MBP	Michael Barclay Partnership	Job Title	Job Number	Sheet Number	Revision
	consulting engineers				
	72-78 Fleet Street,London EC4Y 1HY	Calculation/sketch Title	Date	Author	Checked
	T 020 7240 1191				
	E london@mbp-uk.com				

Loading to the side elevation



Assume the party walls support the roof and the timber floors.

Dead load of solid brick wall Dead load of timber floors Dead load of roof	$\begin{array}{l} G_{k1} = 6.21 \text{kN}/\text{m}^2 \\ G_{k2} = 0.85 \text{kN}/\text{m}^2 \\ G_{k3} = 1.25 \text{kN}/\text{m}^2 \end{array}$
Roof load (no access) Cat A occupancy	$Q_{k1} = 0.70 kN/m^2$ $Q_{k2} = 1.50 kN/m^2$
Height of party wall Loaded width of floor Span of roof supported	H = 11.00m $L_F = 5.50m$ $L_R = 5.50m$
Dead load from wall Dead load from floors Dead load from roof Total dead load	$\begin{split} W_{\rm Gk1} &= 6.21 \text{kN/m2 x } 11.00 \text{m} = 68.31 \text{kN/m} \\ W_{\rm Gk2} &= 0.85 \text{kN/m2 x } 5.50 \text{m} \text{ x } 2 \text{ floors} = 9.35 \text{kN/m} \\ W_{\rm Gk3} &= 1.25 \text{kN/m2 x } 5.50 \text{m} = 6.88 \text{kN/m} \\ \textbf{W}_{\rm Gk} &= \textbf{84.54 \text{kN/m}} \end{split}$
Imposed load from floors Imposed load from roof Total imposed load	$ W_{_{Qk1}} = 1.50 kN/m2 \ x \ 5.50m \ x \ 2 \ floors = 16.50 kN/m \\ W_{_{Qk2}} = 0.70 \ x \ 5.50m = 3.85 kN/m \\ W_{_{Qk}} = 20.35 kN/m $

Tekla Tedds	Project Jac 11-15 King's Terrace 8				Job Ref. 8292	
	Section Propped under	pin analysis and	design for party	wall	Sheet no./rev. 1	
	Calc. by JE	Date	Chk'd by MB	Date	App'd by	Date

RETAINING WALL ANALYSIS

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.12

Retaining wall details	
Stem type	Propped cantilever
Stem height	hstem = 2800 mm
Prop height	h _{prop} = 2800 mm
Stem thickness	t _{stem} = 300 mm
Angle to rear face of stem	α = 90 deg
Stem density	$\gamma_{\text{stem}} = 25 \text{ kN/m}^3$
Toe length	Itoe = 1000 mm
Base thickness	t _{base} = 350 mm
Base density	$\gamma_{\text{base}} = 25 \text{ kN/m}^3$
Height of retained soil	h _{ret} = 2800 mm
Angle of soil surface	$\beta = 0 \deg$
Depth of cover	d _{cover} = 0 mm
Height of water	h _{water} = 1550 mm
Water density	γw = 9.8 kN/m ³
Retained soil properties	
Soil type	Firm silty clay
Moist density	γmr = 19.5 kN/m ³
Saturated density	γsr = 19.5 kN/m ³
Characteristic effective shear resistance angle	φ'r.k = 24 deg
Characteristic wall friction angle	$\delta_{r,k} = 12 \text{ deg}$
Base soil properties	
Soil type	Firm silty clay
Soil density	$\gamma_{\rm b}$ = 19.5 kN/m ³
Characteristic effective shear resistance angle	φ'ь.k = 24 deg
Characteristic wall friction angle	$\delta b.k = 12 \text{ deg}$
Characteristic base friction angle	δbb.k = 16 deg
Presumed bearing capacity	Pbearing = 150 kN/m ²
Loading details	
Variable surcharge load	Surchargeo = 10 kN/m ²
Vertical line load at 1150 mm	P _{G1} = 85 kN/m
	Pq1 = 21 kN/m

Tekla Tedds	Project Jr 11-15 King's Terrace 8					
	Section Propped under	pin analysis and	design for party	wall	Sheet no./rev. 2	
	Calc. by JE	Date	Chk'd by MB	Date	App'd by	Date



