Ref: 11/17802 August 2016

Basement Impact Assessment (Hydrogeological and Structural)

At

Ornan Court, 2 Ornan Road, London, NW3 4PT

For

**Ornan Court Limited** 

# VOLUME 8 of 8

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**8.1 Structural Calculations** 

28.04.16 Date Sheet No. Martin Redston Associates Eng. 140 Consulting Civil & Structural Engineers JOD NO. 16-280 3 Edward Square, London N1 0SP Tel: 020 7837 5377 Fax: 020 7837 3211 V 6 Hale Lane, London NW7 3NX ORNAN COURT Tel: 020 8959 1666 Fax: 020 8906 8503 Email: martin@redston.org It is proposed to excavate a Basement level existing building. The Party will & Frant pelmi the walls will be underprimed with Engineering Brick stears pod footiligs. reinforced concrete The 0A Flank wall is to be underpined The reinforced concrete stem and with pad pasement does not affeid the The. Nor will and a reinforced carchete refairing wall will be constructed webe in. sections to support the earth beyond. wide beaun will be required to Steel support the was bearing walls above which be supported on sheel allumous Which will be supported on mass concrete tum pad foundations 10ADS LIVE DEAS 107242 4.8 m/m 2 Bichwork (225) 4.8 (330) 7.5 7.5 Connote Flion 15 2.5 5 Pitched Nort 0.75 1.35 2.1 Flast Vant 0.75 1.05 1.8 Timber Floor (Gand) 1.5 017 2:2 Blacket Stud 0.5 0.5

28.04.16 Date Sheet No. Martin Redston Associates Eng. GO 161 Consulting Civil & Structural Engineers 3 Edward Square, London N1 0SP Job No. Tel: 020 7837 5377 Fax: 020 7837 3211 ORNAN COURT 6 Hale Lane, London NW7 3NX Tel: 020 8959 1666 Fax: 020 8906 8503 Email: martin@redston.org Beam G6-G7 5 m = 3.6 m and-3nl 5 (4) x 4 = 40 m/m Brich LMA (10 (0.89) ( 12.35) = 50.4 M/m -90 m/m Muna = 146 KNm R = 220,5KN 12×1-5 = 219/W/m Hertice length = 6.4m Ty 252 x 252 uc 89 Monrable manual = 284 KNm > 219 - .: ok Check Deflection S= 5 × 90 (3600)" = 6.6 mm .: 04 Beam 63-64 , spen = 1.6 m Load = 90 KN/m MMM= 29KNM R=72KN the 254×257 4C 89. Ol my inspection

Date 28:04:16 Sheet No. Martin Redston Associates 162 Eng. Consulting Civil & Structural Engineers Job No. 16. 781 3 Edward Square, London N1 0SP Tel: 020 7837 5377 Fax: 020 7837 3211 ORNAN COURT 6 Hale Lane, London NW7 3NX Tel: 020 8959 1666 Fax: 020 8906 8503 Email: martin@redston.org Bellin BG-B7, -similar to GG-G7 Been 86 - 66, spon = 3.95m Point loud from B6-B7 & B5-B6 =90 / 63) = 283 KN Nominal Floor 2.7 /0.3) = 0.81 M/m 283kN 0.81m/m 10.8 3.15 1 0.81 (3.95)2 + 283 (0.8) = Rs 3.95 227.3 0.65 Re= 58.9KN 126.6 RA = 227.3W MNMX = 181.6KNm Win = 93KN/m 123×15=272.4KNm Afertice Oryth = 3.15m Ty 254 x254 UC89 Allowable manert = 316 >272 mm : Ok Check Dellection d = 5 × 93 (3950) 4 = 10mm - . OH

<u>28.04.16</u> 150 Date Sheet No. Martin Redston Associates Consulting Civil & Structural Engineers 163 Eng. Job No. 16. 280 3 Edward Square, London N1 0SP Tel: 020 7837 5377 Fax: 020 7837 3211 ORNAN COURT 6 Hale Lane, London NW7 3NX Tel: 020 8959 1666 Fax: 020 8906 8503 Email: martin@redston.org check Columns Mune: 227 (0,2) = 45.4Mm Woul = 227W Ty 203 × 203 11666  $f = \frac{2700}{51c1} = 53$  pc = 131 Dz= 18.5 plx = 179 W = <u>22.7×103</u> = 38.6 58.7×102 = 38.6  $M = \frac{45.6 \times 10^3}{449} = 101$  $\frac{101}{179} + \frac{38.6}{131} = 0.86 < 1.104$ Allowable Ground Beening Prenue = 250 m/m2 . Use 1200×1200 Achual (iBP = 227 = 157 KN/m2 : 10h

28.04.16 Date Sheet No. Martin Redston Associates 60 Eng. 164 Consulting Civil & Structural Engineers 16:280 Job No. 3 Edward Square, London N1 0SP V Tel: 020 7837 5377 Fax: 020 7837 3211 6 Hale Lane, London NW7 3NX ORNAN COURT Tel: 020 8959 1666 Fax: 020 8906 8503 Email: martin@redston.org Bay Window Support 330 Brid 7.5 (0.75) (13) : 73 W/m Part 211 (0.8) = 0.84 Flour  $5\left(\frac{0.8}{2}\right) \times 4 = 4$ Grow Flor 2.2 (0.1) = 0.88 -79/m Max spin = 2.9m Mmar = 82.8Mm R-115W for X 115 = 12.4 Wm Achine length = 2-9m : the 254,254 UC 73 Res Moment = 273 W/m > 124 - 04 S= 5 × 79 (2900)4 = 3mm indu side Reams, span = 1.2m the 254 UC73 Ok by hypection <u>HUMN - try 152 × 152 UC37</u> -= <u>2700</u> = 70 pc = 115 Munk = 115 (0:175) = 20 M/m 2700 plac = 167 D = 14 WA = 115×103 = 2414 73 + 24.4 = 0:65 × 1 . . . 0k - 73

Date 28.0616 Sheet No. Martin Redston Associates Consulting Civil & Structural Engineers 165 Eng. 6.280 Job No. 3 Edward Square, London N1 0SP 1 Tel: 020 7837 5377 Fax: 020 7837 3211 ORNAN COURT 6 Hale Lane, London NW7 3NX Tel: 020 8959 1666 Fax: 020 8906 8503 Email: martin@redston.org Blam FLE-F7, spin=6.6m Timber Flor 2.7 ( 6 ) = 8.1 M/m Shud Wall 0.5 (2.9) = 1.35 9.45 hN/m Mmw = 5115KNm R= 31KN DIL 115= 77-1 Mm Afertine spr = 7m i. the 203 x 203 UC 60 Allowable numerit: 142Mm > 77.1 i.de J= 5 × 9.45/6600) + = 18mm - in

Date 28.04.16 Sheet No. Martin Redston Associates Consulting Civil & Structural Engineers Eng. 166 Job No. 16.20 3 Edward Square, London N1 0SP V Tel: 020 7837 5377 Fax: 020 7837 3211 DRNAN COURT 6 Hale Lane, London NW7 3NX Tel: 020 8959 1666 Fax: 020 8906 8503 Email: martin@redston.org Bearing along Rid Line 4, mine span = 2.9m At Grownell Landa Care Floor 5 (10:4) × 3 = 78 m/m Timber Flux 2:7/10:4) ×2 = 28 Print 2.1 ( 10:4) = 10.92 Ground Flow 2.7 (15) = 2 170 kN/m 205 Bille 4.8/0.15/12.5 = Point load from F4-F7 = 31 M 170 kulm Z 31hu 170 (2.9) + 31 (2.4) = Ro 2.9 252 Re= 272 M Rr = 252 KN 216 pix15= 280,5Hlm MMMR = 187 W/m Nay= 178/11/m Effective length = 2.5m .: the 203×203 UC86 Monsable manent = 259 M/m >178 M/m J= 5 × 178 (2900)4 = 8mm . '. ON

Date 28.04.16 Sheet No. Martin Redston Associates Consulting Civil & Structural Engineers 167 Eng. 16.280 Job No. 3 Edward Square, London N1 0SP 4 Tel: 020 7837 5377 Fax: 020 7837 3211 ORNAN COURT 6 Hale Lane, London NW7 3NX Tel: 020 8959 1666 Fax: 020 8906 8503 Email: martin@redston.org Column F4  $\frac{1}{12} \frac{1}{12} = 272 + 170 \left(\frac{12}{2}\right) = 374 \text{ keV}$   $\frac{1}{12} \frac{1}{2} = 374 \text{ keV}$   $\frac{1}{12} \frac{1}{2} = 372 \left(\frac{1}{2}\right) = 32(12) = 32(14)$ Ty 203 x 203 NC 60 L = 2700 = 52 pc = 131 D = 14.7 T = 376+103 17 76.4 plac = 180 = 4819  $\frac{M}{2} = \frac{521.410^3}{584}$ = 93.15 

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Date 28.04.16 Sheet No. Martin Redston Associates Eng. GO Consulting Civil & Structural Engineers 168 Job No. 16.280 3 Edward Square, London N1 0SP 4 Tel: 020 7837 5377 Fax: 020 7837 3211 6 Hale Lane, London NW7 3NX DANAN COURT Tel: 020 8959 1666 Fax: 020 8906 8503 Email: martin@redston.org LOADS ON GRID LINE 7 (At Ground Flow level) 330 Brich 7.5 (0.75) (12.5) = 70.3 KN/m Conc Floor 5/63) - 3= 47.25 Timber Plan 2. 2/ 5.3) x 2 = 17.01 P. P. P. 2.1 ( 5.3 ) = 6.6 1(5) . 5 146M/m Mansord Steels for temperary opening, now span = 4.15m MMAK= 314.KNm fork 15= 471 Mm Ty 2/ 406 × 178 UB60 Allowable manert = 240x2 = 480Mm > 471 . Ok d= 5 × 146 (4150) = 6mm - '.OK FOUNDATIONS ON GRID LINE 7 LOAD = 146 M/m BASAMENT WALL 9.6(3)(0185) = 24.5 Une 1200 wide Footings GBP = 171 = 142 m/m² < 250 m/m²

Date 28:04.16 Sheet No. Martin Redston Associates Eng. 60 169 Consulting Civil & Structural Engineers 16.280 Job No. 3 Edward Square, London N1 0SP Tel: 020 7837 5377 Fax: 020 7837 3211 ORNAN COMRT 6 Hale Lane, London NW7 3NX Tel: 020 8959 1666 Fax: 020 8906 8503 Email: martin@redston.org LOASS FROM LOADS ON GRID CINE A/L(At Ground Floor level) Brick 7.5(17) = 127.5KN/m NEIBUBARES Conc Fluxs 5 ( 4 ) × 3: 30 30 5.4 Timber Floors 2.7(4) = 5.4 163 W 198.44 LOADS ON GRID LINE ? (At Ground level) 33 Brich 7.5 (0.85) (12.5) = 79.6 Care Flow 5 ( 4 ) = 3 = 30 Timber Floor 2.7 (4) > 2 = 10.8 P. Put 2. (-4) = 412 Manuard 1(5) = 5 130 Ker/m .



