**Structural Calculations** 

for new basement at

**10, Downside Crescent** 

Belsize Park, London

NW3

## rodriguesassociates 1 Amwell Street London EC1R 1UL

Telephone 020 7837 1133 www.rodriguesassociates.com October 2016 **Structural Calculations** 

for

10, Downside Crescent Belsize Park, London NW3 for

Bow Tie Construction Ltd Unit 86, Cressex Enterprise Centre, Lincoln Road High Wycombe, Bucks HP12 3RL

Job No 1411

Rev	Date	Notes
-	12.10.16	Structural package

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# **1. CALCULATION PLAN**

This report contains the structural engineering calculations for the proposed new basement for 10, Downside Crescent.

The development consists of an existing semi-detached house rear extension. The extension will be composed by two levels: basement and ground floor. The access from the main building will be provided creating a new opening in the existing building back wall.

## **1.1. SUMMARY OF STRUCTURE**

Proposed plan area – extension

Maximum plan dimensions	6.4m by 6.5m, say
Footprint area	41.6m <sup>2</sup>
Storeys	Basement and Ground floor
Maximum height	3m over ground level

## **1.2. IMPOSED LOADS**

The following imposed loads have been used

Typical imposed loads on pitched roofs	0.75 kN/m <sup>2</sup>
Typical imposed loads on floors	1.50 kN/m <sup>2</sup>
Partitions loads on floors (as imposed loads)	1.00 kN/m <sup>2</sup>
Typical imposed loads on flat roofs allowing for maintenance	1.50 kN/m <sup>2</sup>

## **1.3. REAR EXTENSION**

The basement of the new extension will be realised with new reinforced concrete walls and slabs. Ground floor walls will be realized with cavity block works and the roof will be mainly constructed in timber elements and steel beams.

# 2. RESOURCES

## 2.1 CODES & REFERENCES

- BS6399 Pt1 Loadings for buildings. Code of practice for dead and imposed loads.
- BS6399 Pt2 Loadings for buildings. Code of practice for wind loads.
- BS6399 Pt3 Loadings for buildings. Code of practice for imposed roof loads.
- BS5269 Pt2 Structural use of Timber. Code of practice for permissible stress design, materials and workmanship.
- BS5628 Pt1 Use of masonry. Structural use of unreinforced masonry.
- BS5950 Pt1 Structural use of steelwork in building. Code of practice for design in simple and continuous construction hot rolled sections.
- BS8110 Pt1 Structural use of concrete

Manual for the design of plain masonry in building structures – The Institution of Structural Engineers. July 1997.

### 2.2 SOFTWARE

Tekla Structural Designer suite of design and analysis tools.

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alculations:	Area loads	Designed:	ab	Date:	11/10/2016	Ckd:
	,					
Existing pitche	<u>d roof</u>					
Dead	Tiles or slates				0.80 kN/m <sup>2</sup>	
	Battens and felt				$0.05 \text{ kN/m}^2$	
	Bafters				$0.15 \text{ kN/m}^2$	
	Insulation				$0.10 \text{ kN/m}^2$	
	Sonvioos				$0.05 \text{ kN/m}^2$	
	Blasterboard and skim cost				$0.05 \text{ kN/m}^2$	
	Flasterboard and skill coal				1.21 kN/m <sup>2</sup>	-
	Roof Angle 45 °				1.71 kN/m²	
Impos	ed				0.75 kN/m <sup>2</sup>	
Existing typica	floor					
Dead	Finishes				0 15 kN/m <sup>2</sup>	
2000	Boarding				$0.14 \text{ kN/m}^2$	
	loists				$0.15 \text{ kN/m}^2$	
	Insulation				$0.05 \text{ kN/m}^2$	
	Services				$0.05 \text{ kN/m}^2$	
	Lath and plactor				$0.05 \text{ kN/m}^2$	
	Latif and plaster				0.79 kN/m <sup>2</sup>	-
Impos	ed				1.50 kN/m <sup>2</sup>	
External brick	wall					
Dead	External render				$0.60 \text{ kN/m}^2$	
Dead	215mm brickwork				$4.72 \text{ kN/m}^2$	
	2 I Shilli Dhekwolk				$4.75 \text{ kN/m}^2$	
	Flaster				5.59 kN/m <sup>2</sup>	_
					0.00 KIN/III	

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Octovelationary	10 Downside Crescent	Desimado		Deter		Olat	
Calculations:	Area loads	Designed:	ab	Date:	11/10/2016	Ска: -	
Proposed grou	und floor slab						
Dead	Finishes				0.15 kN/m	2	
	Screed				1.80 kN/m	2	
	Insulation				0.05 kN/m	2	
	175mm slab				4 20 kN/m	2	
	Services				0.15 kN/m	2	
					6 35 kN/m	2	
					0.55 KW/III		
Impos	sed				1.50 kN/m	2	
	Partitions				1.00 kN/m	2	
					2.50 kN/m	2	
Proposed bas	ement floor slab						
Dead	Finishes				0.15 kN/m	2	
Deau	Scrood				1.80 kN/m	2	
					0.05 kN/m	2	
	F00mm alab				10.00 kN/m	2	
	Sources				0.15 kN/m	2	
	Services				14.15 kN/m	2	
Impos	sed				1.50 kN/m	2	
	Partitions				1.00 kN/m	2	
					2.50 kN/m	2	
Proposed flat	roof						
Deed					0.15 kN/m	2	
Dead	Fibre glass waterproofing				0.15 KIN/III	2	
	Boarding				0.14 KN/III	2	
	Insulation				0.05 KN/III	.2	
	JOISIS				0.15 KN/III	2	
	Services				0.05 KN/III	.2	
	Plasterboard and skim coat				0.15 KN/III	2	
					0.69 KN/m		
Impos	sed (allowing for maintenance of st	ructure ab	ove)		1.50 kN/m	2	
			,				
<u>Glazing</u>							
Dood	Glazing (Doublo)				0 65 60/~	2	
Dead	Giaziliy (DOUDIE)					1 2	
	Framing					2	
					0.85 KN/M	I	
Impos	sed (for horizontal glazing accounti	ng for sno	w)		0.75 kN/m	2	
		3.0.010	-,				

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Calculations:	Area loads	Designed:	ab	Date: 11/10	0/2016	Ckd: -
<u>Proposed exter</u> Dead	nal wall External render 100mm blockwork Insulation 100mm block work Plasterboard and skim coat				0.60 kN/m <sup>2</sup> 1.50 kN/m <sup>2</sup> 0.05 kN/m <sup>2</sup> 1.50 kN/m <sup>2</sup> 0.15 kN/m <sup>2</sup>	
					3.80 kN/m <sup>2</sup>	







![](_page_11_Figure_0.jpeg)

![](_page_12_Figure_0.jpeg)

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		<b>A</b>	1	14/1-111-					Duint	1	1
Beam & Load	Span	Area	loads	vviatn	LOCa	ation			Point	loads	-
description			LL		Irom	10					
		KIN/III <sup>2</sup>	KIN/111-	mm	mm		KIN/III	KIN/III	KIN	KIN	
Phase 3 - new	basemer	nt front v	wall and	slab ca	st in pla	ce					
ROPALEN	SECTION	B-B									
TILLION	50000	5.0	1-11-	-	COWER AREA I TETRPOR	EXCAUL IN ORI CARY U	HIDN W SR TO DAUNG	PUACE BEAM	BALETVE TOP S 7 PR	NT .ops.	
11			1	L -	COWER	EXCA	NOTION	AGAI	V is Pe	ARE	
	2 2 2 2 246 @ 200 240 Anger	CAL IN.	SEE TON	1500 K	COWER REINFO BASEM	AGAIN DELETIE ENT !	N EXCA ENT AN SLAB, TONS TONS	UATION DE CAST - STARTI FOR R	NG BAR	s y wan	N.
Peopoles D	SECTION	UBAR IN	INIOC N	-	RETLOUS AND CA	E GOTT	ot hor shent	190NTA	r Peops		
		e e .		-	AND OUTSUD HORITCO FREE)	D CONC D AR O E FALL D NTA I	RETE FOI 200 cle E), 4 BARS (1	2 300, BARS DAO 0 NGAOE	IN THE (INSID 200 c/c AND 0	E HUD	5
Phase 4 - new ex Load on new ex Ground floor	ground f tension g 6400x38	iloor sla pround flo 500 6.35 eet 5.5 fr	b cast ir por slab 2.50 o ground	1000	ab calcul	ations	6.35	2.50			

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Beam & Load	Span	Area	loads	Width	Loca	ation	U	DL	Point	loads	
description		DI	11		from	to	DI	11	DI		
accomption	mm	kN/m <sup>2</sup>	kN/m <sup>2</sup>	mm	mm	mm	kN/m	kN/m	kN		
	111111	KIN/111 <sup>-</sup>	KIN/111 <sup>-</sup>	111111	111111	111111	KIN/111	KIN/111	<b>NIN</b>		
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and a contraction	- L -	1000									
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Beam B0 1											
<u>Boam Born</u>	6400										
	0400	0.05	0.50	4750				4.00			
Ground floor		6.35	2.50	1750	trianguia	ar Ioad	11.11	4.38			
Direct load on be	eam	2.15	2.50	500			1.08	1.25			
Beam B0.2											
	6400										
Cround floor	0400	C 05	0.50	1750	ار به مرمنا ا	w lood	44.44	4.00			
Ground lloor		6.35	2.50	1750	Inangula	ar ioad	11.11	4.38			
		6.35	1.50	400	2000	4400	2.54	0.60			
Direct load on be	am	2.15	2.50	500	0	4400	1.08	1.25			
Glazed door		0.85		2300	0	4400	1.96				
Cavity wall above	2	3 80		2500	4400	6400	9 50				
Poof	, 1600	0.00	1 50	2500	4400	0400	0.00		2.76	6.00	
	1000	0.09	1.50	2000	4400				2.70	0.00	
	See She	et 5.6 a	nd 5.7 fo	r beam I	30.1 and	B0.2 ca	lculation	S			
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![](_page_15_Figure_0.jpeg)

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TEDDS calculation version 1.2.01.06

#### RETAINING WALL ANALYSIS (BS 8002:1994)

![](_page_16_Figure_2.jpeg)

#### Wall details

Retaining wall type Height of retaining wall stem Thickness of wall stem Length of toe Length of heel Overall length of base Thickness of base Depth of downstand Position of downstand Thickness of downstand Height of retaining wall Depth of cover in front of wall Depth of unplanned excavation Height of ground water behind wall Height of saturated fill above base Density of wall construction Density of base construction Angle of rear face of wall Angle of soil surface behind wall Effective height at virtual back of wall

## **Retained material details**

Mobilisation factor Moist density of retained material

## Cantilever propped at both h<sub>stem</sub> = **3000** mm twall = 315 mm I<sub>toe</sub> = **450** mm $I_{heel} = \mathbf{0} mm$ $I_{\text{base}} = I_{\text{toe}} + I_{\text{heel}} + t_{\text{wall}} = 765 \text{ mm}$ t<sub>base</sub> = **300** mm $d_{ds} = 0 \text{ mm}$ lds = **15** mm t<sub>ds</sub> = **300** mm $h_{wall} = h_{stem} + t_{base} + d_{ds} = 3300 \text{ mm}$ $d_{cover} = \mathbf{0} mm$ d<sub>exc</sub> = **200** mm $h_{water} = 0 mm$ $h_{sat} = max(h_{water} - t_{base} - d_{ds}, 0 mm) = 0 mm$ $\gamma_{wall} = 23.6 \text{ kN/m}^3$ γ<sub>base</sub> = 23.6 kN/m<sup>3</sup> $\alpha = 90.0 \text{ deg}$ $\beta = 0.0 \text{ deg}$ $h_{eff} = h_{wall} + I_{heel} \times tan(\beta) = 3300 \text{ mm}$

M = **1.5** γ<sub>m</sub> = **18.0** kN/m<sup>3</sup>

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Saturated density of retained ma	aterial	γ <sub>s</sub> = <b>21.0</b> kN	J/m <sup>3</sup>		·	·
Design shear strength		∳' = <b>18.6</b> de	g			
Angle of wall friction		$\delta = 0.0 \text{ deg}$	-			
Base material details						
Firm clay						
Moist density		$\gamma_{mb} = 18.0 \text{ k}$	xN/m <sup>3</sup>			
Design shear strength		∳'₅ = <b>16.5</b> d	eg			
Design base friction		δ <sub>b</sub> = <b>18.6</b> de	eq			
Allowable bearing pressure		P <sub>bearing</sub> = 10	<b>0</b> kN/m²			
Using Coulomb theory						
Active pressure coefficient for re	etained material					
$K_a = sin(\alpha$	+ $\phi'$ ) <sup>2</sup> / (sin( $\alpha$ ) <sup>2</sup> ×	$\sin(\alpha - \delta) \times [1 + \delta]$	$\sqrt{(\sin(\phi' + \delta) \times s)}$	$\sin(\phi' - \beta) / (\sin(\alpha))$	- $\delta$ ) × sin( $\alpha$ +	3)))] <sup>2</sup> ) = <b>0.516</b>
Passive pressure coefficient for	base material					
	$K_p = sin(9)$	0 - φ' <sub>b</sub> )² / (sin(90	- $\delta_b$ ) × [1 - $\sqrt{(sin)}$	$h(\phi'_{b} + \delta_{b}) \times \sin(\phi')$	b) / (sin(90 + δ	b)))] <sup>2</sup> ) = <b>2.835</b>
At-rest pressure						
At-rest pressure for retained ma	terial	$K_0 = 1 - sin$	(φ') = <b>0.681</b>			
Loading details						
Surcharge load on plan		Surcharge	= <b>5.0</b> kN/m²			
Applied vertical dead load on wa	all	$W_{dead} = 41.$	<b>0</b> kN/m			
Applied vertical live load on wall		W <sub>live</sub> = 7.4 k	kN/m			
Position of applied vertical load	on wall	l <sub>load</sub> = <b>558</b> n	nm			
Applied horizontal dead load on	wall	F <sub>dead</sub> = <b>0.0</b>	κN/m			
Applied horizontal live load on w	all	$F_{live} = 0.0 \text{ kl}$	N/m			
Height of applied horizontal load	on wall	110ad = <b>U</b> 1111	n			
	A.8 99.5	Prop -	Image: Second secon			
				Loads shown	ı in kN/m, pressure	s shown in kN/m²

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1 AMWELL STREET		exist back wa	ll underpinning	)	5	.1. 3			
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Vertical forces on wall									
Wall stem		$w_{wall} = h_{stem}$	$\times t_{wall} \times \gamma_{wall} =$	<b>22.3</b> kN/m					
Wall base		$w_{\text{base}} = I_{\text{base}}$	$ imes$ t <sub>base</sub> $ imes$ $\gamma$ <sub>base</sub>	= <b>5.4</b> kN/m					
Applied vertical load		$W_v = W_{dead}$	+ Wlive = <b>48.4</b>	kN/m					
Total vertical load		$W_{total} = W_{wa}$	$w_{base} + W_v =$	= <b>76.1</b> kN/m					
Horizontal forces on wall									
Surcharge		$F_{sur} = K_a \times F_{sur}$	Surcharge × h	<sub>eff</sub> = <b>8.5</b> kN/m					
Moist backfill above water table	е	F <sub>m_a</sub> = 0.5 >	$ imes K_{\mathrm{a}}  imes \gamma_{\mathrm{m}}  imes (h_{\mathrm{eff}})$	f - h <sub>water</sub> ) <sup>2</sup> = <b>50.6</b>	kN/m				
Total horizontal load		F <sub>total</sub> = F <sub>sur</sub> ·	+ F <sub>m_a</sub> = <b>59.1</b> k	κN/m					
Calculate total propping force	e								
Passive resistance of soil in fro	ont of wall	$F_p = 0.5 \times I$	$K_{p}  imes \cos(\delta_{b})  imes (\delta_{b})$	dcover + tbase + dds	s - $d_{exc})^2 \times \gamma_{mb} =$	• <b>0.2</b> kN/m			
Propping force		$F_{prop} = max$	$F_{prop} = max(F_{total} - F_{p} - (W_{total} - W_{iive}) \times tan(\delta_{b}), 0 \text{ kN/m})$						
		F <sub>prop</sub> = <b>35.8</b>	F <sub>prop</sub> = <b>35.8</b> kN/m						
Overturning moments									
Surcharge		M <sub>sur</sub> = F <sub>sur</sub> >	$<$ (h <sub>eff</sub> - 2 $\times$ d <sub>ds</sub>	) / 2 = <b>14.1</b> kNm/	/m				
Moist backfill above water table	е	Mm a = Fm a	$h_{a} \times (h_{eff} + 2 \times h_{f})$	, <sub>water</sub> - 3 × d <sub>ds</sub> ) / 3	= <b>55.7</b> kNm/m				
Total overturning moment		M <sub>ot</sub> = M <sub>sur</sub> +	$+ M_{m_a} = 69.7 \text{ kNm/m}$						
Restoring moments									
Wall stem		$M_{wall} = W_{wall}$	$\times$ (I <sub>toe</sub> + t <sub>wall</sub> / 2	) = <b>13.5</b> kNm/m					
Wall base		M <sub>base</sub> = w <sub>bas</sub>	$s_{e} \times I_{base} / 2 = 2$	. <b>1</b> kNm/m					
Design vertical dead load		$M_{dead} = W_{dead}$	$had \times had = 22.9$	<b>)</b> kNm/m					
Total restoring moment		M <sub>rest</sub> = M <sub>wall</sub>	$M_{rest} = M_{wall} + M_{base} + M_{dead} = 38.5 \text{ kNm/m}$						
Check bearing pressure									
Total vertical reaction		$R = W_{total} =$	<b>76.1</b> kN/m						
Distance to reaction		$x_{bar} = I_{base}$ /	2 = <b>383</b> mm						
Eccentricity of reaction		e = abs((I <sub>ba</sub>	<sub>se</sub> / 2) - x <sub>bar</sub> ) = (	<b>0</b> mm					
				Reaction acts	within middle	e third of ba			
Bearing pressure at toe		$p_{toe} = (R / I_t)$	$_{\rm base}$ ) - (6 $ imes$ R $ imes$	e / I <sub>base</sub> ²) = <b>99.5</b>	kN/m²				
Bearing pressure at heel		$p_{heel} = (R / $	$_{base}$ ) + (6 × R >	< e / I <sub>base</sub> <sup>2</sup> ) = <b>99.5</b>	kN/m²				
	_	100 14 1			allassable bes				

Propping force to base of wall

$$\begin{split} F_{prop\_top} = (M_{ot} - M_{rest} + R \times I_{base} / 2 - F_{prop} \times t_{base} / 2) / (h_{stem} + t_{base} / 2) = \textbf{17.460} \text{ kN/m} \\ F_{prop\_base} = F_{prop} - F_{prop\_top} = \textbf{18.299} \text{ kN/m} \end{split}$$

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1 AMWELL STREET		exist back wa	all underpinning	]	5	.1. 4			
LONDON EC1R 1UL	Calcs by ab	Calcs date 12/10/2016	Checked by	Checked date	Approved by	Approved date			
RETAINING WALL DESIGN (	BS 8002:1994)	)							
Ultimate limit state load fact	ors				TEDDS calculatio	n version 1.2.01.06			
Dead load factor		$\gamma_{f_d} = 1.4$							
Live load factor		γ <sub>f_l</sub> = <b>1.6</b>							
Earth and water pressure factor	or	$\gamma_{f_e} = 1.4$							
Factored vertical forces on v	vall								
Wall stem		$W_{wall_f} = \gamma_{f_d}$	imes h <sub>stem</sub> $ imes$ t <sub>wall</sub> $ imes$	γ <sub>wall</sub> = <b>31.2</b> kN/n	n				
Wall base		Wbase $f = \gamma f$	$_{ m d}  imes {\sf I}_{ m base}  imes {\sf t}_{ m base}  imes$	<γ <sub>base</sub> = <b>7.6</b> kN/r	n				
Applied vertical load		W <sub>v</sub> f=γfd2	×W <sub>dead</sub> + γ <sub>f</sub> ι×	W <sub>live</sub> = <b>69.2</b> kN/n	n				
Total vertical load		$W_{total_f} = W_v$	vall_f + Wbase_f +	W <sub>v_f</sub> = <b>108</b> kN/m					
Factored horizontal at-rest for	orces on wall								
Surcharge		$F_{sur} f = \gamma_{f}$	< K₀ × Surchard	ae × h <sub>eff</sub> = <b>18</b> kN/	′m				
Moist backfill above water tabl	e	$F_{m,n,f} = \gamma_{f,c}$	$F_{m,n,t} = v_{t,n,k} + 0.5 \times K_0 \times v_{m,k} \times (h_{m,t} - h_{m,t,n})^2 = 93.4 \text{ kNl/m}$						
Total horizontal load	•	$F_{\text{total } f} = F_{\text{su}}$	r f + Fm a f = <b>11</b>	<b>1.4</b> kN/m	•••••				
Calculate total prophing for	<u>م</u>								
Passive resistance of soil in fro	ont of wall	$F_{n,f} = \gamma_{f,n} \times$	$0.5 \times K_{\rm p} \times \cos$	$(\delta_{\rm b}) \times (d_{\rm cover} + t_{\rm ball})$	$a_{0} + d_{d_{0}} - d_{a_{0}})^{2}$	$\times \gamma_{mh} = 0.3$			
kN/m		∎ p_i — }i_e ∧	0.0 × 10 × 000		se + dus dexc)	× 1110 - 0.0			
Propping force	$\begin{split} F_{\text{prop}\_f} &= max(F_{\text{total}\_f} - F_{p\_f} - (W_{\text{total}\_f} - \gamma_{f\_l} \times W_{\text{live}}) \times tan(\delta_b), \ 0 \ \text{kN/m} \\ F_{\text{prop}\_f} &= \textbf{78.7} \ \text{kN/m} \end{split}$			kN/m)					
Factored overturning mome	nts								
Surcharge		$M_{sur_f} = F_{sur}$	$r_f \times (h_{eff} - 2 \times c)$	d <sub>ds</sub> ) / 2 = <b>29.7</b> kN	m/m				
Moist backfill above water tabl	e	$M_{m_a_f} = F_m$	$M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3 = \textbf{102.8} \text{ kNm/m}$						
Total overturning moment		$M_{ot_f} = M_{sur}$	$_{f} + M_{m_a_f} = 13$	<b>2.5</b> kNm/m					
Restoring moments									
Wall stem		$M_{wall_f} = W_w$	all_f $\times$ (Itoe + twall)	/ 2) = <b>19</b> kNm/m					
Wall base		$M_{base_f} = W_{base_f}$	$_{\text{base_f} \times \text{l}_{\text{base}}} / 2 =$	= <b>2.9</b> kNm/m					
Design vertical load		$M_{v_f} = W_{v_f}$	$M_{v f} = W_{v f} \times I_{load} = 38.6 \text{ kNm/m}$						
Total restoring moment		$M_{rest_f} = M_w$	$M_{rest_f} = M_{wall_f} + M_{base_f} + M_{v_f} = \textbf{60.5} \text{ kNm/m}$						
Factored bearing pressure									
Total vertical reaction		$R_f = W_{total}$	= <b>108.0</b> kN/m						
Distance to reaction		X <sub>bar_f</sub> = I <sub>base</sub>	$x_{bar_{f}} = I_{base} / 2 = 383 \text{ mm}$						
Eccentricity of reaction		$e_f = abs((I_b$	<sub>ase</sub> / 2) - x <sub>bar_f</sub> ) =	= <b>0</b> mm					
				Reaction acts	within middle	e third of base			
Bearing pressure at toe		$p_{toe_f} = (R_f)$	$I_{base}$ ) - (6 $\times$ R <sub>f</sub>	$\times e_f / I_{base}^2$ ) = 141	<b>I.2</b> kN/m <sup>2</sup>				
Bearing pressure at heel	$p_{heel_f} = (R_f / I_{base}) + (6 \times R_f \times e_f / I_{base}^2) = 141.2 \text{ kN/m}^2$								
Rate of change of base reaction	on	$rate = (p_{toe})$	_f - $p_{\text{heel}_f}$ / $I_{\text{base}}$	= <b>0.00</b> kN/m <sup>2</sup> /m					
Bearing pressure at stem / toe		$p_{stem\_toe\_f} =$	max(p <sub>toe_f</sub> - (ra	te × I <sub>toe</sub> ), 0 kN/m <sup>2</sup>	<sup>2</sup> ) = <b>141.2</b> kN/n	1 <sup>2</sup>			
Bearing pressure at mid stem	m $p_{stem_mid_f} = max(p_{toe_f} - (rate \times (I_{toe} + t_{wall} / 2)), 0 \text{ kN/m}^2) = 141.2 \text{ kN/m}^2$					<b>141.2</b> kN/m <sup>2</sup>			
Bearing pressure at stem / hee	$Bearing \ pressure \ at \ stem \ / \ heel \ p_{stem\_heel\_f} = max(p_{toe\_f} - (rate \times (I_{toe} + t_{wall})), \ 0 \ kN/m^2) = 141.2 \ kN/m^2$								
Calculate propping forces to	top and base	e of wall							
Propping force to top of wall									
	F <sub>prop_top_f</sub> :	= (M <sub>ot_f</sub> - M <sub>rest_f</sub> + R	$_{\rm f}  imes {\rm I}_{\rm base}$ / 2 - ${\rm F}_{\rm pr}$	<sub>op_f</sub> × t <sub>base</sub> / 2) / (ł	n <sub>stem</sub> + t <sub>base</sub> / 2)	= <b>32.225</b> kN/m			