Calculate retaining wall geometry

Base length	Ibase = Itoe + tstem = 1300 mm
Saturated soil height	hsat = hwater + dcover = 1550 mm
Moist soil height	hmoist = hret - hwater = 1250 mm
Length of surcharge load	Isur = Iheel = 0 mm
- Distance to vertical component	xsur_v = Ibase - Iheel / 2 = 1300 mm
Effective height of wall	heff = hbase + dcover + hret = 3150 mm
- Distance to horizontal component	x _{sur_h} = h _{eff} / 2 = 1575 mm
Area of wall stem	Astem = $h_{stem} \times t_{stem} = 0.84 \text{ m}^2$
- Distance to vertical component	Xstem = Itoe + tstem / 2 = 1150 mm
Area of wall base	Abase = Ibase \times tbase = 0.455 m ²
- Distance to vertical component	xbase = Ibase / 2 = 650 mm
Using Coulomb theory	
At rest pressure coefficient	$K_0 = 1 - \sin(\phi'_{r,k}) = 0.593$
Passive pressure coefficient	$K_{P} = \sin(90 - \phi'_{b.k})^{2} / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k})} \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k})) \times (\sin(90 + \delta_{b.k}) \times \sin(\phi'_{b.k})) $
	+ δ _{b.k}))]] ²) = 3.337
Bearing pressure check	
Vertical forces on wall	
Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} = 21 \text{ kN/m}$
Wall base	$F_{\text{base}} = A_{\text{base}} \times \gamma_{\text{base}} = 11.4 \text{ kN/m}$

Tekla Tedds	Project 11-15 King	Project 11-15 King's Terrace				Job Ref. 8292			
	Section Propped ur	nderpin analysi	s and design for pa	arty wall	Sheet no./rev. 3				
	Calc. by JE	Date	Chk'd by MB	Date	App'd by	Date			
Line loads		$F_{P_v} = P_{G_1}$	+ Pq1 = 106 kN/m	ו					
Total		$F_{total_v} = F$	stem + Fbase + FP_v -	+ F _{water_v} = 138	.4 kN/m				
Horizontal forces on wall									
Surcharge load		$F_{sur_h} = K_0$	$\infty \times \cos(\delta_{r.k}) \times Surc$	hargeq × heff =	18.3 kN/m				
Saturated retained soil	Saturated retained soil Water Moist retained soil			$F_{\text{sat}_h} = K_0 \times \cos(\delta_{r.k}) \times (\gamma_{\text{sr}} - \gamma_w) \times (h_{\text{sat}} + h_{\text{base}})^2 / 2 = 10.1 \text{ kN/m}$					
Water				$F_{water_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 = 17.7 \text{ kN/m}$					
Moist retained soil				$F_{moist_h} = K_0 \times cos(\delta_{r.k}) \times \gamma_{mr} \times ((heff - h_{sat} - h_{base})^2 / 2 + (heff - h_{sat} - h_{base}) \times (h_{sat})^2 / 2 + (heff - h_{sat} - h_{base}) \times (h_{sat} - h_{base})^2 / 2 + (heff - h_{sat} - h_{base}) \times (h_{sat} - h_{base})^2 / 2 + (heff - h_{sat} - h_{base}) \times (h_{sat} - h_{base})^2 / 2 + (heff - h_{sat} - h_{base})^2 / 2 + (heff - h_{sat} - h_{base}) \times (h_{sat} - h_{base})^2 / 2 + (heff - h_{sat} - h_{base})^2 / 2 + (heff - h_{sat} - h_{base})^2 / 2 + (heff - h_{sat} - h_{base})^2 / 2 + (h_{sat} - h_{base})^2 / 2 + (h$					
		+ h _{base})) =	35.7 kN/m						
Base soil		$F_{pass_h} = -$	$K_{P} \times cos(\delta_{b,k}) \times \gamma_{b}$	imes (d _{cover} + h _{base})² / 2 = -3.9 kN/m				
Total		$F_{total_h} = F$	sur_h + Fsat_h + Fwate	er_h + Fmoist_h +	F _{pass_h} = 78 kN/m				
Moments on wall									
Wall stem		Mstem = Fs	$tem \times Xstem = 24.2 k$	Nm/m					
Wall base		Mbase = Fb	$ase \times Xbase = 7.4 \text{ kN}$	lm/m					
Surcharge load		$M_{sur} = -F_{st}$	ur_h × Xsur_h = -28.8	kNm/m					
Line loads		MP = (PG1	+ Pq1) × p1 = 121 .	. 9 kNm/m					
Saturated retained soil		Msat = -Fsa	at_h × X sat_h = -6.4 k	Nm/m					
Water		M _{water} = -F	water_h × Xwater_h = -	11.2 kNm/m					
Moist retained soil		Mmoist = -F	moist_h × Xmoist_h = -4	46 kNm/m					
Total		Mtotal = Ms	tem + Mbase + Msur +	- MP + Msat + N	Nwater + Mmoist = 61	kNm/m			
Check bearing pressure									
Propping force to stem		Fprop_stem =	= ($F_{total_v} \times I_{base} / 2 \cdot$	• Mtotal) / (hprop ·	+ t _{base}) = 9.2 kN/m				
Propping force to base		Fprop_base =	= Ftotal_h - Fprop_stem	= 68.8 kN/m					
Moment from propping force		$M_{prop} = F_p$	rop_stem $ imes$ (hprop + tba	_{ase}) = 28.9 kNn	n/m				
Distance to reaction		$\overline{\mathbf{x}} = (\mathbf{M}_{\text{total}})$	ıl + M prop) / Ftotal v =	650 mm					

