MBP	Michael Barclay Partnership
	consulting engineers
	1 Lancaster Place WC2E 7ED
	T 020 7240 1191
	E london@mbp-uk.com
	www.mbp-uk.com

25 OAKHILL AVENUE, LONDON, NW3 7RD

Structural Engineer's Calculations for Planning

October 2022

Revision P1- Planning



Revision	Issued For	Date	Author
P1	PLANNING	20.10.2021	AZ

8536 - 25 OAKHILL AVENUE, LONDON NW3 7ED Structural Calculations

CONTENTS:

- 1 INTRODUCTION
- 2 RELEVANT DOCUMENTS
- 3 MBP STRUCTURAL DRAWINGS
- 4 SECOND FLOOR STRUCTURE
- 5 FIRST FLOOR STRUCTURE
- 6 GROUND FLOOR SLAB STRUCTURE
- 7 BASEMENT RETAINING WALLS

1. INTRODUCTION:

This project covers the design of the refurbishment and the new basement extension at No 25 Oakhill Avenue. The current calculation includes the design of a reinforced concrete basement slab, underpinning of party walls, design of lining walls and new concrete slab above basement to accommodate new Ground Floor layouts as well as the structural elements for the upper floors.

2. RELEVANT DOCUMENTS:

- Site geological investigation carried out by GEA Geotechnical Engineers.
- MBP's Construction Method Statement
- MBP's Specification for the works
- MBP's Structural Drawings for the works

3. STRUCTURAL DRAWINGS:

- MBP-8536-100- PROPOSED BASEMENT GENERAL ARRANGEMENT
- MBP-8536-101- PROPOSED GROUND FLOOR GENERAL ARRANGEMENT
- MBP-8536-102- PROPOSED FIRST FLOOR GENERAL ARRANGEMENT
- MBP-8536-103- PROPOSED SECOND FLOOR GENERAL ARRANGEMENT
- MBP-8536-104- PROPOSED THIRD FLOOR GENERAL ARRANGEMENT
- MBP-8536-103- PROPOSED ROOF GENERAL ARRANGEMENT
- MBP-8536-200- PROPOSED SECTION 1-1 GENERAL ARRANGEMENT
- MBP-8236-210- PROPOSED SECTION A-A GENERAL ARRANGEMENT
- MBP-8236-211- PROPOSED SECTION B-B GENERAL ARRANGEMENT

4. SECOND FLOOR STRCUTURE

The analysis and design of steel and timber elements has been carried out using TEDDS. The steel beams and timber joists has been designed to carry out the domestic loads. Results can be found in Section 4.

5. FIRST FLOOR STRUCTURE

The analysis and design of the Flat Roof steel elements has been carried out using TEDDS. The steel beams have been designed to support existing walls above and new flat roof structure with rooflights. Results can be found in Section 5.

6. GROUND FLOOR SLAB STRUCTURE

The ground floor will be reinforced concrete spanning between liner walls and internal loadbering elements, the maximum span for the slab to be 6.0m the verification has been carried out using TEDDS. Results can be found in Section 6.

7. BASEMENT RETAINING WALLS

The analysis and design of the RC liner walls, has been carried out using TEDDS. The liner walls have been designed to support ground floor load. Results can be found in Section 8.

MBP	Michael Barclay Partnership	Job Title 25 OAKHILL AVENUE, LONDON	Job Number 8536	Sheet Number 4.1	Revision P1
	consulting engineers				
	1 Lancaster Place WC2E 7ED	Calculation/sketch Title	Date	Author	Checked
	T 020 7240 1191 F 020 7240 2241	SECTION 4	OCT 2022	AZ	тн
	E london@mbp-uk.com	SECOND FLOOR STRUCTURE			

GEOMETRY



SECOND FLOOR PLAN

MBP	Michael Barclay Partnership	Job Title 25 OAKHILL AVENUE, LONDON		Sheet Number 4.2	Revision P1
	consulting engineers				
	1 Lancaster Place WC2E 7ED	Calculation/sketch Title	Date	Author	Checked
	T 020 7240 1191 F 020 7240 2241	SECTION 4	OCT 2022	AZ	тн
	E london@mbp-uk.com	SECOND FLOOR STRUCTURE			

	DEAD LOAD			
	0.50			
Floor finishes	0.50			
Boarding	0.15			
Timber joists	0.25			
Ceiling and services	0.25			
TOTAL	1.15	kN/m ²		
Solid brick wall	4.30	kN/m ²		
	IMPOSED LOAD			
Imposed Load (Including Partitions)	2.50	kN/m²		

M B P	Michael Barclay Partnership	Job Title 25 OAKHILL AVENUE, LONDON	Job Number 8536	Sheet Number 4.3	Revision P1
	consulting engineers				
	1 Lancaster Place WC2E 7ED	Calculation/sketch Title	Date	Author	Checked
	T 020 7240 1191 F 020 7240 2241	SECTION 4	OCT 2022	AZ	тн
	E london@mbp-uk.com	SECOND FLOOR STRUCTURE			

TYPICAL TIMBER JOISTS

Max. span 3.3m

Dead I	oad (excluding	self weight)

Imposed load 2.5 kN/m²

Loading

0.9 kN/m²

Tekla.	Tedds	Project 25 Oakhil	l Avenue					Job Ref. 8536		
		Section SECTION	14					Sheet no./rev. 4.4		
	Calc. by AZ	Date 03/	, 10/2022	Ch	k'd by	Date	App'd by TH	Date 03/10/202		
TIMBER JOIST A	NALYSIS & DE th EN1995-1-1	SIGN (EN19 2004 + A2:2	995-1-1:20 2014 inco	004) orporatir	ıg corr	rigendu	m June 2006 and th	e UK natio Tedds cal	nal annex culation version 1	
Description			17	v 200 C	18 timb	or inists				
			47	~ 200 C	mm		2			
			SJoi	st – 330						
				-3300						
Forces input on .	loist									
Vertical permanen	t load on joist		F _G	_Joist = 0.9	0 kN/m	1 ²				
Vertical imposed lo	oad on joist		Fq_	_Joist= 2.5	0 kN/n	1 ²				
Joist loading det	ails									
Distributed loads										
Vertical permanen	t load on joist		p _G	= F _{G_Joist}	imes SJoist	= 0.32 k	:N/m			
Vertical imposed lo	oad on joist		pq	= F _{Q_Joist}	imes S _{Joist}	= 0.88 k	:N/m			
ANALYSIS										
I a a dia a								Tedds cal	culation version 1	
Loading Self weight include	d (Permanent	x 1)								
		× 1)								
	II TACLOIS									
Load	combination		Permanen	Imposed	Snow	Wind				
35G + 1.50Q (Streng	gth)		1.35	1.50	0.00	0.00				
0G + 1.00Q (Servio	ce)		1.00	1.00	0.00	0.00				
Member Loads		Load	d Type	Orient	ation	Descr	ription			
Member Loads Member	Load case					0.001				
Member Loads Member Member	Load case Permanent	U	DL	Glob	balZ	0.32 K	(N/m at 0 m to 3.3 m)			

1.35G + 1.50Q (Strength) - Total deflection

1

× Z





Tekla Tedds	Project 25 Oakhill Ave	enue			Job Ref. 8536	
	Section SECTION 4	Sheet no./rev. 4. 7				
Calc. byDateChk'd byDateAZ03/10/202203/10/202203/10/2022				App'd by TH	Date 03/10/2022	
4—47—1	•	·		·		



47x200 timber section
Cross-sectional area, A, 9400 mm ²
Section modulus, W _v , 313333.3 mm ³
Section modulus, W _z , 73633 mm ³
Second moment of area, I, 31333333 mm ⁴
Second moment of area, Iz, 1730383 mm4
Radius of gyration, i _v , 57.7 mm
Radius of gyration, i _z , 13.6 mm
Timber strength class C18
Characteristic bending strength, f _{m.k} , 18 N/mm ²
Characteristic shear strength, f _{v.k} , 3.4 N/mm ²
Characteristic compression strength parallel to grain, f 18 N/mm ²
Characteristic compression strength perpendicular to grain, f. 90.k, 2.2 N/mm ²
Characteristic tension strength parallel to grain, f _{L0.k} , 10 N/mm ²
Mean modulus of elasticity, E _{0.mean} , 9000 N/mm ²
Fifth percentile modulus of elasticity, E _{0.05} , 6000 N/mm ²
Shear modulus of elasticity, G _{mean} , 560 N/mm ²
Characteristic density, ρ _k , 320 kg/m³
Mean density, ρ _{mean} , 380 kg/m³

Span details

Bearing length

Lb	=	100	mm
Lb	-	100	

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Bearing stress	N/mm ²	1.5	0.6	0.419	PASS
Bending stress	N/mm ²	12.2	7.7	0.634	PASS
Shear stress	N/mm ²	2.3	0.7	0.304	PASS
Deflection	mm	13.2	12.7	0.962	PASS

Consider Combination 1 - 1.35G + 1.50Q (Strength)

Modification factors

Duration of load and moisture content - Table 3.1	k _{mod} = 0.8
Deformation factor - Table 3.2	k _{def} = 0.8
Bending stress re-distribution factor - cl.6.1.6(2)	km = 0.7
Crack factor for shear resistance - cl.6.1.7(2)	k _{cr} = 0.67
System strength factor - cl.6.6	k _{sys} = 1.1

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis	F _{c,y,90,d} = 2.933 kN
Effective contact length	L _{b,ef} = L _b = 100 mm
Design perpendicular compressive stress - exp.6.4	$\sigma_{c,y,90,d}$ = F _{c,y,90,d} / (b × L _{b,ef}) = 0.624 N/mm ²
Design perpendicular compressive strength	$f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c.90.k} \ / \ \gamma_M = \textbf{1.489} \ N/mm^2$
	σ _{c,y,90,d} / (k _{c,90} × f _{c,y,90,d}) = 0.419

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7	
Design shear force	F _{y,d} = 2.933 kN
Design shear stress - exp.6.60	$\tau_{y,d} = 1.5 \times F_{y,d} \text{ / } (k_{cr} \times b \times h) = \textbf{0.699} \text{ N/mm}^2$
Design shear strength	$f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v.k} \ / \ \gamma_M = \textbf{2.302} \ N/mm^2$

Tekla Tedds	Project 25 Oakhill A	Avenue		Job Ref. 8536		
	Section SECTION 4	Section SECTION 4			Sheet no./rev. 4. 8	
	Calc. by AZ	Date 03/10/2022	Chk'd by	Date	App'd by TH	Date 03/10/2022
		$\tau_{y,d} / f_{v,y,d} = 0.3$	304	aboar atrana	th avaada daa	ian aboar atraa
Check design 1650 mm along s	span_	F	ASS - Design	snear streng	in exceeds desi	ign shear stres
Check bending moment - Sect	ion 6.1.6					
Design bending moment		M _{y,d} = 2.42 kM	۱m			
Design bending stress		$\sigma_{m,y,d} = M_{y,d} / T$	Wy = 7.723 N/r	mm²		
Design bending strength		$f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m,k} / \gamma_M = 12.185 \text{ N/mm}^2$				
		$\sigma_{mvd} / f_{mvd} = 0.634$				
		$\sigma_{m,y,d}$ / $f_{m,y,d}$ =	0.634			
		σ _{m,y,d} / f _{m,y,d} = PASS -	0.634 - Design bend	ling strength	exceeds design	bending stres
Consider Combination 2 - 1.00	G + 1.00Q (Se	σ _{m,y,d} / f _{m,y,d} = PASS · rvice)	0.634 - Design bend	ling strength	exceeds design	bending stres
Consider Combination 2 - 1.00 Check design 1650 mm along	G + 1.00Q (Se span	σ _{m,y,d} / f _{m,y,d} = PASS · rvice)	0.634 - Design bena	ling strength	exceeds design	bending stres
Consider Combination 2 - 1.00 Check design 1650 mm along Check y-y axis deflection - Sec	G + 1.00Q (Se span stion 7.2	σ _{m,y,d} / f _{m,y,d} = PASS · rvice)	0.634 - Design bend	ling strength	exceeds design	bending stres
Consider Combination 2 - 1.00 Check design 1650 mm along Check y-y axis deflection - Sec Instantaneous deflection	<u>G + 1.00Q (Sei</u> span ction 7.2	σ _{m,y,d} / f _{m,y,d} = <i>PASS</i> · <u>rvice)</u> δ _y = 7.1 mm	0.634 - Design bena	ling strength	exceeds design	bending stres
Consider Combination 2 - 1.00 Check design 1650 mm along Check y-y axis deflection - Sec Instantaneous deflection Quasi-permanent variable load fa	<mark>G + 1.00Q (Sel</mark> <u>span</u> s tion 7.2 actor	$\sigma_{m,y,d} / f_{m,y,d} =$ <i>PASS</i> <i>rvice</i>) $\delta_y = 7.1 \text{ mm}$ $\psi_2 = 0.3$	0.634 - Design bend	ling strength	exceeds design	bending stres
Consider Combination 2 - 1.00 Check design 1650 mm along = Check y-y axis deflection - Sec Instantaneous deflection Quasi-permanent variable load fa Final deflection with creep	G + 1.00Q (Se span tion 7.2 actor	$\sigma_{m,y,d} / f_{m,y,d} = PASS$ rvice) $\delta_y = 7.1 \text{ mm}$ $\psi_2 = 0.3$ $\delta_{y,Final} = \delta_y \times ($	0.634 - <i>Design bend</i> 1 + k _{def}) = 12.7	ling strength	exceeds design	bending stres
Consider Combination 2 - 1.00 Check design 1650 mm along Check y-y axis deflection - Sec Instantaneous deflection Quasi-permanent variable load fa Final deflection with creep Allowable deflection	G + 1.00Q (Se span stion 7.2 actor	$\sigma_{m,y,d} / f_{m,y,d} = PASS$ rvice) $\delta_y = 7.1 \text{ mm}$ $\psi_2 = 0.3$ $\delta_{y,Final} = \delta_y \times ($ $\delta_{y,Allowable} = L_m$	0.634 - Design bend 1 + k _{def}) = 12.7 1_s1 / 250 = 13 .	ing strength mm .2 mm	exceeds design	bending stres
Consider Combination 2 - 1.00 Check design 1650 mm along = Check y-y axis deflection - Sec Instantaneous deflection Quasi-permanent variable load fa Final deflection with creep Allowable deflection	G + 1.00Q (Se span tion 7.2 actor	$\sigma_{m,y,d} / f_{m,y,d} = PASS$ rvice) $\delta_y = 7.1 \text{ mm}$ $\psi_2 = 0.3$ $\delta_{y,Final} = \delta_y \times ($ $\delta_{y,Allowable} = L_m$ $\delta_{y,Final} / \delta_{y,Allowable}$	0.634 - <i>Design bend</i> 1 + k _{def}) = 12.7 1_s1 / 250 = 13 able = 0.962	ling strength mm .2 mm	exceeds design	bending stres