Propping force to base of wall

F<sub>prop\_base\_f</sub> = F<sub>prop\_f</sub> - F<sub>prop\_top\_f</sub> = **46.487** kN/m

Tekia Tedds RODRIGUES ASSOCIATES 1 AMWELL STREET LONDON EC1R 1UL	Project	10 Downsi	Job no. 1411			
	Calcs for	exist back wa	Start page no./Revision 5.1. 5			
	Calcs by ab	Calcs date 12/10/2016	Checked by	Checked date	Approved by	Approved date
Design of reinforced concrete retaining wall toe (BS 8002:1994)						
Material properties Characteristic strength of concre	f <sub>cu</sub> = <b>30</b> N/n	nm²				

Characteristic strength of reinforcement	f <sub>y</sub> = <b>500</b> N/mm <sup>2</sup>
Base details	
Minimum area of reinforcement	k = <b>0.13</b> %
Cover to reinforcement in toe	c <sub>toe</sub> = <b>45</b> mm
Calculate shear for toe design	
Shear from bearing pressure	$V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times I_{toe} / 2 = \textbf{63.6 kN/m}$
Shear from weight of base	$V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times I_{toe} \times t_{base} = \textbf{4.5 kN/m}$
Total shear for toe design	$V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} = 59.1 \text{ kN/m}$

## Calculate moment for toe design

Moment from bearing pressure Moment from weight of base Total moment for toe design

![](_page_20_Figure_4.jpeg)

 $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (I_{toe} + t_{wall} / 2)^2 / 6 = 26.1 \text{ kNm/m}$  $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (I_{toe} + t_{wall} / 2)^2 / 2) = 1.8 \text{ kNm/m}$  $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} = \textbf{24.2} \text{ kNm/m}$ 

**↓**100-

Check toe in bending	
Width of toe	b = <b>1000</b> mm/m
Depth of reinforcement	$d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) = 251.0 \text{ mm}$
Constant	$K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) = 0.013$
	Compression reinforcement is not required
Lever arm	$z_{toe} = min(0.5 + \sqrt{(0.25 - (min(K_{toe}, 0.225) / 0.9)), 0.95)} \times d_{toe}$
	z <sub>toe</sub> = <b>238</b> mm
A () · · ( ) · · ·	

Area of tension reinforcement required Minimum area of tension reinforcement Area of tension reinforcement required Reinforcement provided Area of reinforcement provided

Check shear resistance at toe Design shear stress Allowable shear stress

From BS8110:Part 1:1997 – Table 3.8 Design concrete shear stress

 $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) = 234 \text{ mm}^2/\text{m}$  $A_{s\_toe\_min} = k \times b \times t_{base} = \textbf{390} \ mm^2/m$  $A_{s\_toe\_req} = Max(A_{s\_toe\_des}, A_{s\_toe\_min}) = 390 \text{ mm}^2/\text{m}$ C503 mesh

 $A_{s_{toe_prov}} = 503 \text{ mm}^2/\text{m}$ 

PASS - Reinforcement provided at the retaining wall toe is adequate

 $v_{toe} = V_{toe} / (b \times d_{toe}) = 0.235 \text{ N/mm}^2$  $v_{adm} = min(0.8 \times \sqrt{(f_{cu} / 1 N/mm^2)}, 5) \times 1 N/mm^2 = 4.382 N/mm^2$ PASS - Design shear stress is less than maximum shear stress

vc\_toe = 0.441 N/mm<sup>2</sup>

*v*<sub>toe</sub> < *v*<sub>c\_toe</sub> - No shear reinforcement required

<b>Tekla</b>	Project 10 Downside Crescent				Job no.				
Tedds					1	411			
RODRIGUES ASSOCIATES	Calcs for				Start page no./R	levision			
		exist back wa	II underpinning	9	5.	1.6			
EC1R 1UL	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date			
	ab	12/10/2016							
Design of reinforced concret	e retaining wa	all stem (BS 8002	:1994)						
Material properties		V	<u>/</u>						
Characteristic strength of conc	rete	f <sub>cu</sub> = <b>30</b> N/r	nm²						
Characteristic strength of reinfo	prcement	f <sub>y</sub> = <b>500</b> N/	mm²						
Wall details									
Minimum area of reinforcement	t	k = <b>0.13</b> %	k = 0.13 %						
Cover to reinforcement in stem		C <sub>stem</sub> = <b>45</b> r	c <sub>stem</sub> = <b>45</b> mm						
Cover to reinforcement in wall		c <sub>wall</sub> = <b>45</b> mm							
Factored horizontal at-rest for	orces on stem	1							
Surcharge	Surcharge			$F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times Surcharge \times (h_{eff} - t_{base} - d_{ds}) = \textbf{16.3 kN/m}$					
Moist backfill above water table	9	$F_{s_m_a_f} = 0$	$F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2 = \textbf{77.2 kN/m}$						
Calculate shear for stem des	ign								
Surcharge		$V_{s\_sur_f} = 5$	$\times F_{s_{s_{s_{s_{s_{s_{s_{s_{s_{s_{s_{s_{s_$	1 <b>0.2</b> kN/m					
Moist backfill above water table	9	$V_{s_m_a_f} = F$	$V_{s_m_a_f} = F_{s_m_a_f} \times b_i \times ((5 \times L^2) - b_i^2) / (5 \times L^3) = 60.2 \text{ kN/m}$						
Total shear for stem design		$V_{stem} = V_{s_s}$	$V_{stem} = V_{s\_sur\_f} + V_{s\_m\_a\_f} = 70.4 \text{ kN/m}$						
Calculate moment for stem d	esign								
Surcharge		$M_{s\_sur} = F_{s\_}$	sur_f × L / 8 = 6	<b>.4</b> kNm/m					
Moist backfill above water table	9	$M_{s_m_a} = F_s$	$M_{s_m_a} = F_{s_m_a_f} \times b_I \times ((5 \times L^2) - (3 \times b_I^2)) / (15 \times L^2) = 35.2 \text{ kNm/m}$						
Total moment for stem design		$M_{stem} = M_s$	$M_{stem} = M_{s\_sur} + M_{s\_m\_a} = \textbf{41.6 kNm/m}$						
Calculate moment for wall de	esign								
Surcharge		$M_{w_sur} = 9 >$	$\langle F_{s\_sur\_f} \times L / 1$	28 = <b>3.6</b> kNm/m					
Moist backfill above water table	9	$M_{w_m_a} = F_s$	 M <sub>w_m_a</sub> = F <sub>s_m_a_f</sub> × 0.577×bi×[(bi³+5×ai×L²)/(5×L³)-0.577²/3] = <b>14.6</b>						
kNm/m									
Total moment for wall design		$M_{wall} = M_{w_s}$	sur + Mw_m_a = <b>1</b>	<b>8.2</b> kNm/m					
	<b>∢</b> -100- <b>▶</b>								

![](_page_21_Figure_1.jpeg)

| ← 100 →

Check wall stem in bending	
Width of wall stem	b = <b>1000</b> mm/m
Depth of reinforcement	$d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2) = 266.0 \text{ mm}$
Constant	$K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu}) = 0.020$
	Compression reinforcement is not required
Lever arm	$z_{\text{stem}} = min(0.5 + \sqrt{(0.25 - (min(K_{\text{stem}}, 0.225) / 0.9)), 0.95)} \times d_{\text{stem}}$
	z <sub>stem</sub> = <b>253</b> mm
Area of tension reinforcement required	$A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem}) = 379 \text{ mm}^2/\text{m}$

<b>Tekla</b>	Project	Project 10 Downside Crescent				Job no. 1411			
BODBIGUES ASSOCIATES	Calcs for				Start page no /F	t page no /Bevision			
1 AMWELL STREET	Oales Iol	exist back wa	II underpinning	g	5.1. 7				
EC1R 1UL	Calcs by ab	Calcs date 12/10/2016	Checked by	Checked date	Approved by	Approved date			
Minimum area of tension reinf	orcement	A <sub>s_stem_min</sub> =	$\mathbf{k} \times \mathbf{b} \times \mathbf{t}_{wall} = \mathbf{k}$	<b>410</b> mm²/m					
Area of tension reinforcement	required	As stem reg =	Max(As stem de	$A_{s,stem min}) = 4$	<b>10</b> mm²/m				
Reinforcement provided	·	C503 mes	h	,					
Area of reinforcement provide	d	As_stem_prov =	= <b>503</b> mm²/m						
		PASS - Reinfo	orcement prov	vided at the reta	nining wall ster	m is adequate			
Check shear resistance at w	all stem								
Design shear stress		v <sub>stem</sub> = V <sub>sten</sub>	$_{n} / (b \times d_{stem}) =$	<b>0.265</b> N/mm <sup>2</sup>					
Allowable shear stress		v <sub>adm</sub> = min(	$v_{adm} = min(0.8 \times \sqrt{(f_{cu}/1 \text{ N/mm}^2)}, 5) \times 1 \text{ N/mm}^2 = 4.382 \text{ N/mm}^2$						
		PASS -	PASS - Design shear stress is less than maximum shear stress						
From BS8110:Part 1:1997 -	Table 3.8		•						
Design concrete shear stress		v <sub>c_stem</sub> = <b>0.427</b> N/mm <sup>2</sup>							
			Vstem	n < Vc_stem - No sl	hear reinforce	ment required			
Check mid height of wall in	bending								
Depth of reinforcement	-	$d_{wall} = t_{wall} - $	- c <sub>wall</sub> – ( $\phi_{wall}$ / 2	2) = <b>266.0</b> mm					
Constant		$K_{wall} = M_{wall}$	$K_{wall} = M_{wall} / (b \times d_{wall}^2 \times f_{cu}) = 0.009$						
			·	Compression re	einforcement is	s not required			
Lever arm		$z_{wall} = Min(0)$	0.5 + √(0.25 - (	(min(K <sub>wall</sub> , 0.225)	/ 0.9)),0.95) × (	d <sub>wall</sub>			
		Z <sub>wall</sub> = <b>253</b>	mm						
Area of tension reinforcement	required	$A_{s_wall_des} =$	$M_{wall}$ / (0.87 $ imes$	$f_y \times z_{wall}$ ) = <b>166</b> m	1m²/m				
Minimum area of tension reinf	orcement	$A_{s_wall_min} =$	$k \times b \times t_{wall} = 4$	<b>10</b> mm²/m					
Area of tension reinforcement	required	As_wall_req =	As wall req = Max(As wall des, As wall min) = <b>410</b> mm <sup>2</sup> /m						
Reinforcement provided	·	C503 mes	C503 mesh						
Area of reinforcement provide	d	$A_{s\_wall\_prov} =$	<b>503</b> mm²/m						
	PASS	S - Reinforcement	t provided to	the retaining wa	all at mid heigl	ht is adequate			
Check retaining wall deflect	ion								
Basic span/effective depth rati	io	ratio <sub>bas</sub> = 2	0						
Design service stress		$f_s = 2 \times f_y \times$	As_stem_req / (3	$\times A_{s\_stem\_prov}) = 2$	<b>71.6</b> N/mm <sup>2</sup>				
Modification factor	factor <sub>tens</sub> = m	in(0.55 + (477 N/m	$m^2 - f_s)/(120 \times$	< (0.9 N/mm <sup>2</sup> + (N	$M_{\rm stem}/(b \times d_{\rm stem}^2)$	))),2) = <b>1.70</b>			
Maximum span/effective depth	n ratio	ratio <sub>max</sub> = ra	$atio_{bas} \times factor_{t}$	tens = <b>34.00</b>					
Actual span/effective depth ra	tio	ratio <sub>act</sub> = h <sub>s</sub>	ratio <sub>act</sub> = h <sub>stem</sub> / d <sub>stem</sub> = <b>11.28</b>						

PASS - Span to depth ratio is acceptable

![](_page_23_Figure_0.jpeg)

![](_page_23_Figure_1.jpeg)

Toe mesh - C503 - (503 mm<sup>2</sup>/m) Wall mesh - C503 - (503 mm<sup>2</sup>/m) Stem mesh - C503 - (503 mm<sup>2</sup>/m)

Tekla <sup>®</sup>	Project				Job no.	
Tedds	10 Downside Crescent				1411	
RODRIGUES ASSOCIATES	Calcs for				Start page no./Revision	
1 AMWELL STREET	new basement back retaining wall				5.2. 1	
LONDON EC1R 1UL	Calcs by ab	Calcs date 12/10/2016	Checked by	Checked date	Approved by	Approved date

TEDDS calculation version 1.2.01.06

#### RETAINING WALL ANALYSIS (BS 8002:1994)

![](_page_24_Figure_2.jpeg)

#### Wall details

Retaining wall type Height of retaining wall stem Thickness of wall stem Length of toe Length of heel Overall length of base Thickness of base Depth of downstand Position of downstand Thickness of downstand Height of retaining wall Depth of cover in front of wall Depth of unplanned excavation Height of ground water behind wall Height of saturated fill above base Density of wall construction Density of base construction Angle of rear face of wall Angle of soil surface behind wall Effective height at virtual back of wall

### **Retained material details**

Mobilisation factor Moist density of retained material

Cantilever propped at both h<sub>stem</sub> = **2500** mm twall = 300 mm  $I_{toe} = 500 \text{ mm}$  $I_{heel} = \mathbf{0} mm$  $I_{\text{base}} = I_{\text{toe}} + I_{\text{heel}} + t_{\text{wall}} = 800 \text{ mm}$ t<sub>base</sub> = **500** mm  $d_{ds} = 0 \text{ mm}$ lds = **15** mm t<sub>ds</sub> = **500** mm  $h_{wall} = h_{stem} + t_{base} + d_{ds} = 3000 \text{ mm}$  $d_{cover} = 0 mm$  $d_{exc} = 0 mm$ h<sub>water</sub> = **3000** mm  $h_{sat} = max(h_{water} - t_{base} - d_{ds}, 0 mm) = 2500 mm$  $\gamma_{wall} = 23.6 \text{ kN/m}^3$ γ<sub>base</sub> = 23.6 kN/m<sup>3</sup>  $\alpha = 90.0 \text{ deg}$  $\beta = 0.0 \text{ deg}$  $h_{eff} = h_{wall} + I_{heel} \times tan(\beta) = 3000 \text{ mm}$ 

M = **1.5**  $\gamma_m$  = **18.0** kN/m<sup>3</sup>

두 Tekla	Project	Job no.				
	TO Downside Crescent				1411	
1 AMWELL STREFT	Calcs for	iew basement h	ack retaining w	vall	Start page no./R	evision
LONDON		Calce date		Checked date	Approved by	
EC1R 1UL	ab	12/10/2016	Checked by	Checked date	Approved by	Approved date
Saturated density of retained m	naterial	γs = <b>21.0</b> kM	N/m <sup>3</sup>			
Design shear strength		φ' = <b>18.6</b> de	eg			
Angle of wall friction		$\delta = 0.0 \text{ deg}$	l			
Base material details						
Firm clay						
Moist density		γmb = <b>18.0</b> k	⟨N/m³			
Design shear strength		φ' <sub>b</sub> = <b>16.5</b> d	leg			
Design base friction		$\delta_{b} = 18.6  d_{c}$	eg			
Allowable bearing pressure		P <sub>bearing</sub> = 10	<b>10</b> kN/m²			
Using Coulomb theory						
Active pressure coefficient for r	etained material					
$K_a = sin(\alpha)$	$(\alpha)^2 + \phi')^2 / (\sin(\alpha)^2 \times \alpha)^2$	$\sin(lpha - \delta)  imes$ [1 +	$-\sqrt{(\sin(\phi' + \delta))}$	$\sin(\phi' - \beta) / (\sin(\alpha$	- $\delta$ ) × sin( $\alpha$ +	β)))]²) = <b>0.516</b>
Passive pressure coefficient fo	r base material					
	$K_p = sin(9)$	0 - φ' <sub>b</sub> )² / (sin(90	$(1 - \delta_b) \times [1 - \sqrt{s}]$	$\ln(\phi_{b} + \delta_{b}) \times \sin(\phi_{b})$	b) / (sin(90 + δ	(b)))] <sup>2</sup> ) = <b>2.835</b>
At-rest pressure						
At-rest pressure for retained ma	aterial	$K_0 = 1 - sir$	n(φ') = <b>0.681</b>			
Loading details						
Surcharge load on plan		Surcharge	= <b>10.0</b> kN/m <sup>2</sup>			
Applied vertical dead load on w	vall	$W_{dead} = 20.$	<b>3</b> kN/m			
Applied vertical live load on wa	II	Wlive = 8.0	kN/m			
Position of applied vertical load	l on wall	l <sub>load</sub> = <b>650</b> r	nm			
Applied horizontal dead load or	n wall	$F_{dead} = 0.0$	kN/m			
Applied horizontal live load on	wall d op woll	$F_{\text{live}} = 0.0 \text{ K}$	N/M			
rieight of applied holizontal loa	u on wan	1110ad = <b>0</b> 1111 28	11			
		Prop	10			
				A		
	Prop					
É	24.2		5.2 0703	29.4		
	69.3	69.3				
	11111					
					in kN/m	a abour in LNI/?
				Loads shown	нн кім/m, pressure	es snown in KN/m²

Teka Tedds RODRIGUES ASSOCIATES 1 AMWELL STREET LONDON EC1R 1UL	Project	10 Downsi	Job no. 1411			
	Calcs for	new basement b	Start page no./Revision 5.2. 3			
	Calcs by ab	Calcs date 12/10/2016	Checked by	Checked date	Approved by	Approved date
Vertical forces on wall Wall stem		w <sub>wall</sub> = h <sub>stem</sub>	$\times t_{wall} \times \gamma_{wall} =$	<b>17.7</b> kN/m		

Wall base	$w_{base} = I_{base} \times t_{base} \times \gamma_{base} = 9.4 \text{ kN/m}$
Applied vertical load	$W_v = W_{dead} + W_{live} = 28.3 \text{ kN/m}$
Total vertical load	$W_{total} = w_{wall} + w_{base} + W_v = 55.5 \text{ kN/m}$
Horizontal forces on wall	
Surcharge	$F_{sur} = K_a \times Surcharge \times h_{eff} = 15.5 \text{ kN/m}$
Saturated backfill	$F_s = 0.5 \times K_a \times (\gamma_{s} - \gamma_{water}) \times h_{water}^2 = 26 \text{ kN/m}$
Water	$F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water} = 44.1 \text{ kN/m}$
Total horizontal load	$F_{total} = F_{sur} + F_s + F_{water} = 85.6 \text{ kN/m}$
Calculate total propping force	
Passive resistance of soil in front	of wall $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb} = 6 \text{ kN/m}$
Propping force	$F_{prop} = max(F_{total} - F_p - (W_{total} - W_{live}) \times tan(\delta_b), 0 \text{ kN/m})$
	F <sub>prop</sub> = <b>63.6</b> kN/m
Overturning moments	
Surcharge	M <sub>sur</sub> = F <sub>sur</sub> × (h <sub>eff</sub> - 2 × d <sub>ds</sub> ) / 2 = <b>23.2</b> kNm/m
Saturated backfill	$M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3 = 26 \text{ kNm/m}$
Water	$M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3 = 44.1 \text{ kNm/m}$
Total overturning moment	$M_{ot} = M_{sur} + M_s + M_{water} = 93.4 \text{ kNm/m}$
Restoring moments	
Wall stem	$M_{wall} = w_{wall} \times (I_{toe} + t_{wall} / 2) = 11.5 \text{ kNm/m}$
Wall base	$M_{base} = w_{base} \times I_{base} / 2 = 3.8 \text{ kNm/m}$
Design vertical dead load	$M_{dead} = W_{dead} \times I_{load} = 13.2 \text{ kNm/m}$
Total restoring moment	$M_{rest} = M_{wall} + M_{base} + M_{dead} = 28.5 \text{ kNm/m}$
Check bearing pressure	
Total vertical reaction	R = W <sub>total</sub> = <b>55.5</b> kN/m
Distance to reaction	x <sub>bar</sub> = I <sub>base</sub> / 2 = <b>400</b> mm
Eccentricity of reaction	$e = abs((I_{base} / 2) - x_{bar}) = 0 mm$
	Reaction acts within middle third of base
Bearing pressure at toe	$p_{toe} = (R / I_{base}) - (6 \times R \times e / I_{base}^2) = 69.3 \text{ kN/m}^2$
Bearing pressure at heel	$p_{heel} = (R / I_{base}) + (6 \times R \times e / I_{base}^2) = 69.3 \text{ kN/m}^2$
	PASS - Maximum bearing pressure is less than allowable bearing pressure
Calculate propping forces to to	p and base of wall
Propping force to top of wall	

 $F_{prop\_top} = (M_{ot} - M_{rest} + R \times I_{base} / 2 - F_{prop} \times t_{base} / 2) / (h_{stem} + t_{base} / 2) = 25.881 \text{ kN/m}$ Propping force to base of wall  $F_{prop\_base} = F_{prop} - F_{prop\_top} = 37.737 \text{ kN/m}$ 