Distance to reaction Eccentricity of reaction Loaded length of base Bearing pressure at toe Bearing pressure at heel Factor of safety

FoS_{bp} = P_{bearing} / max(q_{toe}, q_{heel}) = **1.409** PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

 $e = \overline{x} - I_{base} / 2 = 0 mm$

 $q_{toe} = F_{total_v} / I_{base} \times (1 - 6 \times e / I_{base}) = 106.4 \text{ kN/m}^2$

 $q_{\text{heel}} = F_{\text{total}_v} / I_{\text{base}} \times (1 + 6 \times e / I_{\text{base}}) = 106.4 \text{ kN/m}^2$

lload = lbase = 1300 mm

RETAINING WALL DESIGN

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 2.9.12

Concrete details - Table 3.1 - Strength and deformation characteristics for concrete

Concrete strength class	C35/45
Characteristic compressive cylinder strength	f _{ck} = 35 N/mm ²
Characteristic compressive cube strength	f _{ck,cube} = 45 N/mm ²
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8 N/mm^2 = 43 N/mm^2$
Mean value of axial tensile strength	f_{ctm} = 0.3 N/mm ² × (f _{ck} / 1 N/mm ²) ^{2/3} = 3.2 N/mm ²
5% fractile of axial tensile strength	$f_{ctk,0.05} = 0.7 \times f_{ctm} = 2.2 \text{ N/mm}^2$
Secant modulus of elasticity of concrete	$E_{cm} = 22 \text{ kN/mm}^2 \times (f_{cm} / 10 \text{ N/mm}^2)^{0.3} = 34077 \text{ N/mm}^2$

Tekla Tedds	Project 11-15 King's Te	errace			Job Ref. 8292	
	Section Sh Propped underpin analysis and design for party wall 4				Sheet no./rev. 4	
	Calc. by JE	Date	Chk'd by MB	Date	App'd by	Date

Partial factor for concrete - Table 2.1N	γc = 1.50
Compressive strength coefficient - cl.3.1.6(1)	αcc = 0.85
Design compressive concrete strength - exp.3.15	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma c = 19.8 \text{ N/mm}^2$
Maximum aggregate size	h _{agg} = 20 mm
Ultimate strain - Table 3.1	εcu2 = 0.0035
Shortening strain - Table 3.1	εсиз = 0.0035
Effective compression zone height factor	$\lambda = 0.80$
Effective strength factor	η = 1.00
Bending coefficient k1	K1 = 0.40
Bending coefficient k2	$K_2 = 1.00 \times (0.6 + 0.0014/\epsilon_{cu2}) = 1.00$
Bending coefficient k3	K ₃ = 0.40
Bending coefficient k4	$K_4 = 1.00 \times (0.6 + 0.0014/\epsilon_{cu2}) = 1.00$
Reinforcement details	
Characteristic yield strength of reinforcement	f _{yk} = 500 N/mm ²
Modulus of elasticity of reinforcement	Es = 200000 N/mm ²
Partial factor for reinforcing steel - Table 2.1N	γs = 1.15
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_S = 435 \text{ N/mm}^2$
Cover to reinforcement	
Front face of stem	csf = 25 mm
Rear face of stem	c _{sr} = 60 mm
Top face of base	c _{bt} = 25 mm
Bottom face of base	c _{bb} = 40 mm



