225) y × z _{stem}) = 1240 0 mm ² /m As_stem_min) = 124 n centres	• forcement is / 0.9)),0.95) × mm²/m 0 mm²/m	<i>not required</i> d _{stem}
Check wall stem in bending Width of wall stem Depth of reinforcement Constant Lever arm Area of tension reinforcement re Minimum area of tension reinforcement re Area of tension reinforcement re Reinforcement provided Area of reinforcement provided	• 175 • equired cement equired	b = 1000 m d _{stem} = t _{wall} - K _{stem} = M _{ste} Z _{stem} = min(Z _{stem} = 323 A _{s_stem_des} = A _{s_stem_min} = A _{s_stem_req} = 20 mm dia A _{s_stem_prov} = PASS - Reinfo	m/m $- c_{stem} - (\phi_{stem} / 2)$ $m / (b \times d_{stem}^2 \times f_0)$ $Contraction (0.5 + \sqrt{0.25} - (n))$ mm $M_{stem} / (0.87 \times f_0)$ $K \times b \times t_{wall} = 52$ $Max(As_{stem_des}, f_0)$ $Max(As_{stem_des$	•) = 340.0 mm bu) = 0.038 pmpression rein hin(K _{stem} , 0.225) y × z _{stem}) = 1240 0 mm ² /m A _{s_stem_min}) = 124 n centres led at the retain	• forcement is / 0.9)),0.95) × mm²/m 0 mm²/m ing wall stem	not required d _{stem}
Check wall stem in bending Width of wall stem Depth of reinforcement Constant Lever arm Area of tension reinforcement re Minimum area of tension reinforcement re Reinforcement provided Area of reinforcement provided Check shear resistance at wal	• 175 • • • • • • • • • • • • • • • • • • •	b = 1000 m d _{stem} = t _{wall} - K _{stem} = M _{ste} Z _{stem} = min(Z _{stem} = 323 A _{s_stem_des} = A _{s_stem_req} = 20 mm dia A _{s_stem_prov} = <i>PASS - Reinfo</i>	$ \int \mathbf{m} d\mathbf{m} = - (\phi_{stem} - (\phi_{stem})^2) $ $ m / (b \times d_{stem}^2 \times f_{c}) $ $ f(0.5 + (0.25 - (n m m m m m m m m m m m m m m m m m m $	•) = 340.0 mm su) = 0.038 pmpression reim nin(Kstem, 0.225) y × Zstem) = 1240 0 mm ² /m As_stem_min) = 124 n centres led at the retain	• forcement is / 0.9)),0.95) × mm²/m 0 mm²/m ing wall stem	not required d _{stem}
Check wall stem in bending Width of wall stem Depth of reinforcement Constant Lever arm Area of tension reinforcement re Minimum area of tension reinforcement re Area of tension reinforcement re Reinforcement provided Area of reinforcement provided Check shear resistance at wall Design shear stress	• 175 equired cement equired	b = 1000 m d _{stem} = t _{wall} - K _{stem} = M _{ste} Z _{stem} = min(Z _{stem} = 323 A _{s_stem_des} = A _{s_stem_min} = A _{s_stem_req} = 20 mm dia A _{s_stem_prov} = PASS - Reinfo	$m/m = c_{stem} - (\phi_{stem} / 2)$ $m / (b \times d_{stem}^2 \times f_0)$ $(0.5 + (0.25 - (n m m m m m m m m m m m m m m m m m m $	•) = 340.0 mm bu) = 0.038 pmpression rein hin(K _{stem} , 0.225) y × z _{stem}) = 1240 0 mm ² /m As_stem_min) = 124 n centres led at the retain 024 N/mm ²	• • • • • • • • • • • • • •	not required d _{stem}