<b>Tekla</b>	Project Job no.				Job no.	o no. 1411			
RODRIGUES ASSOCIATES	Calcs for		Start page no./Revision						
1 AMWELL STREET		new basement b	ack retaining v	vall	5.2. 4				
LONDON	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date			
EC1R 1UL	ab	12/10/2016							
	l	1	Į		4				
RETAINING WALL DESIGN (	BS 8002:1994	)							
`		-			TEDDS calculation	version 1.2.01.06			
Ultimate limit state load facto	ors								
Dead load factor		$\gamma_{f\_d} = 1.4$							
Live load factor		γ <sub>f_l</sub> = <b>1.6</b>							
Earth and water pressure facto	or	$\gamma_{f_e} = 1.4$							
Factored vertical forces on v	vall								
Wall stem		$w_{wall\_f} = \gamma_{f\_d}$	imes h <sub>stem</sub> $ imes$ t <sub>wall</sub> $ imes$	γ <sub>wall</sub> = <b>24.8</b> kN/m	ı				
Wall base		$W_{base_f} = \gamma_{f_d}$	imes I <sub>base</sub> $ imes$ t <sub>base</sub> $ imes$	$\gamma_{\text{base}} = 13.2 \text{ kN}/$	′m				
Applied vertical load		$W_{v_f} = \gamma_{f_d} \times$	$W_{dead} + \gamma_{f_l} \times V_{dead}$	<i>N</i> <sub>live</sub> = <b>41.2</b> kN/m	า				
Total vertical load		$W_{total_f} = w_w$	$all_f + W_{base_f} + V$	N <sub>v_f</sub> = <b>79.2</b> kN/m	l				
Factored horizontal at-rest for	orces on wall								
Surcharge		$F_{sur_f} = \gamma_{f_l} \times$	$K_0 \times Surchargenergy$	$le  imes h_{eff} = 32.7 \; kl$	N/m				
Saturated backfill		$F_{s\_f} = \gamma_{f\_e} \times$	$0.5  imes K_0  imes (\gamma_{s} - \gamma_{s})$	$\gamma_{water}$ ) $\times$ $h_{water}^2$ = 4	<b>48</b> kN/m				
Water		$F_{water_f} = \gamma_{f_e}$	$h \times 0.5  imes h_{water}^2$	$\times \gamma_{water} = 61.8 \text{ k}$	N/m				
Total horizontal load		$F_{total_f} = F_{sur}$	_f + Fs_f + Fwater	_f = <b>142.5</b> kN/m					
Calculate total propping forc	e								
Passive resistance of soil in fro	ont of wall	$F_{p_f} = \gamma_{f_e} \times$	$0.5  imes K_p  imes \cos \theta$	$(\delta_b)  imes (d_{cover} + t_{bac})$	se + d <sub>ds</sub> - d <sub>exc</sub> ) <sup>2</sup> :	×γ <sub>mb</sub> = <b>8.5</b>			
kN/m									
Propping force		$F_{prop_f} = ma$	x(F <sub>total_f</sub> - F <sub>p_f</sub> -	$(W_{total_f} - \gamma_{f_l} \times W)$	$_{ m live})  imes tan(\delta_{ m b}), 0$	kN/m)			
		F <sub>prop_f</sub> = <b>111</b>	<b>.7</b> kN/m						
Factored overturning mome	nts								
Surcharge		$M_{sur_f} = F_{sur_f}$	$_{f} \times (h_{eff} - 2 \times d)$	<sub>ds</sub> ) / 2 = <b>49</b> kNm	/m				
Saturated backfill		$M_{s_f} = F_{s_f} \times$	$M_{s_f} = F_{s_f} \times (h_{water} - 3 \times d_{ds}) / 3 = 48 \text{ kNm/m}$						
Water		$M_{water_f} = F_w$	$_{ m ater_f}  imes$ (h <sub>water</sub> - 3	$3 \times d_{ds}) / 3 = 61.8$	<b>3</b> kNm/m				
Total overturning moment		$M_{ot_f} = M_{sur_}$	$f + M_{s_f} + M_{wate}$	r_f = <b>158.8</b> kNm/r	m				
Restoring moments									
Wall stem		$M_{wall_f} = W_{wall_f}$	$II_f  imes (I_{toe} + t_{wall} /$	2) = <b>16.1</b> kNm/r	m				
Wall base		$M_{base_f} = w_{bs}$	$M_{base_f} = w_{base_f} \times I_{base} / 2 = 5.3 \text{ kNm/m}$						
Design vertical load		$M_{v\_f} = W_{v\_f}$	× I <sub>load</sub> = <b>26.8</b> kM	Nm/m					
Total restoring moment		$M_{rest_f} = M_{wa}$	$M_{f} + M_{base_f} + N_{f}$	/lv_f = <b>48.2</b> kNm/r	m				
Factored bearing pressure									
Total vertical reaction		$R_{f} = W_{total\_f}$	= <b>79.2</b> kN/m						
Distance to reaction	the to reaction $x_{bar_f} = I_{base} / 2 = 400 \text{ mm}$								
Eccentricity of reaction		$e_f = abs((I_{ba}$	<sub>lse</sub> / 2) - x <sub>bar_f</sub> ) =	• <b>0</b> mm					
Reaction acts within mide			within middle	third of base					
Dearing pressure at toe		$p_{\text{toe}_f} = (R_f / D_f)$	ubase) - (ט × Kf)	$(e_{\rm f} / I_{\rm base}) = 99.$	$1 \text{ KIN/III}^{-1}$				
$p_{\text{heel}f} = (n_f / l_{\text{base}}) + (0 \times n_f \times e_f / l_{\text{base}}) = 33.1 \text{ KIV/III}^2$ Rate of change of base reaction $rate = (n_f / e_{\text{base}}) + (0 \times n_f \times e_f / l_{\text{base}}) = 33.1 \text{ KIV/III}^2$									
$alle = (p_{\text{toe}_{f}} - p_{\text{heel}_{f}}) /  b_{\text{ase}} = \textbf{U}.\textbf{U} \textbf{K} N/(11^{-}/11)$ $Bearing pressure at stem / toe p_{\text{toe}_{f}} - max(p_{\text{toe}_{f}} - (r_{\text{ate}_{f}} \times h_{\text{toe}}) - 0 \textbf{K} N/(11^{-}/11)$									
Bearing pressure at mid stem		$p_{\text{sterm}\_\text{toe}\_\text{t}} = 1$	$p_{stem_{toe}} = max(p_{toe_{f}} - (rate \times t_{toe}), U KIV/II^{-}) = 99.1 KIV/II^{-}$						
Bearing pressure at this stem / here	j	pstem_mid_t = IIIax(ptoe_t - (Iate × (Itoe + Iwall / 2)), 0 KIV/III <sup>2</sup> ) = <b>33.1</b> KIV/III <sup>2</sup> pstem_bool t = max(ptoe_t - (Iate × (Itoe + Iwall / 2)), 0 KIV/III <sup>2</sup> ) = <b>00.1</b> KIV/III <sup>2</sup>							
			av(hine_i - (i a	$\sim \alpha (100 \pm 100),$	5 KUVIII ) <b>– 33.</b>	. (\\\\/111			
Calculate propping forces to	top and base	e of wall							

Propping force to top of wall

Tedds		10 Downside Crescent				1411	
RODRIGUES ASSOCIATES	Calcs for				Start page no./F	Revision	
1 AMWELL STREET		new basement b	ack retaining v	wall	5	.2. 5	
LONDON EC1R 1UL Eccalcs by ab	Calcs by ab	Calcs date 12/10/2016	Checked by	Checked date	Approved by	Approved date	
	F <sub>prop_top_f</sub> =	= (M <sub>ot_f</sub> - M <sub>rest_f</sub> + R	<sub>f</sub> × I <sub>base</sub> / 2 - F <sub>pr</sub>	$_{op_f} \times t_{base} / 2) / (h$	I <sub>stem</sub> + t <sub>base</sub> / 2)	= <b>41.608</b> kN/m	
Propping force to base of wall		Fprop_base_f =	= Fprop_f - Fprop_t	op_f = <b>70.070</b> kN/	n		
Design of reinforced concret	e retaining wa	all toe (BS 8002:1	994)				
Material properties							
Characteristic strength of conc	rete	$f_{cu} = 40 \text{ N/m}$	nm²				
Characteristic strength of reinfo	prcement	f <sub>y</sub> = <b>500</b> N/r	mm²				
Base details							
Minimum area of reinforcement	t	k = <b>0.13</b> %					
Cover to reinforcement in toe		c <sub>toe</sub> = <b>45</b> m	m				
Calculate shear for toe desig	n						
Shear from bearing pressure		$V_{toe\_bear} = (p$	D <sub>toe_f</sub> + p <sub>stem_toe_</sub>	$I_{toe} / 2 = 49.5$	kN/m		
Shear from weight of base		V <sub>toe_wt_base</sub> =	= $\gamma_{f_d}  imes \gamma_{base}  imes I_t$	$be \times t_{base} = 8.3 \text{ kN}$	l/m		
Total shear for toe design		$V_{toe} = V_{toe\_b}$	ear - V <sub>toe_wt_base</sub>	= <b>41.3</b> kN/m			
Calculate moment for toe des	sign						
Moment from bearing pressure	!	$M_{toe\_bear} = (2)$	$2 \times p_{toe_f} + p_{ster}$	$_{m_{mid_f}} \times (I_{toe} + t_{wa})$	∥ / 2)² / 6 = <b>20.</b>	<b>9</b> kNm/m	
Moment from weight of base	$M_{toe\_wt\_base} = (\gamma_{t\_d} \times \gamma_{base} \times t_{base} \times (I_{toe} + t_{wall} / 2)^2 / 2) = 3.5 \text{ kNm/m}$						
Ũ			N.4	17 / L/N lune / ma			
Total moment for toe design		M <sub>toe</sub> = M <sub>toe</sub> _	bear - Mtoe_wt_bas	e = 17.4 KINM/M			
Total moment for toe design	-	M <sub>toe</sub> = M <sub>toe</sub>	bear − IVItoe_wt_bas	•	•		
Total moment for toe design	•	M <sub>toe</sub> = M <sub>toe</sub>	∮ear - Mitoe_wt_bas	•	•		
Total moment for toe design	<ul> <li>●</li> <li>100</li> <li>100</li></ul>	M <sub>toe</sub> = M <sub>toe_</sub>	€	•	•		
Total moment for toe design	►   <b>4</b> 200	• • • •	ear - Mitoe_wt_bas	•	•		
Total moment for toe design	<ul> <li>●</li> <li>100</li> <li>100</li></ul>	$M_{toe} = M_{toe}$ $b = 1000 \text{ m}$ $d_{toe} = t_{base} - b_{toe}$	• Im/m - Ctoe - (\$\$toe / 2)	• = <b>447.0</b> mm	•		
Total moment for toe design	►   <b>4</b> 200	$\mathbf{M}_{\text{toe}} = \mathbf{M}_{\text{toe}}$ $\mathbf{b} = 1000 \text{ m}$ $\mathbf{d}_{\text{toe}} = \mathbf{t}_{\text{base}} - \mathbf{K}_{\text{toe}} = \mathbf{M}_{\text{toe}} / \mathbf{K}_{\text{toe}}$	ear - Μίτοe_wt_bas	• = 447.0 mm ) = 0.002	•		
Total moment for toe design	<ul> <li>●</li> <li>1</li> <li>●</li> </ul>	$M_{toe} = M_{toe}$ $b = 1000 \text{ m}$ $d_{toe} = t_{base} - K_{toe} = M_{toe} / K_{toe}$	m/m - $c_{toe} - (\phi_{toe} / 2)$ / $(b \times d_{toe}^2 \times f_{cu})$	• = 447.0 mm ) = 0.002 Compression re	• inforcement is	s not required	
Total moment for toe design	►   <b>4</b> 200	$M_{toe} = M_{toe}$ $b = 1000 \text{ m}$ $d_{toe} = t_{base} - K_{toe} = M_{toe} / Z_{toe} = min(0)$	$\frac{1}{2} \frac{1}{2} \frac{1}$	• = <b>447.0</b> mm ) = <b>0.002</b> <i>Compression re</i> min(K <sub>toe</sub> , 0.225) /	• • inforcement is 0.9)),0.95) × c	<b>s not required</b> հւշe	
Total moment for toe design	<ul> <li>►</li> <li>1</li> <li>200</li> </ul>	$M_{toe} = M_{toe}$ $b = 1000 \text{ m}$ $d_{toe} = t_{base} - K_{toe} = M_{toe} / Z_{toe} = \min(0 Z_{toe} = 425 \text{ m})$	$\int \frac{d}{dt} = \frac{1}{2} \int \frac{dt}{dt} = \frac{1}{2}$	• = 447.0 mm ) = 0.002 Compression re min(K <sub>toe</sub> , 0.225) /	• • inforcement is 0.9)),0.95) × c	<b>s not required</b>	
Total moment for toe design	►   ← 200	$M_{toe} = M_{toe}$ $b = 1000 \text{ m}$ $d_{toe} = t_{base} - K_{toe} = M_{toe} / Z_{toe} = min(0)$ $z_{toe} = 425 \text{ m}$ $A_{s\_toe\_des} = 1$	m/m - $c_{toe} - (\phi_{toe} / 2)$ $(b \times d_{toe}^2 \times f_{cu})$ $(b \times d_{toe}^2 \times f_{cu})$ $(0.5 + \sqrt{0.25} - (m))$ $M_{toe} / (0.87 \times f_{y})$	• = 447.0 mm ) = 0.002 Compression re min(K <sub>toe</sub> , 0.225) / (x × Ztoe) = <b>94</b> mm <sup>2</sup>	• • inforcement is 0.9)),0.95) × c	<b>s not required</b> հեշe	
Total moment for toe design	► • required orcement	$M_{toe} = M_{toe}$ $b = 1000 \text{ m}$ $d_{toe} = t_{base} - K_{toe} = M_{toe} / Z_{toe} = min(0)$ $Z_{toe} = 425 \text{ m}$ $A_{s_toe_des} = 1$ $A_{s_toe_min} = 1$	$\mathbf{f}_{\text{tote}} = \mathbf{M}_{\text{tote}} = \mathbf{M}$	• = 447.0 mm ) = 0.002 Compression re min(K <sub>toe</sub> , 0.225) / x × Z <sub>toe</sub> ) = 94 mm <sup>2</sup> 550 mm <sup>2</sup> /m	• • • • • • • • • • • • • • • • • • •	s not required	
Total moment for toe design	<ul> <li>P</li> <li>P</li></ul>	$M_{toe} = M_{toe}$ $b = 1000 \text{ m}$ $d_{toe} = t_{base} - K_{toe} = M_{toe} / K_{toe$	$m/m$ $- c_{toe} - (\phi_{toe} / 2)$ $(b \times d_{toe}^2 \times f_{cu})$ $(b \times d_{toe}^2 \times f_{cu})$ $(b \times d_{toe}^2 \times f_{cu})$ $(c)$	• = 447.0 mm ) = 0.002 Compression re min(K <sub>toe</sub> , 0.225) / x × z <sub>toe</sub> ) = 94 mm <sup>2</sup> 550 mm <sup>2</sup> /m A <sub>s_toe_min</sub> ) = 650 r	• <i>inforcement is</i> (0.9)),0.95) × c <sup>2</sup> /m nm <sup>2</sup> /m	<b>s not required</b>	
Total moment for toe design	► Performance	$M_{toe} = M_{toe}$ $b = 1000 \text{ m}$ $d_{toe} = t_{base} - K_{toe} = M_{toe} / Z_{toe} = M_{toe} / Z_{toe} = 425 \text{ m}$ $A_{s\_toe\_des} =   A_{s\_toe\_des} =   A_{s\_toe\_req} =   16 \text{ mm dia}$	$m/m$ $- c_{toe} - (\phi_{toe} / 2)$ $(b \times d_{toe}^2 \times f_{cu})$ $(b \times d_{toe}^2 \times f_{cu})$ $(b \times d_{toe}^2 \times f_{cu})$ $(c)$ $(b \times d_{toe} / (0.25 - (i))$ $(c)$	• = 447.0 mm ) = 0.002 Compression remin(K <sub>toe</sub> , 0.225) / × z <sub>toe</sub> ) = 94 mm <sup>2</sup> 550 mm <sup>2</sup> /m A <sub>s_toe_min</sub> ) = 650 m nm centres	• • • • • • • • • • • • • • • • • • •	<b>s not required</b> հ <sub>toe</sub>	

<b>Tekla</b> Tedds	Project	10 Downsi	de Crescent	Job no. 1411				
RODRIGUES ASSOCIATES	Calcs for				Start page no./Re	evision		
1 AMWELL STREET		new basement back retaining wall			5.2	2. 6		
EC1R 1UL	Calcs by ab	Calcs date 12/10/2016	Checked by	Checked date	Approved by	Approved date		
Check shear resistance at to	e							
Design shear stress		$v_{toe} = V_{toe}$ /	$(b \times d_{toe}) = 0.09$	<b>92</b> N/mm²				
Allowable shear stress	Allowable shear stress v <sub>adm</sub> = mir				nm² = <b>5.000</b> N/	/mm²		
		PASS -	Design shear	stress is less th	an maximum	shear stress		
From BS8110:Part 1:1997 – T	able 3.8							
Design concrete shear stress		Vc_toe = <b>0.45</b>	<b>0</b> N/mm²					
			Vto	e < Vc_toe - No she	ear reinforcen	nent required		
Design of reinforced concret	e retaining wa	all stem (BS 8002	:1994)					
Material properties								
Characteristic strength of concl	rete	f <sub>cu</sub> = <b>40</b> N/n	nm²					
Characteristic strength of reinfo	prcement	f <sub>y</sub> = <b>500</b> N/r	nm²					
Wall details								
Minimum area of reinforcement	t	k = <b>0.13</b> %						
Cover to reinforcement in stem		$c_{\text{stem}} = 45 \text{ mm}$						
Cover to reinforcement in wall		c <sub>wall</sub> = <b>45</b> m	m					
Factored horizontal at-rest fo	orces on stem							
Surcharge		$F_{s,sur} f = \gamma_{f}$	$\times$ K <sub>0</sub> $\times$ Surcha	$rae \times (h_{eff} - t_{base} - t_{base})$	d <sub>ds</sub> ) = <b>27.2</b> kN/	m		
Saturated backfill		$F_{n,n} = (f_1 \times K_0 \times Surtharge \times (herr - t_{base} - u_{ds}) = 27.2 \text{ km/m}$ $F_{n,n} = (1.5 \times M_{n,n} \times K_0 \times (M_{n-1} \times h_{bast}) \times h_{cn}t^2 = 33.3 \text{ kN/m}$						
Water		$F_{s water f} = 0$	$.5 \times \gamma_{\rm fe} \times \gamma_{\rm water}$	$\times h_{sat}^2 = 42.9 \text{ kN}/$	′m			
Calculate shear for stem des	ian		1 1					
Surcharge	ign	$V_{a} = 5$	< Fa. aur. f / 8 – 1	7 kN/m				
Saturated backfill		$V_{s_sur_l} = 5$	$(1 s_{sur_{-1}} / 0 - 1)$	(5 × 1) - a) / (20 ×	1 <sup>3</sup> ))) – <b>26 7</b> kl	N/m		
Water		$V_{s}s_{1} = 1 s_{s}$	$V_{s\_s\_f} = F_{s\_s\_f} \times (1 - (a_i^2 \times ((5 \times L) - a_i) / (20 \times L^2))) = 20.7 \text{ KIV/III}$					
Total shear for stem design		$V_{s_water_1} = V_{s_s}$	$v_{s} water_{T} = i_{s} water_{T} \land (i - (a_{1} \land ((J \land L) - a_{1}) / (2U \land L^{-}))) = 34.3 \text{ KIV/III}$ $V_{stem} = V_{s} \sup_{T} f + V_{s} \sup_{S} f + V_{s} water_{T} f = 78 \text{ kN/m}$					
Coloulate memort for stars d	ocian	• stent — • 5_5	<b>.</b>					
	esign	М — Е	( <b>x</b>   / <b>9</b> - <b>0</b>	1 kNm/m				
Saturated backfill		M = E	$sur_{T} \wedge \Box / O = 3$	$= \operatorname{KIN}(20\times 1^2)$	/(60×12) - <b>12</b> 2	kNm/m		
Water		$M_{s_s} = F_{s_s_f} \times a \times ((3 \times a^2) \cdot (15 \times a \times L) + (20 \times L^2))/(60 \times L^2) = 12.2 \text{ kNm/m}$						
kNm/m		$V_{s_water} = \Gamma_{s}$	s_water_t ^a ^(()×	a j-(13^a ×L)+(20	//////////////////////////////////////	- 13.7		
Total moment for stem design		M <sub>stem</sub> = M <sub>s</sub>	sur + Ms s + M∝	<sub>water</sub> = <b>37.3</b> kNm/n	า			
Calculate moment for well de	eian							
Surcharge	Sign	M Q ~	F	28 – <b>5 3</b> kNm/m				
Saturated backfill		$M_{\rm sur} = 9 \times$	$s_{sur_{1}} \sim E / 12$	)-a)/(20~13)_(v.h.)	) <sup>3</sup> /(3×2,2)1 - F	5 kNm/m		
Water		$M = F_{S_s}$		/(5×1)-2.)/(20×13)	$(x_{b})^{3} / (2 \sim 2^{2})$	1 = 7  kNm/m		
Total moment for wall decign		$M_{max} = M$	s_water_t $\land$ [a] $\land$ X	$\frac{178 \text{ kNm/r}}{178 \text{ kNm/r}}$	n (∧-∪i) /(u∧a⊢) n			
rotar moment for wall design		IVIWall = IVIW_s	ur + IVIW_S + IVIW_	water = $17.0$ KINIII/I				

![](_page_30_Figure_0.jpeg)

Tekka Tedds RODRIGUES ASSOCIATES 1 AMWELL STREET LONDON EC1R 1UL	Project Calcs for	Project 10 Downside Crescent Calcs for			Job no. 1411 Start page no./Revision	
	Calcs by ab	Calcs date 12/10/2016	Checked by	Wall Checked date	Approved by	2.8
Reinforcement provided Area of reinforcement provided	PASS	12 mm dia A <sub>s_wall_prov</sub> = S - Reinforcement	.bars @ 200 r 565 mm²/m t provided to	mm centres the retaining wa	ll at mid heigl	nt is adequate
Check retaining wall deflection Basic span/effective depth ration Design service stress Modification factor	factor <sub>tens</sub> = m	ratio <sub>bas</sub> = $2$ f <sub>s</sub> = 2 × f <sub>y</sub> × n(0.55 + (477 N/m	) A <sub>s_stem_req</sub> / (3 um² - f <sub>s</sub> )/(120 ×	$\times A_{s\_stem\_prov}) = 22$ $< (0.9 \text{ N/mm}^2 + (\text{N}))$	2 <b>9.9</b> N/mm² I <sub>stem</sub> /(b × d <sub>stem</sub> ²)	))),2) = <b>1.92</b>
Maximum span/effective depth	ratio	ratio <sub>max</sub> = ra	$ratio_{max} = ratio_{bas} \times factor_{tens} = 38.42$			