Tekla Tedds	Project 11-15 King's	Terrace			Job Ref. 8292	
	derpin analysi	s and design for p	arty wall	Sheet no./rev. 5		
Calc. by JE		Date	Chk'd by MB	Date	App'd by	Date
-oading details	- Combination No 2 - kN/m ²	Shear force - Combination I	ia.2-kNim Bending mom	ent - Combination No.2 - kNm/m		
	15.97 15.97		16.3	-12.8 21.3	7.5	
Check stem design at 1595 mm Depth of section	n	h = 300 n	ım			
Rectangular section in flexure	- Section 6.1					
Design bending moment combin	ation 1	M = 15.2	kNm/m	-		
Depth to tension reinforcement		$d = h - C_{sf}$	$-\phi_{sx} - \phi_{sfM} / 2 = 25$	9 mm		
		K = M / (C	$1^2 \times f_{ck}$ = 0.006	(S K)//0 - K		
		$K' = (2 \times 1)$	1 × αcc/γC)×(1 - λ × 7	$(0 - K_1)/(2 \times K_2)$	$)) \times (\lambda \times (0 - K_1))/(2$	2 × N2))
		11 - 01201	K' > K	- No compres	sion reinforcen	nent is required
Lever arm		z = min(0	.5 + 0.5 × (1 - 2 ×	Κ / (η × αcc / γc)	$)^{0.5}, 0.95) \times d = 2$	246 mm
Depth of neutral axis		x = 2.5 ×	(d – z) = 32 mm			
Area of tension reinforcement re-	quired	AsfM.req =	$M / (f_{yd} \times z) = 142 r$	mm²/m		
Tension reinforcement provided		12 dia.ba	rs @ 200 c/c			
Area of tension reinforcement pro	ovided	AsfM.prov =	$\pi imes \phi_{sfM^2}$ / (4 $ imes$ SsfM) = 565 mm²/m		
Minimum area of reinforcement -	exp.9.1N	$A_{sfM.min} = I$	$max(0.26 \times f_{ctm} / f_{y})$	k, 0.0013) × d =	432 mm²/m	
Maximum area of reinforcement	- cl.9.2.1.1(3)	AsfM.max =	0.04 × h = 12000	mm²/m		
		max(AsfM.	eq, AsfM.min) / AsfM.pr	ov = 0.765	c c	
	PASS - Area	a of reinforce	ement provided is	s greater than a	area of reinforc Library item: Rec	ement required tangular single output
Deflection control - Section 7.4	1					
Reference reinforcement ratio		$\rho_0 = \sqrt{f_{ck}}$	′ 1 N/mm²) / 1000	= 0.006		
Required tension reinforcement	ratio	$\rho = A_{sfM.re}$	a / d = 0.001			
Required compression reinforcer	ment ratio	$\rho' = A_{sfM.2}$	req / d2 = 0.000			
Structural system factor - Table 7	7.4N	K _b = 1				
Reinforcement factor - exp.7.17		K₅ = min(500 N/mm ² / (fyk ×	AsfM.req / AsfM.prov	/), 1.5) = 1.5	

Limiting span to depth ratio - exp.7.16.a

Actual span to depth ratio

$$\begin{split} & \mathsf{K}_{s} = \min(500 \ \text{N/mm}^{2} \ / \ (\mathsf{f}_{yk} \times \mathsf{A}_{sfM.req} \ / \ \mathsf{A}_{sfM.prov}), \ 1.5) = \textbf{1.5} \\ & \min(\mathsf{K}_{s} \times \mathsf{K}_{b} \times [11 + 1.5 \times \sqrt{(\mathsf{f}_{ck} \ / \ 1 \ \text{N/mm}^{2})} \times \rho_{0} \ / \ \rho + 3.2 \times \sqrt{(\mathsf{f}_{ck} \ / \ 1 \ \text{N/mm}^{2})} \times (\rho_{0} \ / \ \rho - 1)^{3/2}], \ 40 \times \mathsf{K}_{b}) = \textbf{40} \\ & \mathsf{h}_{prop} \ / \ d = \textbf{10.8} \end{split}$$

Tekla Tedds	Project 11-15 King's Te	errace			Job Ref. 8292	
	Section Si Propped underpin analysis and design for party wall Si				Sheet no./rev. 6	
	Calc. by JE	Date	Chk'd by MB	Date	App'd by	Date