Check wall stem in bending Width of wall stem Depth of reinforcement Constant Lever arm Area of tension reinforcement reinforcement reinforcement provided Area of tension reinforcement reinforcement provided Area of reinforcement provided Check shear resistance at wall Design shear stress Allowable shear stress	• 175 • 175	b = 1000 m d _{stem} = t _{wall} - K _{stem} = M _{ste} Z _{stem} = min(Z _{stem} = 323 A _{s_stem_des} = A _{s_stem_min} = A _{s_stem_req} = 20 mm dia A _{s_stem_prov} = PASS - Reinfo V _{stem} = V _{stem} V _{adm} = min($m/m = c_{stem} - (\phi_{stem} / 2)$ $m / (b \times d_{stem}^2 \times f_{c})$ $(0.5 + (0.25 - (n m m m m m m m m m m m m m m m m m m $	• • • • • • • • • • • • • •	• • • • • • • • • • • • • •	not required dstem o is adequate
Check wall stem in bending Width of wall stem Depth of reinforcement Constant Lever arm Area of tension reinforcement re Minimum area of tension reinforcement re Area of tension reinforcement re Reinforcement provided Area of reinforcement provided Area of reinforcement provided Exect shear resistance at wall Design shear stress Allowable shear stress	• 175 • 175	b = 1000 m d _{stem} = t _{wall} - K _{stem} = M _{ste} z _{stem} = min(z _{stem} = 323 A _{s_stem_des} = A _{s_stem_min} = A _{s_stem_req} = 20 mm dia A _{s_stem_prov} = PASS - Reinfo V _{stem} = V _{stem} V _{adm} = min(PASS -	$\int \frac{1}{2} \int $	•) = 340.0 mm z_u) = 0.038 mpression reim nin(K _{stem} , 0.225) y × Z _{stem}) = 1240 0 mm ² /m As_stem_min) = 124 n centres led at the retain 024 N/mm ² mm ²), 5) × 1 N/m stress is less that	• forcement is / 0.9)),0.95) × mm²/m 0 mm²/m ing wall stem m² = 5.000 N/ an maximum	not required d _{stem} n is adequate 'mm ² shear stress
Check wall stem in bending Width of wall stem Depth of reinforcement Constant Lever arm Area of tension reinforcement re Minimum area of tension reinforcement re Area of tension reinforcement re Reinforcement provided Area of reinforcement provided Check shear resistance at wall Design shear stress Allowable shear stress	• 175 • 175	b = 1000 m d _{stem} = t _{wall} - K _{stem} = M _{ste} Z _{stem} = min(Z _{stem} = 323 A _{s_stem_min} = A _{s_stem_req} = 20 mm dia A _{s_stem_prov} = PASS - Reinfo V _{stem} = V _{stem} V _{adm} = min(PASS - V _c stem = 0.6	$\int \frac{1}{2} \int $	• • • • • • • • • • • • • •	• forcement is / 0.9)),0.95) × mm²/m 0 mm²/m ing wall stem m² = 5.000 N/ an maximum	not required d _{stem} n is adequate 'mm ² shear stress
Check wall stem in bending Width of wall stem Depth of reinforcement Constant Lever arm Area of tension reinforcement re Minimum area of tension reinford Area of tension reinforcement re Reinforcement provided Area of reinforcement provided Area of reinforcement provided Exercision reinforcement rest Reinforcement provided Area of reinforcement provided From BS8110:Part 1:1997 – Ta Design concrete shear stress	• 175 • 175	b = 1000 m d _{stem} = t _{wall} - K _{stem} = M _{ste} Z _{stem} = M _{ste} Z _{stem} = 323 A _{s_stem_des} = A _{s_stem_min} = A _{s_stem_req} = 20 mm dia A _{s_stem_prov} = PASS - Reinfo V _{stem} = V _{ster} V _{adm} = min(PASS - V _{c_stem} = 0.6	$\int_{-C_{stem}} (\phi_{stem} / 2)$ $\int_{-C_{stem}} (b \times d_{stem}^2 \times f_0)$ $\int_{-C_{stem}} (b \times d_{stem}^2 \times f_0)$ $\int_{-C_{stem}} (0.87 \times f_0)$ $\int_{-C_{stem}} (0.87 \times f_0)$ $\int_{-C_{stem}} (0.87 \times f_0)$ $\int_{-C_{stem}} (b \times d_{stem}) = 0.