Actual span/effective depth ratio

ratio<sub>act</sub> =  $h_{stem} / d_{stem} = 10.04$ 

PASS - Span to depth ratio is acceptable

<b>Tekla</b> Tedds	Project 10 Downside Crescent			Job no. 1411		
RODRIGUES ASSOCIATES 1 AMWELL STREET	Calcs for r	new basement b	ack retaining wa	.11	Start page no./Re 5.2	vision 2. 9
ECIR 1UL	Calcs by ab	Calcs date 12/10/2016	Checked by	Checked date	Approved by	Approved date

![](_page_32_Figure_1.jpeg)

Toe bars - 16 mm dia.@ 200 mm centres -  $(1005 \text{ mm}^2/\text{m})$ Wall bars - 12 mm dia.@ 200 mm centres -  $(565 \text{ mm}^2/\text{m})$ Stem bars - 12 mm dia.@ 200 mm centres -  $(565 \text{ mm}^2/\text{m})$ 

	-					
Tekla <sup>®</sup>	Project				Job no.	
Tedds	10 Downside Crescent				1411	
RODRIGUES ASSOCIATES	Calcs for			Start page no./Revision		
1 AMWELL STREET		new basem	ent side wall		5.3	3. 1
EC1R 1UL	Calcs by ab	Calcs date 12/10/2016	Checked by	Checked date	Approved by	Approved date

TEDDS calculation version 1.2.01.06

#### RETAINING WALL ANALYSIS (BS 8002:1994)

![](_page_33_Figure_2.jpeg)

#### Wall details

Retaining wall type Height of retaining wall stem Thickness of wall stem Length of toe Length of heel Overall length of base Thickness of base Depth of downstand Position of downstand Thickness of downstand Height of retaining wall Depth of cover in front of wall Depth of unplanned excavation Height of ground water behind wall Height of saturated fill above base Density of wall construction Density of base construction Angle of rear face of wall Angle of soil surface behind wall Effective height at virtual back of wall

### **Retained material details**

Mobilisation factor Moist density of retained material

Cantilever propped at both h<sub>stem</sub> = **2500** mm twall = 300 mm  $I_{toe} = 500 \text{ mm}$  $I_{heel} = \mathbf{0} mm$  $I_{\text{base}} = I_{\text{toe}} + I_{\text{heel}} + t_{\text{wall}} = 800 \text{ mm}$ t<sub>base</sub> = **500** mm  $d_{ds} = 0 \text{ mm}$ lds = **300** mm t<sub>ds</sub> = **500** mm  $h_{wall} = h_{stem} + t_{base} + d_{ds} = 3000 \text{ mm}$  $d_{cover} = 0 mm$  $d_{exc} = 0 mm$ h<sub>water</sub> = **3000** mm  $h_{sat} = max(h_{water} - t_{base} - d_{ds}, 0 mm) = 2500 mm$  $\gamma_{wall} = 23.6 \text{ kN/m}^3$ γ<sub>base</sub> = 23.6 kN/m<sup>3</sup>  $\alpha = 90.0 \text{ deg}$  $\beta = 0.0 \text{ deg}$  $h_{eff} = h_{wall} + I_{heel} \times tan(\beta) = 3000 \text{ mm}$ 

M = **1.5** γ<sub>m</sub> = **18.0** kN/m<sup>3</sup>

<b>F</b> Tekla	Project Job no.					
Tedds		10 Downsie	de Crescent		14	411
RODRIGUES ASSOCIATES	Calcs for				Start page no./Re	evision
LONDON		new basem	ent side wall		5.	3. 2
EC1R 1UL	Calcs by	Calcs date 12/10/2016	Checked by	Checked date	Approved by	Approved date
	40	12/10/2010				
Saturated density of retained m	naterial	γ <sub>s</sub> = <b>21.0</b> kM	N/m <sup>3</sup>			
Design shear strength		¢' = <b>18.6</b> d€	eg			
Angle of wall friction		$\delta = 0.0 \text{ deg}$				
Base material details						
Firm clay						
Moist density		γ <sub>mb</sub> = <b>18.0</b> k	⟨N/m³			
Design shear strength		¢'₅ = <b>16.5</b> d	eg			
Design base friction		$\delta_{b} = $ <b>18.6</b> de	eg			
Allowable bearing pressure		P <sub>bearing</sub> = 10	<b>0</b> kN/m <sup>2</sup>			
Using Coulomb theory						
Active pressure coefficient for r	retained material					
K <sub>a</sub> = sin(o	$(\alpha + \phi')^2 / (\sin(\alpha)^2 \times$	$\sin(\alpha - \delta) \times [1 +$	$\sqrt{(\sin(\phi' + \delta) \times s)}$	sin(φ' - β) / (sin(α	$-\delta$ ) × sin( $\alpha$ +	β)))]²) = <b>0.516</b>
Passive pressure coefficient fo	r base material	· · · -			, , , , ,	
	$K_p = sin(9)$	0 - φ' <sub>b</sub> )² / (sin(90	$(-\delta_b)  imes [1 - \sqrt{(sir)}]$	$n(\phi'_{b} + \delta_{b}) \times sin(\phi')$	b) / (sin(90 + δ	(b)))] <sup>2</sup> ) = <b>2.835</b>
At-rest pressure						
At-rest pressure for retained m	aterial	K <sub>0</sub> = 1 – sir	n(φ') = <b>0.681</b>			
l oading details						
Surcharge load on plan		Surcharge	= <b>10.0</b> kN/m <sup>2</sup>			
Applied vertical dead load on w	vall	$W_{dead} = 10.$	<b>3</b> kN/m			
Applied vertical live load on wa	.11	W <sub>live</sub> = <b>0.0</b>	<n m<="" th=""><th></th><th></th><th></th></n>			
Position of applied vertical load	d on wall	l <sub>load</sub> = <b>650</b> n	nm			
Applied horizontal dead load or	n wall	F <sub>dead</sub> = <b>0.0</b>	kN/m			
Applied horizontal live load on	wall	F <sub>live</sub> = <b>0.0</b> k	N/m			
Height of applied horizontal loa	id on wall	$h_{load} = 0 mr$	n			
		10 ↓ [[]]]	10			
		Prop		l		
	A TININ					
	Prop -					
<u> </u>						
<u> </u>	24.2 46.8	46.8	5.2 0703	29.4		
				Loads shown	i in kN/m, pressure	es shown in kN/m <sup>2</sup>

<b>Tekla</b> Tedds	Project	10 Downsid	Job no. 1411			
RODRIGUES ASSOCIATES 1 AMWELL STREET	Calcs for	new basem	ent side wall		Start page no./Re 5.3	evision 3. 3
EC1R 1UL	Calcs by ab	Calcs date 12/10/2016	Checked by	Checked date	Approved by	Approved date

Vertical forces on wall	
Wall stem	$w_{wali} = h_{stem} \times t_{wali} \times \gamma_{wali} = 17.7 \ kN/m$
Wall base	$w_{\text{base}} = I_{\text{base}} \times t_{\text{base}} \times \gamma_{\text{base}} = \textbf{9.4} \text{ kN/m}$
Applied vertical load	$W_v = W_{dead} + W_{live} = 10.3 \text{ kN/m}$
Total vertical load	$W_{total} = w_{wall} + w_{base} + W_v = 37.4 \text{ kN/m}$
Horizontal forces on wall	
Surcharge	$F_{sur} = K_a \times Surcharge \times h_{eff} = 15.5 \text{ kN/m}$
Saturated backfill	$F_s = 0.5 \times K_a \times (\gamma_{s} - \gamma_{water}) \times h_{water}^2 = 26 \text{ kN/m}$
Water	$F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water} = 44.1 \text{ kN/m}$
Total horizontal load	$F_{total} = F_{sur} + F_s + F_{water} = 85.6 \text{ kN/m}$
Calculate total propping force	
Passive resistance of soil in front of wall	$F_{p} = 0.5 \times K_{p} \times cos(\delta_{b}) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^{2} \times \gamma_{mb} = \textbf{6} \text{ kN/m}$
Propping force	$F_{prop} = max(F_{total} - F_p - (W_{total}) \times tan(\delta_b), 0 \text{ kN/m})$
	F <sub>prop</sub> = <b>67.0</b> kN/m
Overturning moments	
Surcharge	$M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2 = 23.2 \text{ kNm/m}$
Saturated backfill	$M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3 = 26 \text{ kNm/m}$
Water	$M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3 = 44.1 \text{ kNm/m}$
Total overturning moment	$M_{ot} = M_{sur} + M_s + M_{water} = 93.4 \text{ kNm/m}$
Restoring moments	
Wall stem	$M_{wall} = w_{wall} \times (I_{toe} + t_{wall} / 2) = 11.5 \text{ kNm/m}$
Wall base	$M_{base} = w_{base} \times I_{base} / 2 = 3.8 \text{ kNm/m}$
Design vertical dead load	$M_{dead} = W_{dead} \times I_{load} = 6.7 \text{ kNm/m}$
Total restoring moment	$M_{rest} = M_{wall} + M_{base} + M_{dead} = 22 \text{ kNm/m}$
Check bearing pressure	
Total vertical reaction	R = W <sub>total</sub> = <b>37.4</b> kN/m
Distance to reaction	x <sub>bar</sub> = I <sub>base</sub> / 2 = <b>400</b> mm
Eccentricity of reaction	$e = abs((I_{base} / 2) - x_{bar}) = 0 mm$
	Reaction acts within middle third of base
Bearing pressure at toe	$p_{toe} = (R / I_{base}) - (6 \times R \times e / I_{base}^2) = 46.8 \text{ kN/m}^2$
Bearing pressure at heel	$p_{heel} = (R / I_{base}) + (6 \times R \times e / I_{base}^2) = 46.8 \text{ kN/m}^2$
PA	ASS - Maximum bearing pressure is less than allowable bearing pressure
Calculate propping forces to top and base	of wall
Propping force to top of wall	

 $F_{prop\_top} = (M_{ot} - M_{rest} + R \times I_{base} / 2 - F_{prop} \times t_{base} / 2) / (h_{stem} + t_{base} / 2) = 25.324 \text{ kN/m}$ Propping force to base of wall  $F_{prop\_base} = F_{prop} - F_{prop\_top} = 41.679 \text{ kN/m}$ 

<b>Tekla</b>	Project	10 Downsi	de Crescent		Job no. 14	411
RODRIGUES ASSOCIATES	Calcs for				Start page no./R	evision
1 AMWELL STREET		new basem	ent side wall		5.	3. 4
LONDON EC1R 1UL	Calcs by ab	Calcs date 12/10/2016	Checked by	Checked date	Approved by	Approved date
RETAINING WALL DESIGN (	BS 8002:1994)	1				
、		-			TEDDS calculation	version 1.2.01.06
Ultimate limit state load facto	ors					
Dead load factor		$\gamma_{f\_d} = 1.4$				
Live load factor		$\gamma_{f\_l} = 1.6$				
Earth and water pressure facto	or	$\gamma_{f_e} = 1.4$				
Factored vertical forces on v	vall					
Wall stem		$w_{wall\_f} = \gamma_{f\_d}$	$ imes$ h <sub>stem</sub> $ imes$ t <sub>wall</sub> $ imes$ $\gamma$	wall = <b>24.8</b> kN/m	ı	
Wall base		$W_{base_f} = \gamma_{f_c}$	$1 \times I_{base} \times t_{base} \times t_{base}$	γ <sub>base</sub> = <b>13.2</b> kN/	m	
Applied vertical load		$W_{v_f} = \gamma_{f_d} >$	$\times W_{dead} + \gamma_{f_l} \times V$	V <sub>live</sub> = <b>14.4</b> kN/m	ı	
Total vertical load		$W_{total_f} = w_w$	$_{all_f} + w_{base_f} + W$	/ <sub>v_f</sub> = <b>52.4</b> kN/m		
Factored horizontal at-rest for	orces on wall					
Surcharge		$F_{sur_f} = \gamma_{f_l} \times$	$K_0 \times Surcharge$	e × h <sub>eff</sub> = <b>32.7</b> kN	N/m	
Saturated backfill		$F_{s_f} = \gamma_{f_e} \times$	$0.5 imes K_0 imes (\gamma_{s}-\gamma_{s})$	$(w_{ater}) \times h_{water}^2 = 4$	<b>I8</b> kN/m	
Water		$F_{water_f} = \gamma_{f_e}$	$0.5 \times h_{water}^2$	<γ <sub>water</sub> = <b>61.8</b> ki	N/m	
Total horizontal load		F <sub>total_f</sub> = F <sub>sur</sub>	_f + Fs_f + Fwater_	<sub>f</sub> = <b>142.5</b> kN/m		
Calculate total propping for	e					
Passive resistance of soil in fro	ont of wall	$F_{p f} = \gamma_{f e} \times$	$0.5 \times K_p \times \cos(6)$	$\delta_{b}) \times (d_{cover} + t_{bas})$	se + d <sub>ds</sub> - d <sub>exc</sub> ) <sup>2</sup> >	< γ <sub>mb</sub> = <b>8.5</b>
kN/m		•_ •_		, (	,	
Propping force		$F_{prop_f} = ma$	x(F <sub>total_f</sub> - F <sub>p_f</sub> - (	$W_{total_f}$ × tan( $\delta_b$ )	), 0 kN/m)	
		F <sub>prop_f</sub> = <b>116</b>	<b>5.4</b> kN/m			
Factored overturning mome	nts					
Surcharge		M <sub>sur f</sub> = F <sub>sur</sub>	$_{\rm f} \times ({\rm h}_{\rm eff} - 2 \times {\rm d}_{\rm c})$	<sub>is</sub> ) / 2 = <b>49</b> kNm/	/m	
Saturated backfill		$M_{s f} = F_{s f} \times$	(h <sub>water</sub> - 3 × d <sub>ds</sub>	) / 3 = <b>48</b> kNm/n	n	
Water		M <sub>water</sub> f = F <sub>v</sub>	vater f × (hwater - 3	× d <sub>ds</sub> ) / 3 = <b>61.8</b>	<b>3</b> kNm/m	
Total overturning moment		M <sub>ot_f</sub> = M <sub>sur_</sub>	_f + Ms_f + Mwater_	_f = <b>158.8</b> kNm/r	n	
Restoring moments						
Wall stem		Mwall f = Wwa	$   f \times ( _{top} + t_{wall})$	2) = <b>16.1</b> kNm/r	n	
Wall base		$M_{\text{base f}} = W_{\text{b}}$	ase f × base / 2 =	5.3 kNm/m		
Design vertical load		$M_{\rm V} f = W_{\rm V} f$	× Iload = <b>9.3</b> kNn	n/m		
Total restoring moment		M <sub>rest</sub> f = M <sub>wa</sub>	all $f + M_{base} f + M$	<sub>v f</sub> = <b>30.7</b> kNm/r	n	
Factored bearing pressure				_		
Total vertical reaction		Rf = Wtotal f	= <b>52</b> .4 kN/m			
Distance to reaction		$x_{\text{bar } f} = I_{\text{base}}$	/ 2 = <b>400</b> mm			
Eccentricity of reaction		$e_{f} = abs((I_{ba}))$	use / 2) - Xbar_f) =	<b>0</b> mm		
			, _,	Reaction acts	within middle	third of base
Bearing pressure at toe		$p_{toe_f} = (R_f / $	$I_{base}$ ) - (6 $ imes$ R <sub>f</sub> $ imes$	$e_f / I_{base}^2) = 65.5$	5 kN/m²	
Bearing pressure at heel		$p_{\text{heel}_f} = (R_f$	/ $I_{base}$ ) + (6 $\times$ R <sub>f</sub>	$\times e_{\rm f} / I_{\rm base}^2) = 65$	5.5 kN/m²	
Rate of change of base reaction	on	$rate = (p_{toe_{-}})$	f - p <sub>heel_f</sub> ) / I <sub>base</sub> =	= <b>0.00</b> kN/m²/m		
Bearing pressure at stem / toe		pstem_toe_f =	max(p <sub>toe_f</sub> - (rate	$e \times I_{toe}$ ), 0 kN/m <sup>2</sup>	) = <b>65.5</b> kN/m <sup>2</sup>	
Bearing pressure at mid stem		pstem_mid_f =	max(p <sub>toe_f</sub> - (rate	$e \times (I_{toe} + t_{wall} / 2)$	)), 0 kN/m²) = <b>6</b>	<b>5.5</b> kN/m²
Bearing pressure at stem / hee	el	pstem_heel_f =	max(p <sub>toe_f</sub> - (rat	$\mathbf{e} \times (\mathbf{I}_{\text{toe}} + \mathbf{t}_{\text{wall}})),$	0 kN/m²) = <b>65.</b>	5 kN/m²
Calculate propping forces to	top and base	of wall				

Propping force to top of wall

	Project				Job no. 1411	
RODRIGUES ASSOCIATES	Calcs for				Start page no./F	Revision
1 AMWELL STREET		new basem	nent side wall		5	.3. 5
LONDON EC1R 1UL Ab	Calcs by ab	Calcs date 12/10/2016	Checked by	Checked date	Approved by	Approved date
	F <sub>prop_top_f</sub> =	= (M <sub>ot_f</sub> - M <sub>rest_f</sub> + R	$_{\rm f}  imes {\sf I}_{\rm base}$ / 2 - ${\sf F}_{\rm pr}$	$_{rop_f} \times t_{base} / 2) / (h$	I <sub>stem</sub> + t <sub>base</sub> / 2)	= <b>43.621</b> kN/m
Propping force to base of wall		F <sub>prop_base_f</sub> =	= F <sub>prop_f</sub> - F <sub>prop_t</sub>	<sub>op_f</sub> = <b>72.797</b> kN/i	m	
Design of reinforced concret	e retaining wa	all toe (BS 8002:1	994)			
Material properties						
Characteristic strength of conc	rete	f <sub>cu</sub> = <b>40</b> N/r	nm²			
Characteristic strength of reinfo	orcement	$f_y = 500 \text{ N/r}$	mm²			
Base details						
Minimum area of reinforcemen	t	k = <b>0.13</b> %				
Cover to reinforcement in toe		c <sub>toe</sub> = <b>45</b> m	m			
Calculate shear for toe desig	n					
Shear from bearing pressure		$V_{toe\_bear} = ($	p <sub>toe_f</sub> + p <sub>stem_toe</sub>	_f) × I <sub>toe</sub> / 2 = <b>32.7</b>	′ kN/m	
Shear from weight of base		V <sub>toe_wt_base</sub> =	= $\gamma_{f_d}  imes \gamma_{base}  imes I_f$	$_{oe} \times t_{base} = 8.3 \text{ kN}$	l/m	
Total shear for toe design		$V_{toe} = V_{toe\_b}$	pear - V <sub>toe_wt_base</sub>	= <b>24.5</b> kN/m		
Calculate moment for toe de	sign					
Moment from bearing pressure	)	$M_{toe\_bear} = ($	$2 \times p_{toe_f} + p_{ster}$	m_mid_f) $ imes$ (Itoe + twa	" / 2)² / 6 = <b>13.</b>	<b>8</b> kNm/m
	$M_{toe\_wt\_base} = (\gamma_{t\_d} \times \gamma_{base} \times t_{base} \times (I_{toe} + t_{wall} / 2)^2 / 2) = \textbf{3.5 kNm/m}$					
Moment from weight of base		IVItoe_wt_base =	()	-babb (100 - 11a)		
Moment from weight of base Total moment for toe design		M <sub>toe</sub> = M <sub>toe</sub>	<sub>bear</sub> - M <sub>toe_wt_bas</sub>	<sub>e</sub> = <b>10.3</b> kNm/m		
Moment from weight of base Total moment for toe design	•	Mitoe_wt_base : Mitoe = Mitoe_	bear - Mtoe_wt_bas	e = 10.3 kNm/m	•	
Moment from weight of base Total moment for toe design	►   <b>4</b> 200	Mitoe_wt_base : Mitoe = Mitoe_ •	bear - Mtoe_wt_bas	•	•	
Moment from weight of base Total moment for toe design	●	• • • •	• mm/m	•	•	
Moment from weight of base Total moment for toe design	►   <b>4</b> 200	■ Mtoe_wt_base = Mtoe = Mtoe_ ■ b = 1000 m dtoe = thase =	e 1m/m - Ctoe - (Φτοe / 2)	• • •	•	
Moment from weight of base Total moment for toe design	<ul> <li>●</li> <li>1</li> <li>1</li> <li>2</li> <li>2</li> </ul>	b = 1000 m dtoe = tbase - Ktoe = Mtoe	• im/m - Ctoe - ( $\phi$ toe / 2) / (b × dtoe <sup>2</sup> × fcu	• • • • •	•	
Moment from weight of base Total moment for toe design	► •  •—200	■ Mitoe_wt_base = Mitoe = Mitoe_ b = 1000 m ditoe = tbase - Kitoe = Mitoe /	nm/m - Ctoe - (\phote / 2) / (b × dtoe <sup>2</sup> × fcu	• • = <b>10.3</b> kNm/m • • • • • • • • • • •	• inforcement is	s not required
Moment from weight of base Total moment for toe design	<ul> <li>●</li> <li>1</li> <li>1</li> <li>2</li> <li>2</li> <li>0</li> </ul>	$b = 1000 \text{ m}$ $b = 1000 \text{ m}$ $d_{\text{toe}} = t_{\text{base}} - K_{\text{toe}} = M_{\text{toe}} / Z_{\text{toe}} = \min(0 - 2t_{\text{toe}}) - 2t_{\text{toe}} = 425 \text{ m}$	$\bullet$ $m/m = -c_{toe} - (\phi_{toe} / 2)$ $(b \times d_{toe}^2 \times f_{cu})$ $0.5 + \sqrt{0.25 - (mm)}$	• • • • • • • • • • • • • • • • • • •	• • • • • • • • • • • • •	s not required
Moment from weight of base Total moment for toe design	► • P 200	b = 1000 m dtoe = base − Ktoe = Mtoe/ Ztoe = min(C ztoe = 425 m As the des =	$m/m = c_{toe} - (\phi_{toe} / 2)$ $(b \times d_{toe}^2 \times f_{cu})$	• • • • • • • • • • • • • • • • • • •	• • • inforcement is (0.9)),0.95) × c	s not required
Moment from weight of base Total moment for toe design	● Performance	b = 1000 m b = 1000 m dtoe = tbase - Ktoe = Mtoe / Ztoe = min(C Ztoe = 425 m As_toe_des = As toe min =	$\mathbf{M}_{\text{toe}} = \mathbf{M}_{\text{toe}_{\text{wt}_{\text{base}}}}$	• • • • • • • • • • • • • • • • • • •	• • • • • • • • • • • • • • • • • • •	s not required
Moment from weight of base Total moment for toe design	Performance of the second	$b = 1000 \text{ m}$ $d_{toe} = b_{ase} = K_{toe}$ $k_{toe} = b_{ase} = K_{toe} = M_{toe} / K_{toe} = M_{toe}$	$m/m = c_{toe} - (\phi_{toe} / 2)$ $(b \times d_{toe}^2 \times f_{cu})$ $(b \times d_{toe}^2 \times f_{cu})$ $(b \times d_{toe}^2 \times f_{cu})$ $(c) = (c_{toe} - (\phi_{toe} / 2))$ $(c) = (c_{toe} - (\phi_{toe} /$	• • = <b>10.3</b> kNm/m • = <b>447.0</b> mm ) = <b>0.001</b> <i>Compression re</i> min(K <sub>toe</sub> , 0.225) / / × z <sub>toe</sub> ) = <b>56</b> mm <sup>2</sup> S <b>50</b> mm <sup>2</sup> /m A <sub>s toe min</sub> ) = <b>650</b> n	• <i>inforcement is</i> <sup>2</sup> /m nm <sup>2</sup> /m	s not required
Moment from weight of base Total moment for toe design	<ul> <li></li> <li><!--</td--><td>b = 1000 m dtoe = tbase - Ktoe = Mtoe/ Ztoe = min(C Ztoe = 425 m As_toe_des = As_toe_req = 1 16 mm dia</td><td><math display="block">m/m</math> <math display="block">- Ctoe - (\phi_{toe} / 2)</math> <math display="block">/ (b \times d_{toe}^2 \times f_{cu})</math> <math display="block">0.5 + \sqrt{(0.25 - (mm))}</math> <math display="block">M_{toe} / (0.87 \times f_{su})</math> <math display="block">M_{toe} / (0.87 \times f_{su})</math></td><td>• • = <b>10.3</b> kNm/m • = <b>10.3</b> kNm/m • • • • • • • • • • • • •</td><td>• • • • • • • • • • • • • • • • • • •</td><td><b>s not requirea</b></td></li></ul>	b = 1000 m dtoe = tbase - Ktoe = Mtoe/ Ztoe = min(C Ztoe = 425 m As_toe_des = As_toe_req = 1 16 mm dia	$m/m$ $- Ctoe - (\phi_{toe} / 2)$ $/ (b \times d_{toe}^2 \times f_{cu})$ $0.5 + \sqrt{(0.25 - (mm))}$ $M_{toe} / (0.87 \times f_{su})$	• • = <b>10.3</b> kNm/m • = <b>10.3</b> kNm/m • • • • • • • • • • • • •	• • • • • • • • • • • • • • • • • • •	<b>s not requirea</b>
Moment from weight of base Total moment for toe design	● ● Performance Performanc	$b = 1000 \text{ m}$ $b = 1000 \text{ m}$ $d_{toe} = t_{base} - K_{toe} = M_{toe}/2$ $z_{toe} = min(02)$ $z_{toe} = 425 \text{ m}$ $A_{s_toe_des} = 1$ $A_{s_toe_req} = 1$ $16 \text{ mm dia}$ $A_{s_toe_prov} = 1$	$\bullet$ $nm/m$ $- c_{toe} - (\phi_{toe} / 2)$ $/ (b \times d_{toe}^2 \times f_{cu})$ $0.5 + \sqrt{(0.25 - (mmm))}$ $M_{toe} / (0.87 \times f_{su})$ $k \times b \times t_{base} = (mmm)$ $M_{toe} / (0.87 \times f_{su})$ $h_{toas} @ 200 m$ $1005 mm^2/m$	• • = <b>10.3</b> kNm/m • = <b>10.3</b> kNm/m • • • • • • • • • • • • •	• <i>inforcement is</i> (0.9)),0.95) × c <sup>2</sup> /m nm <sup>2</sup> /m	s not required