MBP	Michael Barclay Partnership	Job Title 25 OAKHILL AVENUE, LONDON	Job Number 8536	Sheet Number 4.9	Revision P1
	consulting engineers				
	1 Lancaster Place WC2E 7ED	Calculation/sketch Title	Date	Author	Checked
	T 020 7240 1191 F 020 7240 2241	SECTION 4	OCT 2022	AZ	тн
	E london@mbp-uk.com	SECOND FLOOR STRUCTURE			

TYPICAL STEEL BEAM B1

Max. span 3.3m

Load attracted from 6.5m/2 = 3.25m

Dead load

Loading $1.15 \times 3.25 = 3.74 \text{ kN/m}$

Imposed load $2.50 \times 3.25 = 8.12 \text{ kN/m}$

Tekla Tedds	Project 25 OAKHILL AVENUE				Job Ref. 8536	
	Section SI SUPER STRUCTURE ELEMENTS 4				Sheet no./rev. 4. 10	
	Calc. by AZ	Date 03/10/2022	Chk'd by	Date	App'd by TH	Date 03/10/2022

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex



Tekla Tedds	Project 25 OAKHILL	Job Ref. 8536	Job Ref. 8536				
	Section SUPER STR	UCTURE ELEME	NTS		Sheet no./rev. 4. 11	Sheet no./rev. 4. 11	
	Calc. by AZ	Date 03/10/2022	Chk'd by	Date	App'd by TH	Date 03/10/2022	
		Support B		Variable > Permane Variable >	< 1.50 nt × 1.35 < 1.50		
Maximum moment Maximum shear Deflection Maximum reaction at support A Unfactored permanent load react Unfactored variable load reaction Maximum reaction at support B	Analysis results Maximum moment Maximum shear Deflection Maximum reaction at support A Unfactored permanent load reaction at support A Unfactored variable load reaction at support A		$M_{max} = 20.1 \text{ kNm}$ $V_{max} = 26.8 \text{ kN}$ $\delta_{max} = 0.9 \text{ mm}$ $R_{A_{max}} = 26.8 \text{ kN}$ $R_{A_{Permanent}} = 6.3 \text{ kN}$ $R_{A_{Variable}} = 12.2 \text{ kN}$		$M_{min} = 0 \text{ kNm}$ $V_{min} = -26.8 \text{ kN}$ $\delta_{min} = 0 \text{ mm}$ $R_{A_{min}} = 26.8 \text{ kN}$		
Unfactored permanent load reac Unfactored variable load reactior	tion at support B n at support B	$R_{B_{Permanent}} = 6$ $R_{B_{Variable}} = 12$	3.3 kN . 2 kN				
Section details Section type Steel grade EN 10025-2:2004 - Hot rolled p Nominal thickness of element Nominal yield strength Nominal ultimate tensile strength Modulus of elasticity	roducts of structure 1_{1}^{1}	UC 203x203x S275 ctural steels t = max(t _f , t _w) = f _y = 275 N/mm f _u = 410 N/mm E = 210000 N	46 (BS4-1) = 11.0 mm h ² h ² /mm ²				
Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to instat Resistance of tensile members to	ility o fracture	γ _{M0} = 1.00 γ _{M1} = 1.00 γ _{M2} = 1.10					
Lateral restraint		Span 1 has fu	ll lateral restra	aint			
Effective length factors Effective length factor in major a	kis	K _y = 1.000					

Tekia . Tedas	Project 25 OAKHIL	L AVENUE			Job Ref. 8536	
	Section		NTS		Sheet no./rev. 4. 12	
	Calc. by AZ	Date 03/10/2022	Chk'd by	Date	App'd by TH	Date 03/10/202
Effective length factor in minor a	xis	K _z = 1.000				
Effective length factor for torsion		K _{LT.A} = 1.000				
		K _{LT.B} = 1.000				
Classification of cross section	s - Section 5.5	; /				
		ε = √[235 N/m	m ² / f _y] = 0.92			
Internal compression parts sul	bject to bendi	ng - Table 5.2 (she	eet 1 of 3)			
Width of section		c = d = 160.8	mm			
		c / $t_w = 24.2 \times$	ε <= /2×ε	Class 1		
Outstand flanges - Table 5.2 (s	heet 2 of 3)					
Width of section		c = (b - t _w - 2 >	< r) / 2 = 88 mi	m		
		c / t _f = $8.7 \times \epsilon$	<= 9 × ε	Class 1	C = =	tion io alo
					Sec	tion is clas
Check shear - Section 6.2.6		h h Out	404.0			
Height of web		$n_w = n - 2 \times t_f$	= 181.2 mm			
Shear area lactor		$\eta = 1.000$	10			
		$\Pi_W / I_W < TZ \times 3$	571	Shear bucklin	a resistance c	an he ionc
Design shear force		V _{Ed} = max(ab	s(V _{max}), abs(V _r	_{min})) = 26.8 kN	g reolotanoe e	un be igne
Shear area - cl 6.2.6(3)		$A_v = max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1698 \text{ mm}^2$				
Design shear resistance - cl 6.2.6	6(2)	$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 269.5 \text{ kN}$				
		PA	SS - Design s	shear resistance	exceeds desi	gn shear fo
Check bending moment major	(y-y) axis - Se	ction 6.2.5				
Design bending moment		M _{Ed} = max(ab	s(M _{s1_max}), abs	s(M _{s1_min})) = 20.1 k	Nm	
Design bending resistance mome	ent - eq 6.13	$M_{c,Rd} = M_{pl,Rd}$	= $W_{pl.y} \times f_y / \gamma_{MO}$	o = 136.8 kNm		
	PA	SS - Design bend	ling resistanc	e moment excee	eds design bei	nding mon
Check vertical deflection - Sec	tion 7.2.1					
	le loads					
Consider deflection due to variab		0 1 1000				
Consider deflection due to variab Limiting deflection		$\delta_{\text{lim}} = L_{s1} / 360$	= 8.3 mm	0.000		
Consider deflection due to variab Limiting deflection Maximum deflection span 1		$\delta_{\text{lim}} = L_{s1} / 360$ $\delta = \max(abs)(\delta - bb)$	= 8.3 mm δ _{max}), abs(δ _{min}))) = 0.893 mm	a not avaard a	laflaction l

MBP	Michael Barclay Partnership	Job Title 25 OAKHILL AVENUE, LONDON	Job Number 8536	Sheet Number 4.13	Revision P1
	consulting engineers				
	1 Lancaster Place WC2E 7ED	Calculation/sketch Title	Date	Author	Checked
	T 020 7240 1191 F 020 7240 2241	SECTION 4	OCT 2022	AZ	тн
	E london@mbp-uk.com	SECOND FLOOR STRUCTURE			

FRAME A

Max. span 1.4m Load attracted from 11m/2 = 5.05mHight of the wall - 2.6m

Loading on top memeber

	Dead load	Imposed load
Floor Loading	$1.15 \times 5.05 = 3.74 \text{ kN/m}$	$2.50 \times 0.5 = 8.12 \text{ kN/m}$
Wall	$4.30 \times 2.60 = 6.90 \text{ kN/m}$	
Tota	10.64 kN/m	8.12 kN/m

Tekla Tedds	Project 25 OAKHILL AVENUE				Job Ref. 8536	
	SectionSISUPER STRUCTURE ELEMENTS4				Sheet no./rev. 4. 14	
	Calc. by AZ	Date 10/10/2022	Chk'd by	Date	App'd by TH	Date 03/10/2022

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex



Tekla Tedds	Project 25 OAKHILL A	VENUE	Job Ref. 8536			
	Section SUPER STRU		INTS		Sheet no./rev 4. 15	
	Calc. by AZ	Date 10/10/2022	Chk'd by	Date	App'd by TH	Date 03/10/2022
Analycis rosults		Support B		Variabl Permai Variabl	e × 1.50 nent × 1.35 e × 1.50	
Maximum moment Maximum shear Deflection Maximum reaction at support A Unfactored permanent load react	tion at support A	$M_{max} = 6.6 \text{ kN}$ $V_{max} = 18.9 \text{ kl}$ $\delta_{max} = 0 \text{ mm}$ $R_{A_max} = 18.9$ $R_{A_permanent} = 7$	m N KN 7.7 kN	$M_{min} = 0$ $V_{min} = -$ $\delta_{min} = 0$ $R_{A_{min}} = -$	0 kNm 18.9 kN 0 mm = 18.9 kN	
Maximum reaction at support B Unfactored permanent load react Unfactored variable load reaction	tion at support B at support B	$RA_Variable = 5.7$ $R_B_max = 18.9$ $R_B_Permanent = 7$ $R_B_Variable = 5.7$	7 KN KN 7.7 KN 7 KN	R _{B_min} =	= 18.9 kN	
Section details Section type Steel grade EN 10025-2:2004 - Hot rolled provide the two sectors of element Nominal thickness of element Nominal yield strength Nominal ultimate tensile strength Modulus of elasticity	Maximum reaction at support B Unfactored permanent load reaction at support B Unfactored variable load reaction at support B Section details Section type Steel grade EN 10025-2:2004 - Hot rolled products of structure Nominal thickness of element Nominal yield strength Nominal ultimate tensile strength Modulus of elasticity		$R_{A_{variable}} = 5.7 \text{ kN}$ $R_{B_{max}} = 18.9 \text{ kN}$ $R_{B_{max}} = 18.9 \text{ kN}$ $R_{B_{min}} = 14$ $R_{B_{permanent}} = 7.7 \text{ kN}$ $UB 254x146x37 (BS4-1)$ $S275$ $UB 254x146x37 (BS4-1)$ $S275$ $t = max(t_{r}, t_{w}) = 10.9 \text{ mm}$ $f_{y} = 275 \text{ N/mm}^{2}$ $f_{u} = 410 \text{ N/mm}^{2}$ $E = 210000 \text{ N/mm}^{2}$			
Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to instab Resistance of tensile members to Lateral restraint	illity o fracture	γ _{M0} = 1.00 γ _{M1} = 1.00 γ _{M2} = 1.10 Span 1 has fu	ıll lateral restr	aint		
Effective length factors Effective length factor in major a	xis	K _y = 1.000				

	Project 25 OAKHILL	AVENUE			Job Ref. 8536		
	Section SUPER STR		NTS		Sheet no./rev. 4. 16		
	Calc. by AZ	Date 10/10/2022	Chk'd by	Date	App'd by TH	Date 03/10/202	
Effective length factor in minor a	xis	K _z = 1.000					
Effective length factor for torsion		K _{LT.A} = 1.000					
		K _{LT.B} = 1.000					
Classification of cross section	s - Section 5.5	1					
		ε = √[235 N/m	m ² / f _y] = 0.92				
Internal compression parts sub	bject to bending	g - Table 5.2 (she	et 1 of 3)				
Width of section		c = d = 219 m	m – –				
		c / $t_w = 37.6 \times$	ε <= 72 × ε	Class 1			
Outstand flanges - Table 5.2 (s	heet 2 of 3)						
Width of section		c = (b - t _w - 2 >	< r) / 2 = 62.5 r	mm			
		c / $t_f = 6.2 \times \epsilon$	<= 9 × ε	Class 1	•		
					Sec	tion is clas	
Check shear - Section 6.2.6							
Height of web		$h_w = h - 2 \times t_f = 234.2 \text{ mm}$					
Shear area factor		$\eta = 1.000$					
		$h_w / t_w < 72 \times \epsilon$	ε/η	Shoorbucklin	a registeres a	on ho iana	
Design shear force		V _{⊏d} = max(abs	s(Vmay) abs(Vr	Silear bucking	g resistance c	an be igno	
Shear area - cl 6.2.6(3)		$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f \cdot n \times h_w \times t_w) = 1759 \text{ mm}^2$					
Design shear resistance - cl 6.2.6	6(2)	$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_v / \sqrt{[3]}) / \gamma_{M0} = 279.3 \text{ kN}$					
0		PA	SS - Design s	shear resistance	exceeds desig	gn shear fo	
Check bending moment major	(y-y) axis - Sec	tion 6.2.5					
Design bending moment		M _{Ed} = max(ab	s(M _{s1_max}), abs	s(M _{s1_min})) = 6.6 kN	١m		
Design bending resistance mome	ent - eq 6.13	$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = \textbf{132.9 kNm}$					
	PAS	SS - Design bena	ling resistanc	e moment excee	eds design bei	nding mon	
Check vertical deflection - Sec	tion 7.2.1						
Consider deflection due to veriab	le loads						
Consider deflection due to variab		S I /000	= 3.9 mm				
Limiting deflection		$\delta_{\text{lim}} = L_{s1} / 360$					
Limiting deflection Maximum deflection span 1		$\delta_{\text{lim}} = L_{s1} / 360$ $\delta = \max(abs)(\delta)$	max), abs(δ _{min}))) = 0.035 mm			

MBP	Michael Barclay Partnership	Job Title 25 OAKHILL AVENUE, LONDON	Job Number 8536	Sheet Number 5.1	Revision P1
	consulting engineers				
	1 Lancaster Place WC2E 7ED	Calculation/sketch Title	Date	Author	Checked
	T 020 7240 1191 F 020 7240 2241	SECTION 5	OCT 2022	AZ	тн
	E london@mbp-uk.com	FIRST FLOOR STRUCTURE			

GEOMETRY



FIRST FLOOR PLAN

MBP	Michael Barclay Partnership	Job Title 25 OAKHILL AVENUE, LONDON	Job Number 8536	Sheet Number 5.2	Revision P1
	consulting engineers				
	1 Lancaster Place WC2E 7ED	Calculation/sketch Title	Date	Author	Checked
	T 020 7240 1191 F 020 7240 2241	SECTION 5	OCT 2022	AZ	тн
	E london@mbp-uk.com	FIRST FLOOR STRUCTURE			

			DEAD LOAD
FLAT ROOF			
Flat roof finishes		0.50	
Waterproofing		0.20	
Insulation		0.10	
Timber joists		0.25	
Ceiling and services		0.25	
	TOTAL	1.30	kN/m ²
FINST FLOOR		0.25	
Poording		0.25	
Timber joists		0.15	
Ceiling and services		0.25	
	TOTAL	0.20	kN/m ²
Solid brick wall		4.30	kN/m²
		IIV	MPOSED LOAD
Imposed Load		2 50	kN/m ²

M B P	Michael Barclay Partnership	Job Title 25 OAKHILL AVENUE, LONDON	Job Number 8536	Sheet Number 5.3	Revision P1
	consulting engineers				
	1 Lancaster Place WC2E 7ED	Calculation/sketch Title	Date	Author	Checked
	T 020 7240 1191 F 020 7240 2241	SECTION 5	OCT 2022	AZ	тн
	E london@mbp-uk.com	FIRST FLOOR STRUCTURE			

TYPICAL TIMBER JOISTS

Max. span 3.3m

	Dead load (excluding self weight)	Imposed load
Loading	0.9 kN/m ²	5.5 kN/m ²

Please refer to Section 4 for Timber Joists calculation.