	PASS - Span to depth ratio is less than deflection control limit
Crack control - Section 7.3	
Limiting crack width	w _{max} = 0.3 mm
Variable load factor - EN1990 – Table A1.1	ψ2 = 0.6
Serviceability bending moment	M _{sis} = 9.7 kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sfM,prov} \times z) = 69.6 \text{ N/mm}^2$
Load duration	Long term
Load duration factor	$k_{t} = 0.4$
Effective area of concrete in tension	A _{c.eff} = min(2.5 × (h - d), (h - x) / 3, h / 2)
	A _{c.eff} = 89208 mm ² /m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm} = 3.2 \text{ N/mm}^2$
Reinforcement ratio	$\rho_{p.eff} = A_{sfM.prov} / A_{c.eff} = 0.006$
Modular ratio	$\alpha_{e} = E_{s} / E_{cm} = 5.869$
Bond property coefficient	k1 = 0.8
Strain distribution coefficient	k ₂ = 0.5
	k ₃ = 3.4
	k ₄ = 0.425
Maximum crack spacing - exp.7.11	Sr.max = $k_3 \times C_{sf} + k_1 \times k_2 \times k_4 \times \phi_{sfM} / \rho_{p.eff} = 407 \text{ mm}$
Maximum crack width - exp.7.8	$W_{k} = Sr.max \times max(\sigma_{s} - k_{t} \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_{e} \times \rho_{p.eff}), 0.6 \times \sigma_{s}) / E_{s}$
	w _k = 0.085 mm
	Wk / Wmax = 0.283
	PASS - Maximum crack width is less than limiting crack width
Check stem design at base of stem	
Depth of section	h = 300 mm
Rectangular section in flexure - Section 6.1	
Design bending moment combination 1	M = 33.1 kNm/m
Depth to tension reinforcement	d = h - csr - фsr / 2 = 234 mm
	$K = M / (d^2 \times f_{ck}) = 0.017$
	$K' = (2 \times \eta \times \alpha_{ccc} / \gamma_{C}) \times (1 - \lambda \times (\delta - K_1) / (2 \times K_2)) \times (\lambda \times (\delta - K_1) / (2 \times K_2))$
	K' = 0.207
	K' > K - No compression reinforcement is required
Lever arm	z = min(0.5 + 0.5 × (1 - 2 × K / (η × α _{cc} / γ c)) ^{0.5} , 0.95) × d = 222 mm
Depth of neutral axis	$x = 2.5 \times (d - z) = 29 \text{ mm}$
Area of tension reinforcement required	$A_{sr.req} = M / (f_{yd} \times z) = 342 \text{ mm}^2/\text{m}$
Tension reinforcement provided	12 dia.bars @ 200 c/c
Area of tension reinforcement provided	$A_{sr,prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = 565 \text{ mm}^2/\text{m}$
Minimum area of reinforcement - exp.9.1N	$A_{sr.min} = max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 391 \text{ mm}^2/\text{m}$
Maximum area of reinforcement - cl.9.2.1.1(3)	Asr.max = 0.04 × h = 12000 mm ² /m
	max(Asr.req, Asr.min) / Asr.prov = 0.691
PASS - Area	of reinforcement provided is greater than area of reinforcement required Library item: Rectangular single output