	Project	10 Downsi	de Crescent	Job no. 1	411			
RODRIGUES ASSOCIATES 1 AMWELL STREET	Calcs for	new basem	Start page no./F	Revision .3. 6				
LONDON EC1R 1UL	Calcs by ab	Calcs date 12/10/2016	Checked by	Checked date	Approved by	Approved date		
Check shear resistance at to	e				- <b>!</b>			
Design shear stress		$v_{toe} = V_{toe} /$	$(b \times d_{toe}) = 0.0$	<b>155</b> N/mm²				
Allowable shear stress		v <sub>adm</sub> = min(	0.8 × √(f <sub>cu</sub> / 1 ľ	$N/mm^2$ ), 5) × 1 N/	/mm <sup>2</sup> = <b>5.000</b> N	N/mm²		
		PASS -	Design shea	r stress is less t	han maximun	n shear stress		
From BS8110:Part 1:1997	Table 3.8							
Design concrete shear stress		Vc_toe = <b>0.4</b>	50 N/mm²					
			Vto	oe < Vc_toe - No sł	near reinforce	ment required		
Design of reinforced concre	te retaining wa	all stem (BS 8002	:1994)					
Material properties		•	<u> </u>					
Characteristic strength of cond	rete	f <sub>cu</sub> = <b>40</b> N/r	nm²					
Characteristic strength of reinf	orcement	$f_{\rm V} = 500 \ {\rm N}/{\rm I}$	mm²					
Wall details		,						
Minimum area of reinforcemer	ıt	k = 0.13 %						
Cover to reinforcement in sten	1	c <sub>stem</sub> = <b>45</b> mm						
Cover to reinforcement in wall		c <sub>wall</sub> = <b>45</b> m	m					
Factored horizontal at-rest f	orces on stem	l						
Surcharge		$F_{s,sur,f} = \gamma_{f}$	$\times$ K <sub>0</sub> $\times$ Surcha	arge $\times$ (h <sub>eff</sub> - t <sub>base</sub> ·	- d <sub>ds</sub> ) = <b>27.2</b> kN	l/m		
Saturated backfill		$F_{s,s,f} = 0.5$	$F_{s,sur_{t}} = \gamma_{t} \times \chi_{0} \times Surcharge \times (herr - base - dds) = 27.2 \text{ kN/m}$ $F_{s,s,t} = 0.5 \times \gamma_{t,s} \times K_{0} \times (\gamma_{t-1} - \gamma_{waster}) \times h_{sar}^{2} = 33.3 \text{ kN/m}$					
Water		$F_{s water f} = 0$	$0.5 \times \gamma_{\rm fe} \times \gamma_{\rm wate}$	r × h <sub>sat</sub> ² = <b>42.9</b> kN	√m			
Calculate chear for stom day	ian		- 1 <u>-</u> - 1					
Surcharge	ngn	$V_{a} = 5$	× Falaur # / 8 – 1	1 <b>7</b> kN/m				
Saturated backfill		$V_{s,sur} = F_{s,r}$	$x + x = (1 - (a)^2 + (1 - (a)^2)^2)$	((5 × 1 ) - a) / (20	×   <sup>3</sup> ))) – <b>26 7</b>	(N/m		
Water		$V_{s} = 1 = 1 = 1$	$= \frac{1}{1} \times \frac{1}{1} = \frac{1}{1}$	$(3 \times 1)^{2} \times ((5 \times 1)^{2} + a_{1})^{2}$	$(20 \times 1^{3})) -$	<b>34 3</b> kN/m		
Total shear for stem design		$V_{stem} = V_{stem}$	$s_{walel_1} \wedge (1)$	$(0 \times 2)^{-1}$	) (20 × E ))) =	<b>04.0</b> KN/III		
Calculate memort for storm	locian							
Surcharge	lesign	M F-		4 kNm/m				
Saturated backfill		M <sub>s</sub> <sub>s</sub> <sub>s</sub> <sub>s</sub> <sub>s</sub>	$\sup_{1 \to \infty}   (3 \times 2)^2 - (3 \times$	15∨avl )⊥(20vl ²)	)/(60×l <sup>2</sup> ) – <b>12</b>	<b>2</b> kNm/m		
Water		Ms_s = 1 s_s_	i ∧ai∧((0∧ai )-(	$(2^2) - (15^2) + (20^2) + (2$	20~L 2))/(60~L 2)	- 15 7		
kNm/m		TVIs_water - T	s_water_t ~al~((0/	(13~a ^E)+(2	.0~L ))/(00~L )	- 13.7		
Total moment for stem design		M <sub>stem</sub> = M <sub>s</sub>	sur + Ms_s + Ms	water = <b>37.3</b> kNm/	'n			
Calculate moment for wall d	esian	oto / 105_						
Surcharge	Sorgin	M., our = 9 >	(Fegur f × I / 1	28 = <b>5.3</b> kNm/m				
Saturated backfill		M	$f \propto [a_2^2 \times x \times ((5 \times 1)^2)]$	-3 = 0.0  km/m	) <sup>3</sup> /(3×a²)1 – 5	5 kNm/m		
Water		Mw_s = 1 s_s	$a_1 \sim [\alpha_1 \sim \alpha_1 (0)]$	-, a,, (_0∧_ ) (∧∩ ×((5×  )-a,)/(20×	$^{3}$ )-(x-b) <sup>3</sup> /(3×a <sup>2</sup> )	(2)] = 7  kNm/m		
Total moment for wall design		$M_{\text{water}} = M_{\text{water}}$	$s_{water_{1}} \wedge [c_{1} \wedge A]$	water = 17.8  kNm	/m	/] — F KINII/III		
retar memorit for war dobigh			,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,					

![](_page_39_Figure_0.jpeg)

<b>M</b> Teka	Project		Job no.	Job no.			
Tedds	10 Downside Crescent				1411		
RODRIGUES ASSOCIATES	Calcs for		Start page no./Re	Start page no./Revision			
1 AMWELL STREET		new basement side wall				5.3. 8	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date	
EGIRIOL	ab	12/10/2016					
	•				•		
Reinforcement provided		12 mm dia	.bars @ 200 ı	mm centres			
Area of reinforcement provided		$A_{s\_wall\_prov} =$	<b>565</b> mm²/m				
	PASS	- Reinforcement	t provided to	the retaining wa	ll at mid heigh	t is adequate	
Check retaining wall deflectio	n						
Basic span/effective depth ratio		ratio <sub>bas</sub> = 2	D				
Design service stress		$f_s = 2 \times f_y \times$	As_stem_req / (3	$\times A_{s\_stem\_prov}) = 22$	<b>9.9</b> N/mm²		

Design service stress Modification factor

 $factor_{tens} = min(0.55 + (477 \text{ N/mm}^2 - f_s)/(120 \times (0.9 \text{ N/mm}^2 + (\text{M}_{stem}/(b \times d_{stem}^2)))),2) = 1.92$ 

Maximum span/effective depth ratio

Actual span/effective depth ratio

 $ratio_{max} = ratio_{bas} \times factor_{tens} = 38.42$  $ratio_{act} = h_{stem} / d_{stem} = 10.04$ 

PASS - Span to depth ratio is acceptable

<b>Tekla</b> Tedds	Project	Job no. 1411				
RODRIGUES ASSOCIATES 1 AMWELL STREET	Calcs for new basement side wall				Start page no./Revision 5.3. 9	
EC1R 1UL	Calcs by ab	Calcs date 12/10/2016	Checked by	Checked date	Approved by	Approved date

![](_page_41_Figure_1.jpeg)

Toe bars - 16 mm dia.@ 200 mm centres -  $(1005 \text{ mm}^2/\text{m})$ Wall bars - 12 mm dia.@ 200 mm centres -  $(565 \text{ mm}^2/\text{m})$ Stem bars - 12 mm dia.@ 200 mm centres -  $(565 \text{ mm}^2/\text{m})$ 

![](_page_42_Figure_0.jpeg)

![](_page_43_Figure_0.jpeg)

<b>Tekla</b> Tedds	Project	10 Downsi	de Crescent		Job no. 1	411
RODRIGUES ASSOCIATES 1 AMWELL STREET	Calcs for	side retaining	wall top bean	n	Start page no./F	Revision 4B. 2
LONDON EC1R 1UL	Calcs by ab	Calcs date 11/10/2016	Checked by	Checked date	Approved by	Approved date
Maximum moment support B Maximum shear support A Maximum shear support A span Maximum shear support B Maximum shear support B span Maximum reaction at support A Unfactored dead load reaction Maximum reaction at support B Unfactored dead load reaction	n 1 at 447 mm n 1 at 5253 mm at support A at support B	$\label{eq:max} \begin{array}{l} M_{B\_max} = 0 \\ V_{A\_max} = 11 \\ V_{A\_s1\_max} = \\ V_{B\_max} = -1 \\ V_{B\_s1\_max} = \\ R_{A} = 111 \ k \\ R_{A\_Dead} = 1 \\ R_{B} = 111 \ k \\ R_{B\_Dead} = 1 \end{array}$	kNm 1 kN 94 kN 11 kN -94 kN N 11 kN N	M <sub>B_red</sub> V <sub>A_red</sub> = V <sub>A_s1_re</sub> V <sub>B_red</sub> = V <sub>B_s1_re</sub>	= 0 kNm = 111 kN ed = 94 kN = -111 kN ed = -94 kN	
Rectangular section details Section width Section depth		b = 300 mr h = 500 mr	n n 0			
Concrete details Concrete strength class Characteristic compressive cub Modulus of elasticity of concret Maximum aggregate size	e strength	<b>C32/40</b> f <sub>cu</sub> = <b>40</b> N/r E <sub>c</sub> = 20kN/ h <sub>agg</sub> = <b>20</b> m	nm² mm² + 200 × f ım	<sub>cu</sub> = <b>28000</b> N/mm	2	
<b>Reinforcement details</b> Characteristic yield strength of Characteristic yield strength of	reinforcement shear reinforcen	f <sub>y</sub> = <b>500</b> N/ nent f <sub>yy</sub> = <b>500</b> N	mm² /mm²			
Nominal cover to reinforcement Nominal cover to top reinforcement Nominal cover to bottom reinforcement Nominal cover to side reinforcement	ent nent rcement ment	Cnom_t = <b>35</b> Cnom_b = <b>35</b> Cnom_s = <b>35</b>	mm mm			
<u>Support A</u>		3 2 3 3 -300→	x 10 <sub>φ</sub> bars x 8 <sub>φ</sub> shear legs a x 20 <sub>φ</sub> bars	t 200 c/c		

🗾 Tekla	Project	Job no.				
Tedds		1411				
RODRIGUES ASSOCIATES	Calcs for	Start page no./Revision				
1 AMWELL STREET		5.4B. 3				
LONDON EC1R 1UL	Calcs by ab	Calcs date 11/10/2016	Checked by	Checked date	Approved by	Approved date

#### Rectangular section in shear

Design shear force span 1 at 447 mm Design shear stress Design concrete shear stress  $(min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$ 

Allowable design shear stress

Value of v from Table 3.7 Design shear resistance required Area of shear reinforcement required Shear reinforcement provided Area of shear reinforcement provided  $V = \max(V_{A\_s1\_max}, V_{A\_s1\_red}) = 94 \text{ kN}$   $v = V / (b \times d) = 0.700 \text{ N/mm}^2$   $v_c = 0.79 \times \min(3, [100 \times A_{s, prov} / (b \times d)]^{1/3}) \times \max(1, (400 / d)^{1/4}) \times$   $v_c = 0.657 \text{ N/mm}^2$   $v_{max} = \min(0.8 \text{ N/mm}^2 \times (f_{cu}/1 \text{ N/mm}^2)^{0.5}, 5 \text{ N/mm}^2) = 5.000 \text{ N/mm}^2$  PASS - Design shear stress is less than maximum allowable  $0.5 \times v_c < v < (v_c + 0.4 \text{ N/mm}^2)$   $v_s = \max(v - v_c, 0.4 \text{ N/mm}^2) = 0.400 \text{ N/mm}^2$   $A_{sv,req} = v_s \times b / (0.87 \times f_{yv}) = 276 \text{ mm}^2/\text{m}$   $2 \times 8\phi \text{ legs at 200 c/c}$ 

A<sub>sv,prov</sub> = **503** mm<sup>2</sup>/m

 $s_{vl,max} = 0.75 \times d = 335 \text{ mm}$ 

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

#### Mid span 1

![](_page_45_Figure_11.jpeg)

#### Design moment resistance of rectangular section (cl. 3.4.4) - Positive moment

Design bending moment	$M = abs(M_{s1 red}) = 159 \text{ kNm}$
Depth to tension reinforcement	$d = h - c_{nom_b} - \phi_v - \phi_{bot} / 2 = 447 \text{ mm}$
Redistribution ratio	$\beta_{b} = min(1 - m_{rs1}, 1) = 1.000$
	$K = M / (b \times d^2 \times f_{cu}) = 0.066$
	K' = 0.156
	K' > K - No compression reinforcement is required
Lever arm	z = min(d × (0.5 + (0.25 - K / 0.9) <sup>0.5</sup> ), 0.95 × d) = <b>411</b> mm
Depth of neutral axis	x = (d - z) / 0.45 = <b>79</b> mm
Area of tension reinforcement required	$A_{s,req} = M / (0.87 \times f_y \times z) = 888 \text{ mm}^2$
Tension reinforcement provided	$3 \times 20\phi$ bars
Area of tension reinforcement provided	A <sub>s,prov</sub> = <b>942</b> mm <sup>2</sup>
Minimum area of reinforcement	$A_{s,min} = 0.0013 \times b \times h = \textbf{195} \ mm^2$
Maximum area of reinforcement	$A_{s,max} = 0.04 \times b \times h = \textbf{6000} \text{ mm}^2$
PASS - Area	of reinforcement provided is greater than area of reinforcement required

Rectangular section in shear	
Shear reinforcement provided	$2 \times 8\phi$ legs at 200 c/c
Area of shear reinforcement provided	A <sub>sv,prov</sub> = <b>503</b> mm <sup>2</sup> /m

(6)									
🚝 Tekla	Project	Project Job no.							
	0   (	TO DOWNSI	Start page no /Bevision						
1 AMWELL STREET	Calcs for	side retaining	wall top beam	5.4B. 4					
LONDON	Calos by	Calos data	Chacked by	Chockod data	Approved by	Approved date			
EC1R 1UL	ab	11/10/2016	Checked by	Checked date	Approved by	Approved date			
Minimum area of shear reinforce	ement (Table	3.7) $A_{sv,min} = 0.4$	$N/mm^2 \times b / (0)$	0.87 × f <sub>yv</sub> ) = <b>276</b> r	nm²/m				
		PASS - Area of sl	hear reinforce	ement provided of	exceeds minin	num required			
Maximum longitudinal spacing (	cl. 3.4.5.5)	$S_{vl,max} = 0.7$	5 × d = <b>335</b> m	m					
	PASS - Lon	gitudinal spacing	g of shear reir	nforcement prov	ided is less th	an maximum			
Design concrete shear stress		$v_{c} = 0.79 N/$	$mm^2 \times min(3,[$	$100 \times A_{s,prov}$ / (b ×	$(d)]^{1/3}) \times max(1)$	, (400mm			
		/d) <sup>1/4</sup> ) × (mi	n(f <sub>cu</sub> , 40N/mm	<sup>2</sup> ) / 25N/mm <sup>2</sup> ) <sup>1/3</sup> /	γ <sub>m</sub> = <b>0.657</b> N/m	1m²			
Design shear resistance provide	ed	$v_{s,prov} = A_{sv,}$	$_{ m prov}  imes 0.87  imes f_{ m yv}$	, / b = <b>0.729</b> N/mr	n²				
Design shear stress provided		Vprov = Vs,pro	$v + V_c = 1.386$	N/mm <sup>2</sup>					
Design shear resistance		V <sub>prov</sub> = v <sub>prov</sub>	$\times$ (b $\times$ d) = <b>18</b>	5.9 kN					
Shear lini	ks provided	valid between 0 n	nm and 5700	mm with tensior	reinforcemer	nt of 942 mm <sup>2</sup>			
Spacing of reinforcement (cl 3	8.12.11)								
Actual distance between bars in	tension	s = (b - 2 ×	$(C_{nom_s} + \phi_v + \phi_v)$	¢ <sub>bot</sub> /2)) /(N <sub>bot</sub> - 1) -	$\phi_{bot} = 77 \text{ mm}$				
Minimum distance between ba	ars in tensio	n (cl 3.12.11.1)							
Minimum distance between bars	s in tension	$s_{min} = h_{agg}$ -	⊦ 5 mm = <b>25</b> m	ım					
			PA	SS - Satisfies the	e minimum sp	acing criteria			
Maximum distance between b	ars in tensio	on (cl 3.12.11.2)							
Design service stress		$f_s = (2 \times f_y)$	$f_{\text{s}} = (2 \times f_{\text{y}} \times A_{\text{s,req}}) \; / \; (3 \times A_{\text{s,prov}} \times \beta_{\text{b}}) = \textbf{313.9} \; \text{N/mm}^2$						
Maximum distance between bar	$s_{max} = min($	s <sub>max</sub> = min(47000 N/mm / f <sub>s</sub> , 300 mm) = <b>150</b> mm							
			PAS	SS - Satisfies the	maximum sp	acing criteria			
Span to depth ratio (cl. 3.4.6)									
Basic span to depth ratio (Table	3.9)	span_to_de	epth <sub>basic</sub> = <b>20.0</b>	)					
Design service stress in tension	In reinforcement $f_s = (2 \times f_y \times A_{s,req})/(3 \times A_{s,prov} \times \beta_b) = 313.9 \text{ N/mm}^2$								
Modification for tension reinforce	ement								
	f <sub>tens</sub> :	= min(2.0, 0.55 + (	477N/mm² - fs	) / (120 × (0.9N/m	$m^{2} + (M / (b \times c))$	d²))))) = <b>0.933</b>			
Modification for compression rei	nforcement								
	f <sub>co</sub>	mp = min(1.5, 1 + (	$100 \times A_{s2,prov}$ /	(b×d)) / (3 + (10	$0 \times A_{s2,prov} / (b >$	< d)))) = <b>1.055</b>			
Modification for span length		$f_{long} = 1.000$	)						
Allowable span to depth ratio		span_to_de	span_to_depthallow = span_to_depthbasic × ftens × fcomp = <b>19.</b>						
Actual span to depth ratio		span_to_de	span_to_depthactual = $L_{s1} / d = 12.8$						
		PASS	- Actual spa	n to depth ratio	s within the a	ilowadie limit			
Support B									
	1 r	<b></b> 3	x 10ø bars						
$\dot{b}$ 2 x $\delta_{\phi}$ shear legs at 200 c/c									
		3	x 20¢ bars						
	-	300►							
Rectangular section in shear									
Design shear force span 1 at 52	53 mm	V = abs(mi	$n(V_{B_s1_max}, V_{B_s})$	_s1_red)) = <b>94</b> kN					
Design shear stress		$v = V / (b \times$	v = V / (b × d) = <b>0.700</b> N/mm <sup>2</sup>						

(9)								
<b>Tekla</b>	Project		Job no.					
Tedds		10 Downsi	de Crescent		1	1411		
RODRIGUES ASSOCIATES	Calcs for				Start page no./F	Start page no./Revision		
1 AMWELL STREET		side retaining	wall top beam	า	5.	4B. 5		
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date		
EC1R 10L	ab	11/10/2016						
	•			• ( ()		( 1) 1 (4)		
Design concrete shear stress		$v_c = 0.79 \times$	$min(3,[100 \times I)$	$A_{s,prov} / (b \times d)$	$\times$ max(1, (400	/d) <sup>1/4</sup> ) ×		
(min(f <sub>cu</sub> , 40) / 25) <sup>1/3</sup> / γ <sub>m</sub>								
		Vc = <b>0.657</b>	N/mm²					
Allowable design shear stress		$v_{max} = min(0.8 \text{ N/mm}^2 \times (f_{cu}/1 \text{ N/mm}^2)^{0.5}, 5 \text{ N/mm}^2) = 5.000 \text{ N/mm}^2$						
		PASS - Design shear stress is less than maximum allowable						
Value of v from Table 3.7		$0.5 \times v_c < v < (v_c + 0.4 \text{ N/mm}^2)$						
Design shear resistance require	ed	v <sub>s</sub> = max(v - v <sub>c</sub> , 0.4 N/mm <sup>2</sup> ) = <b>0.400</b> N/mm <sup>2</sup>						
Area of shear reinforcement red	quired	$A_{sv,req} = v_s \times b / (0.87 \times f_{yv}) = 276 \text{ mm}^2/\text{m}$						
Shear reinforcement provided		$2  imes 8\phi$ legs	at 200 c/c					
Area of shear reinforcement pro	A <sub>sv,prov</sub> = <b>503</b> mm <sup>2</sup> /m							
		PASS - Area of sl	hear reinforce	ement provided	exceeds mini	mum required		
Maximum longitudinal spacing		Svl,max = 0.7	5 × d = <b>335</b> m	m				
	PASS - Lon	gitudinal spacing	g of shear reir	nforcement prov	rided is less t	han maximum		