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## **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 2.4.08

Retaining wall details	
Stem type	Cantilever
Stem height	h <sub>stem</sub> = <b>1850</b> mm
Prop height	h <sub>prop</sub> = <b>1360</b> mm
Stem thickness	t <sub>stem</sub> = 660 mm
Angle to rear face of stem	α <b>= 90</b> deg
Stem density	$\gamma_{\text{stem}} = 25 \text{ kN/m}^3$
Toe length	I <sub>toe</sub> = <b>1350</b> mm
Base thickness	t <sub>base</sub> = <b>600</b> mm
Base density	$\gamma_{\text{base}} = 25 \text{ kN/m}^3$
Height of retained soil	h <sub>ret</sub> = <b>1850</b> mm
Angle of soil surface	$\beta = 0 \deg$
Depth of cover	d <sub>cover</sub> = 0 mm
Height of water	h <sub>water</sub> = 1000 mm
Water density	γw = <b>9.8</b> kN/m <sup>3</sup>
Retained soil properties	
Soil type	Organic clay
Moist density	$\gamma_{mr}$ = <b>15</b> kN/m <sup>3</sup>
Saturated density	$\gamma_{sr}$ = <b>15</b> kN/m <sup>3</sup>
Characteristic effective shear resistance angle	φ'r.k = <b>18</b> deg
Characteristic wall friction angle	δ <sub>r.k</sub> = <b>9</b> deg
Base soil properties	
Soil type	Organic clay
Moist density	$\gamma_{mb}$ = <b>15</b> kN/m <sup>3</sup>
Characteristic effective shear resistance angle	φ'ь.κ <b>= 18</b> deg
Characteristic wall friction angle	δ <sub>b.k</sub> = <b>9</b> deg
Characteristic base friction angle	δ <sub>bb.k</sub> = <b>12</b> deg
Presumed bearing capacity	$P_{\text{bearing}} = 250 \text{ kN/m}^2$
Loading details	
Permanent surcharge load	Surcharge <sub>G</sub> = 1 kN/m <sup>2</sup>
Variable surcharge load	Surchargeo = 1.5 kN/m <sup>2</sup>





# Using Coulomb theory

Active pressure coefficient

Passive pressure coefficient

**Bearing pressure check** 

Vertical forces on wall

Wall stem

Wall base

Line loads

$$\begin{split} &\mathsf{K}_{\mathsf{A}} = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k})} \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]]^2) = \mathbf{0.483} \\ &\mathsf{K}_{\mathsf{P}} = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k})} \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]]^2) = \mathbf{2.359} \end{split}$$

 $F_{stem} = A_{stem} \times \gamma_{stem} = 30.5 \text{ kN/m}$   $F_{base} = A_{base} \times \gamma_{base} = 30.2 \text{ kN/m}$  $F_{P_v} = P_{G1} + P_{Q1} = 198 \text{ kN/m}$ 

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Total	$F_{total_v} = F_{stem} + F_{base} + F_{water_v} + F_{P_v} = 258.7 \text{ kN/m}$							
Horizontal forces on wall								
Surcharge load		$F_{sur_h} = K_A$	× $\cos(\delta_{r.d})$ × (S	urcharge <sub>G</sub> + Suro	chargea) × heff :	= <b>2.9</b> kN/m		
Saturated retained soil		F <sub>sat_h</sub> = K <sub>A</sub> :	× $\cos(\delta r.d)$ × ( $\gamma$ s	r - $\gamma_w$ ) $ imes$ (hsat + hba	ase) <sup>2</sup> / 2 <b>= 3.2</b> kl	N/m		
Water		$F_{water_h} = \gamma_w$	$ \times (h_{water} + d_{cov}) $	<sub>er</sub> + h <sub>base</sub> ) <sup>2</sup> / 2 = 1	<b>2.6</b> kN/m			
Moist retained soil		F <sub>moist_h</sub> = KA	$\mathbf{A} \times \mathbf{COS}(\delta_{r.d}) \times \gamma$	mr × ((heff - hsat - h	$(base)^2 / 2 + (h_{eff})^2$	- $h_{sat}$ - $h_{base}$ ) ×		
	$(h_{sat} + h_{base})) = 12.3 \text{ kN/m}$							
Total	F <sub>total_h</sub> = F <sub>sat_h</sub> + F <sub>moist_h</sub> + F <sub>water_h</sub> + F <sub>sur_h</sub> = <b>31</b> kN/m							
Moments on wall								
Wall stem		M <sub>stem</sub> = F <sub>ste</sub>	m × Xstem = 51.3	₿ kNm/m				
Wall base		M <sub>base</sub> = F <sub>bas</sub>	se × x <sub>base</sub> = 30.3	<b>8</b> kNm/m				
Surcharge load		Msur = -Fsur_	$h \times \mathbf{X}_{sur_h} = -3.6$	<b>6</b> kNm/m				
Line loads		M <sub>P</sub> = (P <sub>G1</sub> +	+ Pq1) × p1 = 32	2 <b>6.7</b> kNm/m				
Saturated retained soil		Msat = -Fsat_	_h × <b>X</b> sat_h = -1.7	′ kNm/m				
Water		M <sub>water</sub> = -F <sub>w</sub>	rater_h × <b>X</b> water_h =	= <b>-6.7</b> kNm/m				
Moist retained soil		Mmoist = -Fm	noist_h × <b>X</b> moist_h =	- <b>12.7</b> kNm/m				
Total		M <sub>total</sub> = M <sub>ste</sub>	m + M <sub>base</sub> + M <sub>sa</sub>	at + M <sub>moist</sub> + M <sub>water</sub>	+ $M_{sur}$ + $M_P$ = 3	<b>383.7</b> kNm/m		
Check bearing pressure								
Propping force		F <sub>prop_base</sub> =	F <sub>total_h</sub> = <b>31</b> kN	/m				
Distance to reaction		$\overline{\mathbf{x}} = (\mathbf{M}_{\text{total}})$	+ Mprop) / Ftotal_v	/ = <b>1483</b> mm				
Eccentricity of reaction		$e = \bar{x} - I_{bas}$	<sub>e</sub> / 2 <b>= 478</b> mm	1				
Loaded length of base		$I_{load} = 2 \times (I_{load})$	$base - \bar{x} = 105$	<b>4</b> mm				
Bearing pressure at toe		$q_{toe} = 0 \text{ kN/}$	′m²					
Bearing pressure at heel		$q_{heel} = F_{total}$	_v / I <sub>load</sub> = <b>245.5</b>	kN/m²				
Factor of safety		$FoS_{bp} = P_{bearing} / max(q_{toe}, q_{heel}) = 1.018$						
	PASS - Allowable bearing pressure exceeds maximum applied bearing pressure							

#### **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 and EN1996-1-1:2005 incorporating Corrigenda dated February 2006 and July 2009 and the UK National Annex

Tedds calculation version 2.4.08

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

Concrete strength class	C30/37
Characteristic compressive cylinder strength	f <sub>ck</sub> = <b>30</b> N/mm <sup>2</sup>
Characteristic compressive cube strength	fck,cube = <b>37</b> N/mm <sup>2</sup>

 $f_{cm} = f_{ck} + 8 \text{ N/mm}^2 = 38 \text{ N/mm}^2$ Mean value of compressive cylinder strength  $f_{ctm}$  = 0.3 N/mm<sup>2</sup> × ( $f_{ck}$  / 1 N/mm<sup>2</sup>)<sup>2/3</sup> = **2.9** N/mm<sup>2</sup> Mean value of axial tensile strength 5% fractile of axial tensile strength f<sub>ctk,0.05</sub> = 0.7 × f<sub>ctm</sub> = **2.0** N/mm<sup>2</sup> E<sub>cm</sub> = 22 kN/mm<sup>2</sup> × (f<sub>cm</sub> / 10 N/mm<sup>2</sup>)<sup>0.3</sup> = **32837** N/mm<sup>2</sup> Secant modulus of elasticity of concrete Partial factor for concrete - Table 2.1N γc **= 1.50** Compressive strength coefficient - cl.3.1.6(1) α<sub>cc</sub> = 0.85 Design compressive concrete strength - exp.3.15  $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_c = 17.0 \text{ N/mm}^2$ Maximum aggregate size h<sub>agg</sub> = **20** mm **Reinforcement details** 

Characteristic yield strength of reinforcement  $f_{yk} = 500 \text{ N/mm}^2$ 

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Modulus of elasticity of reinforc	ement	Es <b>= 20000</b>	<b>0</b> N/mm <sup>2</sup>				
Partial factor for reinforcing stee	el - Table 2.1N	γs <b>= 1.15</b>					
Design yield strength of reinford	cement	f <sub>yd</sub> = f <sub>yk</sub> / γs	<b>= 435</b> N/mm <sup>2</sup>				
Cover to reinforcement							
Top face of base		c <sub>bt</sub> = <b>50</b> mm	l				
Bottom face of base		с <sub>ьь</sub> = <b>75</b> mr	n				
Masonry details - Section 3.1							
Masonry type		Clav with w	ater absorption	of < 7% - Grour	o 1		
Normalised mean compressive	strength	f₀ <b>= 50</b> N/m	m <sup>2</sup>				
Characteristic flexural strength	- Table NA.6	f <sub>xk</sub> = <b>0.5</b> N/r	nm²				
Initial shear strength - Table NA	۹.5	fvko <b>= 0.15</b> №	N/mm <sup>2</sup>				
Mortar details - Section 3.2							
Mortar type		General pu	rpose - M6, pres	scribed mix			
Compressive strength of morta	r	f <sub>m</sub> = 6 N/mr	n <sup>2</sup>				
Illtimate limit states - Table N	ΙΔ 1						
Class of execution control		1					
Category of manufacture control	ol	1					
Partial factor for direct or flexur	al compression	γ <sub>Mc</sub> = <b>2.3</b>					
Partial factor for flexural tensior	י ז	γ <sub>Mt</sub> = <b>2.3</b>					
Partial factor for shear		γ <sub>Mv</sub> = <b>2.5</b>					
Chack stem design at hase o	fstom	·					
Depth of section	1 Stell	t = <b>660</b> mm					
Masonry characteristics							
Compressive strength constant	s - Table NA.4	K = 0.5					
Characteristic compressive stre	ength - cl.3.6.1.2(1	) $f_k = K \times f_b^{0.7}$	$\times  \mathrm{fm}^{0.3} = 13.234$	N/mm <sup>2</sup>			
Design compressive strength		$f_d = f_k / \gamma_{Mc}$ =	= <b>5.754</b> N/mm <sup>2</sup>				
Design flexural strength		$f_{xd} = f_{xk} / \gamma_{Mt}$	= 0.217 N/mm <sup>2</sup>				
Height of masonry		h <sub>wt</sub> = h <sub>stem</sub> =	<b>1850</b> mm				
Compressive axial force combined	nation 1	<b>F = (</b> γ <sub>Gf</sub> × (γ	$_{ m stem}  imes h_{ m wt}  imes t + Po$	<sub>G1</sub> ) + γ <sub>Qf</sub> × P <sub>Q1</sub> ) =	= <b>198.5</b> kN/m		
Eccentricity of axial load		e <b>= 25.4</b> mr	n				
Capacity reduction factor - exp.	6.4	$\Phi$ = 1 - 2 ×	e / t = <b>0.923</b>				
Design vertical resistance - exp	0.6.2	$N_{Rd} = \Phi \times t$	× fd = 3505.5 kM	N/m			
Design vertical compressive str	ess	$\sigma_d = \min(F)$	/ t, 0.15 $ imes$ N <sub>Rd</sub> / t	t) = <b>0.301</b> N/mm	1 <sup>2</sup>		
Apparent design flexural streng	th - exp.6.16	$f_{xd,app} = f_{xd} +$	- σd <b>= 0.518</b> N/m	1m²			
Characteristic shear strength -	exp.3.5	fvk = min(fvk	$_{ m o}$ + 0.4 $ imes$ $\sigma_{ m d}$ , 0.0	65 × fb) = <b>0.27</b> N	N/mm <sup>2</sup>		
Design shear strength		$f_{vd} = f_{vk} / \gamma_{Mv}$	<b>- = 0.108</b> N/mm <sup>2</sup>	2			
Unreinforced masonry walls	subjected to late	ral loadin <u>g</u> - S	ection 6.3				
Design bending moment combi	nation 1	M = <b>14.289</b>	kNm/m				
Elastic section modulus of wall		$Z = t^2 / 6 =$	<b>72600000</b> mm³/	'n			
Moment of resistance of masor	ıry - exp.6.15	$M_{Rd} = f_{xd,app}$	× Z <b>= 37.62</b> kN	m/m			
		M / M <sub>Rd</sub> = 0	M / M <sub>Rd</sub> = 0.380				
		PASS	Moment of res	sistance excee	ds applied de	esign momen	
Unreinforced masonry walls	subjected to shea	ar loading - Se	ection 6.2				
Design shear force		V = 23.173	kN/m				
Maximum compressive stress		$\sigma_{max}$ = F / t	+ M / Z = <b>0.498</b>	N/mm <sup>2</sup>			

