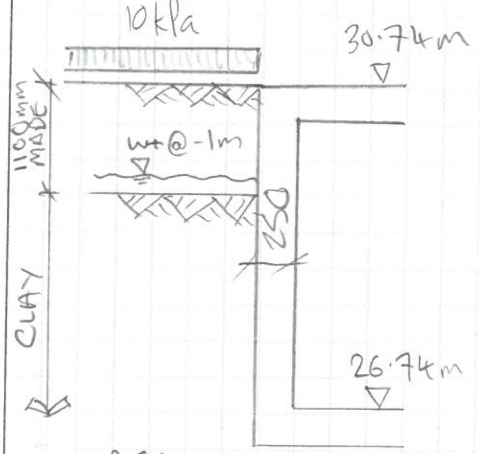
Calculation Sheet

parmarbrook

t. +44 (0)207 839 3999 | f. +44 (0)207 956 2001 | www.parmarbrook.com | 2nd Floor, 345 Old Street, London, EC1V 9LL

PROJECT: 176 PRINCE OF WALES ROAD					
SHEET TITLE: RETAING WALL DESIGN	PROJECT NO.: 1691	DATE: 10.05.17	BY: RSC	CHECKED: BP	SHEET/REV: 0/0

REFERENCE | CALCULATIONS | OUTPUT



$h = 4.000\,m$
$h_{CLEAR} = 3.725\,m$

MADE $\gamma = 1700\ kg/m^3$
$c' = 0$
$\phi' = 27°$

CLAY $\gamma = 1950\ kg/m^3$
$c' = 0$
$\phi' = 24°$
$K_0 = 1 - \sin\phi = 0.6$

ASSUME SIMPLY SUPPORTED WALL

CLAY DEPTH RANGE BETWEEN −0.3m to −1.1m

TAKE −0.3m AS CRITICAL CASE AS ϕ' IS CRITICAL.

LL = 10kPa → ROAD ON WEST

TAKE Ko as WALL DEFLECTION < 0.25%h
= 10mm

$\sigma_v' = \int \gamma\,\delta z + q - u$

CONSERVATIVLEY ASSUME CLAY TO FULL HEIGHT FOR LAT PRESSURE

$\int \gamma\,\delta z = 4 \times 19.5\ kN/m^3 = 78\ kPa$

$q = 10\,kPa$

$u = \gamma_w (z - z_w) = 10 \times 3m = 30\,kPa$ [WATER TO 1m BGL]

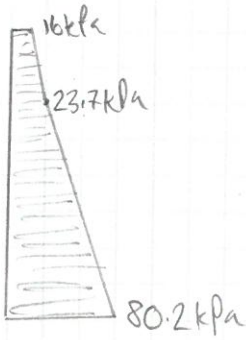
$\sigma_v' = 1.2$ EARTH + WATER + 1.6 LIVE = 57.6 + 16 = 73.6kPa

$\sigma'_{0H} = K_0 \sigma'_v + u = 0.6[73.6] + 1.2[30\,kPa] = 80.2\,kPa$

σ'_{0H} @ 0m BGL = 16kPa

σ'_{0H} @ 1m BGL = 23.7kPa

σ'_{0H} @ 4m BGL = 80.2kPa



$M = 89.7\ kNm/m$
$\tau = 122.4\ kN/m$ $\quad \sigma = 0.49\ MPa$
H16 @ 150 c/c $\quad$ vc = 0.775 < 1.10 OK//
SEE TEDDS

8.1.1 Retaining Walls

The following parameters are suggested for the design of the permanent basement retaining walls.

Stratum	Bulk Density (kg/m^3)	Effective Cohesion (c' – kN/m^2)	Effective Friction Angle (Φ' – degrees)
Made ground	1700	Zero	27
London Clay	1950	Zero	24

Groundwater has been measured at depths of between 0.80 m and 3.55 m and may therefore be encountered during basement excavation. Further groundwater monitoring and trial excavations should be undertaken, but at this stage it is recommended that in order to address the risk of surface water perched groundwater building up behind the retaining walls, a design groundwater level of two-thirds the retained height should be adopted. Reference should be made to BS8102:2009[9] with regard to requirements for waterproofing and design with respect to groundwater pressures.

8.1.2 Basement Heave

Formation level of the 4.7 m deep basement is likely to be within the stiff weathered London Clay, and the excavation will result in a net unloading of around 90 kN/m^2, which will result in elastic heave and long term swelling of the London Clay. The effects of the longer term swelling movement will to a certain extent be counteracted by the applied loads from the development, but further consideration is given to heave movements within the ground movement analysis, in Part 3 of this report.