$	•) = 340.0 mm su) = 0.038 pmpression reim nin(K _{stem} , 0.225) y × Z _{stem}) = 1240 0 mm ² /m As_stem_min) = 124 n centres led at the retain 024 N/mm ² mm ²), 5) × 1 N/m stress is less that v_{c_stem} - No she	• forcement is / 0.9)),0.95) × mm²/m 0 mm²/m 10 mm²/m 11 mm² = 5.000 N/ 12 mm² = 5.000 N/ 13 mm² = 5.000 N/ 14 mm² = 5.000 N/ 15 mm² = 5.000 N/ 16 mm² = 5.000 N/ 17 mm² = 5	not required d _{stem} is adequate 'mm ² shear stress
Check wall stem in bending Width of wall stem Depth of reinforcement Constant Lever arm Area of tension reinforcement reinford Area of tension reinforcement reinford Area of tension reinforcement reinford Area of tension reinforcement reinford Area of reinforcement provided Check shear resistance at wall Design shear stress Allowable shear stress From BS8110:Part 1:1997 – Ta Design concrete shear stress	• 175 • • • • • • • • • • • • • • • • • • •	b = 1000 m d _{stem} = t _{wall} - K _{stem} = M _{ste} Z _{stem} = min(Z _{stem} = 323 A _{s_stem_min} = A _{s_stem_req} = 20 mm dia A _{s_stem_prov} = PASS - Reinfo V _{stem} = V _{stem} V _{adm} = min(PASS - V _{c_stem} = 0.6	$\int \frac{1}{\sqrt{b^2 + b^2}} dx = \frac{1}{\sqrt{b^2 + b^2}} + $	•) = 340.0 mm z_{u}) = 0.038 pmpression reim $din(K_{stem}, 0.225)$ $y \times z_{stem}$) = 1240 0 mm ² /m $A_{s_stem_min}$) = 124 n centres led at the retain 024 N/mm ² mm ²), 5) × 1 N/m stress is less that v_{c_stem} - No she	• forcement is / 0.9)),0.95) × mm ² /m 0 mm ² /m ing wall stem m ² = 5.000 N/ an maximum par reinforcem	not required d _{stem} n is adequate 'mm ² shear stress nent required

Tekla Tedds	Project	38 Froa	Job no. 20080			
TAK Structures	Calcs for		Start page no./Revision			
10 Kennington Park Place	3.3m high retaining wall					7
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
5ETT 4A5	RH	12/05/2021				
Design service stress		$f_{n} = 2 \vee f_{n} \vee$	Δ. atom / /3. ·	$\Delta_{\rm extern} = -220$	1 2 N/mm ²	
Modification factor	factor = min((ns = 2 × 1y ×	$m_{s_{s_{e}}}^{2} = f_{s_{e}} / (120 \times 10^{2})$	$\Lambda s_{stem_prov} = 230$	$(b \times d_{44})$	() 2) = 1.40
Maximum span/effective denth	racionens – minio	ratio = ra	$r = r s / (r \ge 0 \times (r \ge 0))$	- = 9.83	tem/(D × Ustem)))),2) = 1 .40
Actual span/effective depth ratio		rationat = ha	$d_{\text{tiobas}} \times 12 \text{Cionten}$	s – 3.03		
		Tatioact Its		PASS - Span to	o depth ratio i	s acceptable
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Tekla Tedds	Project	38 Froc	unal I ane		Job no.	0080				
TAK Structures 10 Kennington Park Place	Calcs for	3.3m high i	Start page no./Revision 8							
London SE11 4AS	Calcs by RH	Calcs date 12/05/2021	Checked by	Checked date	Approved by	Approved date				
Indicativo rotaining wall roinfo	Indicative retaining wall reinforcement diagram									
Indicative retaining wall reinforcement diagram										
Tao minformenat				 ■ Stem rei 	nforcement					
Toe reinforcement-										
_										
Toe bars - 16 mm dia.@ 200 mn Stem bars - 20 mm dia.@ 175 m	n centres - (10	05 mm²/m) 795 mm²/m)								