TYPICAL STEEL BEAM B1

Max. span 2.9m Load attracted from 6.5m/2 = 3.25m

Dead load		Imposed load		
Loading	$1.15 \times 3.25 = 3.74 \text{ kN/m}$	2.50 x 3.25 = 8.12 kN/m		

Please refer to Section 4 for similar beam B1 callculation.

MBP	Michael Barclay Partnership	Job Title 25 OAKHILL AVENUE, LONDON	Job Number 8536	Sheet Number 5.4	Revision P1
	consulting engineers				
	1 Lancaster Place WC2E 7ED	Calculation/sketch Title	Date	Author	Checked
	T 020 7240 1191 F 020 7240 2241	SECTION 5	OCT 2022	AZ	тн
	E london@mbp-uk.com	FIRST FLOOR STRUCTURE			

2No STEEL BEAM B2

Max. span 4.0m Load attracted from timber floor 3.2m/2 = 1.6mLoad attracted from Flat Roof 1.9m/2 = 0.8m

-	TOTAL	2.64 kN/m	6.25 kN/m
Flat Roof	$1.3 \times 0.9 =$	1.20 kN/m	$2.50 \times 0.9 = 2.25 \text{ kN/m}$
First Floor	$0.9 \times 1.6 =$	1.44 kN/m	$2.50 \times 1.6 = 4.00 \text{ kN/m}$
	Dead load		Imposed load

Solid brick wall $4.30 \times 6.5m = 28.00 \text{ kN/m}$

Tekla Tedds	Project 25 OAKHILL AVENUE				Job Ref. 8536	
	Section FIRST FLOOR STRUCTURE			Sheet no./rev. 5. 5		
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 03/10/2022

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex



Tekla Tedds	Project Job Ref. 25 OAKHILL AVENUE 8536					
	Section FIRST FLOO	R STRUCTURE			Sheet no./rev. 5. 6	
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 03/10/2022
		Support B		Variable Perman Variable	e × 1.50 ent × 1.35 e × 1.50	
Analysis results Maximum moment Maximum shear Deflection Maximum reaction at support A Unfactored permanent load react Unfactored variable load reaction Maximum reaction at support B	tion at support A n at support A	$M_{max} = 103.4 \text{ k}$ $V_{max} = 103.4 \text{ k}$ $\delta_{max} = 0.9 \text{ mm}$ $R_{A_max} = 103.4 \text{ k}$ $R_{A_max} = 103.4 \text{ k}$ $R_{A_Permanent} = 6 \text{ k}$ $R_{A_Variable} = 12 \text{ k}$ $R_{B_max} = 103.4 \text{ k}$	kNm KN 62.7 kN 5 kN 4 kN	Mmin = 0 Vmin = -1 δmin = 0 RA_min = RB_min =	kNm 03.4 kN mm 103.4 kN 103.4 kN	
Unfactored permanent load reaction Unfactored variable load reaction Section details	n at support B	RB_Permanent = C RB_Variable = 12	.5 kN			
Section type Steel grade EN 10025-2:2004 - Hot rolled p	roducts of struc	2 x UB 254x1 S275 tural steels	46x37 (BS4-1)		
Nominal thickness of element Nominal yield strength Nominal ultimate tensile strength Modulus of elasticity		$\begin{split} t &= \max(t_{\rm f}, t_{\rm w}) : \\ f_y &= 275 \; \text{N/mm} \\ f_u &= 410 \; \text{N/mm} \\ \text{E} &= 210000 \; \text{N} \end{split}$	= 10.9 mm 1 ² 1 ² /mm ²			
	→ → → 256 → 10.9	→ ←5.3				
	+	—146.4 —— ►				
Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to instab Resistance of tensile members to	ility o fracture	γмо = 1.00 γм1 = 1.00 γм2 = 1.10				
Lateral restraint		Span 1 has fu	ll lateral restra	int		
Effective length factors Effective length factor in major as	kis	K _y = 1.000				

	25 OAKHILL A	VENUE			Job Ref. 8536		
	Section				Sheet no./rev.		
	FIRST FLOOP	RSTRUCTURE	1 1		5. 7		
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 03/10/2022	
Effective length factor in minor a	xis	Kz = 1.000					
Effective length factor for torsion		KLT.A = 1.000					
		Klt.b = 1.000					
Classification of cross sectior	s - Section 5.5						
		ε = √[235 N/m	m ² / f _y] = 0.92				
Internal compression parts su	bject to bending	- Table 5.2 (she	et 1 of 3)				
Width of section		c = d = 219 m	m				
		c / tw = 37.6 \times	ε <= 72 × ε	Class 1			
Outstand flanges - Table 5.2 (s	sheet 2 of 3)						
Width of section		c = (b - t _w - 2 >	r) / 2 = 62.5 mr	n			
		c / tf = $6.2 \times \epsilon$	<= 9 × ε	Class 1			
					Sec	ion is class 1	
Check shear - Section 6.2.6							
Height of web		$h_w = h - 2 \times t_f =$	= 234.2 mm				
Shear area factor		η = 1.000					
		h _w / t _w < 72 × ε	: / ŋ				
				Shear buckling	g resistance ca	an be ignored	
Design shear force		V _{Ed} = max(abs	(Vmax), abs(Vmin)) = 103.4 kN		<u>_</u>	
Shear area - cl 6.2.6(3)		$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1759 \text{ mm}^2$					
Design shear resistance - cl 6.2.	6(2)	$V_{c,Rd} = V_{pl,Rd} = N \times A_V \times (f_y / V[3]) / \gamma_{M0} = 558.7 \text{ KN}$					
		PA	55 - Design she		exceeds desig		
Check bending moment major	(y-y) axis - Secti	on 6.2.5			le N Iroo		
Design bending moment	opt og 6 12	$M_{Ed} = M_{AE} (dDS(Ms1_max), dDS(Ms1_min)) = 103.4 \text{ kNM}$					
Design bending resistance mon	PASS	S - Design bending resistance moment exceeds design bending moment					
		2 conginisoria	ing roototanoo		ao aoongni bon	aniginoni	
Chack vortical deflection - Sec	tion 7 2 1						
Check vertical deflection - Sec	tion 7.2.1						
Check vertical deflection - Sec Consider deflection due to varial Limiting deflection	tion 7.2.1 ble loads	δlim = Ls1 / 360	= 11.1 mm				
Check vertical deflection - Sec Consider deflection due to varial Limiting deflection Maximum deflection span 1	t ion 7.2.1 ble loads	$\delta_{\text{lim}} = L_{s1} / 360$ $\delta = max(abs(\delta))$	= 11.1 mm _{max}), abs(δ _{min})) =	0.896 mm			

MBP	Michael Barclay Partnership	Job Title 25 OAKHILL AVENUE, LONDON	Job Number 8536	Sheet Number 5.8	Revision P1
	consulting engineers				
	1 Lancaster Place WC2E 7ED	Calculation/sketch Title	Date	Author	Checked
	T 020 7240 1191 F 020 7240 2241	SECTION 5	OCT 2022	AZ	тн
	E london@mbp-uk.com	FIRST FLOOR STRUCTURE			

TYPICAL STEEL BEAM B3

Max. span 5.6m

Load attracted from 4.6m/2 = 2.3m

Dead load

Loading 1.30 x 2.30 = 3.00 kN/m

Imposed load $2.50 \times 2.30 = 5.75 \text{ kN/m}$

Tekla Tedds	Project 25 OAKHILL AVENUE				Job Ref. 8536	
	Section FIRST FLOOR	STRUCTURE		Sheet no./rev. 5. 9		
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 21/10/2022

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex



Ə Tekla Tedds	Project 25 OAKHILL A	AVENUE			Job Ref. 8536		
	Section FIRST FLOOF	R STRUCTURE			Sheet no./rev. 5. 10		
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 21/10/2022	
Analysis results		Support B		Variabl Permar Variabl	e × 1.50 nent × 1.35 e × 1.50		
Maximum moment Maximum shear Deflection Maximum reaction at support A Unfactored permanent load react Unfactored variable load reaction Maximum reaction at support B Unfactored permanent load react	ion at support A at support A ion at support B	$M_{max} = 52.1 \text{ kl}$ $V_{max} = 37.2 \text{ kl}$ $\delta_{max} = 7.7 \text{ mm}$ $R_{A_max} = 37.2$ $R_{A_Permanent} = 9$ $R_{A_Variable} = 16$ $R_{B_max} = 37.2$ $R_{B_Permanent} = 9$	Nm N KN 0.7 kN .1 kN KN 0.7 kN	$\begin{split} M_{min} &= 0 \\ V_{min} &= - \\ \delta_{min} &= 0 \\ R_{A_min} &= \\ R_{B_min} &= \end{split}$	0 kNm 37.2 kN mm = 37.2 kN = 37.2 kN		
Unfactored variable load reaction Section details Section type Steel grade EN 10025-2:2004 - Hot rolled pr Nominal thickness of element	at support B	R _{B_Variable} = 16 UC 203x203x S275 tural steels t = max(t _f , t _w):	.1 kN 46 (BS4-1) = 11.0 mm				
Nominal yield strength Nominal ultimate tensile strength Modulus of elasticity		f _y = 275 N/mm f _u = 410 N/mm E = 210000 N	1 ² 1 ² /mm ²				
			.2				
Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to instab Resistance of tensile members to	,∎ ility o fracture	γ _{M0} = 1.00 γ _{M1} = 1.00 γ _{M2} = 1.10		-1			
Lateral restraint Effective length factors	in	Span 1 has fu	ll lateral restra	aint			

	25 OAKHIL	Project 25 OAKHILL AVENUE				Job Ref. 8536	
	Section FIRST FLC	OR STRUCTURE			Sheet no./rev. 5. 11		
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 21/10/202	
Effective length factor in minor a	kis	K _z = 1.000					
Effective length factor for torsion		K _{LT.A} = 1.000 K _{LT.B} = 1.000					
Classification of cross section	s - Section 5.	5					
		ε = √[235 N/m	1m ² / f _y] = 0.92				
Internal compression parts sul	oject to bendi	ng - Table 5.2 (sh	eet 1 of 3)				
Width of section		c = d = 160.8	mm				
		c / $t_w = 24.2 \times$	$\varepsilon <= /2 \times \varepsilon$	Class 1			
Outstand flanges - Table 5.2 (s	heet 2 of 3)						
Width of section		c = (b - t _w - 2 :	× r) / 2 = 88 m	m			
		c / t _f = $8.7 \times \epsilon$	<= 9 × ε	Class 1	0		
					260	ction is clas	
Check shear - Section 6.2.6			404.0				
Height of web		$n_w = n - 2 \times t_f$	= 181.2 mm				
Shear area factor		$\eta = 1.000$	- m				
		$\Pi_W / I_W < TZ \times S$	8/1	Shear bucklin	a resistance i	ran he iana	
Design shear force		V _{Ed} = max(ab	s(V _{max}), abs(V _l	min)) = 37.2 kN	g resistance (an be igne	
Shear area - cl 6.2.6(3)		$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1698 \text{ mm}^2$					
Design shear resistance - cl 6.2.0	6(2)	$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 269.5 \text{ kN}$					
		PA	SS - Design s	shear resistance	exceeds desi	gn shear fo	
Check bending moment major	(y-y) axis - Se	ection 6.2.5					
Design bending moment		M _{Ed} = max(ab	s(M _{s1_max}), abs	s(M _{s1_min})) = 52.1 k	٨Mm		
Design bending resistance mome	ent - eq 6.13	$M_{c,Rd} = M_{pl,Rd}$	= $W_{pl.y} \times f_y / \gamma_{M0}$	₀ = 136.8 kNm			
	PA	ASS - Design bend	ding resistand	e moment excee	eds design be	nding mon	
Check vertical deflection - Sec	tion 7.2.1						
Consider deflection due to variab	le loads						
Limiting deflection		$\delta_{\text{lim}} = L_{s1} / 360$) = 15.6 mm				
		δ = max(abs(ð	δ_{max}), abs(δ_{min})) = 7.676 mm			
Maximum deflection span 1							