<b>Tekla</b> Tedds	Project	10 Downsi	Job no. 1411			
RODRIGUES ASSOCIATES 1 AMWELL STREET	Calcs for	Start page no./Revision 5.5.1				
LONDON EC1R 1UL	Calcs by ab	Calcs date 12/10/2016	Checked by	Checked date	Approved by	Approved date

RC SLAB DESIGN (BS8110:PART1:1997)	TEDDS calculation version 1.0.04
Overall depth of slab $h = 175$ mm	
Outer sagging steel	
Cover to outer tension reinforcement resisting sagging $c_{sag} = 25$ mm	
Trial bar diameter D <sub>trvx</sub> = <b>12</b> mm	
Depth to outer tension steel (resisting sagging)	
$d_x = h - c_{sag} - D_{troy}/2 = 144 \text{ mm}$	
Inner sagging steel	
Trial bar diameter $D_{\text{troy}} = 12 \text{ mm}$	
Depth to inner tension steel (resisting sagging)	
$d_{v} = h - c_{eag} - D_{trav} - 2 = 132 \text{ mm}$	
Materials	
Characteristic strength of reinforcement $f_{y} = 500 \text{ N/mm}^{2}$	
Characteristic strength of concrete $f_{cu} = 40 \text{ N/mm}^2$	
Asy Nominal 1 m width Asx Shorter Span	
h Asy Nominal 1 m width Asx	
Longer Span	
Two-way spanning slab (simple)	
MAXIMUM DESIGN MOMENTS	
Length of shorter side of slab $I_x = 3.500 \text{ m}$	
Length of longer side of slab $l_y = 6.400 \text{ m}$	
Design ultimate load per unit area $n_s = 12.9 \text{ kN/m}^2$	

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Moment coefficients

 $\alpha_{sx} = (I_y / I_x)^4 / (8 \times (1 + (I_y / I_x)^4)) = 0.115$ 

 $\alpha_{sy} = (I_y / I_x)^2 / (8 \times (1 + (I_y / I_x)^4)) = 0.034$ 

Maximum moments per unit width - simply supported slabs

 $m_{sx} = \alpha_{sx} \times n_s \times l_x^2 =$  **18.1** kNm/m

 $m_{sy} = \alpha_{sy} \times n_s \times I_x{}^2 = \textbf{5.4} \text{ kNm/m}$ 

#### CONCRETE SLAB DESIGN - SAGGING - OUTER LAYER OF STEEL (CL 3.5.4)

Design sagging moment (per m width of slab) m<sub>sx</sub> = 18.1 kNm/m

Moment Redistribution Factor  $\beta_{bx} = 1.0$ 

#### Area of reinforcement required

 $K_x = abs(m_{sx}) / (d_x^2 \times f_{cu}) = 0.022$ 

 $K'_x = min~(0.156~,~(0.402\times(\beta_{bx}$  - 0.4)) -  $(0.18\times(\beta_{bx}$  -  $0.4)^2~)) = \textbf{0.156}$ 

#### Outer compression steel not required to resist sagging

#### Slab requiring outer tension steel only - bars (sagging)

 $z_x = min ((0.95 \times d_x), (d_x \times (0.5 + \sqrt{(0.25 - K_x/0.9)}))) = 137 mm$ 

Neutral axis depth  $x_x = (d_x - z_x) / 0.45 = 16 \text{ mm}$ 

Area of tension steel required

 $A_{sx\_req} = abs(m_{sx}) / (1/\gamma_{ms} \times f_y \times z_x) = \textbf{305} \text{ mm}^2/\text{m}$ 

#### **Tension steel**

Provide 12 dia bars @ 200 centres outer tension steel resisting sagging

 $A_{sx\_prov} = A_{sx} = 565 \text{ mm}^2/\text{m}$ 

Area of outer tension steel provided sufficient to resist sagging

Design sagging moment (per m width of slab)  $m_{sy} = 5.4 \text{ kNm/m}$ 

Moment Redistribution Factor  $\beta_{by} = 1.0$ 

Area of reinforcement required

 $K_y = abs(m_{sy}) / (d_y^2 \times f_{cu}) = 0.008$ 

 $K'_{y} = min~(0.156$  ,  $(0.402 \times (\beta_{by}$  - 0.4)) -  $(0.18 \times (\beta_{by}$  -  $0.4)^2$  )) = 0.156

Inner compression steel not required to resist sagging

Slab requiring inner tension steel only - bars (sagging)

 $z_y = min ((0.95 \times d_y), (d_y \times (0.5 + \sqrt{(0.25 - K_y/0.9)}))) = 125 mm$ 

Neutral axis depth  $x_y = (d_y - z_y) / 0.45 = 15 \text{ mm}$ 

Area of tension steel required

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A <sub>sy_req</sub> = abs(m <sub>sy</sub> ) / (1/γ <sub>r</sub> Tension steel <u>Provide 12 dia bars @ 200 d</u> A <sub>sy_prov</sub> = A <sub>sy</sub> = 565 mm	$f_{ms} \times f_y \times z_y) = 99$ <u>centres</u> <u>inner to</u> <sup>2</sup> /m	mm²/m ension steel res Area of	sisting sagging	g steel provided s	sufficient to re	sist sagging
Check min and max areas of	steel resisting	sagging				
Total area of concrete $A_c = h$	= <b>175000</b> mm²/n	n				
Minimum % reinforcem	ient k = 0.13 %					
$A_{st\_min} = k \times A_c = 228 m$	ım²/m					
$A_{st_max} = 4 \% \times A_c = 700$	<b>00</b> mm²/m					
Steel defined:						
Outer steel resisting sa	agging $A_{sx\_prov} =$	<b>565</b> mm²/m				
Inner steel resisting se	aging A	565 mm <sup>2</sup> /m		Area of outer s	teel provided	(sagging) OK
initer steel resisting sa	ggilig Asy_prov = -	<b>JUJ</b> IIIII /III		Area of inner st	eel provided (	sagging) OK
SHEAR RESISTANCE OF CO	NCRETE SI AR	S (CL 3 5 5)				
Outer tension steel resisting	sagging mome	nts				
Depth to tension steel	from compressio	on face d <sub>x</sub> = <b>144</b>	mm			
Area of tension reinford	cement provided	(per m width of	slab) A <sub>sx_prov</sub> =	<b>565</b> mm²/m		
Design ultimate shear	force (per m wid	th of slab) $V_x = 4$	<b>41</b> kN/m			
Characteristic strength	of concrete fcu :	= <b>40</b> N/mm <sup>2</sup>				
Applied shear stress						
$v_x = V_x / d_x = 0.29 \text{ N/mm}^2$						
Check shear stress to clause	3.5.5.2					
$v_{\text{allowable}} = \min ((0.8 \text{ N}^{1/2}/\text{mm}) \times 10^{1/2})$	√(f <sub>cu</sub> ), 5 N/mm² )	= <b>5.00</b> N/mm <sup>2</sup>				
					Shea	nr stress - OK
Shear stresses to clause 3.5.	5.3					
Design shear stress						
$f_{cu_ratio} = if (f_{cu} > 40 N/m)$	m² , 40/25 , f <sub>cu</sub> /(2	25 N/mm²)) = <b>1.6</b>	600			
$v_{cx} = 0.79 \text{ N/mm}^2 \times \text{min}$	$n(3,100 \times A_{sx\_pro})$	$(v / d_x)^{1/3} \times max(0)$	.67,(400 mm / o	d <sub>x</sub> ) <sup>1/4</sup> ) / 1.25 × f <sub>cu_</sub>	_ratio <sup>1/3</sup>	
v <sub>cx</sub> = <b>0.70</b> N/mm <sup>2</sup>						
Applied shear stress						
v <sub>x</sub> = <b>0.29</b> N/mm <sup>2</sup>						
				No sh	ear reinforcen	nent required

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SHEAR RESISTANCE OF CO Inner tension steel resisting Depth to tension steel	NCRETE SLA sagging mom	<b>BS (CL 3.5.5)</b> <b>Nents</b> sion face $d_y = 132$	mm	<b>- 565</b> mm <sup>2</sup> /m		

Design ultimate shear force (per m width of slab)  $V_y = 26 \text{ kN/m}$ 

Characteristic strength of concrete f<sub>cu</sub> = 40 N/mm<sup>2</sup>

#### **Applied shear stress**

 $v_y = V_y / d_y = 0.20 \text{ N/mm}^2$ 

#### Check shear stress to clause 3.5.5.2

 $v_{allowable} = min ((0.8 \text{ N}^{1/2}/mm) \times \sqrt{(f_{cu})}, 5 \text{ N}/mm^2) = 5.00 \text{ N}/mm^2$ 

## Shear stress - OK

#### Shear stresses to clause 3.5.5.3

#### **Design shear stress**

$$\begin{split} f_{cu\_ratio} &= if \; (f_{cu} > 40 \; N/mm^2 \;, \; 40/25 \;, \; f_{cu}/(25 \; N/mm^2)) = \textbf{1.600} \\ v_{cy} &= 0.79 \; N/mm^2 \; \times \; min(3,100 \; \times \; A_{sy\_prov} \; / \; d_y)^{1/3} \; \times \; max(0.67,(400 \; mm) \; / \; d_y)^{1/4} \; / \; 1.25 \; \times \; f_{cu\_ratio}^{1/3} \\ v_{cy} &= \textbf{0.73} \; N/mm^2 \\ \text{Applied shear stress} \\ v_y &= \textbf{0.20} \; N/mm^2 \end{split}$$

#### No shear reinforcement required

#### CONCRETE SLAB DEFLECTION CHECK (CL 3.5.7)

Slab span length  $l_x = 3.500 \text{ m}$ 

Design ultimate moment in shorter span per m width  $m_{sx} = 18 \text{ kNm/m}$ 

Depth to outer tension steel  $d_x = 144 \text{ mm}$ 

#### **Tension steel**

Area of outer tension reinforcement provided  $A_{sx\_prov} = 565 \text{ mm}^2/\text{m}$ 

Area of tension reinforcement required Asx\_req = 305 mm<sup>2</sup>/m

Moment Redistribution Factor  $\beta_{bx} = 1.00$ 

#### **Modification Factors**

Basic span / effective depth ratio (Table 3.9) ratio<sub>span\_depth</sub> = 20

The modification factor for spans in excess of 10m (ref. cl 3.4.6.4) has not been included.

 $f_{s} = 2 \times f_{y} \times A_{sx\_req} / (3 \times A_{sx\_prov} \times \beta_{bx}) = 179.8 \text{ N/mm}^{2}$ 

 $factor_{tens} = min (2, 0.55 + (477 \text{ N/mm}^2 - f_s) / (120 \times (0.9 \text{ N/mm}^2 + m_{sx} / d_x^2))) = 1.946$ 

#### Calculate Maximum Span

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This is a simplified approach and further attention should be given where special circumstances exist. Refer to clauses 3.4.6.4 and 3.4.6.7.

Maximum span  $I_{max} = ratio_{span depth} \times factor_{tens} \times d_x = 5.60 m$ 

#### Check the actual beam span

Actual span/depth ratio  $I_x / d_x = 24.31$ 

Span depth limit  $ratio_{span\_depth} \times factor_{tens} = 38.91$ 

#### Span/Depth ratio check satisfied

### CHECK OF NOMINAL COVER (SAGGING) - (BS8110:PT 1, TABLE 3.4)

Slab thickness h = 175 mm

Effective depth to bottom outer tension reinforcement  $d_x = 144.0 \text{ mm}$ 

Diameter of tension reinforcement  $D_x = 12 \text{ mm}$ 

Diameter of links  $L_{diax} = 0 \text{ mm}$ 

Cover to outer tension reinforcement

 $c_{tenx} = h - d_x - D_x / 2 = 25.0 \text{ mm}$ 

Nominal cover to links steel

 $c_{nomx} = c_{tenx} - L_{diax} = \textbf{25.0} mm$ 

Permissable minimum nominal cover to all reinforcement (Table 3.4)

c<sub>min</sub> = **25** mm

Cover over steel resisting sagging OK

![](_page_53_Figure_0.jpeg)

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#### Analysis results

Maximum moment support A	M <sub>A_max</sub> = <b>-74</b> kNm	M <sub>A_red</sub> = <b>-74</b> kNm
Maximum moment span 1 at 3200 mm	M <sub>s1_max</sub> = <b>42</b> kNm	M <sub>s1_red</sub> = <b>42</b> kNm
Maximum moment support B	M <sub>B_max</sub> = <b>-74</b> kNm	M <sub>B_red</sub> = <b>-74</b> kNm
Maximum shear support A	$V_{A_{max}} = 61 \text{ kN}$	$V_{A\_red} = 61 \text{ kN}$
Maximum shear support A span 1 at 199 mm	$V_{A_{s1}_{max}} = 59 \text{ kN}$	V <sub>A_s1_red</sub> = <b>59</b> kN
Maximum shear support B	V <sub>B_max</sub> = <b>-61</b> kN	V <sub>B_red</sub> = <b>-61</b> kN
Maximum shear support B span 1 at 6201 mm	$V_{B_{s1}_{max}} = -59 \text{ kN}$	$V_{B_s1_{red}} = -59 \text{ kN}$
Maximum reaction at support A	R <sub>A</sub> = <b>61</b> kN	
Unfactored dead load reaction at support A	R <sub>A_Dead</sub> = <b>31</b> kN	
Unfactored imposed load reaction at support A	R <sub>A_Imposed</sub> = <b>11</b> kN	
Maximum reaction at support B	R <sub>B</sub> = <b>61</b> kN	
Unfactored dead load reaction at support B	R <sub>B_Dead</sub> = <b>31</b> kN	
Unfactored imposed load reaction at support B	$R_{B_{Imposed}} = 11 \text{ kN}$	

#### **Rectangular section details**

Section width Section depth

![](_page_54_Picture_5.jpeg)

![](_page_54_Figure_6.jpeg)

![](_page_54_Figure_7.jpeg)

![](_page_54_Figure_8.jpeg)

### **Concrete details**

Concrete strength class Characteristic compressive cube strength Modulus of elasticity of concrete Maximum aggregate size

#### C32/40

$$\label{eq:fcu} \begin{split} f_{cu} &= \textbf{40} \; N/mm^2 \\ E_c &= 20 k N/mm^2 + 200 \times f_{cu} = \textbf{28000} \; N/mm^2 \\ h_{agg} &= \textbf{20} \; mm \end{split}$$

#### **Reinforcement details**

 $\begin{array}{ll} \mbox{Characteristic yield strength of reinforcement} & f_y = {\color{black}{500}} \ \mbox{N/mm}^2 \\ \mbox{Characteristic yield strength of shear reinforcement} & f_{yv} = {\color{black}{500}} \ \mbox{N/mm}^2 \end{array}$ 

#### Nominal cover to reinforcement

Nominal cover to top reinforcement	$c_{nom_t} = 35 \text{ mm}$
Nominal cover to bottom reinforcement	Cnom_b = <b>35</b> mm
Nominal cover to side reinforcement	Cnom_s = <b>35</b> mm

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Support A								
1	6			5 x 16 bars				
0				$2 \times 8_{\phi}$ shear legs at	: 100 c/c			
	•	<u> </u>		$5 \times 16_{\varphi}$ bars				
▼								
-		—500— <b>—</b>						
Rectangular section in flexu	re (cl.3.4.4)							
Design bending moment		$M = abs(M_{A_{red}}) =$	<b>74</b> kNm					
Depth to tension reinforcement	t	$d = h - c_{nom_t} - \phi_v - \phi_v$	φ <sub>top</sub> / 2 =	= <b>199</b> mm				
Redistribution ratio		$\beta_b = min(1 - m_{rA}, T)$	) = 1.00	00				
		$K = M / (b \times d^2 \times f_{cu}) = 0.094$						
		K' = 0.156						
			(' > K - )	No compression	n reinforceme	ent is require		
Lever arm		$z = \min(d \times (0.5 +$	(0.25 -	K / 0.9) <sup>0.5</sup> ), 0.95 :	× d) = <b>175</b> mm			
Depth of neutral axis		x = (d - z) / 0.45 =	52 mm					
Area of tension reinforcement	required	A <sub>s,req</sub> = M / (0.87 >	$f_y \times z) =$	= <b>974</b> mm²				
Tension reinforcement provide	d	$5 \times 16\phi$ bars						
Area of tension reinforcement	provided	$A_{s,prov} = 1005 \text{ mm}^2$						
Minimum area of reinforcemen	it .	$A_{s,min} = 0.0013 \times b \times h = 163 \text{ mm}^2$						
Maximum area of reinforcement	nt PASS - Area	$A_{s,max} = 0.04 \times b$	n = 500 ded is o	JO mm² preater than area	a of reinforce	ment require		
Rectangular section in shear	r		lea le g			inent require		
Design shear force span 1 at 1	99 mm	$V = max(V_{A s1 max})$	VA s1 re	d) = <b>59</b> kN				
Design shear stress		$v = V / (b \times d) = 0$	<b>593</b> N/n	nm²				
Design concrete shear stress		$v_c = 0.79 \times min(3,[100 \times A_{s,orov} / (b \times d)]^{1/3}) \times max(1, (400 / d)^{1/4}) \times$						
(min(f <sub>cu</sub> , 40) / 25) <sup>1/3</sup> / γ <sub>m</sub>				· · · · · · ·				
		v <sub>c</sub> = <b>0.883</b> N/mm <sup>2</sup>						
Allowable design shear stress		$v_{max} = min(0.8 \text{ N/mm}^2 \times (f_{cu}/1 \text{ N/mm}^2)^{0.5}, 5 \text{ N/mm}^2) = 5.000 \text{ N/mm}^2$						
Value of v from Table 3.7		0 5 × v < v < (v -	. 0 4 N/r	$m^{2}$	s man maxin	ium anowad		
Design shear resistance requir	red	$v_{c} = \max(v_{c} - v_{c})$	1 N/mm <sup>2</sup>	<sup>2</sup> ) – <b>0 400</b> N/mm <sup>2</sup>	2			
Area of shear reinforcement re	auired	$v_s = max(v - v_c, 0.4 \text{ N/mm}^2) = 0.400 \text{ N/mm}^2$						
Shear reinforcement provided	quilou	$2 \times 8\phi$ legs at 100 c/c						
Area of shear reinforcement or	rovided	$A_{\text{sy prov}} = 1005 \text{ mm}$	<sup>2</sup> /m					
		PASS - Area of shear re	inforce	ment provided of	exceeds mini	mum require		
Maximum longitudinal spacing		S <sub>vl,max</sub> = 0.75 × d =	<b>149</b> mr	n				
· · ·	PASS - Lon	gitudinal spacing of sh	ear rein	forcement prov	ided is less t	han maximu		
Spacing of reinforcement (cl	3.12.11)							