Deflection control -	Section 7.4
----------------------	-------------

Reference reinforcement ratio

 $\rho_0 = \sqrt{(f_{ck} / 1 \text{ N/mm}^2) / 1000} = 0.006$

Tekla. Tedds	Project 11-15 King	s Terrace Job Ref. 8292					
	Section		Sheet no./rev.				
	Propped ur	iderpin analysis	and design for pa	arty wall	7		
	Calc. by JE	Date	Chk'd by MB	Date	App'd by	Date	
Required tension reinforcement ration	0	$\rho = A_{sr.req} /$	d = 0.001				
Required compression reinforcement	nt ratio	$\rho' = A_{sr.2.red}$	/ d ₂ = 0.000				
Structural system factor - Table 7.4	N	Kb = 1					
Reinforcement factor - exp.7.17		K₅ = min(5	00 N/mm ² / (f _{yk} \times	Asr.req / Asr.prov),	1.5) = 1.5		
Limiting span to depth ratio - exp.7.	16.a	min(K₅ × K	йь × [11 + 1.5 × √(1	ck / 1 N/mm²) >	< ρ₀ / ρ + 3.2 × √	(fck / 1 N/mm²) ×	
		(ρ₀ / ρ - 1) ³	^{3/2}], $40 \times \text{K}_{b}$) = 40				
Actual span to depth ratio		$h_{prop} / d = $	12				
		P	ASS - Span to de	epth ratio is le	ess than deflect	ion control limi	
Crack control - Section 7.3							
Limiting crack width		Wmax = 0.3	mm				
Variable load factor - EN1990 - Tal	ble A1.1	$\psi_2 = 0.6$					
Serviceability bending moment		Msis = 21.6 kNm/m					
Tensile stress in reinforcement		$\sigma_{s} = M_{sls} / (A_{sr, prov} \times z) = 171.7 \text{ N/mm}^{2}$					
Load duration		Long term					
Load duration factor		$k_t = 0.4$					
Effective area of concrete in tensior	ì	$A_{c.eff} = min(2.5 \times (h - d), (h - x) / 3, h / 2)$					
		Ac.eff = 902	50 mm²/m	m			
Mean value of concrete tensile stren	ngth	$f_{ct.eff} = f_{ctm} =$	= 3.2 N/mm ²				
Reinforcement ratio		ρp.eff = Asr.prov / Ac.eff = 0.006					
Modular ratio		$\alpha_{e} = E_{s} / E_{cm} = 5.869$					
Bond property coefficient		k1 = 0.8					
Strain distribution coefficient		k ₂ = 0.5					
		k3 = 3.4					
		k4 = 0.425					
Maximum crack spacing - exp.7.11		Sr.max = k3 >	\times Csr + k ₁ \times k ₂ \times k ₄	$\times \phi_{sr} / \rho_{p.eff} = 5$	30 mm		
Maximum crack width - exp.7.8		Wk = Sr.max	$\times \max(\sigma_s - k_t \times (f_c$	t.eff / $ ho_{p.eff}$ × (1 ·	+ $\alpha_{e} \times \rho_{p.eff}$), 0.6	× σs) / Es	
		Wk = 0.273	mm				
		Wk / Wmax =	0.909				
		Р	ASS - Maximum	crack width is	s less than limit	ting crack width	
Rectangular section in shear - Se	ction 6.2						
Design shear force		V = 69.4 k	N/m				
		$C_{\text{Rd,c}} = 0.1$	8 / γc = 0.120				
		k = min(1 -	+ √(200 mm / d), 2	2) = 1.925			
Longitudinal reinforcement ratio		ρι = min(As	sr.prov / d, 0.02) = 0	.002			
		Vmin = 0.03	$5 \text{ N}^{1/2}/\text{mm} \times \text{k}^{3/2} \times$	fck ^{0.5} = 0.553 N	N/mm²		
Design shear resistance - exp.6.2a	& 6.2b	V _{Rd.c} = max	$x(C_{Rd.c} \times k \times (100))$	$N^2/mm^4 \times \rho \times 1$	f_{ck}) ^{1/3} , Vmin) × d		
		V _{Rd.c} = 129	.4 kN/m		, . ,		
		$V / V_{Rd.c} =$	0.536				
			PASS - Design s	hear resistan	ce exceeds des	sign shear force	
Check stem design at prop			-				
Depth of section		h = 300 m	m				
Pactangular soction in sheer So	ction 6 2						
Design shear force		\/ _ 21 6 k	N/m				
Design Shear 10100		v = 21.0 K	IN/111				