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Minimum compressive stress		$\sigma_{min} = F / t$	- M / Z <b>= 0.104</b>	N/mm <sup>2</sup>						
Thickness of wall in compression	on	t <sub>c</sub> = t = 660	mm							
Design shear resistance - exp.	6.13	$V_{Rd} = f_{vd} \times f_{vd}$	tc = <b>71.364</b> kN/	'n						
		V / V <sub>Rd</sub> = 0.	325							
		PASS - Desi	gn shear resi	stance exceeds	applied desig	gn shear force				
Check stem design at 460 m	m									
Depth of section		t = <b>660</b> mm	1							
Masonry characteristics										
Compressive strength constant	ts - Table NA.4	K = <b>0.5</b>								
Characteristic compressive stre	ength - cl.3.6.1.2	$2(1)  f_k = K \times f_b^{0.7}$	′ × f <sub>m</sub> <sup>0.3</sup> = <b>13.2</b> 3	<b>34</b> N/mm <sup>2</sup>						
Design compressive strength		$f_d = f_k / \gamma_{Mc}$	= <b>5.754</b> N/mm <sup>2</sup>	2						
Design flexural strength	Design flexural strength f <sub>x</sub>			f <sub>xd</sub> = f <sub>xk</sub> / γ <sub>Mt</sub> = <b>0.217</b> N/mm <sup>2</sup>						
Height of masonry		h <sub>wt</sub> = h <sub>stem</sub> -	1wt = h <sub>stem</sub> - 460 mm = <b>1390</b> mm							
Compressive axial force combi	nation 1	$F = (\gamma_{Gf} \times (\gamma_{ff}))$	$F = (\gamma_{Gf} \times (\gamma_{stem} \times h_{wt} \times t + P_{G1}) + \gamma_{Qf} \times P_{Q1}) = 190.9 \text{ kN/m}$							
Eccentricity of axial load		e = <b>26.4</b> m	m							
Capacity reduction factor - exp	.6.4	$\Phi$ = 1 - 2 ×	e / t = <b>0.92</b>							
Design vertical resistance - exp	0.6.2	$N_{Rd}$ = $\Phi  imes t$	× fd = 3493.8	kN/m						
Design vertical compressive st	ress	$\sigma_d = \min(F)$	/ t, $0.15 \times N_{Rd}$	/ t) = <b>0.289</b> N/mn	n <sup>2</sup>					
Apparent design flexural streng	gth - exp.6.16	$f_{xd,app} = f_{xd}$ -	⊦ σd <b>= 0.507</b> N/	/mm²						
Characteristic shear strength -	exp.3.5	f <sub>vk</sub> = min(f <sub>vk</sub>	$_{\rm to}$ + 0.4 $ imes$ $\sigma_{\rm d}$ , 0.	065 × f <sub>b</sub> ) = <b>0.266</b>	N/mm <sup>2</sup>					
Design shear strength		$f_{vd} = f_{vk} / \gamma_M$	f <sub>vd</sub> = f <sub>vk</sub> / γ <sub>Mv</sub> = <b>0.106</b> N/mm <sup>2</sup>							
Unreinforced masonry walls	subjected to la	teral loading - S	ection 6.3							
Design bending moment comb	ination 1	M = <b>6.166</b>	kNm/m							
Elastic section modulus of wall		$Z = t^2 / 6 =$	72600000 mm	<sup>3</sup> /m						
Moment of resistance of masor	nry - exp.6.15	$M_{Rd} = f_{xd,app}$	M <sub>Rd</sub> = f <sub>xd,app</sub> × Z = <b>36.785</b> kNm/m							
		$M / M_{Rd} = 0$	.168							
		PASS	- Moment of r	esistance excee	eds applied d	esign moment				
Unreinforced masonry walls	subjected to sl	hear loading - S	ection 6.2							
Design shear force		∨ = 12.731	kN/m							
Maximum compressive stress		$\sigma_{max} = F / t$	+ M / Z = 0.37	<b>4</b> N/mm <sup>2</sup>						
Minimum compressive stress		$\sigma_{min} = F / t$	- M / Z = 0.204	N/mm <sup>2</sup>						
Thickness of wall in compression	on	tc = t = <b>660</b>	mm							
Design shear resistance - exp.	6.13	$V_{Rd} = f_{vd} \times f_{vd}$	tc = <b>70.15</b> kN/n	า						
		$V / V_{Rd} = 0$	181							

PASS - Design shear resistance exceeds applied design shear force

#### Check stem design at 920 mm

Depth of section

#### t = **660** mm

#### Masonry characteristics

Compressive strength constants - Table NA.4K = 0.5Characteristic compressive strength - cl.3.6.1.2(1) $f_k = K \times f_b^{0.7} \times f_m^{0.3} = 13.234 \text{ N/mm}^2$ Design compressive strength $f_d = f_k / \gamma_{Mc} = 5.754 \text{ N/mm}^2$ Design flexural strength $f_{xd} = f_{xk} / \gamma_{Mt} = 0.217 \text{ N/mm}^2$ Height of masonryhwt = hstem - 920 mm = 930 mmCompressive axial force combination 1 $F = (\gamma_{Gf} \times (\gamma_{Stem} \times h_{wt} \times t + P_{G1}) + \gamma_{Qf} \times P_{Q1}) = 183.3 \text{ kN/m}$ Eccentricity of axial loade = 27.5 mm

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London N1 0SP	Calcs by GO	Calcs date 10/08/2016	Checked by	Checked date	Approved by	Approved d		
Capacity reduction factor - exp.6	6.4	Φ = 1 - 2 ×	e / t = <b>0.917</b>					
Design vertical resistance - exp.	.6.2	$N_{Rd} = \Phi \times t$	× fd = <b>3481.3</b>	kN/m				
Design vertical compressive stre	ess	$\sigma_d = \min(F)$	/ t, 0.15 $\times$ N <sub>Rd</sub>	/ t) = <b>0.278</b> N/mr	m²			
Apparent design flexural strengt	h - exp.6.16	$f_{xd,app} = f_{xd}$ -	⊦ <sub>ರd</sub> = <b>0.495</b> N	l/mm <sup>2</sup>				
Characteristic shear strength - e	exp.3.5	f <sub>vk</sub> = min(f <sub>vk</sub>	$\infty$ + 0.4 $ imes$ $\sigma$ d, 0	0.065 × fb) = <b>0.26</b> <sup>2</sup>	I N/mm <sup>2</sup>			
Design shear strength		$f_{vd} = f_{vk} / \gamma_M$	v = <b>0.104</b> N/m	m <sup>2</sup>				
Unreinforced masonry walls s	ubiected to la	iteral loading - S	Section 6.3					
Design bending moment combin	nation 1	M = 2.039	kNm/m					
Elastic section modulus of wall		$Z = t^2 / 6 =$	72600000 mm	n³/m				
Moment of resistance of mason	ry - exp.6.15	M <sub>Rd</sub> = f <sub>xd,app</sub>	$M_{Rd} = f_{xd,app} \times Z = 35.951 \text{ kNm/m}$					
		M / M <sub>Rd</sub> = <b>0.057</b>						
		PASS	- Moment of	resistance exce	eds applied d	esign mom		
Unreinforced masonry walls s	ubjected to s	hear loading - S	ection 6.2					
Design shear force		∨ = 5.798 I	kN/m					
Maximum compressive stress		$\sigma_{max}$ = F / t	+ M / Z = 0.30	<b>)6</b> N/mm²				
Minimum compressive stress		$\sigma_{min}$ = F / t	- M / Z = <b>0.25</b>	<b>0</b> N/mm²				
Thickness of wall in compression	n	tc = t = <b>660</b>	mm					
Design shear resistance - exp.6	.13	V <sub>Rd</sub> = f <sub>vd</sub> × t <sub>c</sub> = <b>68.935</b> kN/m						
		V / V <sub>Rd</sub> = 0.084						
		PASS - Desi	gn shear res	istance exceeds	applied desig	gn shear fo		
Check base design at toe								
Depth of section		h = <b>600</b> mr	n					
Rectangular section in flexure	e - Section 6.1							
Design bending moment combin	nation 1	M = <b>18.1</b> k	Nm/m					
Depth to tension reinforcement		d = h - c <sub>bb</sub> - φ <sub>bb</sub> / 2 = <b>519</b> mm						
		$K = M / (d^2)$	× f <sub>ck</sub> ) = <b>0.002</b>					
		K' = <b>0.207</b>						
			K'>K-	No compressio	n reinforcem	ent is requ		
Lever arm		z = min(0.5)	o + 0.5 × (1 − 3	$3.53 \times \text{K})^{0.3}, 0.95)$	× d = 493 mm			
Depth of neutral axis		$x = 2.5 \times (c$	1 – z) = <b>65</b> mm	1				
Area of tension reinforcement re	equired	$A_{bb.req} = M$	$(t_{yd} \times z) = 84$	mm²/m				
I ension reinforcement provided		12 dia.bars	a (2) 100 c/c					
Area of tension reinforcement p	rovided	$A_{bb.prov} = \pi$	$\times \phi_{bb}^2 / (4 \times S_b)$	<sub>b</sub> ) = <b>1131</b> mm <sup>2</sup> /m				
Minimum area of reinforcement	- exp.9.1N	$A_{bb.min} = ma$	$ax(0.26 \times f_{ctm})$	$f_{yk}$ , 0.0013) × d =	<b>782</b> mm²/m			
Maximum area of reinforcement	: - cl.9.2.1.1(3)	$A_{bb.max} = 0.$	04 × h = <b>2400</b>	<b>0</b> mm²/m				

max(Abb.req, Abb.min) / Abb.prov = 0.691

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3Limiting crack width $W_{max} = 0.3 \text{ mm}$ Variable load factor - EN1990 - Table A1.1 $\psi_2 = 0.6$ Serviceability bending moment $M_{sls} = 13.3 \text{ kNm/m}$ Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{bb,prov} \times z) = 23.8 \text{ N/mm}^2$ Load durationLong termLoad duration factor $k_t = 0.4$ Effective area of concrete in tension $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2) = 178375 \text{ mm}^2/m$ 