MBP	Michael Barclay Partnership	Job Title 25 OAKHILL AVENUE, LONDON	Job Number 8536	Sheet Number 6.1	Revision P1
	consulting engineers				
	1 Lancaster Place WC2E 7ED	Calculation/sketch Title	Date	Author	Checked
	T 020 7240 1191 F 020 7240 2241	SECTION 6	OCT 2022	AZ	тн
	E london@mbp-uk.com	GROUND FLOOR SLAB STRUCTUF	RE		

GEOMETRY



GROUND FLOOR PLAN

MBP	Michael Barclay Partnership	Job Title 25 OAKHILL AVENUE, LONDON	Job Number 8536	Sheet Number 6.2	Revision P1	
	consulting engineers					
	1 Lancaster Place WC2E 7ED	Calculation/sketch Title	Date	Author	Checked	
	T 020 7240 1191 F 020 7240 2241	SECTION 6	OCT 2022	AZ	тн	
	E london@mbp-uk.com	GROUND FLOOR SLAB STRUCTURE				

		DEAD LOAD			
Floor finishes	0.50				
100mm Screed	2.20				
250mm RC Slab	6.00				
Ceiling and services	0.50				
TOTAL	9.20	kN/m ²			
New cavity wall	4.88	kN/m²			
IMPOSED LOAD					
Imposed Load (Including Partitions)	2.50	kN/m ²			
······································	•				

Tekla Tedds	Project 25 OAKHILL AVENUE				Job Ref. 8536		
	Section GROUND FLO	Section GROUND FLOOR STRUCTURE				Sheet no./rev. 6. 3	
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 21/10/2022	

RC SLAB DESIGN

In accordance with EN1992-1-1:2004 incorporating corrigendum January 2008 and the UK national annex

Tedds calculation version 1.0.21

Design summary					
Description	Unit	Provided	Required	Utilisation	Result
Short span					
Reinf. at midspan	mm²/m	1005	311	0.309	PASS
Bar spacing at midspan	mm	200	300	0.667	PASS
Shear at discont. supp	kN/m	103.9	23.8	0.229	PASS
Deflection ratio		27.83	39.31	0.708	PASS
Long span					
Reinf. at midspan	mm²/m	1005	261	0.260	PASS
Bar spacing at midspan	mm	200	300	0.667	PASS
Shear at discont. supp	kN/m	99.0	23.8	0.240	PASS
Cover					
Min cover bottom	mm	30	26	0.867	PASS



|--|

Slab reference name	250mm RC Slab
Type of slab	Two way spanning with restrained edges
Overall slab depth	h = 250 mm
Shorter effective span of panel	l _x = 5900 mm
Longer effective span of panel	l _y = 9500 mm
Support conditions	Four edges discontinuous
Bottom outer layer of reinforcement	Short span direction
Loading	
Characteristic permanent action	G _k = 3.2 kN/m ²
Characteristic variable action	Q _k = 2.5 kN/m ²
Partial factor for permanent action	γ _G = 1.35
Partial factor for variable action	γ _Q = 1.50
Quasi-permanent value of variable action	$\psi_2 = 0.30$
Design ultimate load	$q = \gamma_G \times G_k + \gamma_Q \times Q_k = \textbf{8.1} \text{ kN/m}^2$
Quasi-permanent load	$q_{\text{SLS}} = 1.0 \times G_k + \psi_2 \times Q_k = \textbf{4.0} \ kN/m^2$

Tekla. Tedds	25 OAKHILL	AVENUE	Job Ref. 8536					
	Section GROUND FL	OOR STRUCTU	RE		Sheet no./rev. 6. 4	Sheet no./rev. 6. 4		
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 21/10/20		
Concrete properties								
Concrete strength class		C25/30						
Characteristic cylinder strength		f _{ck} = 25 N/mm	2					
Partial factor (Table 2.1N)		γc = 1.50						
Compressive strength factor (cl. 3	8.1.6)	α _{cc} = 0.85						
Design compressive strength (cl.	3.1.6)	f _{cd} = 14.2 N/m	m²					
Mean axial tensile strength (Table	9.1)	f _{ctm} = 0.30 N/n	$1000 m^2 imes$ (f _{ck} / 1 N/r	mm²) ^{2/3} = 2.6 ۱	N/mm²			
Maximum aggregate size		d _g = 20 mm						
Reinforcement properties								
Characteristic yield strength		f _{yk} = 500 N/mr	n²					
Partial factor (Table 2.1N)		γs = 1.15						
Design yield strength (fig. 3.8)		$f_{yd} = f_{yk} / \gamma_S = 4$	34.8 N/mm ²					
Concrete cover to reinforcemer	nt							
Nominal cover to outer bottom rei	nforcement	c _{nom_b} = 30 mr	n					
Fire resistance period to bottom o	f slab	$R_{btm} = 60 \text{ min}$						
Axia distance to bottom reinft (Table 5.8)		$a_{f_{1}b} = 15 \text{ mm}$						
Min. btm cover requirement with r	egard to bond	c _{min,b_b} = 16 m	m					
Reinforcement fabrication		Not subject to QA system						
Cover allowance for deviation Min. required nominal cover to bottom reinft		$\Delta c_{dev} = 10 \text{ mm}$						
		PASS - There is sufficient cover to the bottom reinforcen						
Reinforcement design at midsp	an in short spa	n direction (cl.6	(1)					
Bending moment coefficient		β _{sx p} = 0.0968	,					
Design bending moment		$M_{x,p} = \beta_{xx,p} \times 0$	a × l _v ² = 27.2 kN	lm/m				
Reinforcement provided		16 mm dia, ba	ars at 200 mm c	entres				
Area provided		$A_{sx,p} = 1005 \text{ n}$	nm²/m					
Effective depth to tension reinforce	ement	$d_{x,p} = h - c_{pom}$	$h - \Phi_{x,p} / 2 = 212$	2.0 mm				
K factor		$K = M_{x n} / (b \times c)$	$d_{x} p^{2} \times f_{ck} = 0.0$	024				
Redistribution ratio		$\delta = 1.0$						
K' factor		$K' = 0.598 \times 8$	$-0.18 \times \delta^2 - 0.2$	21 = 0 208				
N lactor		K = 0.590 × 0	<i>K < K' - 0</i>	21 – 0.200 Compression	reinforcement	is not reau		
l ever arm		z = min(0.95)	(dv a dv a/2 × ($1 + \sqrt{1 - 353}$	× K))) = 201 4 m	m		
Area of reinforcement required for	r bending	$\Delta = \min(0.30)$	$(4x_{p}, 4x_{p}/2 \times ($	mm^{2}/m	× (())) = 201. 4 fi			
Area of reinforcement required for bending		$A_{sx_p} = M_{x_p} / (T_{yd} \times Z) = 311 \text{ mm}^2/\text{m}$						
Minimum area of reinforcement required		$A_{sx_p_min} = max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{x_p}, 0.0013 \times b \times d_{x_p}) = 283 \text{ mm}^2/\text{m}$						
Area or remorcement required		A _{sx_p_req} – max PAS	s - Area of rein	nin) – 311 mm-/ nforcement pr	rovided exceed	s area requ		
Check reinforcement spacing				-				
		$\sigma_{sx p} = (f_{vk} / \gamma_s)$)×min((A _{sx pm} /	A _{sx p}), 1.0) × a	lsLs / q = 65.8 N	/mm ²		
Reinforcement service stress			, (<u>· ·ov_p_</u> iii)	··, ···-, ···				
Reinforcement service stress Maximum allowable spacing (Tab	le 7.3N)	$S_{max x p} = 300$	mm					
Reinforcement service stress Maximum allowable spacing (Tab Actual bar spacing	le 7.3N)	s _{max_x_p} = 300 s _{x_p} = 200 mm	mm					

Tekla Tedds	Project 25 OAKHILL AVENUE {					Job Ref. 8536	
	Section GROUND FLOOR STRUCTURE				Sheet no./rev. 6. 5		
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 21/10/2022	

Reinforcement design at midspan in long sp	oan direction (cl.6.1)
Bending moment coefficient	β _{sy_p} = 0.0560
Design bending moment	$M_{y_p} = \beta_{sy_p} \times q \times I_x^2 = 15.7 \text{ kNm/m}$
Reinforcement provided	16 mm dia. bars at 200 mm centres
Area provided	$A_{sy_p} = 1005 \text{ mm}^2/\text{m}$
Effective depth to tension reinforcement	d _{y_p} = h - c _{nom_b} - φ _{x_p} - φ _{y_p} / 2 = 196.0 mm
K factor	$K = M_{y_p} / (b \times d_{y_p}^2 \times f_{ck}) = 0.016$
Redistribution ratio	δ = 1.0
K' factor	K' = $0.598 \times \delta - 0.18 \times \delta^2 - 0.21 = 0.208$
	K < K' - Compression reinforcement is not required
Lever arm	z = min(0.95 × d _{y_p} , d _{y_p} /2 × (1 + $\sqrt{(1 - 3.53 \times K))}$ = 186.2 mm
Area of reinforcement required for bending	$A_{sy_p} = M_{y_p} / (f_{yd} \times z) = 194 \text{ mm}^2/\text{m}$
Minimum area of reinforcement required	$A_{sy_p_{min}} = max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{y_p}, 0.0013 \times b \times d_{y_p}) = 261 \text{ mm}^2/\text{m}$
Area of reinforcement required	A _{sy_p_req} = max(A _{sy_p_m} , A _{sy_p_min}) = 261 mm ² /m
	PASS - Area of reinforcement provided exceeds area required
Check reinforcement spacing	
Reinforcement service stress	$\sigma_{sy_p} = (f_{yk} / \gamma_s) \times min((A_{sy_p_m}/A_{sy_p}), 1.0) \times q_{sLS} / q = 41.1 \text{ N/mm}^2$
Maximum allowable spacing (Table 7.3N)	s _{max_y_} = 300 mm
Actual bar spacing	s _{y_p} = 200 mm

PASS - The reinforcement spacing is acceptable

Shear capacity check at short span discontinuous support

Shear force	$V_{x_d} = q \times I_x / 2 = 23.8 \text{ kN/m}$
Reinforcement provided	8 mm dia. bars at 200 mm centres
Area provided	A _{sx_d} = 251 mm²/m
Effective depth	$d_{x_d} = h - c_{nom_b} - \phi_{x_d} / 2 = 216.0 \text{ mm}$
Effective depth factor	k = min(2.0, 1 + (200 mm / d _{x_d}) ^{0.5}) = 1.962
Reinforcement ratio	$\rho_{I} = min(0.02, A_{sx_d} / (b \times d_{x_d})) = 0.0012$
Minimum shear resistance	V_{Rd,c_min} = 0.035 N/mm ² × k ^{1.5} × (f _{ck} / 1 N/mm ²) ^{0.5} × b × d _{x_d}
	V _{Rd,c_min} = 103.9 kN/m
Shear resistance constant (cl. 6.2.2)	$C_{Rd,c} = 0.18 \text{ N/mm}^2 / \gamma_c = 0.12 \text{ N/mm}^2$
Shear resistance	

$V_{Rd,c_x_d} = max(V_{Rd,c_min},$	$C_{\text{Rd},\text{c}} \times k \times (100 \times \rho_{\text{l}} \times (f_{\text{ck}}/1 \text{ N/mm}^2))^{0.333} \times b \times d_{x_d}) = \textbf{103.9 kN/m}$
	PASS - Shear capacity is adequate (0.229)

Shear capacity check at long span discontinuous support

Shear force	$V_{y_d} = q \times I_x / 2 = 23.8 \text{ kN/m}$
Reinforcement provided	8 mm dia. bars at 200 mm centres
Area provided	A _{sy_d} = 251 mm ² /m
Effective depth	$d_{y_d} = h - c_{nom_b} - \phi_{x_p} - \phi_{y_d} / 2 = 200.0 \text{ mm}$
Effective depth factor	k = min(2.0, 1 + (200 mm / d _{y_d}) ^{0.5}) = 2.000
Reinforcement ratio	$\rho_l = min(0.02, A_{sy_d} / (b \times d_{y_d})) = 0.0013$
Minimum shear resistance	$V_{\text{Rd,c_min}} = 0.035 \text{ N/mm}^2 \times k^{1.5} \times (f_{\text{ck}} \ / \ 1 \ \text{N/mm}^2)^{0.5} \times b \times d_{\text{y_d}}$
	V _{Rd,c_min} = 99.0 kN/m
Shear resistance constant (cl. 6.2.2)	$C_{Rd,c}$ = 0.18 N/mm ² / γ_{C} = 0.12 N/mm ²

Tekla , Tedds	Project 25 OAKHILL A	VENUE			Job Ref. 8536	Job Ref. 8536	
	Section GROUND FLC	DOR STRUCTURE			Sheet no./rev. 6. 6		
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 21/10/2022	
Shear resistance							
	V_{Rd,c_y_d} =	= max(V _{Rd,c_min} , ($C_{Rd,c} \times k \times (10)$	$0 \times \rho_{I} \times (f_{ck}/1 N)$	J/mm ²)) ^{0.333} × b × d	_{y_d}) = 99.0 kN/m	
				PA55 - 51	lear capacity is a	dequate (0.240)	
Basic span-to-depth deflection ra	tio check (cl.	7.4.2)					
Reference reinforcement ratio		$\rho_0 = (f_{ck} / 1 N /$	2) ^{0.5} / 1000) = 0.0050			
Required tension reinforcement ration	ρ = max(0.0035, A _{sx_p_req} / (b × d _{x_p})) = 0.0035						
Required compression reinforcement	ed compression reinforcement ratio $\rho' = A_{scx p req} / (b \times d_{x p}) = 0.0000$						
Stuctural system factor (Table 7.4N)	K _δ = 1.0					
Basic limit span-to-depth ratio (Exp.	7.16)						
	ratio _{lim x bas} =	• K _δ × [11 +1.5×(f _{ck} /1 N/mm ²) ^{0.}	⁵ ×ρ₀/ρ + 3.2×(f _{ck} /1 N/mm²) ^{0.5} ×(ρ ₀	/ρ -1) ^{1.5}] = 26.20	
Mod span-to-depth ratio limit		-	,		,	, ,	
	ratio _{lim x} = r	nin(40 × K₅, min	(1.5, (500 N/n	nm ² / f_{yk}) × (A _s)	(p/A _{sx pm}))×ratio	Dlim x bas) = 39.31	
Actual span-to-eff, depth ratio	_	ratio _{act x} = $I_x /$	dx n = 27.83			/	
· · · · · · · · · · · · · · · · · · ·		······	PASS - Actu	al span-to-ef	fective depth rati	o is acceptable	
Reinforcement summary							
Midspan in short span direction		16 mm dia. b	ars at 200 mi	m centres B1			
Midspan in long span direction		16 mm dia. b	ars at 200 mi	m centres B2			
Discontinuous support in short spar	n direction	8 mm dia. ba	rs at 200 mm	centres B1			
Discontinuous support in long span direction 8 r			8 mm dia. bars at 200 mm centres B2				

Reinforcement sketch

The following sketch is indicative only. Note that additional reinforcement may be required in accordance with clauses 9.2.1.2, 9.2.1.4 and 9.2.1.5 of EN 1992-1-1:2004 to meet detailing rules.