Project Job Ref. 11-15 King's Terrace 8292								
	Section	Sheet no./rev.						
	Propped under	erpin analysis an	in analysis and design for party wall 8					
	Calc. by JE	Date	Chk'd by MB	Date	App'd by	Date		
		C _{Rd,c} = 0.18 / -	γc = 0.120					
		k = min(1 + √(200 mm / d), 2) = 1.925				
Longitudinal reinforcement ratio		ρι = min(Asr1.pr	ov / d, 0.02) = 0	.002				
		v _{min} = 0.035 N	$^{1/2}/\text{mm} \times \text{k}^{3/2} \times$	fck ^{0.5} = 0.553	N/mm²			
Design shear resistance - exp.6.2	a & 6.2b	V _{Rd.c} = max(C	Rd.c × \mathbf{k} × (100	$N^2/mm^4 \times \rho \times$	fck) ^{1/3} , Vmin) $ imes$ d			
		VRd.c = 129.4	kN/m					
		$V / V_{Rd.c} = 0.1$	67					
		PA	SS - Design s	hear resistar	nce exceeds des	sign shear force		
Horizontal reinforcement parall	el to face of ste	em - Section 9.6						
Minimum area of reinforcement -	cl.9.6.3(1)	$A_{sx.req} = max(0)$	$0.25 \times A_{sr.prov}, 0$.001 × t _{stem}) =	300 mm²/m			
Maximum spacing of reinforceme	Ssx_max = 400 mm							
Transverse reinforcement provide	ed	10 dia.bars @ 200 c/c						
Area of transverse reinforcement	provided	$A_{\text{sx.prov}} = \pi \times \phi$	$sx^2 / (4 \times Ssx) = 3$	393 mm²/m				
	PASS - Area	of reinforcemer	nt provided is	greater than	area of reinford	ement required		
Check base design at toe								
Depth of section		h = 350 mm						
Rectangular section in flexure -	Section 6.1							
Design bending moment combination 1M =Depth to tension reinforcementd =		M = 67.2 kNm	/m					
		d = h - c _{bb} - φ _b	ь / 2 = 304 mm					
		$K = M / (d^2 \times f$	ck) = 0.021					
		K' = (2 × η × α K' = 0.207	ιcc/γc)×(1 - λ × ((δ - K1)/(2 × K	2))×(λ × (δ - K1)/(2 × K2))		
l ever arm		z = min(0.5 +	K' > K 0 5 × (1 - 2 × k	- No compres	ssion reinforcer	nent is required		
Depth of neutral axis		$x = 2.5 \times (d = 1)$	z) = 38 mm		, 0.00) × u =			
Area of tension reinforcement rec	wired	$A = 2.0 \times (G)$	∠) – 30 mm	m²/m				
Tension reinforcement provided	uncu	12 dia bars @	100 c/c					
Area of tension reinforcement provided	wided	Abb prov = $\pi \times \phi$	$h^{2}/(4 \times Shh) =$	1131 mm ² /m				
Minimum area of reinforcement -	exp 9 1N	$A_{bb min} = max(0)$	$0.26 \times f_{ctm} / f_{vk}$	0 0013) × d =	507 mm²/m			
Maximum area of reinforcement -	cl 9 2 1 1(3)	$A_{bbmax} = 0.04$	× h = 14000 m	m²/m				
	01.0.2.111(0)	max(Abb reg. At	\times m = 14000 m	: 0.473				
	PASS - Area	of reinforcemer	nt provided is	greater than	area of reinford	ement required		
				5	Library item: Red	ctangular single output		
Crack control - Section 7.3								
Limiting crack width		w _{max} = 0.3 mm	ı					
Variable load factor - EN1990 - T	able A1.1	ψ2 = 0.6						
Serviceability bending moment		Msis = 48.8 kN	m/m					
Tensile stress in reinforcement		$\sigma_s = M_{sls} / (A_{bb})$	$prov \times z) = 149.$	5 N/mm ²				
Load duration		Long term						
Load duration factor		$k_t = 0.4$						
Effective area of concrete in tensi	on	$A_{c.eff} = min(2.5)$	5 × (h - d), (h - :	x) / 3, h / 2)				
		Ac.eff = 104000) mm²/m					
.								