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Minimum distance between	have in tension		·				
Minimum distance between	bars in tension	1 (CI 3.12.11.1)	5 mm <b>- 25</b> n	200			
		Smin = Hagg -	το πημ. = 23 η ΡΔ	SS - Satisfies th	e minimum sr	nacina cri	
Maximum diatanaa haturaan	have in tensis	- (-1.0.10.11.0)			e mininani op	uonig on	
Design convice stross	bars in tensio	f (ci 3.12.11.2)	$(\mathbf{A})/(2\times\mathbf{I})$	<b>\ x</b> B. <b>) = 333</b>	1 N/mm <sup>2</sup>		
Maximum distance between b	are in tonsion	$I_s = (2 \times I_y)$	× A <sub>s,req</sub> ) / (3 × <i>r</i>	$(f_{s,prov} \times p_b) = 323.$	15 mm		
		Smax = IIIII(	47000 Ν/ΠΠΠ/ <b>ΡΔ</b>	$r_s, 300 rmm) = 1-$	+J IIIII e maximum er	nacina cri	
				55 - Salishes line	e maximum sp		
Mid span 1							
<b>↑</b>				<b>5</b> ( <b>0</b> )			
		• •	1	$5 \times 16 \phi$ bars			
20-				$2 \times 8_{\phi}$ shear legs a	it 100 c/c		
5				T U			
	<b>L</b>	• •		$5 \times 16_{\varphi}$ bars			
<u> </u>							
		500					
		300					
Design moment resistance of	of rootongular	antion (al. 2.4.4)					
	Ji reclanyular	section (cl. 3.4.4)	) - Positive me	oment			
Design bending moment		$M = abs(M_{\rm s})$	) - Positive me <sub>s1_red</sub> ) = 42 kNr	<b>oment</b> m			
Design bending moment Depth to tension reinforcement	nt	$M = abs(M_s)$ $d = h - c_{norm}$	) - Positive mo <sub>s1_red</sub> ) = <b>42</b> kNr η_b - φ <sub>v</sub> - φ <sub>bot</sub> / 2	oment m = <b>199</b> mm			
Design bending moment Depth to tension reinforcemen Redistribution ratio	nt	$M = abs(M_s)$ $d = h - c_{norr}$ $\beta_b = min(1)$	) - Positive ma s1_red) = 42 kNr n_b - φ <sub>v</sub> - φ <sub>bot</sub> / 2 - m <sub>rs1</sub> , 1) = <b>1.0</b>	oment m = 199 mm 000			
Design bending moment Depth to tension reinforcemen Redistribution ratio	nt	$M = abs(M)$ $d = h - c_{nom}$ $\beta_b = min(1)$ $K = M / (b > b)$	$\begin{array}{l} \textbf{-Positive mo}_{s1\_red} = \textbf{42} \ \text{kNr}_{s1\_red} = \textbf{42} \ \text{kNr}_{1\_b} - \phi_{V} - \phi_{bot} / 2 \\ \textbf{-m}_{rs1}, 1) = \textbf{1.0} \\ \textbf{-d}^{2} \times f_{cu}) = \textbf{0.0} \end{array}$	oment n = 199 mm 000 053			
Design bending moment Depth to tension reinforcemen Redistribution ratio	nt	$M = abs(M)$ $d = h - c_{nom}$ $\beta_b = min(1)$ $K = M / (b >$ $K' = 0.156$	) - <b>Positive m</b> s1_red) = <b>42</b> kNr a_b - φ <sub>v</sub> - φ <sub>bot</sub> / 2 - m <sub>rs1</sub> , 1) = <b>1.0</b> × d <sup>2</sup> × f <sub>cu</sub> ) = <b>0.0</b>	oment m = 199 mm 000 053			
Design bending moment Depth to tension reinforcemen Redistribution ratio	nt	$M = abs(M)$ $d = h - c_{nom}$ $\beta_b = min(1)$ $K = M / (b > K' = 0.156$	$\begin{aligned} \mathbf{Positive mo}_{s1\_red} &= 42 \text{ kNr} \\ \mathbf{x}_{1\_e} - \mathbf{\phi}_{v} - \mathbf{\phi}_{bot} / 2 \\ - \mathbf{m}_{rs1}, 1) &= 1.0 \\ \times d^2 \times f_{cu}) &= 0.0 \\ \mathbf{K'} &> \mathbf{K'} - \mathbf{K'} \end{aligned}$	oment n = 199 mm 000 053 <i>No compressio</i>	n reinforceme	ent is requ	
Design bending moment Depth to tension reinforcemen Redistribution ratio	nt	$M = abs(M)$ $d = h - c_{nom}$ $\beta_b = min(1)$ $K = M / (b > K' = 0.156)$ $z = min(d > b)$	$\begin{array}{l} \textbf{(-) - Positive mass}\\ \textbf{(s1_red)} = 42 \ \text{kNr}\\ \textbf{(s1_red)} = \phi_{V} - \phi_{bot} \ / \ 2\\ \textbf{(-)} - \textbf{(rs1, 1)} = 1.0\\ \textbf{(c)} < d^2 \times \textbf{(rcu)} = 0.0\\ \hline \textbf{(c)} \\ \textbf{(c)} < \textbf{(c)} \\ \textbf{(c)} $	oment n = 199 mm 000 053 <i>No compressio</i> K / 0.9) <sup>0.5</sup> ), 0.95	<i>n reinforceme</i> × d) = <b>186</b> mm	ent is requ	
Design bending moment Depth to tension reinforcemen Redistribution ratio Lever arm Depth of neutral axis	nt	$M = abs(M)$ $M = abs(M)$ $d = h - c_{nom}$ $\beta_b = min(1)$ $K = M / (b > C)$ $K' = 0.156$ $z = min(d > C)$ $x = (d - z) / C$	(0) - <b>Positive m</b> s1_red) = <b>42</b> kNr h_b - φ <sub>V</sub> - φ <sub>bot</sub> / 2 - m <sub>rs1</sub> , 1) = <b>1.0</b> × d <sup>2</sup> × f <sub>cu</sub> ) = <b>0.0</b> <i>K' &gt; K</i> - (0.5 + (0.25 - 0.45 = <b>28</b> mm	oment m = <b>199</b> mm 000 053 <i>No compressio</i> K / 0.9) <sup>0.5</sup> ), 0.95	o <b>n reinforceme</b> × d) = <b>186</b> mm	ent is requ	
Design bending moment Depth to tension reinforcemen Redistribution ratio Lever arm Depth of neutral axis Area of tension reinforcement	required	$M = abs(M)$ $d = h - c_{nom}$ $\beta_b = min(1)$ $K = M / (b > K' = 0.156)$ $z = min(d > x = (d - z) / A_{s,req} = M / (b > z)$	$\begin{array}{l} \textbf{(0.87 \times f_y \times z)} = \textbf{42 kNr}\\ \textbf{(0.81_red)} = \textbf{42 kNr}\\ \textbf{(0.81 \times fcu)} = \textbf{42 kNr}\\ \textbf{(0.87 \times fcu)} = \textbf{1.0}\\ \textbf{(0.87 \times fy \times z)} \end{array}$	oment n = 199 mm 000 053 <i>No compressio</i> K / 0.9) <sup>0.5</sup> ), 0.95 n = <b>518</b> mm <sup>2</sup>	o <b>n reinforceme</b> × d) = <b>186</b> mm	ent is requ	
Design bending moment Depth to tension reinforcemen Redistribution ratio Lever arm Depth of neutral axis Area of tension reinforcement Tension reinforcement provide	required	$\begin{aligned} & M = abs(M, M) \\ & M = abs(M, M) \\ & d = h - c_{norm} \\ & \beta_b = min(1) \\ & K = M / (b + b) \\ & K' = 0.156 \\ & z = min(d \times b) \\ & x = (d - z) / \\ & A_{s,req} = M / \\ & 5 \times 16\phi bar \end{aligned}$	$\begin{array}{l} \textbf{-Positive mas}\\ \textbf{s1_red}) = \textbf{42} \ \text{kNr}\\ \textbf{s1_red}) = \textbf{42} \ \text{kNr}\\ \textbf{s1_red} = \textbf{\phi}_{V} - \textbf{\phi}_{bot} \ / \ 2\\ - \ m_{rs1}, \ 1) = \textbf{1.0}\\ \times \ d^2 \times f_{cu}) = \textbf{0.0}\\ \hline \textbf{K' > K} - \textbf{c}\\ < (0.5 + (0.25 - 0.45 = \textbf{28} \ \text{mm}\\ (0.87 \times f_{y} \times z) \ \text{rs} \end{array}$	oment m = 199 mm 000 053 <i>No compressio</i> K / 0.9) <sup>0.5</sup> ), 0.95 n = <b>518</b> mm <sup>2</sup>	<i>m reinforceme</i> × d) = <b>186</b> mm	ent is requ	
Design bending moment Depth to tension reinforcemen Redistribution ratio Lever arm Depth of neutral axis Area of tension reinforcement Tension reinforcement provide Area of tension reinforcement	required provided	$\begin{aligned} &M = abs(M, a) \\ &M = abs(M, b) \\ &d = h - c_{nom} \\ &\beta_b = min(1) \\ &K = M / (b) \\ &K' = 0.156 \\ &z = min(d) \\ &X = (d - z) / \\ &A_{s,req} = M / \\ &5 \times 16\phi \ bar \\ &A_{s,prov} = 10 \end{aligned}$	$\begin{aligned} \mathbf{F}_{s1\_red} &= 42 \text{ kNr}_{s1\_red} \\ \mathbf{F}_{s1\_red} &= 42 \text{ kNr}_{s1\_red} \\ \mathbf{F}_{s1\_red} &= 42 \text{ kNr}_{s1} \\ \mathbf{F}_{s1\_red} &= 1.0 \\ \mathbf{F}_{s1\_red\_red\_red\_red\_red\_red\_red\_red\_red\_red$	oment n = <b>199</b> mm 000 053 <i>No compressio</i> K / 0.9) <sup>0.5</sup> ), 0.95 n = <b>518</b> mm <sup>2</sup>	o <b>n reinforceme</b> × d) = <b>186</b> mm	ent is requ	
Design bending moment Depth to tension reinforcemen Redistribution ratio Lever arm Depth of neutral axis Area of tension reinforcement Tension reinforcement provide Area of tension reinforcement Minimum area of reinforcemen	required ed provided	$M = abs(M, d = h - c_{nom})$ $\beta_b = min(1)$ $K = M / (b > c_{nom})$ $K' = 0.156$ $z = min(d > c_{nom})$ $x = (d - z) / (b > c_{nom})$ $A_{s,req} = M / (b > c_{nom})$ $A_{s,req} = 100$ $A_{s,min} = 0.00$	$\begin{array}{l} \textbf{(-) Positive mass}\\ (-) Positive $	oment n = 199 mm 000 053 <i>No compressio</i> K / 0.9) <sup>0.5</sup> ), 0.95 = <b>518</b> mm <sup>2</sup>	<i>m reinforceme</i> × d) = <b>186</b> mm	ent is requ	
Design bending moment Depth to tension reinforcemen Redistribution ratio Lever arm Depth of neutral axis Area of tension reinforcement Tension reinforcement provide Area of tension reinforcement Minimum area of reinforcemen	required ed provided nt	$\begin{aligned} &M = abs(M, d = h - c_{nom}) \\ &B_b = min(1) \\ &K = M / (b > K' = 0.156) \\ &Z = min(d > K' = 0.156) \\ &Z = min(d > K' = 0.156) \\ &Z = (d - Z) / \\ &A_{s,req} = M / \\ &5 > 16\phi ban \\ &A_{s,prov} = 100 \\ &A_{s,min} = 0.00 \\ &A_{s,max} = 0.00 \end{aligned}$	$\begin{aligned} \mathbf{F}_{1} &= \mathbf{Positive ma}\\ \mathbf{s}_{1-red} &= 42 \text{ kNr}\\ \mathbf{s}_{1-b} &= \phi_{V} - \phi_{bot} / 2\\ - m_{rs1}, 1) &= 1.0\\ \times d^{2} \times f_{cu} &= 0.0\\ \mathbf{K'} &= \mathbf{K'}\\ \times d^{2} \times f_{cu} &= 0.0\\ \mathbf{K'} &= \mathbf{K'}\\ \times \mathbf{K'} &= \mathbf{K'}\\ \times \mathbf{K'} &= \mathbf{K'}\\ \mathbf{K'} &$	oment m = 199 mm 000 053 <i>No compressio</i> K / 0.9) <sup>0.5</sup> ), 0.95 h = 518 mm <sup>2</sup> 163 mm <sup>2</sup> 00 mm <sup>2</sup>	o <b>n reinforceme</b> × d) = <b>186</b> mm	ent is requ	
Design bending moment Depth to tension reinforcemen Redistribution ratio Lever arm Depth of neutral axis Area of tension reinforcement Tension reinforcement provide Area of tension reinforcement Minimum area of reinforcemer Maximum area of reinforcemer	required ed provided nt <b>PASS - Area</b>	$M = abs(M, d = h - c_{nom})$ $\beta_b = min(1)$ $K = M / (b > c_{nom})$ $K' = 0.156$ $z = min(d > c_{nom})$ $x = (d - z) / (b > c_{nom})$ $A_{s,req} = M / (b > c_{nom})$ $A_{s,req} = 0.00$ $A_{s,max} = 0.00$ of reinforcements	$\begin{aligned} \mathbf{F}_{i_{red}} &= 42 \text{ kNr} \\ \mathbf{f}_{i_{red}} &= 42 \text{ kNr} \\ \mathbf{f}_{i_{red}} &= \mathbf{\phi}_{v} - \mathbf{\phi}_{bot} / 2 \\ - \mathbf{m}_{rs1}, 1) &= 1.0 \\ \times \mathbf{d}^{2} \times \mathbf{f}_{cu}) &= 0.0 \\ \mathbf{K'} &> \mathbf{K} - \\ \times (0.5 + (0.25 - 0.45 = 28 \text{ mm} \\ (0.87 \times \mathbf{f}_{y} \times \mathbf{z}) \\ \mathbf{rs} \\ 0.5 \text{ mm}^{2} \\ 013 \times \mathbf{b} \times \mathbf{h} &= 50 \\ \mathbf{t} \text{ provided is get} \end{aligned}$	oment m = 199 mm 000 053 <i>No compressio</i> K / 0.9) <sup>0.5</sup> ), 0.95 = 518 mm <sup>2</sup> 163 mm <sup>2</sup> 00 mm <sup>2</sup> greater than are	n reinforceme × d) = <b>186</b> mm	ent is requ ment requ	
Design bending moment Depth to tension reinforcemen Redistribution ratio Lever arm Depth of neutral axis Area of tension reinforcement Tension reinforcement provide Area of tension reinforcement Minimum area of reinforcemer Maximum area of reinforceme	required ed provided nt <b>PASS - Area</b> <b>r</b>	Section (cf. 3.4.4) $M = abs(M, d = h - c_{nom})$ $\beta_b = min(1)$ K = M / (b > 2) K' = 0.156 z = min(d > 2) x = (d - 2) / 2 $A_{s,req} = M / 5 > 16\phi$ bas $A_{s,prov} = 100$ $A_{s,min} = 0.00$ $A_{s,max} = 0.00$ of reinforcements	$\begin{aligned} \mathbf{F}_{1} &= \mathbf{Positive ma} \\ \mathbf{s}_{1} &= \mathbf{q}_{2} \text{ kNr} \\ \mathbf{s}_{1} &= \mathbf{q}_{v} - \mathbf{q}_{bot} / 2 \\ - \mathbf{m}_{rs1}, 1) &= 1.0 \\ \times & \mathbf{q}^{2} \times \mathbf{f}_{cu}) &= 0.0 \\ \mathbf{K}' &= \mathbf{K} - \mathbf{q}_{2} \\ \times & \mathbf{q}_{2} \times \mathbf{f}_{cu}) &= 0.0 \\ \mathbf{K}' &= \mathbf{K} - \mathbf{q}_{2} \\ \times & \mathbf{q}_{2} \times \mathbf{f}_{cu}) &= 0.0 \\ \mathbf{K}' &= \mathbf{K} - \mathbf{q}_{2} \\ \times & \mathbf{q}_{2} \times \mathbf{f}_{cu} \\ \mathbf{K}' &= \mathbf{K} - \mathbf{q}_{2} \\ \mathbf{K} &= \mathbf{K} \\ \mathbf{K} &= \mathbf{K} - \mathbf{q}_{2} \\ \mathbf{K} &= \mathbf{K} \\ \mathbf{K} \\ \mathbf{K} &= \mathbf{K} \\ \mathbf{K} \\$	oment m = 199 mm 000 053 <i>No compressio</i> K / 0.9) <sup>0.5</sup> ), 0.95 = 518 mm <sup>2</sup> 163 mm <sup>2</sup> 00 mm <sup>2</sup> greater than are	on reinforceme × d) = <b>186</b> mm	ent is requ ment requ	
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Design bending moment Depth to tension reinforcemen Redistribution ratio Lever arm Depth of neutral axis Area of tension reinforcement Tension reinforcement provide Area of tension reinforcement Minimum area of reinforcemer Maximum area of reinforcemer Rectangular section in shea Shear reinforcement provided Area of shear reinforcement p Minimum area of shear reinfor Maximum longitudinal spacing Design concrete shear stress	required ed provided nt <b>PASS - Area</b> r rovided rcement (Table (cl. 3.4.5.5) <b>PASS - Lon</b>	Section (cl. 3.4.4) $M = abs(M, d = h - c_{nom})$ $\beta_b = min(1)$ K = M / (b > K' = 0.156) $z = min(d × x = (d - z) / A_{s,req} = M / 5 × 16\phi$ bance $A_{s,rreq} = M / 5 × 16\phi$ bance $A_{s,rreq} = M / 5 × 16\phi$ bance $A_{s,rreq} = 0.00$ $A_{s,max} = 0.00$ $A_{sv,prov} = 100$ $A_{sv,prov} = 100$ $A_{sv,pr$		pment         m         = 199 mm         000         053         No compression $K / 0.9)^{0.5}$ ), 0.95         = 518 mm²         163 mm²         00 mm²         greater than are         0.87 × fyv) = 460 m         m         forcement provided         m         f100 × As,prov / (b ×         2) / 25N/mm²) <sup>1/3</sup> /	n reinforceme × d) = 186 mm a of reinforced mm²/m exceeds minit vided is less th × d)] <sup>1/3</sup> ) × max(' ′ γm = 0.883 N/r	ent is requ ment requ mum requ han maxir 1, (400mm nm <sup>2</sup>	
Design bending moment Depth to tension reinforcement Redistribution ratio Lever arm Depth of neutral axis Area of tension reinforcement Tension reinforcement provide Area of tension reinforcement Minimum area of reinforcement Maximum area of reinforcement Rectangular section in shea Shear reinforcement provided Area of shear reinforcement p Minimum area of shear reinfor Maximum longitudinal spacing Design concrete shear stress Design shear resistance provide	required ed provided nt nt <b>PASS - Area</b> r rovided cement (Table (cl. 3.4.5.5) <b>PASS - Lon</b> ded	Section (cf. 3.4.4) $M = abs(M, d = h - c_{nom})$ $\beta_b = min(1)$ $K = M / (b > C_{rom})$ K' = 0.156 $z = min(d > C_{rom})$ $x = (d - z) / (b > C_{rom})$ $A_{s,req} = M / (c > C_{rom})$ $A_{s,req} = M / (c > C_{rom})$ $A_{s,req} = 0.0$ $A_{s,max} = 0.0$ $A_{s,max} = 0.0$ $A_{s,max} = 0.7$ $S_{vl,max} = 0.7$ gitudinal spacing $v_c = 0.79N / (d)^{1/4}) × (min)$ $v_{s,prov} = A_{sv}$	) - Positive ma $s_1_{red}$ ) = 42 kNr $r_b - \phi_v - \phi_{bot} / 2$ $- m_{rs1}, 1) = 1.0$ $\times d^2 \times f_{cu}$ ) = 0.0 $K' > K - d^2 \times f_{cu}$ $(0.5 + (0.25 - 0.45 - 28 mm^2))$ $(0.87 \times f_y \times z)$ rs $0.5 mm^2$ $0.13 \times b \times h = 50$ $t provided is gravity t x b \times h = 50t provided is gravityt provided is gravityt provided is gravityt provided is gravity t p = 50t p = 50t$	pment         m         = 199 mm         000         053         No compression         K / 0.9) <sup>0.5</sup> ), 0.95         = 518 mm <sup>2</sup> 163 mm <sup>2</sup> 00 mm <sup>2</sup> greater than are         0.87 × fyy) = 460 m         ement provided         m         flox × fyy) = 25N/mm <sup>2</sup> ) <sup>1/3</sup> /         (100 × As,prov / (b > 2) / 25N/mm <sup>2</sup> ) <sup>1/3</sup> /         y / b = 0.875 N/mi	$rm reinforcement \times d) = 186 mmrm^2/mrm^2/mrided is less thrided is less th(\gamma_m = 0.883 N/rm^2)$	ent is requ ment requ mum requ han maxir 1, (400mm nm <sup>2</sup>	
Design bending moment Depth to tension reinforcemen Redistribution ratio Lever arm Depth of neutral axis Area of tension reinforcement Tension reinforcement provide Area of tension reinforcement Minimum area of reinforcemer Maximum area of reinforcemer Rectangular section in shea Shear reinforcement provided Area of shear reinforcement p Minimum area of shear reinfor Maximum longitudinal spacing Design concrete shear stress Design shear resistance provided	required ed provided nt <b>PASS - Area</b> r rovided rcement (Table g (cl. 3.4.5.5) <b>PASS - Lon</b> ded	Section (cl. 3.4.4) $M = abs(M, d = h - c_{nom})$ $\beta_b = min(1)$ K = M / (b > K' = 0.156) z = min(d > K' = 0.156) $A_{s,req} = M / (b > K' = 0.156)$ $A_{s,req} = M / (b > K' = 0.156)$ $A_{s,req} = M / (b > K' = 0.156)$ $A_{s,req} = M / (b > K' = 0.156)$ $A_{s,min} = 0.00$ $A_{s,min} = 0.00$ $A_{s,min} = 0.00$ $A_{s,max} = 0.00$ $A_{s,max} = 0.00$ $A_{s,max} = 0.00$ $A_{s,max} = 0.00$ $A_{s,max} = 0.00$ $A_{s,min} = 0.00$	p - Positive masses (1 - Pos	pment         m         = 199 mm         000         053         No compression $K / 0.9)^{0.5}$ ), 0.95         = 518 mm²         163 mm²         00 mm²         greater than are         0.87 × fyv) = 460 m         ement provided         m         forcement provided         m         for b = 0.875 N/mi         N/mm²	m reinforceme × d) = 186 mm a of reinforced mm <sup>2</sup> /m exceeds minin vided is less th × d)] <sup>1/3</sup> ) × max( $\gamma_m = 0.883$ N/r m <sup>2</sup>	ent is requ ment requ mum requ han maxin 1, (400mm nm <sup>2</sup>	

	Project	10 Downs	ide Crescent		Job no.	1411
	Calcs for	hoa	m R0 1	Start page no./Revision		
LONDON		Deal				.0. 5
EC1R 1UL	Calcs by ab	Calcs date 12/10/2016	Checked by	Checked date	Approved by	Approved date
Shear lin	ks provided v	alid between 0 m	nm and 6400 n	nm with tension	reinforcemer	nt of 1005 mm <sup>2</sup>
Spacing of reinforcement (cl	3.12.11)					
Actual distance between bars	n tension	s = (b - 2 >	< (Cnom_s + \$\phi_v\$ +	¢ <sub>bot</sub> /2)) /(N <sub>bot</sub> - 1)	- φ <sub>bot</sub> = <b>84</b> mm	
Minimum distance between	bars in tensio	n (cl 3.12.11.1)				
Minimum distance between ba	rs in tension	$S_{min} = N_{agg}$	+ 5 mm = 25 n <i>PA</i>	nm <b>SS - Satisfies th</b>	e minimum s	pacing criteria
Maximum distance between	bars in tensio	on (cl 3.12.11.2)				<b>j</b>
Design service stress		$f_s = (2 \times f_y)$	imes A <sub>s,req</sub> ) / (3 $ imes$ A	$A_{s,prov} \times \beta_b) = 171.$	.6 N/mm²	
Maximum distance between ba	ars in tension	s <sub>max</sub> = min	(47000 N/mm /	( f <sub>s</sub> , 300 mm) = <b>2</b> 7	74 mm	
			PAS	SS - Satisfies the	e maximum s <sub>i</sub>	pacing criteria
Span to depth ratio (cl. 3.4.6)	)					
Basic span to depth ratio (Tab	e 3.9)	span_to_d	lepth <sub>basic</sub> = 20.0	)		
Design service stress in tensio	n reinforcemer	nt $f_s = (2 \times f_y)$	$\times$ A <sub>s,req</sub> )/ (3 $\times$ A	$(s, prov \times \beta_b) = 171.0$	6 N/mm²	
Modification for tension reinfor	cement	$-\min(2,0,0,55)$	$(477N)/mm^2$ f	) / (120 v (0 0N/n	$m^2 + (M / h)$	$(d^{2})))) = 1.303$
Modification for compression re	einforcement	= mm(2.0, 0.55 +	(4//IN/IIIIIIs	) / (120 × (0.914/1)	IIIII <sup>-</sup> + (IVI / (D ×	(u-))))) = <b>1.393</b>
· ·	f <sub>co</sub>	<sub>mp</sub> = min(1.5, 1 + (	$(100 \times A_{s2,prov} / $	$(b \times d)) / (3 + (10))$	$00  imes A_{s2,prov}$ / (b	×d)))) = <b>1.252</b>
Modification for span length		$f_{long} = 1.00$	0			
Allowable span to depth ratio		span_to_d	lepth <sub>allow</sub> = spai	n_to_depth <sub>basic</sub> $\times$	$f_{tens} \times f_{comp} = 34$	4.9
Actual span to depth ratio		span_to_d	$lepth_{actual} = L_{s1}$	/ d = <b>32.2</b>		
		PAS	S - Actual spa	n to depth ratio	is within the a	allowable limit
Support B						
1				5 v 16 + bars		
		•••	1	5 x 10 y bais		
-250-				$2 \times 8\phi$ shear legs a	t 100 c/c	
	L .	• •		5 x 16₀ bars		
•		—500———				
			·			
Rectangular section in flexu	re (cl.3.4.4)					
Design bending moment		M = abs(N	l <sub>B_red</sub> ) = <b>74</b> kNn	n		
Depth to tension reinforcement	t	$d = h - c_{nor}$	m_t -	= <b>199</b> mm		
Redistribution ratio		$\beta_{b} = min(1$	- $m_{rB}$ , 1) = <b>1.0</b>	00		
		K = M / (b	$\times d^2 \times f_{cu}) = 0.0$	)94		
		K' = 0.156		N		
l over arm		$z = \min(d)$	K'>K- √ (0.5 ↓ (0.25		r n reinforcements $r$ $r$ $r$ $r$ $r$	ent is required
Depth of neutral axis		$z = \min(d, z)$	× (0.5 + (0.25 - / 0.45 <b>- 52</b> mm	N / 0.9)**), 0.95	× u) = 175 mm	1
Area of tension reinforcement	required	$\mathbf{X} = (\mathbf{u} - \mathbf{Z})$	(0.45 = 52  mm)	- <b>974</b> mm <sup>2</sup>		
Tension reinforcement provide	d	5 × 16d ba	$(0.07 \land 1y \land Z)$	- 317 11111		
Area of tension reinforcement	- provided	$A_{a prov} = 10$	0 <b>5</b> mm <sup>2</sup>			
Minimum area of reinforcement	t	$A_{s \min} = 0.0$	$0013 \times b \times h = 10000$	<b>163</b> mm²		
	•	/ is,nim = 0.0				