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	60	10/08/2010						
Mean value of concrete tensile	strength	$f_{ct.eff} = f_{ctm} =$	<b>2.9</b> N/mm <sup>2</sup>					
Reinforcement ratio		$\rho_{p.eff}$ = A <sub>bb.pr</sub>	ov / A <sub>c.eff</sub> = <b>0.006</b>					
Modular ratio		$\alpha_{e} = E_{s} / E_{cr}$	m <b>= 6.091</b>					
Bond property coefficient	k1 = <b>0.8</b>							
Strain distribution coefficient		k <sub>2</sub> = 0.5						
		k <sub>3</sub> = <b>3.4</b>						
		k4 = <b>0.425</b>						
Maximum crack spacing - exp.7	'.11	$s_{r.max} = k_3 \times$	$s_{r.max}$ = $k_3 \times c_{bb}$ + $k_1 \times k_2 \times k_4 \times \phi_{bb}$ / $\rho_{p.eff}$ = 577 mm					
Maximum crack width - exp.7.8		$w_{k} = s_{r.max} \times max(\sigma_{s} - k_{t} \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_{e} \times \rho_{p.eff}), 0.6 \times \sigma_{s}) / E_{s}$						
		w <sub>k</sub> = <b>0.041</b>	mm					
		$w_k / w_{max} =$	0.137					
		PASS	- Maximum cra	ck width is less	s than limiting	g crack width		
Rectangular section in shear	- Section 6.2							
Design shear force		V = 92.2 kN	l/m					
		$C_{Rd,c} = 0.18$	/ γc <b>= 0.120</b>					
		k = min(1 +	√(200 mm / d),	2) <b>= 1.621</b>				
Longitudinal reinforcement ratio	)	ρι = min(Abb	.prov / d, 0.02) =	0.002				
		vmin = 0.035	$N^{1/2}/mm \times k^{3/2} >$	< fck <sup>0.5</sup> = <b>0.396</b> N	/mm²			
Design shear resistance - exp.6	6.2a & 6.2b	$V_{\text{Rd.c}}$ = max( $C_{\text{Rd.c}} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_I \times f_{\text{ck}})^{1/3}$ , $v_{\text{min}}) \times d$						
		V <sub>Rd.c</sub> = <b>205.3</b> kN/m						
		$V / V_{Rd.c} = 0$	.449					
		PAS	S - Design shea	ar resistance ex	ceeds desigi	n shear force		
Secondary transverse reinfor	cement to base	- Section 9.3						
Minimum area of reinforcement	– cl.9.3.1.1(2)	$A_{bx.req} = 0.2$	$\times$ Abb.prov = 226	mm²/m				
Maximum spacing of reinforcem	nent – cl.9.3.1.1(3	5) Sbx_max = <b>45</b>	<b>0</b> mm					
Transverse reinforcement provi	ded	8 dia.bars (	2) 200 c/c					
Area of transverse reinforcement	nt provided	$A_{bx.prov} = \pi >$	$\langle \phi_{bx}^2 / (4 \times s_{bx}) =$	= <b>251</b> mm²/m				
	PASS - Area of	reinforcement	provided is gro	eater than area	of reinforcem	nent required		







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## **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 2.4.08

Retaining wall details	
Stem type	Cantilever
Stem height	h <sub>stem</sub> = <b>1850</b> mm
Prop height	h <sub>prop</sub> = <b>1850</b> mm
Stem thickness	t <sub>stem</sub> = <b>350</b> mm
Angle to rear face of stem	$\alpha$ = 90 deg
Stem density	$\gamma_{\text{stem}} = 25 \text{ kN/m}^3$
Toe length	I <sub>toe</sub> = <b>920</b> mm
Heel length	I <sub>heel</sub> = <b>480</b> mm
Base thickness	t <sub>base</sub> = 600 mm
Base density	$\gamma_{\text{base}} = 25 \text{ kN/m}^3$
Height of retained soil	h <sub>ret</sub> = <b>1850</b> mm
Angle of soil surface	$\beta = 0 \deg$
Depth of cover	d <sub>cover</sub> = <b>0</b> mm
Height of water	h <sub>water</sub> = <b>1000</b> mm
Water density	γw <b>= 9.8</b> kN/m <sup>3</sup>
Retained soil properties	
Retained soil properties Soil type	Medium dense well graded sand
<b>Retained soil properties</b> Soil type Moist density	Medium dense well graded sand γmr = <b>21</b> kN/m <sup>3</sup>
Retained soil properties Soil type Moist density Saturated density	Medium dense well graded sand $\gamma_{mr} = 21 \text{ kN/m}^3$ $\gamma_{sr} = 23 \text{ kN/m}^3$
Retained soil properties Soil type Moist density Saturated density Characteristic effective shear resistance angle	Medium dense well graded sand $\gamma_{mr} = 21 \text{ kN/m}^3$ $\gamma_{sr} = 23 \text{ kN/m}^3$ $\phi'_{r.k} = 30 \text{ deg}$
Retained soil properties Soil type Moist density Saturated density Characteristic effective shear resistance angle Characteristic wall friction angle	Medium dense well graded sand $\gamma_{mr} = 21 \text{ kN/m}^3$ $\gamma_{sr} = 23 \text{ kN/m}^3$ $\phi'_{r.k} = 30 \text{ deg}$ $\delta_{r.k} = 0 \text{ deg}$
Retained soil properties Soil type Moist density Saturated density Characteristic effective shear resistance angle Characteristic wall friction angle Base soil properties	Medium dense well graded sand $\gamma_{mr} = 21 \text{ kN/m}^3$ $\gamma_{sr} = 23 \text{ kN/m}^3$ $\phi'_{r.k} = 30 \text{ deg}$ $\delta_{r.k} = 0 \text{ deg}$
Retained soil propertiesSoil typeMoist densitySaturated densityCharacteristic effective shear resistance angleCharacteristic wall friction angleBase soil propertiesSoil type	Medium dense well graded sand $\gamma_{mr} = 21 \text{ kN/m}^3$ $\gamma_{sr} = 23 \text{ kN/m}^3$ $\phi'_{r,k} = 30 \text{ deg}$ $\delta_{r,k} = 0 \text{ deg}$ Medium dense well graded sand
Retained soil propertiesSoil typeMoist densitySaturated densityCharacteristic effective shear resistance angleCharacteristic wall friction angleBase soil propertiesSoil typeMoist density	Medium dense well graded sand $\gamma_{mr} = 21 \text{ kN/m}^3$ $\gamma_{sr} = 23 \text{ kN/m}^3$ $\phi'_{r,k} = 30 \text{ deg}$ $\delta_{r,k} = 0 \text{ deg}$ Medium dense well graded sand $\gamma_{mb} = 18 \text{ kN/m}^3$
Retained soil propertiesSoil typeMoist densitySaturated densityCharacteristic effective shear resistance angleCharacteristic wall friction angleBase soil propertiesSoil typeMoist densityCharacteristic effective shear resistance angle	Medium dense well graded sand $\gamma_{mr} = 21 \text{ kN/m}^3$ $\gamma_{sr} = 23 \text{ kN/m}^3$ $\phi'_{r,k} = 30 \text{ deg}$ $\delta_{r,k} = 0 \text{ deg}$ Medium dense well graded sand $\gamma_{mb} = 18 \text{ kN/m}^3$ $\phi'_{b,k} = 30 \text{ deg}$
<ul> <li>Retained soil properties</li> <li>Soil type</li> <li>Moist density</li> <li>Saturated density</li> <li>Characteristic effective shear resistance angle</li> <li>Characteristic wall friction angle</li> <li>Base soil properties</li> <li>Soil type</li> <li>Moist density</li> <li>Characteristic effective shear resistance angle</li> <li>Characteristic effective shear resistance angle</li> </ul>	Medium dense well graded sand $\gamma_{mr} = 21 \text{ kN/m}^3$ $\gamma_{sr} = 23 \text{ kN/m}^3$ $\phi'_{r.k} = 30 \text{ deg}$ $\delta_{r.k} = 0 \text{ deg}$ Medium dense well graded sand $\gamma_{mb} = 18 \text{ kN/m}^3$ $\phi'_{b.k} = 30 \text{ deg}$ $\delta_{b.k} = 15 \text{ deg}$
Retained soil propertiesSoil typeMoist densitySaturated densityCharacteristic effective shear resistance angleCharacteristic wall friction angleBase soil propertiesSoil typeMoist densityCharacteristic effective shear resistance angleCharacteristic effective shear resistance angleCharacteristic effective shear resistance angleCharacteristic effective shear resistance angleCharacteristic base friction angle	Medium dense well graded sand $\gamma_{mr} = 21 \text{ kN/m}^3$ $\gamma_{sr} = 23 \text{ kN/m}^3$ $\phi'_{r,k} = 30 \text{ deg}$ $\delta_{r,k} = 0 \text{ deg}$ Medium dense well graded sand $\gamma_{mb} = 18 \text{ kN/m}^3$ $\phi'_{b,k} = 30 \text{ deg}$ $\delta_{b,k} = 15 \text{ deg}$ $\delta_{bb,k} = 30 \text{ deg}$
Retained soil propertiesSoil typeMoist densitySaturated densityCharacteristic effective shear resistance angleCharacteristic wall friction angleBase soil propertiesSoil typeMoist densityCharacteristic effective shear resistance angleCharacteristic effective shear resistance angleCharacteristic effective shear resistance angleCharacteristic effective shear resistance angleCharacteristic base friction anglePresumed bearing capacity	Medium dense well graded sand $\gamma_{mr} = 21 \text{ kN/m}^3$ $\gamma_{sr} = 23 \text{ kN/m}^3$ $\phi'_{r.k} = 30 \text{ deg}$ $\delta_{r.k} = 0 \text{ deg}$ Medium dense well graded sand $\gamma_{mb} = 18 \text{ kN/m}^3$ $\phi'_{b.k} = 30 \text{ deg}$ $\delta_{b.k} = 15 \text{ deg}$ $\delta_{bb.k} = 30 \text{ deg}$ $P_{bearing} = 250 \text{ kN/m}^2$
Retained soil propertiesSoil typeMoist densitySaturated densityCharacteristic effective shear resistance angleCharacteristic wall friction angleBase soil propertiesSoil typeMoist densityCharacteristic effective shear resistance angleCharacteristic effective shear resistance anglePresumed bearing capacityLoading details	Medium dense well graded sand $\gamma_{mr} = 21 \text{ kN/m}^3$ $\gamma_{sr} = 23 \text{ kN/m}^3$ $\phi'_{r.k} = 30 \text{ deg}$ $\delta_{r.k} = 0 \text{ deg}$ Medium dense well graded sand $\gamma_{mb} = 18 \text{ kN/m}^3$ $\phi'_{b.k} = 30 \text{ deg}$ $\delta_{b.k} = 15 \text{ deg}$ $\delta_{b.k} = 30 \text{ deg}$ $P_{bearing} = 250 \text{ kN/m}^2$