MBP	Michael Barclay Partnership	Job Title 25 OAKHILL AVENUE, LONDON	Job Number 8536	Sheet Number 7.1	Revision P1
	consulting engineers				
	1 Lancaster Place WC2E 7ED	Calculation/sketch Title	Date	Author	Checked
	T 020 7240 1191 F 020 7240 2241	SECTION 7	OCT 2022	AZ	тн
	E london@mbp-uk.com	RETAINING WALL CALCULATION			

GEOMETRY



SECTION THROUGT RETAINING WALL

MBP	Michael Barclay Partnership	Job Title 25 OAKHILL AVENUE, LONDON	Job Number 8536	Sheet Number 7.2	Revision P1
	consulting engineers				
	1 Lancaster Place WC2E 7ED	Calculation/sketch Title	Date	Author	Checked
	T 020 7240 1191 F 020 7240 2241	SECTION 7	OCT 2022	AZ	тн
	E london@mbp-uk.com	RETAINING WALL CALCULATION			

575mm THICK RETAINING WALL

	DEAD LOAD				
VERTICAL LUAD:	$1.05 \pm 10/m^2 \approx 1.1m = 0.15 \pm 10/m$				
Roof	$1.05 \text{ kN/m}^2 \times 1.1\text{m} = 2.15 \text{ kN/m}$				
Second Floor	$0.90 \text{ kN/m}^2 \text{ x } 1.9 \text{m} = 1.71 \text{ kN/m}$				
Flat Roof	$1.30 \text{ kN/m}^2 \text{ x } 1.9 \text{m} = 2.50 \text{ kN/m}$				
Ground Floor	$6.80 \text{ kN/m}^2 \text{ x } 3.4 \text{m} = 23.12 \text{ kN/m}$				
TOTAL:	29.50 kN/m				
Existing Wall	$6.31 \text{ kN/m}^2 \text{ x } 10\text{m} = 63.10 \text{ kN/m}$				
GROUND FORCE:	(trapeziondal force) height 4.4m, $\gamma =$ 18.5 kN/m ³				
	IMPOSED LOAD				
VERTICAL LOAD:					
Roof	$0.6 \text{ kN/m}^2 \text{ x } 1.1 \text{m} = 0.66 \text{ kN/m}$				
Second Floor	$2.5 \text{ kN/m}^2 \text{ x } 1.9 \text{m} = 4.75 \text{ kN/m}$				
Flat Roof	$1.5 \text{ kN/m}^2 \times 1.9 \text{m} = 2.85 \text{ kN/m}$				
Ground Floor	$2.5 \text{ kN/m}^2 \times 3.4 \text{m} = 8.50 \text{ kN/m}$				
τοται·	16 76 kN/m				
SURCHARGE:	5 kN/m ²				
WATER:	Full high water level 3.0m above slab				

575 mm thick Lining wall to be propped at the bottom by RC slab.

Tekla Tedds Mbp	Project 25 OAK HILL F	Road, London	1		Job Ref. 8415	
	Section RETAINING WALL			Sheet no./rev. 7. 3		
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 21/10/2022



Retained material details Mobilisation factor	Section RETAININ Calc. by AZ	G WALL Date 21/10/2022	Chk'd by	Date	Sheet no./rev. 7.4 App'd by TH	Date 21/10/20
Retained material details Mobilisation factor	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 21/10/20
Retained material details Mobilisation factor						
Mobilisation factor						
		M = 1.5				
Moist density of retained material		γm = 18.5 kN/r	m ³			
Saturated density of retained mate	erial	γ _s = 21.5 kN/n	n ³			
Design shear strength		φ' = 34.0 deg				
Angle of wall friction		$\delta = 0.7 \text{ deg}$				
Base material details						
Stiff clay						
Moist density		γ _{mb} = 18.5 kN/	/m³			
Design shear strength	ar strength $\phi'_{b} = 34.0 \text{ deg}$					
Design base friction	$\delta_{\rm b} = 0.7 \deg$					
Allowable bearing pressure		$P_{\text{bearing}} = 300 \text{ kN/m}^2$				
Using Coulomb theory Active pressure coefficient for reta	ined materia	I				
K _a = sin($(\alpha + \phi')^2 / (\sin \phi)^2$	$(\alpha)^2 \times \sin(\alpha - \delta) \times [1]$	+ √(sin(φ' + δ	$) \times \sin(\phi' - \beta) / $	$(\sin(\alpha - \delta) \times \sin(\alpha))$	$(\alpha + \beta)))]^2) = 0$
Passive pressure coefficient for ba	ise material					
	K _p =	$sin(90 - \phi'_b)^2 / (sin(90 - \phi'_b)^2)$	90 - δ _b) × [1 - γ	$\sqrt{(\sin(\phi_b + \delta_b))}$	sin(φ'₅) / (sin(90	+ $\delta_{b})))]^{2}) = 3$
At-rest pressure						
At-rest pressure for retained mater	rial	$K_0 = 1 - \sin(\phi)$	') = 0.441			
Loading details						
Surcharge load on plan		Surcharge = !	5.0 kN/m²			
Applied vertical dead load on wall		W _{dead} = 92.6	κN/m			
Applied vertical live load on wall		W _{live} = 16.8 ki	N/m			
Position of applied vertical load on	wall	l _{load} = 2250 m	m			
Applied horizontal dead load on wa	all	F _{dead} = 0.0 kN	/m			
	I	Flive = 0.0 kN/	m			
Applied horizontal live load on wal						



Tekla Tedds Mbp	Project 25 OAK HILL ROAD, LONDON				Job Ref. 8415	
	Section RETAINING WALL				Sheet no./rev. 7. 6	
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 21/10/2022

Wall base	M _{base} = w _{base} × I _{base} / 2 = 27.8 kNm/m
Design vertical dead load	$M_{dead} = W_{dead} \times I_{load} = 208.4 \text{ kNm/m}$
Total restoring moment	M _{rest} = M _{wall} + M _{base} + M _{dead} = 324.6 kNm/m
Check bearing pressure	
Design vertical live load	$M_{live} = W_{live} \times I_{load} = 37.7 \text{ kNm/m}$
Total moment for bearing	M _{total} = M _{rest} - M _{ot} + M _{live} = 287.1 kNm/m
Total vertical reaction	R = W _{total} = 169.6 kN/m
Distance to reaction	x _{bar} = M _{total} / R = 1693 mm
Eccentricity of reaction	e = abs((l _{base} / 2) - x _{bar}) = 405 mm
	Reaction acts within middle third of base
Bearing pressure at toe	$p_{toe} = (R / I_{base}) - (6 \times R \times e / I_{base}^2) = 3.7 \text{ kN/m}^2$
Bearing pressure at heel	$p_{heel} = (R / I_{base}) + (6 \times R \times e / I_{base}^2) = 128.1 \text{ kN/m}^2$
	PASS - Maximum bearing pressure is less than allowable bearing pressure

Iekla. ledds	Project 25 OAK HILL ROAD, LONDON				Job Ker. 8415			
Mbp			Sheet no./rev	Sheet no./rev.				
	RETAININ	G WALL	G WALL					
	Calc. by	Date	Chk'd by	Date	App'd by	Date		
	AZ	21/10/2022			TH	21/10/20		
RETAINING WALL DESIGN (B	<u> 8002:1994)</u>				TEDDS calcula	ation version 1.2		
Ultimate limit state load factor	S							
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	II							
Wall stem		wwall_f = $\gamma_{f_d} \times$	$h_{\text{stem}} imes t_{\text{wall}} imes \gamma_{\text{t}}$	wall = 54.1 kN/m	n			
Wall base		$W_{base_f} = \gamma_{f_d} \times$	$ _{base} \times t_{base} \times \gamma$	/base = 30.3 kN/	/m			
Applied vertical load		$W_{v_f} = \gamma_{f_d} \times V$	V_{dead} + $\gamma_{f_l} \times W$	/ _{live} = 156.5 kN/	′m			
Total vertical load		W _{total_f} = w _{wall}	f + W _{base_f} + W	/ _{v_f} = 240.8 kN/i	m			
Factored horizontal at-rest for	ces on wall							
Surcharge		$F_{sur_{f}} = \gamma_{f_{l}} \times k$	$K_0 imes$ Surcharge	e × h _{eff} = 11.1 kl	N/m			
Moist backfill below water table		$F_{m_b_f} = \gamma_{f_e} \times$	$F_{m b} f = \gamma_{f e} \times K_0 \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water} = 0 \text{ kN/m}$					
Saturated backfill		$F_{s_f} = \gamma_{f_e} \times 0.5 \times K_0 \times (\gamma_{s_r} - \gamma_{water}) \times h_{water}^2 = 35.8 \text{ kN/m}$						
Water		 F _{water f} = γ _{fe} ≻	$F_{water_f} = \gamma_{f_e} \times 0.5 \times h_{water}^2 \times \gamma_{water} = 68.1 \text{ kN/m}$					
Total horizontal load		F _{total_f} = F _{sur_f}	$F_{total_f} = F_{sur_f} + F_{m_b_f} + F_{s_f} + F_{water_f} = 115 \text{ kN/m}$					
Calculate propping force								
Passive resistance of soil in fron	t of wall	$F_{p f} = \gamma_{f e} \times 0$	$.5 imes K_p imes \cos(\delta)$	ób) × (d _{cover} + t _{ba}	se + d _{ds} - d _{exc}) ² ×	γ _{mb} = 5.7 kN		
Propping force		$F_{prop f} = max($	Ftotal f - Fp f - (Notal f - γfι×W	$_{live}$) × tan(δ_{b}), 0 k	N/m)		
		F _{prop_f} = 106. 7	7 kN/m					
Factored overturning moment	S							
Surcharge		$M_{sur_f} = F_{sur_f}$	imes (h _{eff} - 2 $ imes$ d _d	s) / 2 = 17.5 kN	m/m			
Moist backfill below water table		$M_{m_b_f} = F_{m_b}$	$_{\rm f} imes$ (h _{water} - 2 $ imes$	d _{ds}) / 2 = 0 kN	m/m			
Saturated backfill		$M_{s_f} = F_{s_f} \times (h_{water} - 3 \times d_{ds}) / 3 = 37.6 \text{ kNm/m}$						
Water		$M_{water_f} = F_{water_f} \times (h_{water} - 3 \times d_{ds}) / 3 = 71.5 \text{ kNm/m}$						
Total overturning moment		$M_{ot_f} = M_{sur_f}$	+ M _{m_b_f} + M _{s_f}	+ M _{water_f} = 126	5.6 kNm/m			
Restoring moments								
Wall stem		$M_{wall_f} = W_{wall_f}$	$_{\rm f} imes$ (I _{toe} + t _{wall} / 2	2) = 123.7 kNm	ı/m			
Wall base		$M_{base_f} = W_{base}$	$_{e_f} \times I_{base} / 2 = 3$	39 kNm/m				
Design vertical load		$M_{v_f} = W_{v_f} \times$	I _{load} = 352 kNn	n/m				
Total restoring moment		M _{rest_f} = M _{wall_}	f + M _{base_f} + M	_{/_f} = 514.8 kNm	ı/m			
Factored bearing pressure								
Total moment for bearing		M _{total_f} = M _{rest_}	_f - M _{ot_f} = 388.	1 kNm/m				
Total vertical reaction		$R_f = W_{total_f} =$	240.8 kN/m					
Distance to reaction		$\mathbf{x}_{bar_f} = \mathbf{M}_{total_f}$	/ R _f = 1612 m	m				
Eccentricity of reaction		e _f = abs((I _{base}	/ 2) - x _{bar_f}) = :	324 mm	, · ·			
		<u> </u>) <i>(</i> 2 =	Reaction a	acts within mid	dle third of		
Bearing pressure at toe		$p_{toe_f} = (R_f / I_b)$	ase) - $(6 \times R_f \times$	$e_f / I_{base^2} = 22.$	9 kN/m ²			
Bearing pressure at heel		$p_{heel_f} = (R_f / I)$	base) + ($6 \times R_f$	$\times \text{ e}_{\text{f}} / \text{ I}_{\text{base}^2} = 16$	54.2 kN/m ²			
Rate of change of base reaction		rate = $(p_{\text{toe } f} -$	Dheel f) / base =	-54.87 kN/m ² /	m			