	Project				lob no				
	Filgect	10 Downsi	de Crescent	1411					
RODRIGUES ASSOCIATES	Calcs for			Start page no /Bevision					
1 AMWELL STREET		bean	n B0.1		5.	.6. 6			
LONDON EC1R 1UL	Calcs by ab	Calcs date 12/10/2016	Checked by	Checked date	Approved by	Approved date			
				<b>2</b>					
Maximum area of reinforcement	ητ PASS - Area	$A_{s,max} = 0.0$	4 × b × h = 50 t provided is (	UU mm² greater than are	a of reinforce	nent required			
Pootongular contion in chas	, ,	or removeenen	provided is			nem reganea			
Design shear force span 1 at 6	201 mm	V – abs(mi	n(Vp., vp.)	(1) – <b>59</b> kN					
Design shear stress	201 1111	$v = U / (b \times$	d) = 0.593 N/i	$mm^2$					
Design concrete shear stress		$v_{0} = 0.79 \times 10^{-10}$	$min(3 [100 \times 1])$	$A_{a, rray} / (b \times d) l^{1/3}$	× max(1 (400	$(d)^{1/4}) \times$			
$(\min(f_{m}, 40) / 25)^{1/3} / \gamma_{m}$		ve = 0.70 ×	11111(0,[100 × 7		× max(1, (100				
		$v_{c} = 0.883$	N/mm <sup>2</sup>						
Allowable design shear stress		v <sub>max</sub> = min(0.8 N/mm <sup>2</sup> × (f <sub>cu</sub> /1 N/mm <sup>2</sup> ) <sup>0.5</sup> , 5 N/mm <sup>2</sup> ) = <b>5.000</b> N/mm <sup>2</sup>							
		PASS - Design shear stress is less than maximum allowable							
Value of v from Table 3.7		$0.5 \times v_c < v$	$0.5 \times v_c < v < (v_c + 0.4 \text{ N/mm}^2)$						
Design shear resistance requir	red	$v_s = max(v)$	- v <sub>c</sub> , 0.4 N/mm	<sup>2</sup> ) = <b>0.400</b> N/mm	2				
Area of shear reinforcement re	quired	$A_{sv,req} = v_s \times b / (0.87 \times f_{yv}) = 460 \text{ mm}^2/\text{m}$							
Shear reinforcement provided		$2 \times 8\phi$ legs at 100 c/c							
Area of shear reinforcement p	rovided	A <sub>sv,prov</sub> = <b>1005</b> mm <sup>2</sup> /m							
		PASS - Area of shear reinforcement provided exceeds minimum required							
Maximum longitudinal spacing		Svl,max = 0.7	5 × d = <b>149</b> m	m					
	PASS - Lon	gitudinal spacing	g of shear reir	nforcement prov	vided is less th	han maximum			
Spacing of reinforcement (cl	3.12.11)								
Actual distance between bars	in tension	$s = (b - 2 \times (c_{nom_s} + \phi_v + \phi_{top}/2)) / (N_{top} - 1) - \phi_{top} = 84 \text{ mm}$							
Minimum distance between	bars in tensior	n (cl 3.12.11.1)							
Minimum distance between ba	rs in tension	S <sub>min</sub> = h <sub>agg</sub> +	⊦ 5 mm = <b>25</b> m	ım					
			PA	SS - Satisfies th	e minimum sp	acing criteria			
Maximum distance between	bars in tensio	n (cl 3.12.11.2)							
Design service stress		$f_s = (2 \times f_y >$	$<$ A <sub>s,req</sub> ) / (3 $\times$ A	$A_{s,prov} \times \beta_b) = 323.$	<b>1</b> N/mm <sup>2</sup>				
Maximum distance between ba	ars in tension	s <sub>max</sub> = min(47000 N/mm / f <sub>s</sub> , 300 mm) = <b>145</b> mm							
			PASS - Satisfies the maximum spacing criteria						

![](_page_59_Figure_0.jpeg)

🗾 Tekla	Project			Job no.		
Tedds		10 Downsi		1	411	
RODRIGUES ASSOCIATES	Calcs for			Start page no./F	Revision	
1 AMWELL STREET		bean	n B0.2		5	.7. 2
EC1R 1UL	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	ab	12/10/2016				
		Span 1		Dead x	1 40	
		opanii		Impose	d v 1 60	
		Support B			1 40	
		Support B			1.40 d v 1.60	
				impose	u × 1.60	
Analysis results						
Maximum moment support A		MA_max = -1	<b>08</b> kNm	MA_red =	-108 kNm	
Maximum moment span 1 at 33	809 mm	$M_{s1_max} = 62$	2 kNm	M <sub>s1_red</sub> =	= 62 kNm	
Maximum moment support B		$M_{B_{max}} = -1$	22 kNm	M <sub>B_red</sub> =	-122 kNm	
Maximum shear support A	4 1 4 0 7	V <sub>A_max</sub> = 87	KN	V <sub>A_red</sub> =	87 KN	
Maximum shear support A spar	1 I at 197 mm	$V_{A_{s1}_{max}} =$	84 KIN	VA_s1_red	= 84 KN	
Maximum shear support B	1 -1 0000	$V_{B_{max}} = -10$		V <sub>B_red</sub> =	-106 KN	
Maximum snear support B spar	1 1 at 6203 mm	$VB_s1_max = 1$	- 1UZ KIN	VB_s1_red	= -102 KN	
Maximum reaction at support A	at auropart A	$R_A = 87 \text{ KN}$	7 61			
Unfactored dead load reaction	al support A	$R_{A}Dead = 4$	1 KIN 19 LAN			
Maximum reaction at support R	on at support A	RA_Imposed =				
Linfactored dead load reaction	at support B		n kNi			
Linfactored imposed load reaction	on at support B	BB_Impaged =	14 kN			
		TB_Imposed —				
Flanged section details		b 500 mm	~			
Section width		b = 500 mm	n			
Section depth Maximum flango width		h = 250  mm	m			
Flange depth		b <sub>f</sub> = <b>175</b> m	m			
		III = <b>173</b> III	11			
T .T						
-0-175						
- 1						
75						
<u>▼</u> <u>▲</u>	l					
	<b>∢</b> —200— <b>▶</b>	•	—500——	►	200	
				I	I	
Concrete details						
Concrete strength class		C32/40	2			
Unaracteristic compressive cub	e strength	t <sub>cu</sub> = <b>40</b> N/n	nm <sup>2</sup>			
Modulus of elasticity of concret	e	Ec = 20kN/i	mm² + 200 × f	<sub>cu</sub> = <b>28000</b> N/mm <sup>2</sup>		
Maximum aggregate size		h <sub>agg</sub> = <b>20</b> m	im			
Reinforcement details						
Characteristic yield strength of	reinforcement	$f_y = 500 \text{ N/r}$	mm²			
Characteristic yield strength of	shear reinforcem	ent f <sub>yv</sub> = <b>500</b> N/	/mm²			
Nominal cover to reinforceme	ent					
Nominal cover to top reinforcen	nent	Cnom_t = <b>35</b>	mm			
Nominal cover to bottom reinfo	rcement	Cnom_b = <b>35</b>	mm			
Nominal cover to side reinforce	ment	Cnom_s = <b>35</b>	mm			

![](_page_61_Figure_0.jpeg)

	FTOJECL	Job no.       10 Downside Crescent       141				411			
RODRIGUES ASSOCIATES	Calcs for				DRIGUES ASSOCIATES Calcs for			Start page no./	Revision
1 AMWELL STREET		bean	n B0.2		5	.7. 4			
EC1R 1UL	Calcs by ab	Calcs date 12/10/2016	Checked by	Checked date	Approved by	Approved			
Maximum distance between	bars in tensio	on (cl 3.12.11.2)	•			-			
Design service stress		$f_s = (2 \times f_y >$	$\times$ A <sub>s,req</sub> ) / (3 $\times$ A	$A_{s,prov} \times \beta_b) = 275$	<b>.8</b> N/mm <sup>2</sup>				
Maximum distance between ba	ars in tension	$s_{max} = min($	47000 N/mm /	/ f <sub>s</sub> , 300 mm) = <b>1</b> 7	<b>70</b> mm				
			PAS	SS - Satisfies the	e maximum sj	pacing cri			
Mid span 1									
0		••••		6 x 20 <sub>0</sub> bars					
52									
75				āx 80 sheas	legs at 100 c/c				
-			_						
<b>←</b> _2	00		▶ < 200 - ▶						
Ĭ	I								
Flanged section in flexure -	Positive mom	ent	) <b>60</b>						
Design bending moment		$M = abs(M_s)$	s1_red) = <b>62</b> kNi	n					
Distance between points of ze	ro moment	$L_0 = 0.85 \times$	L <sub>s1</sub> = <b>5440</b> m	m					
Effective flange width		b <sub>eff</sub> = min(0	$0.2 \times I_0 + b, b_f$	= 900 mm					
Depth to tension reinforcement		$d = h - C_{nom}$	_b - φ <sub>v</sub> - φ <sub>bot</sub> / 2	= <b>197</b> mm					
Percentage redistribution		$m_{rs1} = M_{s1_r}$	$m_{rs1} = M_{s1\_red} / M_{s1\_max} - 1 = 0\%$						
Redistribution ratio		$\beta_{b} = min(1)$	$\beta_{\rm b} = \min(1 - m_{\rm rs1}, 1) = 1.000$						
		K = M / (b <sub>ef</sub>	$f \times d^2 \times f_{cu} = 0$	0.045					
		K' = 0.156	= 0.156	No comprossio	n rainfaraam	nt in roa			
Lovor orm		z min/d v	$\mathbf{N} > \mathbf{N}^{-}$		v d) 197 mm	ent is requ			
Dopth of noutral axis		$z = \min(d \times z)/d$	(0.5 + (0.25 - 0.45 -	K / 0.9) <sup>55</sup> ), 0.95	× u) = <b>107</b> mm	1			
Area of tension reinforcement	required	$\mathbf{x} = (\mathbf{u} - \mathbf{z}) / \mathbf{A}$	(0.45 = 23)	I - <b>769</b> mm <sup>2</sup>					
Tonsion reinforcement provide	iequireu ad	$A_{s,req} = M7$	(0.07 × 1y × 2)	= 709 mm					
Area of tension reinforcement	provided	5 × 20φ bai	5 71 mm <sup>2</sup>						
Minimum area of reinforcement	provided	$A_{s,prov} = 15$	/ I IIIII 013 ∨ b ∨ b – '	<b>163</b> mm <sup>2</sup>					
Maximum area of reinforcement	nt	$A_{s,min} = 0.00$	$A_{s,min} = 0.0013 \times b \times n = 163 \text{ mm}^2$						
	DASS - Aroa	As,max = 0.0	t provided is	areater than are	a of reinforce	mont roau			
	1 A00 - Alea	or remorcement	provided is	greater than are		ment requ			
Rectangular section in shea	r								
Shear reinforcement provided	un dala al	2 × 8¢ legs	at 100 c/c						
Area of snear reinforcement p	rovided	$A_{sv,prov} = 10$	105 mm²/m	0.07					
Minimum area of shear reinfor	cement (Table	$3.7)  A_{sv,min} = 0.4$	HN/IIIII <sup>-</sup> × D / (	$(0.87 \times I_{yv}) = 400$	ovoodo mini	mum roau			
Maximum langitudinal anaging		PASS - Area of Si		ment provided	exceeds mini	mum requ			
Maximum longitudinal spacing	PASS - Lor	Svi,max = 0.7	$3 \times 0 = 140$ m	nforcomont prov	vidad is lass t	han mavi			
	FA33 - LUI		$mm^2 \times min(2)$	$100 \times \Lambda \qquad / (b)$	(1000  is its)	1 (400mm			
Design concrete aboar stress		$v_c = 0.79 N/$	nnii × nnin(ə,	$[100 \times \text{As,prov} / (D)]$	× u)] <sup></sup> ) × max(	1, (4001111 mm <sup>2</sup>			
Design concrete shear stress		/ɑ) <sup>,,,</sup> *) × (mi	11(I <sub>cu</sub> , 40IN/MM	-) / ZOIN/ITIM <sup>2</sup> ) <sup>1/3</sup> /	γ <sub>m</sub> = 1.031 IN/	1111-			
Design concrete shear stress	dad	^			100 (				
Design concrete shear stress Design shear resistance provide	ded	$v_{s,prov} = A_{sv}$	$prov \times 0.87 \times f_y$	v / b = <b>0.875</b> N/m	m²				
Design concrete shear stress Design shear resistance provided Design shear stress provided	ded	$v_{s,prov} = A_{sv},$ $v_{prov} = v_{s,prov}$	$prov \times 0.87 \times f_{y}$ $v + V_c = 1.906$	v / b = <b>0.875</b> N/m N/mm²	m²				

(9)							
<b>Tekla</b>	Project	10 Downsi	de Crescent	Job no. 1411			
		TO DOWNSI					
1 AMWELL STREET	Calcs for	hean	5 7 5				
LONDON							
EC1R 1UL	ab	12/10/2016	Спескеа by	Checked date	Approved by	Approved date	
0	-	-1	<u>н</u>	- I	ļ		
Actual distance between bars in	<b>3.12.11)</b> n tension	s = (h - 2 ×	$(C_{nom} + \Phi_i + 0)$		•		
Minimum distance between b		(a) 2 10 11 1)					
Minimum distance between ha	rs in tension	(CI 3.12.11.1) Smin - haag d	5 mm – <b>25</b> m	ım			
			PA:	SS - Satisfies the	e minimum sp	oacing criteria	
Maximum distance between	bars in tension	(cl 3.12.11.2)				0	
Design service stress		$f_s = (2 \times f_v)$	$\langle A_{s reg} \rangle / (3 \times A)$	$(s prov \times \beta_b) = 163.$	<b>1</b> N/mm²		
Maximum distance between ba	rs in tension	$s_{max} = min(4)$	47000 N/mm /	$f_{s}$ , 300 mm) = <b>28</b>	8 mm		
			PAS	S - Satisfies the	e maximum sp	oacing criteria	
Span to depth ratio (cl. 3.4.6)							
Basic span to depth ratio (Table	e 3.9)	span to de	$epth_{basic} = 17.5$				
Design service stress in tension	n reinforcement	$f_s = (2 \times f_y)$	$< A_{s,req})/(3 \times A_{s})$	$\beta_{s,prov} \times \beta_b) = 163.1$	N/mm²		
Modification for tension reinford	ement						
	f <sub>tens</sub> = m	in(2.0, 0.55 + (47	77N/mm² - f <sub>s</sub> ) /	(120 × (0.9N/mn	n <sup>2</sup> + (M / (b <sub>eff</sub> $\times$	d²))))) = <b>1.523</b>	
Modification for compression re	einforcement						
	$f_{comp} = r$	nin(1.5, 1 + (100	$ imes A_{s2,prov}$ / (b <sub>eff</sub>	×d)) / (3 + (100	$ imes A_{s2,prov}$ / (b <sub>eff</sub>	× d)))) = <b>1.262</b>	
Modification for span length		$f_{long} = 1.000$	)				
Allowable span to depth ratio		$span\_to\_depth_{allow} = span\_to\_depth_{basic} \times f_{tens} \times f_{comp} = \textbf{33.6}$					
Actual span to depth ratio		span_to_de	epth <sub>actual</sub> = L <sub>s1</sub> /	d = <b>32.5</b>			
		PASS	S - Actual spai	n to depth ratio	is within the a	llowable limit	
Support B							
<b>1</b>	-			6 x 20 bara			
175				0 × 20φ bais			
				5 00 1			
75	<b>•</b> ••	<u> </u>		₹x 88% heas i	egs at 100 c/c		
<b>†</b>							
<b>∢</b> —20	0	—500———	▶ 200				
Rectangular section in flexur	e (cl.3.4.4)		) <b>100</b> I-NI				
Design bending moment		$M = abs(M_{E})$	B_red) = 122 KN	m 107			
Depth to tension reinforcement		$\mathbf{d} = \mathbf{n} - \mathbf{C}_{\text{nom}}$	_t - Øv - Øtop / 2	= 197 mm			
Redistribution ratio		$\beta_b = min(1)$	$-m_{rB}, 1) = 1.00$	JU 			
		K = M / (b >	$\times d^2 \times t_{cu}$ = 0.1	57			
		K = 0.150	K ~ 1	(' - Compressio	n reinforceme	ont is required	
l ever arm		$z = d \times (0.5)$	5 + (0 25 - K' / )	$(9)^{0.5} = 153 \text{ mm}$		in is required	
Depth of neutral axis		x = (d - z) / (d - z)	0.45 = 98  mm	, ) <b>– 100</b> mm			
Depth of compression reinforce	ement	$d_2 = C_{nom b}$	$+ \phi_v + \phi_{bot} / 2 =$	53 mm			
Area of compression reinforcer	nent reauired	$A_{s2.reg} = (K$	- K') $\times$ f <sub>cu</sub> $\times$ b $\times$	$d^2$ / (0.87 × $f_v$ × (	d - d <sub>2</sub> )) = <b>9</b> mn	1 <sup>2</sup>	
Compression reinforcement pro	ovided	5 × 20¢ bar	'S	- ( ) (			
Area of compression reinforcer	nent provided	$A_{s2,prov} = 15$	5 71 mm <sup>2</sup>				
Maximum area of reinforcemen	it (cl.9.2.1.1(3))	$A_{s,max} = 0.0$	4 × b × h = <b>50</b>	<b>00</b> mm²			
	PASS - Area o	f reinforcement	provided is <u>c</u>	greater than area	a of reinforce	ment required	
Area of tension reinforcement r	equired	$A_{s,req} = K' \times$	$f_{cu} \times b \times d^2 / (0)$	$0.87 \times f_y \times z) + A_s$	<sub>2,req</sub> = <b>1828</b> mr	n <sup>2</sup>	

	Project	10 Downsi	de Crescent	Job no. 1411				
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EC1R 1UL	Calcs by ab	Calcs date 12/10/2016	Checked by	Checked date	Approved by	Approved date		
Tension reinforcement provide	d	6 × 20ø bar	S					
Area of tension reinforcement	provided	A <sub>s,prov</sub> = <b>188</b>	<b>35</b> mm²					
Minimum area of reinforcemen	t (exp.9.1N)	$A_{s,min} = 0.00$	013 × b × h = <b>1</b>	<b>63</b> mm²				
	PASS - Area	of reinforcement	provided is g	reater than area	a of reinforcen	nent required		
Rectangular section in shear								
Design shear force span 1 at 6	203 mm	V = abs(mi	n(V <sub>B s1 max</sub> , V <sub>B</sub>	s1_red)) = <b>102</b> kN				
Design shear stress		$v = V / (b \times$	d) = <b>1.032</b> N/n	nm²				
Design concrete shear stress		$v_c = 0.79 \times$	min(3,[100 × A	$(b \times d)^{1/3}$	× max(1, (400)	/d) <sup>1/4</sup> ) ×		
(min(f <sub>cu</sub> , 40) / 25) <sup>1/3</sup> / γ <sub>m</sub>						, ,		
		v <sub>c</sub> = <b>1.095</b>	V/mm²					
Allowable design shear stress		$v_{max} = min($	$0.8 \text{ N/mm}^2 \times (f_0$	<sub>cu</sub> /1 N/mm <sup>2</sup> ) <sup>0.5</sup> , 5	N/mm <sup>2</sup> ) = <b>5.00</b>	<b>0</b> N/mm²		
5		PASS - Design shear stress is less than maximum allowable						
Value of v from Table 3.7		$0.5 \times v_c < v < (v_c + 0.4 \text{ N/mm}^2)$						
Design shear resistance requir	ed	$v_s = max(v)$	- v <sub>c</sub> , 0.4 N/mm <sup>2</sup>	<sup>2</sup> ) = <b>0.400</b> N/mm	2			
Area of shear reinforcement re	quired	$A_{sv,req} = v_s >$	< b / (0.87 × f <sub>yv</sub> )	) = <b>460</b> mm²/m				
Shear reinforcement provided		2×8¢ legs	at 100 c/c					
Area of shear reinforcement pr	ovided	A <sub>sv,prov</sub> = <b>1005</b> mm <sup>2</sup> /m						
	F	PASS - Area of sl	hear reinforce	ment provided	exceeds minin	num required		
Maximum longitudinal spacing		$S_{vl,max} = 0.7$	5 × d = <b>148</b> mr	n				
	PASS - Long	gitudinal spacing	of shear rein	forcement prov	rided is less th	an maximum		
Spacing of reinforcement (cl	3.12.11)							
Actual distance between bars i	n tension	s = (b - 2 ×	$(C_{nom_s} + \phi_v + \phi_v)$	o <sub>top</sub> /2)) /(N <sub>top</sub> - 1) ·	•			
Minimum distance between b	oars in tension	ı (cl 3.12.11.1)						
Minimum distance between ba	rs in tension	s <sub>min</sub> = h <sub>agg</sub> +	- 5 mm = <b>25</b> m	m				
			PAS	SS - Satisfies th	e minimum sp	acing criteria		
Maximum distance between	bars in tensio	n (cl 3.12.11.2)						
Design service stress		$f_s = (2 \times f_y)$	$<$ A <sub>s,req</sub> ) / (3 $\times$ A	$s, prov  imes \beta_b) = 323.$	<b>3</b> N/mm²			
Maximum distance between ba	rs in tension	$s_{max} = min(47000 \text{ N/mm} / f_s, 300 \text{ mm}) = 145 \text{ mm}$						
		,	PASS - Satisfies the maximum spacing criteria					

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10 Downside Crescent											
Water uplift check			Designed:	ab	<sup>Date:</sup> 11/10/2016			Ckd: _			
·											
	0	A	1	VA/ shi	1				D. J.	1	1
Beam & Load	Span	Area	loads	width	LOC	ation	0		Point	loads	
description		DL	LL		from	to	DL	LL	DL	LL	
	mm	kN/m²	kN/m²	mm	mm	mm	kN/m	kN/m	kN	kN	
Water uplift force	<u>e</u>										
3m high	7000	30.00		7300					1533.0		
water table											
Gravitational loa	d										
Boof dood load	7000	0 60		5000					24.2		
	7000	0.03		0000					24.2		
GF dead load	7000	6.35		6300					280.0		
Basement slab	7000	14.15		/300					/23.1		
Basement wall S	SW	7.20		71500	sqmm				514.8		
тот									1542.1	PASS	
							1		1		1