Variable surcharge load Vertical line load at 1165 mm Surcharge<sub>Q</sub> = **5** kN/m<sup>2</sup> P<sub>G1</sub> = **124** kN/m P<sub>Q1</sub> = **40** kN/m



#### Area of saturated soil

- Distance to vertical component
- Distance to horizontal component Area of water
- Distance to vertical component
- Distance to horizontal component Area of moist soil
- Distance to vertical component
- Distance to horizontal component

Asat =  $h_{sat} \times I_{heel} = 0.48 \text{ m}^2$ Xsat\_v = Ibase - (hsat × I\_{heel}^2 / 2) / Asat = 1510 mm Xsat\_h = (hsat + hbase) / 3 = 533 mm Awater =  $h_{sat} \times I_{heel} = 0.48 \text{ m}^2$ Xwater\_v = Ibase - (hsat × I\_{heel}^2 / 2) / Asat = 1510 mm Xwater\_h = (hsat + hbase) / 3 = 533 mm Amoist =  $h_{moist} \times I_{heel} = 0.408 \text{ m}^2$ Xmoist\_v = Ibase - (hmoist × I\_{heel}^2 / 2) / Amoist = 1510 mm Xmoist\_v = I\_{base} - (hmoist × I\_{heel}^2 / 2) / Amoist = 1510 mm Xmoist\_h = (hmoist × (t\_{base} + h\_{sat} + h\_{moist} / 3) / 2 + (h\_{sat} + t\_{base})^2/2) / (h\_{sat} + t\_{base} + h\_{moist} / 2) = 1027 mm

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N1 0SP	GO GO	03/05/2016	Checked by	Checked date	Approved by	Approved date
Using Coulomb theory						
Active pressure coefficient		$K_A = sin(\alpha)$	+ $\phi'_{r.k}$ ) <sup>2</sup> / (sin( $\alpha$	) <sup>2</sup> × sin( $\alpha$ - $\delta_{r.k}$ ) ×	[1 + √[sin(¢'r.ĸ -	ר δr.k <b>) × sin(</b> φ'r.k
		- β) / (sin(o	$\alpha$ - $\delta_{r.k}$ ) × sin( $\alpha$ ·	+ β))]] <sup>2</sup> ) = <b>0.333</b>		
Passive pressure coefficient		K <sub>P</sub> = sin(90	) - φ' <sub>b.k</sub> )² / (sin(§	90 + $\delta_{b.k}$ ) × [1 - $\sqrt{s}$	sin(φ' <sub>b.k</sub> + δ <sub>b.k</sub> ) >	< <b>sin(</b> \phi'_b.k) /
		(sin(90 + δ	b.k))]] <sup>2</sup> ) = <b>4.977</b>			
Bearing pressure check						
Vertical forces on wall						
Wall stem		F <sub>stem</sub> = A <sub>ste</sub>	m × γ <sub>stem</sub> = <b>16.2</b>	kN/m		
Wall base		Fbase = Abas	e × γ <sub>base</sub> = <b>26.3</b>	kN/m		
Surcharge load	$F_{sur_v}$ = (Surcharge <sub>G</sub> + Surcharge <sub>Q</sub> ) × I <sub>heel</sub> = <b>21.6</b> kN/m					
Line loads	F <sub>P_v</sub> = P <sub>G1</sub> + P <sub>Q1</sub> = <b>164</b> kN/m					
Saturated retained soil	$F_{sat_v} = A_{sat} \times (\gamma_{sr} - \gamma_w) = 6.3 \text{ kN/m}$					
Water	$F_{water_v} = A_{water} \times \gamma_w = 4.7 \text{ kN/m}$					
Moist retained soil	$F_{moist_v} = A_{moist} \times \gamma_{mr} = 8.6 \text{ kN/m}$					
Total	F <sub>total_v</sub> = F <sub>stem</sub> + F <sub>base</sub> + F <sub>sat_v</sub> + F <sub>moist_v</sub> + F <sub>water_v</sub> + F <sub>sur_v</sub> + F <sub>P_v</sub> = <b>247.6</b> kN/m					
Horizontal forces on wall						
Surcharge load		Fsur_h <b>= K</b> A	× <b>(Surcharge</b> G	+ Surcharge <sub>Q</sub> ) $\times$	h <sub>eff</sub> <b>= 36.8</b> kN/	m
Saturated retained soil		F <sub>sat_h</sub> = K <sub>A</sub>	$ imes$ ( $\gamma_{ m sr}$ - $\gamma_{ m w}$ ) $ imes$ (hs	<sub>at</sub> + h <sub>base</sub> ) <sup>2</sup> / 2 = 5	<b>.6</b> kN/m	
Water		$F_{water_h} = \gamma_w$	$u \times (h_{water} + d_{cov})$	<sub>er</sub> + h <sub>base</sub> ) <sup>2</sup> / 2 = 1	<b>2.6</b> kN/m	
Moist retained soil		Fmoist_h = K	$A \times \gamma_{mr} \times ((h_{eff} -$	hsat - h <sub>base</sub> ) <sup>2</sup> / 2 +	(heff - hsat - hbas	se) × (hsat +
		h <sub>base</sub> )) = <b>12</b>	kN/m			
Total		$F_{total_h} = F_{sa}$	at_h + F <sub>moist_h</sub> +	F <sub>water_h</sub> + F <sub>sur_h</sub> = 6	67 kN/m	
Moments on wall						
Wall stem		Mstem = Fste	m × Xstem = 17.7	<b>/</b> kNm/m		
Wall base		M <sub>base</sub> = F <sub>ba</sub>	se × x <sub>base</sub> = 23 k	«Nm/m		
Surcharge load		$M_{sur} = F_{sur}$	$v  imes \mathbf{x}_{sur_v} - \mathbf{F}_{sur_v}$	n × xsur_h = -12.4 k	Nm/m	
Line loads		M <sub>P</sub> = (P <sub>G1</sub> -	+ P <sub>Q1</sub> ) × p <sub>1</sub> = <b>1</b>	91.1 kNm/m		
Saturated retained soil		Msat = Fsat_	$v  imes \mathbf{X}$ sat_v - $\mathbf{F}$ sat_h	× <b>x</b> sat_h <b>= 6.6</b> kNr	n/m	
Water		$M_{water} = F_{water}$	ater_v × Xwater_v -	F <sub>water_h</sub> × <b>x</b> <sub>water_h</sub> =	<b>0.4</b> kNm/m	
Moist retained soil		$M_{moist} = F_{mo}$	$bist_v  imes \mathbf{X}_{moist_v} -  $	F <sub>moist_h</sub> × <b>x</b> <sub>moist_h</sub> =	<b>0.6</b> kNm/m	
Total		M <sub>total</sub> = M <sub>ste</sub>	m + M <sub>base</sub> + M <sub>sa</sub>	at + M <sub>moist</sub> + M <sub>water</sub>	+ $M_{sur}$ + $M_P$ = 2	<b>226.9</b> kNm/m
Check bearing pressure						
Propping force		Fprop_base =	Ftotal_h = 67 kN	/m		

Distance to reaction Eccentricity of reaction Loaded length of base Bearing pressure at toe Bearing pressure at heel Factor of safety  $\overline{\mathbf{x}} = (\mathbf{M}_{\text{total}} + \mathbf{M}_{\text{prop}}) / F_{\text{total}\_v} = \mathbf{916} \text{ mm}$   $\mathbf{e} = \overline{\mathbf{x}} - \mathbf{I}_{\text{base}} / 2 = \mathbf{41} \text{ mm}$   $\mathbf{I}_{\text{load}} = 2 \times (\mathbf{I}_{\text{base}} - \overline{\mathbf{x}}) = \mathbf{1668} \text{ mm}$   $\mathbf{q}_{\text{toe}} = \mathbf{0} \text{ kN/m}^2$   $\mathbf{q}_{\text{heel}} = F_{\text{total}\_v} / \mathbf{I}_{\text{load}} = \mathbf{148.5} \text{ kN/m}^2$   $\mathbf{FoS}_{\text{bp}} = \mathbf{P}_{\text{bearing}} / \max(\mathbf{q}_{\text{toe}}, \mathbf{q}_{\text{heel}}) = \mathbf{1.684}$ 

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

Tedds	Project	Project Ornan Court				Job no. 16.280	
Martin Redston Associates 4 Edward Square London N1 0SP	Calcs for Flank Wall Underpin Stem				Start page no./Revision 4		
	Calcs by GO	Calcs date 03/05/2016	Checked by	Checked date	Approved by	Approved date	

### **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 2.4.08

Concrete details - Table 3.1 - Strength and deto	craciar characteristics for concrete
Concrete strength class	$f_{\rm c} = 20  \rm N/mm^2$
Characteristic compressive cube strength	$f_{ck} = 30 \text{ N/mm}^2$
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8 N/mm^2 = 38 N/mm^2$
Mean value of axial tensile strength	$f_{ctm} = 0.3 \text{ N/mm}^2 \times (f_{ctv} / 1 \text{ N/mm}^2)^{2/3} = 2.9 \text{ N/mm}^2$
5% fractile of axial tensile strength	$f_{\text{ctk}, 0.05} = 0.7 \times f_{\text{ctm}} = 2.0 \text{ N/mm}^2$
Secant modulus of elasticity of concrete	$F_{cm} = 22 \text{ kN/mm}^2 \times (f_{cm} / 10 \text{ N/mm}^2)^{0.3} = 32837 \text{ N/mm}^2$
Partial factor for concrete - Table 2 1N	$v_c = 1.50$
Compressive strength coefficient - $cl 3 1 6(1)$	$   \alpha_{cc} = 0.85 $
Design compressive concrete strength - exp 3 15	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_c = 17.0 \text{ N/mm}^2$
Maximum aggregate size	$h_{ang} = 20 \text{ mm}$
Poinforcomont dotaile	
Characteristic vield strength of reinforcement	$f_{\rm tr} = 500  \rm N/mm^2$
Modulus of elasticity of reinforcement	$F_s = 200000 \text{ N/mm}^2$
Partial factor for reinforcing steel - Table 2.1N	vs = 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	c <sub>sf</sub> = <b>40</b> mm
Rear face of stem	c <sub>sr</sub> = <b>50</b> mm
Top face of base	c <sub>bt</sub> = <b>50</b> mm
Bottom face of base	c <sub>bb</sub> = <b>75</b> mm
Check stem design at base of stem	
Depth of section	h = <b>350</b> mm
Rectangular section in flexure - Section 6.1	
Design bending moment combination 1	M = <b>46.7</b> kNm/m
Depth to tension reinforcement	d = h - c <sub>sr</sub> - φ <sub>sr</sub> / 2 = <b>292</b> mm
	$K = M / (d^2 \times f_{ck}) = 0.018$
	K' = <b>0.207</b>
	K' > K - No compression reinforcement is required
Lever arm	z = min(0.5 + 0.5 × (1 – 3.53 × K) <sup>0.5</sup> , 0.95) × d = <b>277</b> mm