Tekla Tedds	Tedds 25 OAK HILL ROAD, LONDON							
р	Section RETAININC	G WALL			Sheet no./rev. 7.8			
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 21/10/2022		
Bearing pressure at stem / toe		p _{stem_toe_f} = ma	ax(p _{heel_f} + (rat	$e \times (I_{heel} + t_{wall}))$), 0 kN/m²) = 132	. 6 kN/m ²		
Bearing pressure at mid stem		p _{stem_mid_f} = m	ax(p _{heel_f} + (rat	$te imes (I_{heel} + t_{wall})$	/ 2)), 0 kN/m²) =	148.4 kN/m²		
Bearing pressure at stem / heel		p _{stem_heel_f} = m	ax(p _{heel_f} + (ra	te $ imes$ I _{heel}), 0 kN	/m²) = 164.2 kN/r	m²		
Design of reinforced concrete	retaining wall	toe (BS 8002:199	4)					
Material properties			<u> </u>					
Characteristic strength of concre	e	f _{cu} = 35 N/mn	1 ²					
Characteristic strength of reinford	ement	f _y = 500 N/mr	n²					
Base details								
Minimum area of reinforcement		k = 0.13 %						
Cover to reinforcement in toe		c _{toe} = 50 mm						
Calculate shear for toe design								
Shear from bearing pressure		V _{toe_bear} = (p _{to}	e_f + p _{stem_toe_f})	× I _{toe} / 2 = 155 .	5 kN/m			
Shear from weight of base		$V_{\text{toe wt base}} = \gamma$	$_{\rm d} imes \gamma_{\rm base} imes {\sf I}_{ m toe}$	imes t _{base} = 23.5 k	N/m			
Total shear for toe design		V _{toe} = V _{toe_bear}	$V_{\text{toe}} = V_{\text{toe} \text{ bear}} - V_{\text{toe} \text{ wt} \text{ base}} = 132 \text{ kN/m}$					
Calculate moment for toe desig	an							
Moment from bearing pressure	•	M _{toe bear} = (2 ;	<ptoe +="" f="" i<="" pstem="" td=""><td>mid f) $imes$ (Itoe + twa</td><td>u / 2)² / 6 = 169.3</td><td>kNm/m</td></ptoe>	mid f) $ imes$ (Itoe + twa	u / 2) ² / 6 = 169.3	kNm/m		
Moment from weight of base		M _{toe} wt base = ($M_{toe_wt_base} = (\gamma_{f_d} \times \gamma_{base} \times t_{base} \times (I_{toe} + t_{wall} / 2)^2 / 2) = 30.8 \text{ kNm/m}$					
Total moment for toe design		M _{toe} = M _{toe_bear} - M _{toe_wt_base} = 138.6 kNm/m						
350	•	•	•	•	•			
	⊲ —_200-	→						
Check toe in bending	∢ —200-	→	1					
Check toe in bending Width of toe	⊲ —200	► b = 1000 mm	/m	290.0 mm				
Check toe in bending Width of toe Depth of reinforcement	 ⊲ —200-	b = 1000 mm d _{toe} = t _{base} − C	/m _{toe} – (φ _{toe} / 2) =	290.0 mm				
Check toe in bending Width of toe Depth of reinforcement Constant	 ⊲ —200	b = 1000 mm d _{toe} = t _{base} − c K _{toe} = M _{toe} / (t	/m _{toe} – (φ _{toe} / 2) = ο × d _{toe} ² × f _{cu}) =	290.0 mm = 0.047 Compressio	n reinforcemen	t is not required		
Check toe in bending Width of toe Depth of reinforcement Constant	 4 −−−200	$b = 1000 \text{ mm}$ $d_{toe} = t_{base} - C$ $K_{toe} = M_{toe} / (k$ $z_{toe} = \min(0.5)$	/m _{toe} – (φ _{toe} / 2) = 0 × d _{toe} ² × f _{cu}) = + √(0, 25 - (mi	290.0 mm = 0.047 Compressio	<i>n reinforcemen</i> ∕ 0 9)) 0 95) ∨ d⊷	t is not required		
Check toe in bending Width of toe Depth of reinforcement Constant Lever arm	 4 −−−200	b = 1000 mm d _{toe} = t _{base} - c K _{toe} = M _{toe} / (k z _{toe} = min(0.5 z _{toe} = 274 mm	/m toe – (φ _{toe} / 2) = o × d _{toe} ² × f _{cu}) = + √(0.25 - (mi	290.0 mm = 0.047 Compressio in(K _{toe} , 0.225) /	<i>n reinforcemen</i> ′ 0.9)),0.95) × d _{too}	t is not required		
Check toe in bending Width of toe Depth of reinforcement Constant Lever arm Area of tension reinforcement red	4 200- ₁uired	$b = 1000 \text{ mm}$ $d_{toe} = t_{base} - c$ $K_{toe} = M_{toe} / (t)$ $z_{toe} = min(0.5)$ $z_{toe} = 274 \text{ mm}$ $A_{s toe} \text{ des} = M_{to}$	/m $t_{toe} - (\phi_{toe} / 2) =$ $0 \times d_{toe}^2 \times f_{cu}) =$ $+ \sqrt{(0.25 - (min))}$ $t_{toe} / (0.87 \times f_v \times t_v)$	= 290.0 mm = 0.047 <i>Compressio</i> in(K _{toe} , 0.225) / : z _{toe}) = 1163 m	<i>n reinforcemen</i> ′ 0.9)),0.95) × dtoa um²/m	t is not required		
Check toe in bending Width of toe Depth of reinforcement Constant Lever arm Area of tension reinforcement ree Minimum area of tension reinforce	4 −−−200- µuired ement	$b = 1000 \text{ mm}$ $d_{toe} = t_{base} - C$ $K_{toe} = M_{toe} / (t)$ $z_{toe} = min(0.5)$ $z_{toe} = 274 \text{ mm}$ $A_{s_toe_des} = M_{to}$ $A_{s_toe_des} = M_{to}$	/m $t_{toe} - (\phi_{toe} / 2) =$ $0 \times d_{toe}^2 \times f_{cu}) =$ $+ \sqrt{(0.25 - (min))}$ $t_{toe} / (0.87 \times f_y \times f_y)$ $t_{tobe} = 45$	290.0 mm = 0.047 <i>Compressio</i> in(K _{toe} , 0.225) / : z _{toe}) = 1163 m 5 mm ² /m	<i>n reinforcemen</i> ′ 0.9)),0.95) × dtoo ⊔m²/m	t is not required		
Check toe in bending Width of toe Depth of reinforcement Constant Lever arm Area of tension reinforcement rea Minimum area of tension reinforce	4 —200 1 200 1 1 2 200	$b = 1000 \text{ mm}$ $d_{toe} = t_{base} - C$ $K_{toe} = M_{toe} / (t)$ $z_{toe} = min(0.5)$ $z_{toe} = 274 \text{ mm}$ $A_{s_toe_des} = M_{toe}$ $A_{s_toe_min} = k >$ $A_{s_toe_min} = M_{toe}$	/m $t_{toe} - (\phi_{toe} / 2) =$ $0 \times d_{toe}^2 \times f_{cu}) =$ $+ \sqrt{(0.25 - (min))}$ $t_{toe} / (0.87 \times f_y \times f_y)$ $t_{to b \times t_{base}} = 45$ $t_{to a toe des.} A_s$	290.0 mm = 0.047 <i>Compressio</i> in(K _{toe} , 0.225) / : z _{toe}) = 1163 m 5 mm ² /m toe min) = 1163	<i>n reinforcemen</i> ′ 0.9)),0.95) × dtor lm²/m mm²/m	t is not required		
Check toe in bending Width of toe Depth of reinforcement Constant Lever arm Area of tension reinforcement rea Minimum area of tension reinforce Area of tension reinforcement rea Reinforcement provided	4 −−−200- quired ement quired	$b = 1000 \text{ mm}$ $d_{toe} = t_{base} - C$ $K_{toe} = M_{toe} / (t)$ $z_{toe} = min(0.5)$ $z_{toe} = 274 \text{ mm}$ $A_{s_toe_des} = M_{to}$ $A_{s_toe_min} = k >$ $A_{s_toe_req} = Ma$ 20 mm dia.b	/m $t_{toe} - (\phi_{toe} / 2) =$ $0 \times d_{toe}^2 \times f_{cu}) =$ $+ \sqrt{(0.25 - (min))}$ $t_{toe} / (0.87 \times f_y \times f_{toe})$ $t_{toe} \times t_{base} = 45$ $t_{toe} \times t_{base} = 45$ $t_{toe} \times t_{base} = 45$ $t_{toe} \times t_{toe}$ $t_{toe} \times t_{to$	290.0 mm = 0.047 <i>Compressio</i> in(K _{toe} , 0.225) / : z _{toe}) = 1163 m 5 mm ² /m _toe_min) = 1163 n centres	<i>n reinforcemen</i> ′ 0.9)),0.95) × dtoo um²/m mm²/m	t is not required		
Check toe in bending Width of toe Depth of reinforcement Constant Lever arm Area of tension reinforcement rea Minimum area of tension reinforc Area of tension reinforcement rea Reinforcement provided Area of reinforcement provided	4 —200. quired ement quired	$b = 1000 \text{ mm}$ $d_{toe} = t_{base} - C$ $K_{toe} = M_{toe} / (t)$ $z_{toe} = min(0.5)$ $z_{toe} = 274 \text{ mm}$ $A_{s_toe_des} = M_{toe}$ $A_{s_toe_req} = M_{toe}$ $20 \text{ mm} \text{ dia.b}$ $A_{s_toe_prov} = 15$	/m $t_{toe} - (\phi_{toe} / 2) =$ $0 \times d_{toe}^2 \times f_{cu}) =$ $+ \sqrt{(0.25 - (min))}$ $t_{toe} / (0.87 \times f_y \times f_y)$ $t_{tob} \times t_{base} = 45$ $t_{tot}(A_{s_toe_des}, A_s)$ $t_{tot}(A_{s_toe_des}, A_s)$ $t_{tot}(A$	290.0 mm = 0.047 <i>Compressio</i> in(K _{toe} , 0.225) / : z _{toe}) = 1163 m 5 mm ² /m _toe_min) = 1163 n centres	<i>n reinforcemen</i> ′ 0.9)),0.95) × dtoo lm²/m mm²/m	t is not required		

Tekla Tedds	Tekla Tedds			. ROAD, LONDON				
цар	Section RETAININ	G WALL			Sheet no./rev. 7.9			
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 21/10/2022		
Check shear resistance at toe								
Design shear stress		$v_{toe} = V_{toe} / (b$	× d _{toe}) = 0.45	5 N/mm²				
Allowable shear stress		v _{adm} = min(0.8	$3 \times \sqrt{(f_{cu} / 1 N/)}$	mm²), 5) × 1 N	l/mm ² = 4.733 N/r	nm²		
		PASS	- Design she	ear stress is l	ess than maxim	um shear stress		
From BS8110:Part 1:1997 – Tak	ole 3.8							
Design concrete shear stress		v _{c_toe} = 0.625	N/mm ²	_				
				V _{toe} < V _{c_toe} - N	No shear reinford	ement required		
Design of reinforced concrete	retaining wall	stem (BS 8002:1	994 <u>)</u>					
Material properties								
Characteristic strength of concret	e	f _{cu} = 35 N/mm ²						
Characteristic strength of reinford	ement	f _y = 500 N/mm ²						
Wall details								
Minimum area of reinforcement		k = 0.13 %						
Cover to reinforcement in stem		c _{stem} = 50 mm						
Cover to reinforcement in wall		c _{wall} = 50 mm						
Factored horizontal at-rest force	es on stem							
Surcharge		$F_{s_sur_f} = \gamma_{f_l} \times$	K ₀ × Surcharg	${\sf ge} imes ({\sf h}_{\sf eff}$ - ${\sf t}_{\sf base}$	- d _{ds}) = 9.9 kN/m			
Moist backfill below water table		$F_{s_mb_f} = \gamma_{f_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat} = 0 \text{ kN/m}$						
Saturated backfill		$F_{s_s_f} = 0.5 \times \gamma_{f_e} \times K_0 \times (\gamma_{s} - \gamma_{water}) \times h_{sat}^2 = 28.3 \text{ kN/m}$						
Water		$F_{s_water_f} = 0.5 \times \gamma_{f_e} \times \gamma_{water} \times h_{sat}^2 = 53.8 \text{ kN/m}$						
Calculate shear for stem desig	n							
Shear at base of stem	-	V _{stem} = F _{s sur f}	+ F _{s m b f} + F _s	ssf +F swaterf	- Fprop f = -14.7 kN	J/m		
Calculate moment for stem dec	ian							
Surcharge		Ms sur = Fs sur	f × (hstem + the	_{se}) / 2 = 15.6 k	Nm/m			
	$M_{2} = F_{2} = F_{2} + F_{2$			0 kNm/m				
Moist backfill below water table		$M_{\mu} = F_{\mu} (x h_{\mu} / 3 - 26.4 \text{ kNm/m})$						
Moist backfill below water table Saturated backfill		Ms_m_b = 1 s_m_ Ms_s = Fs_s_f×	$h_{sat}/3 = 26.4$	kNm/m				
Moist backfill below water table Saturated backfill Water		$Ms_m_b = F_{s_s_f} \times M_{s_s_s_f} = F_{s_s_f} \times M_{s_s_s_f} = F_{s_s_f}$	$h_{sat} / 3 = 26.4$	kNm/m = 50.2 kNm/m				