8.2 Spread Foundations

Moderate width pad or strip foundations excavated from basement level and bearing in the stiff London Clay may be designed to apply a net allowable bearing pressure of 160 kN/m^2. This value incorporates an adequate factor of safety against bearing capacity failure and should ensure that settlement remains within normal tolerable limits.

8.3 Basement Floor Slab

Following the excavation of the basement, it is likely that the floor slab for the proposed basement will need to be suspended over a void to accommodate the anticipated heave, unless the slab can be suitably reinforced to cope with these movements. Further consideration is given to heave movements in Part 3 of this report.

8.4 Shallow Excavations

On the basis of the borehole and trial pit findings it is considered likely that it will be feasible to form relatively shallow excavations for services extending through the made ground without the requirement for lateral support, although localised instabilities may occur. However, should deeper excavations be considered or if excavations are to remain open for prolonged periods it is recommended that provision be made for battered side slopes or lateral support. Where personnel are required to enter excavations, a risk assessment should be carried out and temporary lateral support or battering of the excavation sides considered in order to comply with normal safety requirements.

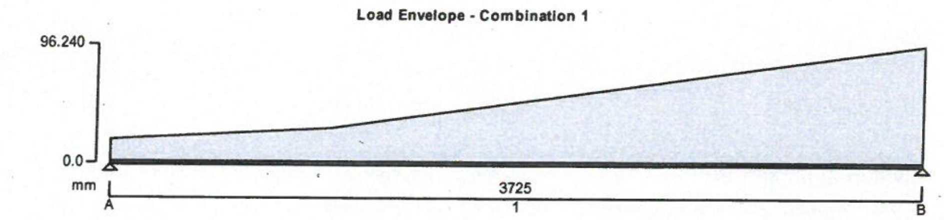
Significant groundwater inflows into shallow excavations are not generally anticipated due to the clayey nature of the underlying soils, although seepages may be encountered from

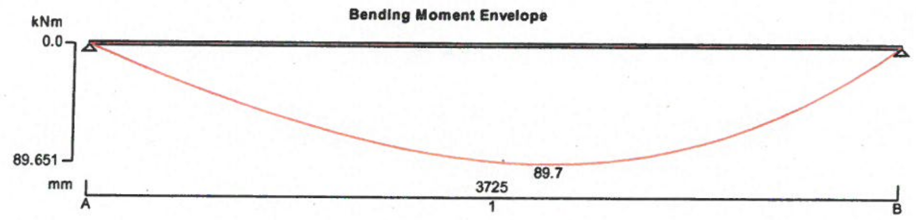
9 BS8102 (2009) *Code of practice for protection of below ground structures against water from the ground*

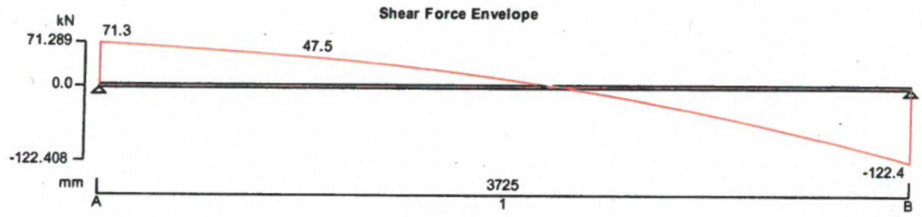
Project				Job no.	
176 Prince of Wales Road				1691	
Calcs for				Start page no./Revision	
Lining Wall Design - With Hydro				1	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
RSC	01/06/2017	BP		CP	

RC BEAM ANALYSIS & DESIGN BS8110

TEDDS calculation version 2.1.12







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Other partial VDL 16 kN/m at 0 mm to 23.7 kN/m at 1000 mm