Depth of neutral axis

Area of tension reinforcement required Tension reinforcement provided Area of tension reinforcement provided Minimum area of reinforcement - exp.9.1N Maximum area of reinforcement - cl.9.2.1.1(3)  $\begin{aligned} x &= 2.5 \times (d - z) = 37 \text{ mm} \\ A_{sr.req} &= M / (f_{yd} \times z) = 387 \text{ mm}^2/\text{m} \\ 16 \text{ dia.bars @ 200 c/c} \\ A_{sr.prov} &= \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = 1005 \text{ mm}^2/\text{m} \\ A_{sr.min} &= \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 440 \text{ mm}^2/\text{m} \\ A_{sr.max} &= 0.04 \times h = 14000 \text{ mm}^2/\text{m} \\ max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} = 0.437 \end{aligned}$ 

PASS - Area of reinforcement provided is greater than area of reinforcement required

**Crack control - Section 7.3** 

Limiting crack width

w<sub>max</sub> = 0.3 mm

P	Project Job no.							
Tedds	Ornan Court					280		
Martin Redston Associates C	Calcs for			Start page no./Re	evision			
4 Edward Square			nderpin Stem			5		
N1 0SP	alcs by GO	Calcs date 03/05/2016	Checked by	Checked date	Approved by	Approved date		
Variable load factor - EN1990 – Ta	able A1.1	ψ2 <b>= 0.6</b>						
Serviceability bending moment		Msis = <b>33.1</b>	kNm/m					
Tensile stress in reinforcement		$\sigma_{s}$ = M <sub>sls</sub> / (	$A_{sr.prov} \times z) = 11$	<b>8.7</b> N/mm²				
Load duration		Long term						
Load duration factor		kt = <b>0.4</b>						
Effective area of concrete in tension	on	A <sub>c.eff</sub> = min(	2.5 × (h - d), (h	- x) / 3, h / 2) = 1	104500 mm²/m	1		
Mean value of concrete tensile stre	ength	f <sub>ct.eff</sub> = f <sub>ctm</sub> =	<b>2.9</b> N/mm <sup>2</sup>					
Reinforcement ratio	-	ρ <sub>p.eff</sub> = A <sub>sr.pro</sub>	ov / A <sub>c.eff</sub> = <b>0.010</b>	)				
Modular ratio		$\alpha_{e}$ = Es / Ec	m = <b>6.091</b>					
Bond property coefficient		k1 <b>= 0.8</b>						
Strain distribution coefficient		k2 <b>= 0.5</b>						
		k <sub>3</sub> = <b>3.4</b>						
		k <sub>4</sub> = <b>0.425</b>						
Maximum crack spacing - exp.7.1	1	$s_{r.max} = k_3 \times$	$\mathbf{C}_{sr}$ + $\mathbf{k}_1 \times \mathbf{k}_2 \times \mathbf{k}_2$	$4 \times \phi_{sr} / \rho_{p.eff} = 45$	<b>3</b> mm			
Maximum crack width - exp.7.8		$w_k = s_{r.max} \times max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$						
		w <sub>k</sub> = <b>0.161</b> mm						
		w <sub>k</sub> / w <sub>max</sub> = <b>0.538</b>						
		PASS	PASS - Maximum crack width is less than limiting crack width					
Rectangular section in shear - S	Section 6.2							
Design shear force		V = <b>59</b> kN/r	n					
C		$C_{Rd,c} = 0.18$	3 / γc = <b>0.120</b>					
		k = min(1 +	√(200 mm / d),	(2) = <b>1.828</b>				
l ongitudinal reinforcement ratio		$\rho_{\rm l} = \min(A_{\rm sf, prov} / d. 0.02) = 0.001$						
		$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times \text{k}^{3/2} \times \text{f}_{ck}^{0.5} = 0.474 \text{ N}/\text{mm}^2$						
Design shear resistance - exp 6.2	a & 6 2h	$V_{\text{Pd}o} = \max(C_{\text{Pd}o} \times k \times (100 \text{ N}^2/\text{mm}^4 \times o) \times f_{\text{ek}})^{1/3} \text{ V}_{\text{min}}) \times d$						
Design shear resistance - exp.0.26		$v_{Ra,c} = max(O_{Ra,c} \times K \times (100 \text{ N} / 11111 \times \text{pl} \times \text{lck})^{-1}, \text{Vmin}) \times U$						
	$V_{Ra,c} = 130.3 \text{ KW/III}$							
		V / VRO.C - U.420 PASS - Design shear resistance evenede design shear force						
Horizontal reinforcement paralle	el to face of s	tem - Section 9	).6					
Minimum area of reinforcement –	cl.9.6.3(1)	$A_{sy reg} = ma$	x(0.25 × Asr nrov	$0.001 \times t_{stem}) = 3$	50 mm²/m			
Maximum spacing of reinforcement	t = cl.9.6.3(2)	Ssy may = 40	0 mm					
Transverse reinforcement provide	d	10 dia.bars	@ 200 c/c					
Area of transverse reinforcement	orovided	$A_{sx,prov} = \pi$	$\langle \phi_{sx}^2 / (4 \times S_{sx}) \rangle$	= <b>393</b> mm²/m				
PA	ASS - Area of	reinforcement	provided is a	reater than area	of reinforcem	ent reauirea		
Chack base design of too			,					
Dopth of costion		h - 600 mm						
		11 <b>- 000</b> 1111	1					

Rectangular section in flexure - Section 6.1	I
Design bending moment combination 1	M = <b>65.8</b> kNm/m
Depth to tension reinforcement	d = h - c <sub>bb</sub> - φ <sub>bb</sub> / 2 = <b>517</b> mm
	$K = M / (d^2 \times f_{ck}) = 0.008$
	K' = 0.207
	K' > K - No compression reinforcement is required
Lever arm	z = min(0.5 + 0.5 × (1 – 3.53 × K) <sup>0.5</sup> , 0.95) × d = <b>491</b> mm
Depth of neutral axis	x = 2.5 × (d − z) = <b>65</b> mm
Area of tension reinforcement required	$A_{bb.req} = M / (f_{yd} \times z) = 308 \text{ mm}^2/\text{m}$
Tension reinforcement provided	16 dia.bars @ 200 c/c

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4 Edward Square		Flank Wall L	Jnderpin Stem		6			
London N1 0SP	Calcs by GO	Calcs date 03/05/2016	Checked by	Checked date	Approved by	Approved date		
Area of tension reinforcement pr	rovided	$\Delta_{\rm hh}  {\rm prov} = \pi$	$\times d_{\rm hb}^2 / (4 \times s_{\rm hb})^2$	$a = 1005 \text{ mm}^2/\text{m}$	•			
Minimum area of reinforcement p		Abb.prov = $\pi$	∧ ψ00 / ( <del>+</del> ∧ 300 αν(0.26 ∨ f/	f 0 0013) × d =	<b>779</b> mm <sup>2</sup> /m			
Maximum area of reinforcement	$-e_{x}p_{.9.11}$		$ax(0.20 \times 1000)$	$1_{\rm yk}, 0.0013) \times 0 =$				
	- 01.9.2.1.1(3)	Abb.max = 0.0	04 × 11 – <b>2400</b>	- 0 775				
	PASS - Area o	f reinforcemen	t provided is g	greater than are	a of reinforce	ment required		
Crack control - Section 7.3				-		·		
Limiting crack width		w <sub>max</sub> = <b>0.3</b>	mm					
Variable load factor - EN1990 –	Table A1.1	ψ2 <b>= 0.6</b>						
Serviceability bending moment		M <sub>sls</sub> = <b>48.1</b>	kNm/m					
Tensile stress in reinforcement		$\sigma_{s}$ = M <sub>sls</sub> / (	$A_{bb.prov} \times z) = 9$	<b>7.3</b> N/mm <sup>2</sup>				
Load duration		Long term						
Load duration factor		k <sub>t</sub> = <b>0.4</b>						
Effective area of concrete in tension		A <sub>c.eff</sub> = min(2.5 × (h - d), (h – x) / 3, h / 2) = <b>178458</b> mm²/m						
Mean value of concrete tensile s	trength	$f_{ct.eff} = f_{ctm} = 2.9 \text{ N/mm}^2$						
Reinforcement ratio		$\rho_{p.eff} = A_{bb.prov} / A_{c.eff} = 0.006$						
Modular ratio		αe = Es / Ecm = <b>6.091</b>						
Bond property coefficient		k1 = <b>0.8</b>						
Strain distribution coefficient		k <sub>2</sub> = <b>0.5</b>						
		k <sub>3</sub> = <b>3.4</b>						
		k4 = <b>0.425</b>						
Maximum crack spacing - exp.7.	11	sr.max = k <sub>3</sub> ×	$\mathbf{c}$ Cbb + $\mathbf{k}_1 \times \mathbf{k}_2 \times \mathbf{k}_2$	$\times$ k4 $\times$ $\phi$ bb / $\rho$ p.eff =	<b>738</b> mm			
Maximum crack width - exp.7.8		w <sub>k</sub> = s <sub>r.max</sub> >	$\times$ max( $\sigma_{s}$ – k <sub>t</sub> $\times$	(f <sub>ct.eff</sub> / $\rho_{p.eff}$ ) × (1	+ $\alpha_e \times \rho_{p.eff}$ ), 0	$.6  imes \sigma_s$ ) / Es		
		w <sub>k</sub> = <b>0.215</b>	mm					
		$w_k / w_{max} =$	0.718					
		PASS	- Maximum c	rack width is le	ss than limitir	ng crack width		
Rectangular section in shear -	Section 6.2							
Design shear force	Design shear force		V = <b>147.7</b> kN/m					
		C <sub>Rd,c</sub> = 0.18 / γ <sub>C</sub> = <b>0.120</b>						
		k = min(1 +	- √(200 mm / d	l), 2) <b>= 1.622</b>				
Longitudinal reinforcement ratio		ρι = min(Ab	b.prov / d, 0.02)	= 0.002				
		v <sub>min</sub> = 0.035	5 N <sup>1/2</sup> /mm × k <sup>3/</sup>	<sup>/2</sup> × f <sub>ck</sub> <sup>0.5</sup> = <b>0.396</b>	N/mm <sup>2</sup>			
Design shear resistance - exp.6.	.2a & 6.2b	V <sub>Rd.c</sub> = max	$(C_{Rd.c} \times k \times (10))$	00 N <sup>2</sup> /mm <sup>4</sup> $\times$ $\rho$ l $\times$	$f_{ck})^{1/3}$ , Vmin) × d			
		V <sub>Rd.c</sub> = <b>204</b>	<b>.7</b> kN/m					
		$V / V_{Rd.c} = 0$	0.721					
		PAS	SS - Design sh	near resistance o	exceeds desig	gn shear force		