Tekla Tedds	L ROAD, LONDO	DN	Job Ref. 8415	Job Ref. 8415			
Section RETA		G WALL			Sheet no./rev 7. 10		
Ca Az	alc. by Z	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 21/10/2022	
575							
		•	•	•	•		
ł	⊲ 200						
Check wall stem in bending							
Width of wall stem		b = 1000 mm	n/m				
Depth of reinforcement		$d_{stem} = t_{wall} - $	c _{stem} − (¢ _{stem} / 2	<u>2)</u> = 517.0 mm			
Constant		K _{stem} = M _{stem}	/ (b $ imes$ d _{stem} ² $ imes$ f	_{cu}) = 0.010			
				Compressio	n reinforcemen	t is not require	
Lever arm		z _{stem} = min(0	.5 + √(0.25 - (r	nin(K _{stem} , 0.225	5) / 0.9)),0.95) × 0	d _{stem}	
		z _{stem} = 491 m	ım		_		
Area of tension reinforcement required	d	As_stem_des = N	$M_{ m stem}$ / (0.87 × f	fy × Z _{stem}) = 432	mm²/m		
Minimum area of tension reinforcemer	nt	$A_{s_{stem_{min}}} = k$	$x \times b \times t_{wall} = 74$	18 mm²/m			
Area of tension reinforcement required	d	$A_{s_stem_req} = Max(A_{s_stem_des}, A_{s_stem_min}) = 748 \text{ mm}^2/\text{m}$					
Area of reinforcement provided		To mm dia.bars (200 mm centres $\Delta = -1005 \text{ mm}^2/\text{m}$					
Area of remorcement provided		PASS - Reir	nforcement pr	rovided at the	retaining wall s	tem is adequate	
Check shear resistance at wall sten	n				C C	·	
Design shear stress		v _{stem} = V _{stem} /	(b × d _{stem}) = -().028 N/mm ²			
Allowable shear stress		$v_{adm} = min(0)$	8 × √(f _{cu} / 1 N/	mm²), 5) × 1 N/	/mm ² = 4.733 N/i	mm²	
		PAS	S - Design she	ear stress is le	ess than maxim	um shear stress	
From BS8110:Part 1:1997 – Table 3.	.8						
Design concrete shear stress		V _{c_stem} = 0.41	0 N/mm ²				
			Vs	tem < Vc_stem - N	o shear reinfor	cement required	
Check retaining wall deflection							
Basic span/effective depth ratio		ratio _{bas} = 7					
Design service stress		$f_s = 2 \times f_y \times A$	$_{\rm s_stem_req}$ / (3 $ imes$	As_stem_prov) = 24	47.9 N/mm²		
Modification factor factor _t	_{ens} = min(0.55 + (477 N/mm	$n^2 - f_s)/(120 \times (0))$	0.9 N/mm² + (N	$I_{stem}/(b \times d_{stem}^2)))$),2) = 2.00	
Maximum span/effective depth ratio		ratio _{max} = rati	$o_{bas} imes factor_{tens}$	_s = 14.00			

ratio_{act} = h_{stem} / d_{stem} = **5.42**

Actual span/effective depth ratio

PASS - Span to depth ratio is acceptable

	Project				Job Ref	
	25 OAK HILL ROAD, LONDON 8415					
qaivi	Section			Sheet no./rev.	Sheet no./rev.	
	RETAINING WALL				7.11	
	AZ	Date 21/10/2022	Chk'd by	Date	TH	Date 21/10/2022
Indicative retaining wall reinfor	cement diagrai	<u>n</u>				
				Stem	reinforcement	
					Tomoroomone	
Toe reinforcement -						
Г Г						
Toe bars - 20 mm dia @ 200 mm	centres - (1571	mm²/m)				
Stem bars - 16 mm dia.@ 200 mm	n centres - (100	5 mm²/m)				

MBP	Michael Barclay Partnership	Job Title 25 OAKHILL AVENUE, LONDON	Job Number 8536	Sheet Number 7.12	Revision P1
	consulting engineers				
	1 Lancaster Place WC2E 7ED	Calculation/sketch Title	Date	Author	Checked
	T 020 7240 1191 F 020 7240 2241	SECTION 7	OCT 2022	AZ	тн
	E london@mbp-uk.com	RETAINING WALL CALCULATION			

250mm THICK RETAINING WALL

	DEAD LOAD
Flat Boof	$1.30 \text{ kN/m}^2 \text{ x} 1.6 \text{m} = 2.10 \text{ kN/m}$
Ground Floor	$6.80 \text{ kN/m}^2 \times 3.4 \text{m} = 23.12 \text{ kN/m}$
	25.22 kN/m
IUTAL.	25.22 KN/III
New Cavity Wall	$4.04 \text{ kN/m}^2 \text{ x } 4.3 \text{m} = 8.34 \text{ kN/m}$
GROUND FORCE:	(trapeziondal force) height 4.4m, $\gamma=$ 18.5 kN/m³
	IMPOSED LOAD
VERTICAL LOAD:	
Flat Roof	$1.5 \text{ kN/m}^2 \text{ x } 1.6 \text{m} = 2.40 \text{ kN/m}$
Ground Floor	$2.5 \text{ kN/m}^2 \text{ x } 3.4 \text{m} = 8.50 \text{ kN/m}$
TOTAL:	10.90 kN/m
SURCHARGE:	5 kN/m ²
WATER:	Full high water level 3.0m above slab

250mm linning wall to be propped at the top and at the bottom by RC slab.

Tekla Tedds	Project	Project				Job Ref.	
	25 OAK HILL	25 OAK HILL ROAD, LONDON				8415	
дам	Section				Sheet no./rev.	Sheet no./rev.	
	RETAINING WALL				7. 14	7. 14	
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 21/10/2022	



Tekla Tedds	Project 25 OAK HILL ROAD, LONDON				Job Ref. 8415		
Мbр	Section RETAINING W	/ALL			Sheet no./rev. 7. 15		
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 21/10/2022	
Moist density of retained material Saturated density of retained material Design shear strength Angle of wall friction Base material details Stiff clay Moist density Design shear strength Design base friction		$\gamma_m =$ 18.5 kN/m $\gamma_s =$ 21.5 kN/m $\phi' =$ 34.0 deg $\delta =$ 0.7 deg $\gamma_{mb} =$ 18.5 kN/r $\phi'_b =$ 34.0 deg $\delta_b =$ 0.7 deg	n ³ 3 N ³				
Allowable bearing pressure Using Coulomb theory Active pressure coefficient for reta K _a = sin(Passive pressure coefficient for ba	ined material α + φ')² / (sin(α)² ase material K _P = sin($P_{\text{bearing}} = 300 \text{ k}$ $\times \sin(\alpha - \delta) \times [1 + (\sin(9 - \phi'_b)^2 / (\sin(9 - \phi'$	N/m² + √(sin(φ' + δ) × 0 - δ _b) × [1 - √(si	sin(φ' - β) / (sin(α n(φ'ь + δь) × sin(α	α - δ) × sin(α + φ' _b) / (sin(90 +	β)))] ²) = 0.281 δ _b)))] ²) = 3.620	
At-rest pressure At-rest pressure for retained mater Loading details Surcharge load on plan Applied vertical dead load on wall Applied vertical live load on wall	rial	K ₀ = 1 – sin(φ') Surcharge = 5 W _{dead} = 33.6 k W _{live} = 10.9 kN) = 0.441 .0 kN/m ² N/m I/m			,, <u>,,</u> ,	
Position of applied vertical load on wain Position of applied vertical load on Applied horizontal dead load on wain Applied horizontal live load on wain Height of applied horizontal load on	ı wall all I n wall	$I_{load} = 2250 \text{ mn}$ $F_{dead} = 0.0 \text{ kN/r}$ $F_{live} = 0.0 \text{ kN/r}$ $h_{load} = 0 \text{ mm}$	n m n 445				
Prog	35.6	Prop —	35.6	0003 30.9			

Tekla Tedds Mbp	Project 25 OAK HILL R	OAD, LONDON	Job Ref. 8415			
	Section RETAINING WALL				Sheet no./rev. 7. 16	
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 21/10/2022

Loads shown in kN/m, pressures shown in kN/m²

Vertical forces on wall	
Wall stem	$w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall} = 16.8 \text{ kN/m}$
Wall base	$w_{base} = I_{base} \times t_{base} \times \gamma_{base} = 18.9 \text{ kN/m}$
Applied vertical load	$W_v = W_{dead} + W_{live} = 44.5 \text{ kN/m}$
Total vertical load	$W_{total} = W_{wall} + W_{base} + W_v = 80.2 \text{ kN/m}$
Horizontal forces on wall	
Surcharge	F_{sur} = K _a × cos(90 - α + δ) × Surcharge × h _{eff} = 4.4 kN/m
Moist backfill below water table	$F_{m_b} = K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_eff - h_water) \times h_water = 0 \; kN/m$
Saturated backfill	$F_{s} = 0.5 \times K_{a} \times cos(90 - \alpha + \delta) \times (\gamma_{s} - \gamma_{water}) \times h_{water}^{2} = 16.3 \text{ kN/m}$
Water	$F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water} = 48.7 \text{ kN/m}$
Total horizontal load	$F_{total} = F_{sur} + F_{m_b} + F_s + F_{water} = 69.4 \text{ kN/m}$
Calculate total propping force	
Passive resistance of soil in front of wall	$F_{p} = 0.5 \times K_{p} \times cos(\delta_{b}) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^{2} \times \gamma_{mb} = 4.1 \text{ kN/m}$
Propping force	$F_{prop} = max(F_{total} - F_p - (W_{total} - W_{live}) \times tan(\delta_b), 0 \text{ kN/m})$
	F _{prop} = 64.4 kN/m
Overturning moments	
Surcharge	$M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2 = 7 \text{ kNm/m}$
Moist backfill below water table	$M_{m_b} = F_{m_b} \times (h_{water} - 2 \times d_{ds}) / 2 = 0 \text{ kNm/m}$
Saturated backfill	$M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3 = 17.1 \text{ kNm/m}$
Water	M _{water} = F _{water} × (h _{water} - 3 × d _{ds}) / 3 = 51.1 kNm/m
Total overturning moment	$M_{ot} = M_{sur} + M_{m_b} + M_s + M_{water} = 75.2 \text{ kNm/m}$
Restoring moments	
Wall stem	M_{wall} = $w_{wall} \times (I_{toe} + t_{wall} / 2)$ = 35.7 kNm/m
Wall base	$M_{base} = w_{base} \times I_{base} / 2 = 21.3 \text{ kNm/m}$
Design vertical dead load	$M_{dead} = W_{dead} \times I_{load} = 75.5 \text{ kNm/m}$
Total restoring moment	M _{rest} = M _{wall} + M _{base} + M _{dead} = 132.5 kNm/m
Check bearing pressure	
Total vertical reaction	R = W _{total} = 80.2 kN/m
Distance to reaction	x _{bar} = I _{base} / 2 = 1125 mm
Eccentricity of reaction	$e = abs((l_{base} / 2) - x_{bar}) = 0 mm$
	Reaction acts within middle third of base
Bearing pressure at toe	$p_{toe} = (R / I_{base}) - (6 \times R \times e / I_{base}^2) = 35.6 \text{ kN/m}^2$
Bearing pressure at heel	$p_{heel} = (R / I_{base}) + (6 \times R \times e / I_{base}^2) = 35.6 \text{ kN/m}^2$
	PASS - Maximum bearing pressure is less than allowable bearing pressure
Calculate propping forces to top and ba	se of wall
Propping force to top of wall	
	$F_{prop_top} = (M_{ot} - M_{rest} + R \times I_{base} / 2 - F_{prop} \times t_{base} / 2) / (h_{stem} + t_{base} / 2) = 7.263 \text{ kN/m}$
Propping force to base of wall	F _{prop_base} = F _{prop} - F _{prop_top} = 57.174 kN/m

Tekla Tedds	25 OAK H	Project 25 OAK HILL ROAD, LONDON				Job Ref. 8415		
Мbр					Sheet no./rev.			
			Chlidhu	Data	1.1/			
	AZ	21/10/2022	Clik d by	Date	TH	21/10/20		
RETAINING WALL DESIGN (B	S 8002:1994)					ation version 1.		
Ultimate limit state load factor	s				TEDDS calcul	ation version 1.2		
Dead load factor	-	γ _{f d} = 1.4						
Live load factor		γ _{f I} = 1.6						
Earth and water pressure factor		γ _{f e} = 1.4						
Factored vertical forces on wa	II							
Wall stem		$W_{wall f} = \gamma_{f d} \times$	$h_{\text{stem}} \times t_{\text{wall}} \times \gamma_{\text{t}}$	wall = 23.5 kN/n	n			
Wall base		– ∙– Wbase f= ∿fd>	ر ۱ hase X thase X	wase = 26.5 kN	/m			
Applied vertical load		$W_{\rm V}$ f = $V_{\rm f}$ d \times V	$N_{dead} + \gamma_{f} \times M$	/live = 64.4 kN/n	n			
Total vertical load		W _{total_f} = W _{wall}	_f + W _{base_f} + W	/ _{v_f} = 114.4 kN/	m			
Factored horizontal at-rest for	ces on wall							
Surcharge		$F_{sur f} = \gamma_{f \perp} \times K_0 \times Surcharge \times h_{eff} = 11.1 \text{ kN/m}$						
Moist backfill below water table		Fm b f = γ _{fe} ×	$F_{m_b_f} = \gamma_{f_e} \times K_0 \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water} = 0 \text{ kN/m}$					
Saturated backfill		$F_{s f} = \gamma_{f e} \times 0.$	$F_{s_f} = \gamma_{f_e} \times 0.5 \times K_0 \times (\gamma_{s} - \gamma_{water}) \times h_{water}^2 = 35.8 \text{ kN/m}$					
Water		 F _{water f} = γ _{fe} >	$F_{water_f} = \gamma_{f_e} \times 0.5 \times h_{water}^2 \times \gamma_{water} = 68.1 \text{ kN/m}$					
Total horizontal load	F _{total_f} = F _{sur_f}	$F_{total_f} = F_{sur_f} + F_{m_b_f} + F_{s_f} + F_{water_f} = 115 \text{ kN/m}$						
Calculate total propping force								
Passive resistance of soil in front of wall		$F_{p_f} = \gamma_{f_e} \times 0$	$.5 imes K_p imes \cos(\delta)$	$(d_{cover} + t_{bac}) \times (d_{cover} + t_{bac})$	$_{ m ase}$ + d _{ds} - d _{exc}) ² ×	γ _{mb} = 5.7 kN		
Propping force		F _{prop_f} = max($F_{prop_f} = max(F_{total_f} - F_{p_f} - (W_{total_f} - \gamma_{f_l} \times W_{live}) \times tan(\delta_{b}), \ 0 \ kN/m)$					
		F _{prop_f} = 108.	1 kN/m					
Factored overturning moment	S							
Surcharge		$M_{sur_f} = F_{sur_f}$	$M_{sur_f} = F_{sur_f} \times (h_{eff} - 2 \times d_{ds}) / 2 = 17.5 \text{ kNm/m}$					
Moist backfill below water table		$M_{m_b_f} = F_{m_b_f}$	$M_{m_b_f} = F_{m_b_f} \times (h_{water} - 2 \times d_{ds}) / 2 = 0 \text{ kNm/m}$					
Saturated backfill		$M_{s_f} = F_{s_f} \times ($	$M_{s_f} = F_{s_f} \times (h_{water} - 3 \times d_{ds}) / 3 = 37.6 \text{ kNm/m}$					
Water		$M_{water_f} = F_{water}$	$M_{water_f} = F_{water_f} \times (h_{water} - 3 \times d_{ds}) / 3 = 71.5 \text{ kNm/m}$					
Total overturning moment		$M_{ot_f} = M_{sur_f}$	+ M _{m_b_f} + M _{s_f}	+ M _{water_f} = 126	5.6 kNm/m			
Restoring moments								
Wall stem		$M_{wall_f} = w_{wall_f} \times (I_{toe} + t_{wall} / 2) = 50 \text{ kNm/m}$						
Wall base		$M_{base_f} = w_{base_f} \times I_{base} / 2 = 29.8 \text{ kNm/m}$						
Design vertical load		$M_{v_f} = W_{v_f} \times$	$M_{v_f} = W_{v_f} \times I_{load} = 145 \text{ kNm/m}$					
Total restoring moment		M _{rest_f} = M _{wall_}	$_{f} + M_{base_{f}} + M_{base_{f}}$	_{/_f} = 224.7 kNm	n/m			
Factored bearing pressure								
Total vertical reaction	$R_f = W_{total_f} =$	$R_f = W_{total_f} = 114.4 \text{ kN/m}$						
Distance to reaction		$X_{bar_f} = I_{base} / 2$	$X_{bar_f} = I_{base} / 2 = 1125 \text{ mm}$					
Eccentricity of reaction		e _f = abs((I _{base}	e / 2) - X _{bar_f}) =	0 mm Position	acts within mid	dla third af		
Bearing pressure at toe		$D_{\text{top}} = (R_f / I_h)$		$e_f / l_{base}^2) = 50$	8 kN/m ²			
Bearing pressure at heel	$P_{\text{toe}_{i}} = (\mathbf{R}_{f} / \mathbf{D}_{i})$	$p_{10e_1} = (1x_1 / 1base) = (0 \times 1x_1 \times C_1 / 1base) = 50.0 \text{ KiV/III}^{-1}$ $p_{10e_1} \in = (R_1 / 1base) + (6 \times R_2 \times e_2 / 1b_{22}) = 50.8 \text{ k/V/m}^2$						
Rate of change of base reaction		$rate = (n_{top} f - n_{heal} f) / f_{haco} = 0.00 k N/m^2/m$						
Bearing pressure at stem / toe	Determine $f = \max(\text{proc}_1 / \text{proc}_2 / \text{proc}_3 / \text{proc}_4 / \text{proc}_3 / \text{proc}_4 / $							
		pstem_toe_t - IIIax(ptoe_t - (Iate × Itoe), U KIV/III-) - DU.O KIV/III-						