Reinforcement ratio Modular ratio Bond property coefficient Strain distribution coefficient Maximum crack spacing - exp.7.11 Maximum crack width - exp.7.8	Section Propped under Calc. by JE	pin analysis and Date $p_{p.eff} = A_{bb.prov} / \alpha_e = E_s / E_{cm} = k_1 = 0.8$ $k_2 = 0.5$ $k_3 = 3.4$ $k_4 = 0.425$ Sr.max = $k_3 \times C_{bt}$ $W_k = Sr.max \times M$	d design for pa Chk'd by MB Ac.eff = 0.011 5.869 + k1 × k2 × k4	arty wall Date	Sheet no./rev. 9 App'd by	Date	
Reinforcement ratio Modular ratio Bond property coefficient Strain distribution coefficient Maximum crack spacing - exp.7.11 Maximum crack width - exp.7.8	Calc. by JE	Date $p_{p.eff} = A_{bb.prov} / \alpha_e = E_s / E_{cm} = k_1 = 0.8 \\ k_2 = 0.5 \\ k_3 = 3.4 \\ k_4 = 0.425 \\ s_{r.max} = k_3 \times C_{bt} \\ w_k = s_{r.max} \times m$	Chk'd by MB Ac.eff = 0.011 5.869	Date	App'd by	Date	
Reinforcement ratio Modular ratio Bond property coefficient Strain distribution coefficient Maximum crack spacing - exp.7.11 Maximum crack width - exp.7.8		$\begin{array}{l} \rho_{p.eff} = A_{bb,prov} / \\ \alpha_e = E_s / E_{cm} = \\ k_1 = 0.8 \\ k_2 = 0.5 \\ k_3 = 3.4 \\ k_4 = 0.425 \\ s_{r.max} = k_3 \times C_{bt} \\ w_k = S_{r.max} \times m \end{array}$	Ac.eff = 0.011 5.869 • + k1 × k2 × k4	x det / 0# =			
Modular ratio Bond property coefficient Strain distribution coefficient Maximum crack spacing - exp.7.11 Maximum crack width - exp.7.8		$\alpha e = E_{s} / E_{cm} =$ $k_{1} = 0.8$ $k_{2} = 0.5$ $k_{3} = 3.4$ $k_{4} = 0.425$ $Sr.max = k_{3} \times Cbt$ $W_{k} = Sr.max \times M$	5.869 + k ₁ × k ₂ × k ₄	x det / 0# = 1			
Bond property coefficient Strain distribution coefficient Maximum crack spacing - exp.7.11 Maximum crack width - exp.7.8		$k_{1} = 0.8$ $k_{2} = 0.5$ $k_{3} = 3.4$ $k_{4} = 0.425$ Sr.max = $k_{3} \times Cbt$ $W_{k} = Sr.max \times M$	• + k 1 × k 2 × k 4	x det / 0# = *			
Strain distribution coefficient Maximum crack spacing - exp.7.11 Maximum crack width - exp.7.8		$k_{2} = 0.5$ $k_{3} = 3.4$ $k_{4} = 0.425$ Sr.max = $k_{3} \times Cbt$ $W_{k} = Sr.max \times M$	• + k 1 × k 2 × k 4	х фил (от <i>т</i> и т			
Maximum crack spacing - exp.7.11 Maximum crack width - exp.7.8		$k_3 = 3.4$ $k_4 = 0.425$ $s_{r.max} = k_3 \times Cbt$ $W_k = S_{r.max} \times m$	• + k 1 × k 2 × k 4	x det / 0 # = *			
Maximum crack spacing - exp.7.11 Maximum crack width - exp.7.8		$k_4 = 0.425$ Sr.max = $k_3 \times Cbb$ Wk = Sr.max $\times Mbb$	\mathbf{h} + $\mathbf{k}_1 \times \mathbf{k}_2 \times \mathbf{k}_4$	× dub / op = # = *			
Maximum crack spacing - exp.7.11 Maximum crack width - exp.7.8		$S_{r.max} = k_3 \times C_{bb}$ $W_k = S_{r.max} \times M_b$	+ $\mathbf{k}_1 \times \mathbf{k}_2 \times \mathbf{k}_4$	× dub / an at - "			
Maximum crack width - exp.7.8		$W_k = S_{r.max} \times M_k$		× ψου / pp.en - •	324 mm		
			$ax(\sigma_s - k_t \times (f_{ct}))$	t.eff / $ ho$ p.eff) $ imes$ (1	+ $\alpha_{e} \times \rho_{p.eff}$), 0.6 \times	σs) / Es	
		w _k = 0.145 mm					
		Wk / Wmax = 0.4	84				
		PASS	S - Maximum	crack width is	s less than limitir	ng crack w	
Rectangular section in snear - Se	ction 6.2						
Design shear force		V = 134.3 kN/i	n				
		$C_{Rd,c} = 0.18 / \gamma$	c = 0.120				
		$\begin{aligned} &k = \min(1 + \sqrt{(200 \text{ mm / d})}, 2) = \textbf{1.811} \\ &\rho_{I} = \min(A_{bb,prov} / d, 0.02) = \textbf{0.004} \\ &v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \textbf{0.505} \text{ N/mm}^{2} \end{aligned}$					
Longitudinal reinforcement ratio							
Design shear resistance - exp.6.2a	& 6.2b	$V_{\text{Rd.c}} = max(C_{\text{Rd.c}} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_I \times f_{\text{ck}})^{1/3}, \text{ v_{min}}) \times d$					
		VRd.c = 155.4 k	N/m				
		V / V _{Rd.c} = 0.86	64				
		PAS	SS - Design s	hear resistan	ice exceeds desig	gn shear fo	
Secondary transverse reinforcem	ent to base - S	ection 9.3					
Minimum area of reinforcement - cl	.9.3.1.1(2)	$A_{bx.req} = 0.2 \times A_{bx.req}$	Abb.prov = 226 n	nm²/m			
Maximum spacing of reinforcement	- cl.9.3.1.1(3)	Sbx_max = 450 n	nm				
Transverse reinforcement provided		10 dia.bars @	200 c/c				
Area of transverse reinforcement pr	ovided	$A_{bx.prov} = \pi \times \phi_{t}$	$x^2 / (4 \times Sbx) =$	393 mm²/m			
	PASS - Area c	of reinforcemer	t provided is	greater than	area of reinforce	ment requ	

Tekla Tedds	Project 11-15 King's Te	Job Ref. 8292				
	Section Propped under	Sheet no./rev. 10				
	Calc. by JE	Date	Chk'd by MB	Date	App'd by	Date



Reinforcement details