**Rectangular section in shear - Section 6.2** 

```
Design shear forceV = 38 \text{ kN/m}C_{Rd,c} = 0.18 / \gamma_C = 0.120k = \min(1 + \sqrt{200 \text{ mm } / d}), 2) = 1.622Longitudinal reinforcement ratio\rho_I = \min(A_{bt,prov} / d, 0.02) = 0.001v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times \text{k}^{3/2} \times f_{ck}^{0.5} = 0.396 \text{ N/mm}^2Design shear resistance - exp.6.2a & 6.2bV_{Rd,c} = \max(C_{Rd,c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_I \times f_{ck})^{1/3}, v_{min}) \times dV_{Rd,c} = 204.7 \text{ kN/m}V / V_{Rd,c} = 0.185PASS - Design shear resistance exceeds design shear forceSecondary transverse reinforcement to base - Section 9.3
```





<b>2</b> 3	Project				Job no.			
Tedds		Ornan Court				16.280		
Martin Redston Associates	Calcs for	Calcs for			Start page no./Revision			
4 Edward Square		External Retaining Wall			1			
London	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date		
N1 0SP	GO	03/05/2016						
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RETAINING WALL ANALYSIS	<u>}</u>							

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

Tedds calculation version 2.4.08

Cantilever
h <sub>stem</sub> = <b>2000</b> mm
h <sub>prop</sub> = <b>2000</b> mm
t <sub>stem</sub> = <b>250</b> mm
$\alpha$ = 90 deg
$\gamma_{\text{stem}} = 25 \text{ kN/m}^3$
I <sub>toe</sub> = <b>1250</b> mm
t <sub>base</sub> = <b>350</b> mm
$\gamma_{\text{base}} = 25 \text{ kN/m}^3$
h <sub>ret</sub> = <b>1800</b> mm
$\beta = 0 \deg$
d <sub>cover</sub> = 0 mm
h <sub>water</sub> = <b>1000</b> mm
γw = <b>9.8</b> kN/m <sup>3</sup>
Medium dense well graded sand
γmr <b>= 21</b> kN/m <sup>3</sup>
$\gamma_{sr} = 23 \text{ kN/m}^3$
φ'r.k = <b>30</b> deg
δ <sub>r.k</sub> = <b>0</b> deg
Medium dense well graded sand
γ <sub>mb</sub> = <b>18</b> kN/m <sup>3</sup>
φ' <sub>b.k</sub> = <b>30</b> deg
δ <sub>b.k</sub> = <b>15</b> deg
δ <sub>bb.k</sub> = <b>30</b> deg
$P_{\text{bearing}} = 250 \text{ kN/m}^2$





## Using Coulomb theory

Active pressure coefficient

Passive pressure coefficient

**Bearing pressure check** 

#### Vertical forces on wall

Wall stem

Wall base

$$\begin{split} \mathsf{K}_{\mathsf{A}} &= \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k})} \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]]^2) = \mathbf{0.333} \\ \mathsf{K}_{\mathsf{P}} &= \sin(90 - \phi'_{\mathsf{b.k}})^2 / (\sin(90 + \delta_{\mathsf{b.k}}) \times [1 - \sqrt{[\sin(\phi'_{\mathsf{b.k}} + \delta_{\mathsf{b.k}})} \times \sin(\phi'_{\mathsf{b.k}}) / (\sin(90 + \delta_{\mathsf{b.k}}))]]^2) = \mathbf{4.977} \end{split}$$

 $F_{stem} = A_{stem} \times \gamma_{stem} = 12.5 \text{ kN/m}$  $F_{base} = A_{base} \times \gamma_{base} = 13.1 \text{ kN/m}$ 

_ 😂	Project Job no.					6 280	
Tedds						Devision	
Martin Redston Associates	Calcs for	External Re	etaining Wall		Start page no./I	Start page no./Revision	
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N1 0SP	Calcs by	03/05/2016	Спескеа by	Checked date	Approved by	Approved date	
		00/00/2010					
Total	F <sub>total_v</sub> = F <sub>stem</sub> + F <sub>base</sub> + F <sub>water_v</sub> = <b>25.6</b> kN/m						
Horizontal forces on wall							
Surcharge load		$F_{sur_h} = K_A$	× Surchargeo	× h <sub>eff</sub> <b>= 3.6</b> kN/m			
Saturated retained soil		Fsat_h = KA	× ( $\gamma$ sr - $\gamma$ w) × (hs	<sub>sat</sub> + h <sub>base</sub> ) <sup>2</sup> / 2 = 4	kN/m		
Water		$F_{water_h} = \gamma_w$	$h \times (h_{water} + d_{cov})$	<sub>er</sub> + h <sub>base</sub> ) <sup>2</sup> / 2 <b>= 8</b>	8 <b>.9</b> kN/m		
Moist retained soil		F <sub>moist_h</sub> = K	$A \times \gamma_{mr} \times ((h_{eff} -$	$h_{sat} - h_{base})^2 / 2 +$	(h <sub>eff</sub> - h <sub>sat</sub> - h <sub>ba</sub>	$_{ m se}$ ) × (h <sub>sat</sub> +	
	h <sub>base</sub> )) = <b>9.8</b> kN/m						
Total	F <sub>total_h</sub> = F <sub>sat_h</sub> + F <sub>moist_h</sub> + F <sub>water_h</sub> + F <sub>sur_h</sub> = <b>26.3</b> kN/m						
Moments on wall							
Wall stem		M <sub>stem</sub> = F <sub>ste</sub>	m × Xstem = 17.2	2 kNm/m			
Wall base		M <sub>base</sub> = F <sub>bas</sub>	se × Xbase = 9.8	kNm/m			
Surcharge load		Msur = -Fsur	$h \times \mathbf{X}_{sur} = -3.9$	<b>9</b> kNm/m			
Saturated retained soil		M <sub>sat</sub> = -F <sub>sat</sub> _	_h × <b>x</b> sat_h = -1.8	<b>3</b> kNm/m			
Water		M <sub>water</sub> = -F <sub>w</sub>	/ater_h × Xwater_h =	= <b>-4</b> kNm/m			
Moist retained soil		M <sub>moist</sub> = -F <sub>m</sub>	noist_h × <b>X</b> moist_h =	= <b>-8.7</b> kNm/m			
Total		M <sub>total</sub> = M <sub>ste</sub>	m + Mbase + Msa	at + Mmoist + Mwater	+ M <sub>sur</sub> = <b>8.6</b> kl	Nm/m	
Check bearing pressure							
Propping force		Fprop_base =	Ftotal_h = <b>26.3</b> k	N/m			
Distance to reaction	$\overline{\mathbf{x}}$ = (M <sub>total</sub> + M <sub>prop</sub> ) / F <sub>total_v</sub> = <b>337</b> mm						
Eccentricity of reaction	e = x - I <sub>base</sub> / 2 = <b>-413</b> mm						
Loaded length of base	$I_{load} = 2 \times \bar{x} = 673 \text{ mm}$						
Bearing pressure at toe		$q_{toe} = F_{total}$	v / Iload = <b>38</b> kN	/m²			
Bearing pressure at heel		q <sub>heel</sub> = 0 kN	l/m²				
Factor of safety		$FoS_{bp} = P_{bp}$	<sub>earing</sub> / max(q <sub>toe</sub>	, q <sub>heel</sub> ) <b>= 6.571</b>			
	PASS -	Allowable bearing	ig pressure ex	xceeds maximu	m applied bea	aring pressure	

## **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 2.4.08

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

oncrete strength class	C30/37
haracteristic compressive cylinder strength	f <sub>ck</sub> = <b>30</b> N/mm <sup>2</sup>
haracteristic compressive cube strength	f <sub>ck,cube</sub> = <b>37</b> N/mm <sup>2</sup>
ean value of compressive cylinder strength	f <sub>cm</sub> = f <sub>ck</sub> + 8 N/mm <sup>2</sup> = <b>38</b> N/mm <sup>2</sup>
ean value of axial tensile strength	f <sub>ctm</sub> = 0.3 N/mm <sup>2</sup> × (f <sub>ck</sub> / 1 N/mm <sup>2</sup> ) <sup>2/3</sup> = <b>2.9</b> N/mm <sup>2</sup>
% fractile of axial tensile strength	$f_{ctk,0.05} = 0.7 \times f_{ctm} = 2.0 \text{ N/mm}^2$
ecant modulus of elasticity of concrete	$E_{cm}$ = 22 kN/mm <sup>2</sup> × (f <sub>cm</sub> / 10 N/mm <sup>2</sup> ) <sup>0.3</sup> = <b>32837</b> N/mm <sup>2</sup>
artial factor for concrete - Table 2.1N	γc <b>= 1.50</b>
ompressive strength coefficient - cl.3.1.6(1)	αcc <b>= 0.85</b>
esign compressive concrete strength - exp.3.15	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_c = 17.0 \text{ N/mm}^2$
aximum aggregate size	h <sub>agg</sub> <b>= 20</b> mm
einforcement details	
haracteristic yield strength of reinforcement	f <sub>yk</sub> = <b>500</b> N/mm <sup>2</sup>
odulus of elasticity of reinforcement	Es <b>= 200000</b> N/mm <sup>2</sup>
artial factor for reinforcing steel - Table 2.1N	$v_{s} = 1.15$
artial factor for reinforcing steel - Table 2.1N	$v_{s} = 1.15$

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	GO	03/05/2016						
Design yield strength of reinford	cement	$f_{yd}$ = $f_{yk}$ / $\gamma_S$	<b>= 435</b> N/mm <sup>2</sup>					
Cover to reinforcement								
Front face of stem		c <sub>sf</sub> = <b>40</b> mn	า					
Rear face of stem		<sub>Csr</sub> = <b>50</b> mn	n					
Top face of base		c <sub>bt</sub> = <b>50</b> mn	n					
Bottom face of base		c <sub>bb</sub> = <b>75</b> mr	n					
Check stem design at base o	f stem							
Depth of section		h = <b>250</b> mr	n					
Rectangular section in flexur	e - Section 6.1							
Design bending moment combi	nation 1	M = <b>14.9</b> kNm/m						
Depth to tension reinforcement		d = h - c <sub>sr</sub> - $\phi_{sr}$ / 2 = <b>192</b> mm						
		$K = M / (d^2)$	× f <sub>ck</sub> ) = 0.013					
		K' <b>= 0.207</b>						
			K' > K -	No compressio	n reinforcemer	nt is required		
Lever arm		z = min(0.5	5 + 0.5 × (1 – 3	0.53 × K) <sup>0.5</sup> , 0.95)	× d = <b>182</b> mm			
Depth of neutral axis		$x = 2.5 \times (c$	l – z) = <b>24</b> mm	I				
Area of tension reinforcement r	equired	$A_{sr.req} = M /$	$A_{sr.req} = M / (f_{yd} \times z) = 187 \text{ mm}^2/\text{m}$					
Tension reinforcement provided	k	16 dia.bars	16 dia.bars @ 200 c/c					
Area of tension reinforcement p	provided	$A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = 1005 \text{ mm}^2/\text{m}$						
Minimum area of reinforcement	:-exp.9.1N	$A_{sr.min} = max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 289 \text{ mm}^2/\text{m}$						
Maximum area of reinforcemen	t - cl.9.2.1.1(3)	A <sub>sr.max</sub> = 0.04 × h = <b>10000</b> mm <sup>2</sup> /m						
		max(A <sub>sr.req</sub> ,	Asr.min) / Asr.prov	v = <b>0.288</b>				
	PASS - Area o	f reinforcement	t provided is g	greater than are	a of reinforcem	ent required		
Crack control - Section 7.3								
Limiting crack width		w <sub>max</sub> = 0.3	mm					
Variable load factor - EN1990 – Table A1.1		ψ2 <b>= 0.6</b>						
Serviceability bending moment		M <sub>sis</sub> = <b>9.6</b> kNm/m						
Tensile stress in reinforcement		$\sigma_{s}$ = M <sub>sls</sub> / (,	$A_{sr.prov} \times z) = 5$	<b>2.5</b> N/mm <sup>2</sup>				
Load duration		Long term						
Load duration factor		kt = <b>0.4</b>						