Tekla Tedds	Project 25 OAK HILL ROAD, LONDON			Job Ref. 8415	Job Ref. 8415				
Мbр	Section RETAINING WALL			Sheet no./rev. 7. 18	Sheet no./rev. 7. 18				
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 21/10/2022			
Bearing pressure at mid stem Bearing pressure at stem / heel Calculate propping forces to tor	p _{stem_mid_f} = ma p _{stem_heel_f} = ma	$p_{stem_mid_f} = max(p_{toe_f} - (rate \times (l_{toe} + t_{wall} / 2)), 0 \text{ kN/m}^2) = 50.8 \text{ kN/m}^2$ $p_{stem_heel_f} = max(p_{toe_f} - (rate \times (l_{toe} + t_{wall})), 0 \text{ kN/m}^2) = 50.8 \text{ kN/m}^2$							
Propping force to top of wall	_								
Propping force to base of wall	F _{prop_top_f} = Propping force to base of wall		□ R _f × I _{base} / 2 - F prop_f - Fprop_top_f	- _{prop_f} × t _{base} / = 104.177 kl	V2) / (h _{stem} + t _{base} /) N/m	2) = 3.934 kN/m			
Design of reinforced concrete re	etaining wall t	oe (BS 8002:1994	<u>4)</u>						
Material properties		(05 N/	2						
Characteristic strength of concrete Characteristic strength of reinforce	ement	f _{cu} = 35 N/mm f _y = 500 N/mm	2 1 ²						
Base details									
Minimum area of reinforcement		k = 0.13 %							
Cover to reinforcement in toe		c _{toe} = 50 mm	c _{toe} = 50 mm						
Calculate shear for toe design Shear from bearing pressure		V _{toe bear} = (p _{toe}	f + Dstem toe f) ×	Itoe / 2 = 10 1	I. 7 kN/m				
Shear from weight of base		$V_{\text{toe}_wt_base} = \gamma_{f_d} \times \gamma_{\text{base}} \times I_{\text{toe}} \times t_{\text{base}} = 23.5 \text{ kN/m}$							
Total shear for toe design	Total shear for toe design		V _{toe} = V _{toe_bear} - V _{toe_wt_base} = 78.2 kN/m						
Calculate moment for toe design	ı								
Moment from bearing pressure	Moment from bearing pressure		$M_{\text{toe}_\text{bear}} = (2 \times p_{\text{toe}_f} + p_{\text{stem}_\text{mid}_f}) \times (I_{\text{toe}} + t_{\text{wall}} / 2)^2 / 6 = 114.8 \text{ kNm/m}$						
Moment from weight of base	Moment from weight of base		/f_d $ imes$ γ base $ imes$ tbase	$e \times (I_{\text{toe}} + t_{\text{wall}})$	/ 2) ² / 2) = 26.6 kN	m/m			
	•	•	•	•	•				
	∢ —200—								
Check toe in bending									
Width of toe	Width of toe		b = 1000 mm/m						
Depth of reinforcement	Constant		$u_{toe} - u_{base} - U_{toe} - (\psi_{toe} / z) - z \exists z. 0 \text{ IIIII}$ $K_{tee} = M_{tee} / (b \times d_{tee}^2 \times f_{ev}) = 0 \text{ 030}$						
Constant	Constant		Compression reinforcement is not reauired						
Lever arm		$z_{\text{toe}} = \min(0.5 + \sqrt{(0.25 - (\min(K_{\text{toe}}, 0.225) / 0.9)), 0.95)} \times d_{\text{toe}}$ $z_{\text{toe}} = 277 \text{ mm}$							
Area of tension reinforcement requ	$A_{s_toe_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) = 731 \text{ mm}^2/\text{m}$								
Minimum area of tension reinforce	$A_{s_toe_min} = k \times b \times t_{base} = 455 \text{ mm}^2/\text{m}$								
Area of tension reinforcement requ	Area of tension reinforcement required			$A_{s_toe_req} = Max(A_{s_toe_des}, A_{s_toe_min}) = 731 \text{ mm}^2/\text{m}$					

Tekla , Tedds	Project 25 OAK HI	Project 25 OAK HILL ROAD, LONDON Section RETAINING WALL				Job Ref. 8415			
Мbр	Section RETAININ					Sheet no./rev. 7. 19			
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 21/10/2022			
Reinforcement provided		16 mm dia.b	ars @ 200 mr	n centres					
Area of reinforcement provided		As_toe_prov = 10)05 mm²/m						
		PASS - Re	inforcement	provided at the	e retaining wall	toe is adequate			
Check shear resistance at toe									
Design shear stress		$v_{toe} = V_{toe} / (b$	\times d _{toe}) = 0.268	3 N/mm²					
Allowable shear stress		v _{adm} = min(0.8	8×√(f _{cu} / 1 N/	mm²), 5) × 1 N/	mm² = 4.733 N/	mm²			
		PASS - Design shear stress is less than maximum shear stres							
From BS8110:Part 1:1997 – Ta	ble 3.8								
Design concrete shear stress		v _{c_toe} = 0.536	N/mm ²						
				$v_{toe} < v_{c_{toe}} - N_{c_{toe}}$	o shear reinfor	cement required			
Design of reinforced concrete	retaining wal	stem (BS 8002:1	994)						
Material properties			<u>.</u>						
Characteristic strength of concre	te	f _{cu} = 35 N/mn	n ²						
Characteristic strength of reinforcement		f _v = 500 N/mr	$f_{\rm cu} = 500 \text{N/mm}^2$						
Wall dataile		.,							
Minimum area of reinforcement		k = 0 13 %							
Cover to reinforcement in stem		c _{stem} = 50 mm							
Cover to reinforcement in stell		c _{wall} = 50 mm							
Eastared barizontal at reat for	nan an atam								
Surpharao	ces on stem	E m . y		ax (ht.	$d_{\rm L}$) = 0.0 kN/m				
		$\Gamma_{s_sur_f} - \gamma_{f_l} \times$		Je × (Neff - Ubase -	$(u_{ds}) = 9.9 \text{ kiv/m}$				
NUISI DACKIIII DEIOW WALEF LADIE		$F_{s_m_b_f} = \gamma_{f_e}$	$\times \mathbf{K}_0 \times \gamma_{\mathrm{m}} \times (\mathrm{n}_{\mathrm{e}})$	ff - Ibase - Ods - As	$sat) \times \Pi sat = \mathbf{U} K \Pi /$	m			
		$F_{s_s} = 0.5 \times r_e \times R_0 \times (r_s - r_water) \times F_{sat} = 20.5 R_V/H$							
Water		Fs_water_f = 0.5	$0 imes \gamma_{f_e} imes \gamma_{water} imes$	< h _{sat} ² = 53.8 kN	I/m				
Calculate shear for stem desig	In								
Surcharge		$V_{s_sur_f} = 5 \times F_{s_sur_f} / 8 = 6.2 \text{ kN/m}$							
Moist backfill below water table		$V_{s_m_b_f} = F_{s_m_b_f} \times (8 - (n^2 \times (4 - n))) / 8 = 0 \text{ kN/m}$							
Saturated backfill		$V_{s_s_f} = F_{s_s_f} \times (1 - (a_1^2 \times ((5 \times L) - a_1) / (20 \times L^3))) = 22.6 \text{ kN/m}$							
Water		$V_{s_water_f} = F_{s_water_f} \times (1 - (a_1^2 \times ((5 \times L) - a_1) / (20 \times L^3))) = 43.1 \text{ kN/m}$							
Total shear for stem design		V _{stem} = V _{s_sur_1}	$_{f}$ + V _{s_m_b_f} + V	s_s_f + Vs_water_f =	= 71.9 kN/m				
Calculate moment for stem de	sign								
Surcharge		$M_{s_sur} = F_{s_sur_f} \times L / 8 = 3.7 \text{ kNm/m}$							
Moist backfill below water table		$M_{s_m_b} = F_{s_m_b_f} \times a_I \times (2 - n)^2 / 8 = 0 \text{ kNm/m}$							
Saturated backfill		M _{s_s} = F _{s_s_f} ×a _i ×((3×a _i ²)-(15×a _i ×L)+(20×L²))/(60×L²) = 11.2 kNm/m							
Water		M _{s_water} = F _{s_water_f} ×a _l ×((3×a _l ²)-(15×a _l ×L)+(20×L²))/(60×L²) = 21.4 kNm/m							
Total moment for stem design		$M_{stem} = M_{s_sur} + M_{s_m_b} + M_{s_s} + M_{s_water} = 36.2 \text{ kNm/m}$							
Calculate moment for wall des	ian	_							
Surcharge		$M_{W sur} = 9 \times F$	s sur f X L / 128	8 = 2.1 kNm/m					
Moist backfill below water table		$M_{w m b} = F_{s m b f} \times a_{l} \times [((8-n^{2}\times(4-n))^{2}/16)-4+n\times(4-n)]/8 = 0 kNm/m$							
Saturated backfill		$M_{w_0} = F_{0,0} \in Y [a_2^2 x Y ((5 \times 1 - a_1))/(20 \times 1 ^3) - (1 \times 1 ^3)/(2 \times 2 ^3)] = 5 k N m/m$							
		$\frac{1}{100} = \frac{1}{100} \frac{1}{100} = \frac{1}{100} \frac{1}{100} \frac{1}{1000} \frac{1}{1000}$							
Total moment for well design	$M_{\text{unit}} = M_{\text{unit}} + M_{\text{unit}} + M_{\text{unit}} + M_{\text{unit}} - 16.6 \text{ k/lm/m}$								
rotar moment for wall design	iviwali — IVIw_sur	י IVIw_m_b + IVIw	_s ⊤ iviw_water − 1	0.0 NINII/III					



Tekla Tedds Mbp	Project 25 OAK HILL ROAD, LONDON				Job Ref. 8415	
	Section RETAINING WALL				Sheet no./rev. 7. 21	
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 21/10/2022

Reinforcement provided		16 mm dia.bars @ 200 mm centres			
Area of reinforcement prov	ided	A _{s_wall_prov} = 1005 mm ² /m			
	PASS -	Reinforcement provided to the retaining wall at mid height is adequate			
Check retaining wall defle	ection				
Basic span/effective depth	ratio	ratio _{bas} = 20			
Design service stress		$f_s = 2 \times f_y \times A_{s_stem_req} / (3 \times A_{s_stem_prov}) = 151.5 \text{ N/mm}^2$			
Modification factor	factor _{tens} = min(0.55	5 + (477 N/mm ² - f _s)/(120 × (0.9 N/mm ² + (M _{stem} /(b × d _{stem} ²)))),2) = 1.99			
Maximum span/effective de	epth ratio	ratio _{max} = ratio _{bas} × factor _{tens} = 39.81			
Actual span/effective depth	ı ratio	ratio _{act} = h _{stem} / d _{stem} = 14.58			
		PASS - Span to depth ratio is acceptable			



Calculations Prepared by:

Name (Engineer) Agnieszka Zajac MSc Eng For Michael Barclay Partnership LLP

Calculations Approved by:

Name (Principal) Tony Hayes BSc (Hons) CEng MIStructE Date 20.10.2022