 $A_{c.eff} = min(2.5 \times (h - d), (h - x) / 3, h / 2) = 75333 mm^2/m$  $f_{ct.eff}$  =  $f_{ctm}$  = 2.9 N/mm<sup>2</sup>

Mean value of concrete tensile strength  $\rho_{p.eff}$  = A<sub>sr.prov</sub> / A<sub>c.eff</sub> = 0.013

 $\alpha_{e}$  = E<sub>s</sub> / E<sub>cm</sub> = 6.091

Bond property coefficient k<sub>1</sub> = **0.8** 

Effective area of concrete in tension

Reinforcement ratio

Modular ratio

Strain distribution coefficient	k <sub>2</sub> = <b>0.5</b>
	k <sub>3</sub> = <b>3.4</b>
	k <sub>4</sub> = <b>0.425</b>
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} = 374 \text{ mm}$
Maximum crack width - exp.7.8	$w_{k} = s_{r.max} \times max(\sigma_{s} - k_{t} \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_{e} \times \rho_{p.eff}), 0.6 \times \sigma_{s}) / E_{s}$
	w <sub>k</sub> = <b>0.059</b> mm
	w <sub>k</sub> / w <sub>max</sub> = <b>0.196</b>
	PASS - Maximum crack width is less than limiting crack width
Rectangular section in shear - Section 6.2	
Design shear force	V = <b>24.7</b> kN/m
	C <sub>Rd,c</sub> = 0.18 / γ <sub>C</sub> = <b>0.120</b>

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	00	03/03/2010						
		k = min(1 +	· √(200 mm / d),	2) = <b>2.000</b>				
Longitudinal reinforcement ratio	)	ρι = min(Ast	.prov / d, 0.02) = (	0.002				
		v <sub>min</sub> = 0.035	$5 \text{ N}^{1/2}/\text{mm} \times \text{k}^{3/2}$	× f <sub>ck</sub> <sup>0.5</sup> = <b>0.542</b> N	/mm²			
Design shear resistance - exp.6	6.2a & 6.2b	V <sub>Rd.c</sub> = max	$(C_{Rd.c} \times k \times (100))$	) N <sup>2</sup> /mm <sup>4</sup> × $\rho_l$ × f <sub>d</sub>	$(k)^{1/3}, V_{min}) \times d$			
		V <sub>Rd.c</sub> = <b>104</b>	. <b>1</b> kN/m					
		$V / V_{Rd.c} = 0$	).237					
		PAS	S - Design she	ar resistance ex	ceeds desig	n shear force		
Horizontal reinforcement para	allel to face of s	tem - Section §	9.6					
Minimum area of reinforcement	– cl.9.6.3(1)	A <sub>sx.req</sub> = ma	$x(0.25 \times A_{sr.prov})$	$0.001 \times t_{stem}) = 2$	2 <b>51</b> mm²/m			
Maximum spacing of reinforcem	nent – cl.9.6.3(2)	S <sub>sx_max</sub> = 40	Ssx_max = <b>400</b> mm					
Transverse reinforcement provi	ded	10 dia.bars	@ 200 c/c					
Area of transverse reinforcement	nt provided	$A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx}) = 393 \text{ mm}^2/\text{m}$						
	PASS - Area of	reinforcement	provided is gr	eater than area	of reinforcer	nent required		
Check base design at toe								
Depth of section		h = <b>350</b> mn	h = <b>350</b> mm					
Rectangular section in flexure	e - Section 6.1							
Design bending moment combi	nation 1	M = <b>22.9</b> kl	Nm/m					
Depth to tension reinforcement		$d = h - c_{bb} - c_$	φ <sub>bb</sub> / 2 <b>= 269</b> m	m				
		$K = M / (d^2 \times f_{ck}) = 0.011$						
		K' = <b>0.207</b>						
			K' > K - N	lo compression	reinforceme	nt is required		
Lever arm		z = min(0.5 + 0.5 × (1 – 3.53 × K) <sup>0.5</sup> , 0.95) × d = <b>256</b> mm						
Depth of neutral axis		$x = 2.5 \times (d)$	x = 2.5 × (d − z) = <b>34</b> mm					
Area of tension reinforcement re	equired	$A_{bb.req} = M$	$A_{bb.req} = M / (f_{yd} \times z) = 206 \text{ mm}^2/\text{m}$					
Tension reinforcement provided	l	12 dia.bars @ 200 c/c						
Area of tension reinforcement p	rovided	$A_{bb.prov} = \pi$	$\times \phi_{bb}^2$ / (4 $\times$ Sbb)	<b>= 565</b> mm²/m				
Minimum area of reinforcement	- exp.9.1N	A <sub>bb.min</sub> = ma	$A_{bb.min}$ = max(0.26 × f <sub>ctm</sub> / f <sub>yk</sub> , 0.0013) × d = 405 mm <sup>2</sup> /m					
Maximum area of reinforcemen	t - cl.9.2.1.1(3)	$A_{bb.max} = 0.0$	A <sub>bb.max</sub> = 0.04 × h = <b>14000</b> mm <sup>2</sup> /m					
				max(A <sub>bb.req</sub> , A <sub>bb.min</sub> ) / A <sub>bb.prov</sub> = <b>0.716</b>				
	PASS - Area of	reinforcement	provided is gr	eater than area	of reinforcer	nent required		
Crack control - Section 7.3								
Limiting crack width		w <sub>max</sub> = <b>0.3</b> I	w <sub>max</sub> = <b>0.3</b> mm					
Variable load factor - EN1990 – Table A1.1		ψ2 <b>= 0.6</b>	$\psi_2 = 0.6$					

Serviceability bending moment $M_{sls} =$ Tensile stress in reinforcement $\sigma_s = N$ Load durationLong

 $ψ_2 = 0.6$   $M_{sls} = 16.6 \text{ kNm/m}$   $σ_s = M_{sls} / (A_{bb.prov} × z) = 114.6 \text{ N/mm}^2$  Long term

Load durat	tion factor	kt = <b>0.4</b>
Effective a	rea of concrete in tension	$A_{c.eff} = min(2.5 \times (h - d), (h - x) / 3, h / 2) = 105458 mm^2/m$
Mean valu	e of concrete tensile strength	$f_{ct.eff} = f_{ctm} = 2.9 \text{ N/mm}^2$
Reinforcer	nent ratio	$\rho_{p.eff} = A_{bb.prov} / A_{c.eff} = 0.005$
Modular ra	tio	α <sub>e</sub> = E <sub>s</sub> / E <sub>cm</sub> = <b>6.091</b>
Bond prop	erty coefficient	k1 = <b>0.8</b>
Strain distr	ibution coefficient	k <sub>2</sub> = <b>0.5</b>
		k <sub>3</sub> = <b>3.4</b>
		k <sub>4</sub> = <b>0.425</b>
Maximum	crack spacing - exp.7.11	$s_{r.max}$ = $k_3 \times c_{bb}$ + $k_1 \times k_2 \times k_4 \times \phi_{bb}$ / $\rho_{p.eff}$ = 635 mm

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London	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date	
N1 0SP	GO	03/05/2016					
Maximum crack width - exp.7.8		Wk = Sr.max >	< max(σs – kt ×	(f <sub>ct.eff</sub> / $\rho_{p.eff}$ ) × (1	+ $\alpha_{e} \times \rho_{p.eff}$ ), 0	$.6  imes \sigma_s) / E_s$	
		w <sub>k</sub> = 0.219	mm				
		w <sub>k</sub> / w <sub>max</sub> =	0.728				
		PASS	- Maximum c	rack width is les	ss than limitin	ng crack width	
Rectangular section in shear	- Section 6.2						
Design shear force		V = <b>19.8</b> kN	V = 19.8  kN/m C <sub>Rd c</sub> = 0.18 / $\gamma_{C} = 0.120$				
		$C_{Rd,c} = 0.18$					
		k = min(1 + 1)	$k = min(1 + \sqrt{200} mm/d) = 1.862$				
Longitudinal reinforcement ratio		$\alpha = \min(A_{\rm b})$	$\alpha = \min(A_{\text{th}} \max(d, 0, 0, 2)) = 0.002$				
			$y_{\text{min}} = 0.035 \text{ N}^{1/2} \text{ /mm} \times k^{3/2} \times f_{10} \cdot 0.5 = 0.487 \text{ N} \text{ /mm}^2$				
Design sheer resistance over 6		Vmin - 0.03	20	$\times 1_{CK} = 0.4071$	n/////// د ۱/۶ مر ۲/۱۰ ما		
Design shear resistance - exp.6.2a & 6.2b		$V_{Rd.c} = max$	$V_{Rd,c} = \Pi dX (C_{Rd,c} \times K \times (100 \text{ N}^2/\Pi \Pi \Pi^2 \times \rho I \times I_{Ck})^{++}, V_{min}) \times U$				
		$V_{Rd.c} = 131$	$V_{Rd,c} = 131.1 \text{ KN/m}$				
	$V / V_{Rd.c} = 0$	$V / V_{Rd,c} = 0.151$					
Secondary transverse reinford	amont to been	PAJ	ss - Design sn		exceeds desig	gii shear iorce	
Minimum area of reinforcement			) v Au – 11	$2 mm^2/m$			
Maximum spacing of reinforcement $-$ cl.9.3.1.1(2)		Abx.req = $0.2$	Abx.req $-0.2 \times \text{Abb.prov} - 113 \text{ mm} / \text{m}$				
Transverse reinforcement provided		3) $Sbx_max = 43$	$S_{bx}_{max} = 430$ [1][[]				
Area of transverse reinforcement provided			$A_{1} = - x + \frac{2}{2} ((4 + x)) - 302 = m^{2}/m$				
Area of transverse reinforcemer		Abx.prov = $\pi$	$\times \phi_{\text{bx}}$ / (4 $\times$ Sbx	) = 393 mm-/m			
	PASS - Area of	reinforcement	t provided is g	greater than area	a of reinforce	ment requirea	
		40	0→   ←→   ←50				
	10	dia.bars @ 200 c/c					
	horiz par	ontal reinforcement allel to face of stem					
			• •				
			• •				
T	10	dia.bars @ 200 c/c		6 dia.bars @ 200 c/c			