Other partial VDL 23.7 kN/m at 1000 mm to 80.2 kN/m at 3725 mm

Load combinations

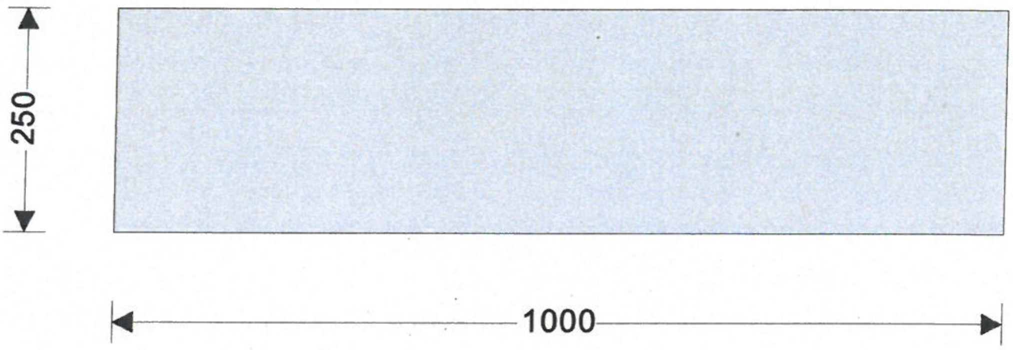
Load combination 1	Support A	Other × 1.20
	Span 1	Other × 1.20
	Support B	Other × 1.20

Analysis results

Maximum moment support A	M_{A_max} = **0** kNm	M_{A_red} = **0** kNm
Maximum moment span 1 at 2120 mm	M_{s1_max} = **90** kNm	M_{s1_red} = **90** kNm
Maximum moment support B	M_{B_max} = **0** kNm	M_{B_red} = **0** kNm
Maximum shear support A	V_{A_max} = **71** kN	V_{A_red} = **71** kN
Maximum shear support A span 1 at 200 mm	$V_{A\_s1\_max}$ = **67** kN	$V_{A\_s1\_red}$ = **67** kN
Maximum shear support B	V_{B_max} = **-122** kN	V_{B_red} = **-122** kN
Maximum shear support B span 1 at 3511 mm	$V_{B\_s1\_max}$ = **-102** kN	$V_{B\_s1\_red}$ = **-102** kN
Maximum reaction at support A	R_A = **71** kN	
Unfactored other load reaction at support A	R_{A_Other} = **59** kN	
Maximum reaction at support B	R_B = **122** kN	
Unfactored other load reaction at support B	R_{B_Other} = **102** kN	

Rectangular section details

Section width	b = **1000** mm
Section depth	h = **250** mm



Concrete details

Concrete strength class	**C32/40**
Characteristic compressive cube strength	f_{cu} = **40** N/mm^2
Modulus of elasticity of concrete	E_c = 20kN/mm^2 + 200 × f_{cu} = **28000** N/mm^2
Maximum aggregate size	h_{agg} = **20** mm

Reinforcement details

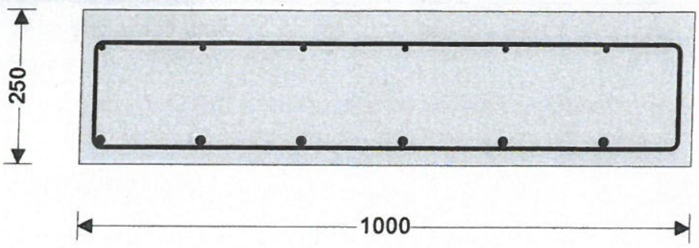
Characteristic yield strength of reinforcement	f_y = **500** N/mm^2
Characteristic yield strength of shear reinforcement	f_{yv} = **500** N/mm^2

Nominal cover to reinforcement

Nominal cover to top reinforcement	c_{nom_t} = **50** mm
Nominal cover to bottom reinforcement	c_{nom_b} = **25** mm
Nominal cover to side reinforcement	c_{nom_s} = **20** mm

Mid span 1

parmarbrook	Project 176 Prince of Wales Road			Job no. 1691	
	Calcs for Lining Wall Design - With Hydro			Start page no./Revision 3	
	Calcs by RSC	Calcs date 01/06/2017	Checked by BP	Checked date	Approved by CP	Approved date



7 x 10ϕ bars

5 x 6ϕ shear legs at 50 c/c

7 x 16ϕ bars

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

Design bending moment	$M = abs(M_{s1\_red})$ = **90** kNm
Depth to tension reinforcement	$d = h - c_{nom\_b} - \phi_v - \phi_{bot} / 2$ = **211** mm
Redistribution ratio	$\beta_b = \min(1 - m_{rs1}, 1)$ = **1.000**
	$K = M / (b \times d^2 \times f_{cu})$ = **0.050**
	$K' = 0.156$

K' > K - No compression reinforcement is required

Lever arm	$z = \min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d)$ = **198** mm
Depth of neutral axis	$x = (d - z) / 0.45$ = **28** mm
Area of tension reinforcement required	$A_{s,req} = M / (0.87 \times f_y \times z)$ = **1039** mm^2
Tension reinforcement provided	7 × 16ϕ bars
Area of tension reinforcement provided	$A_{s,prov}$ = **1340** mm^2
Minimum area of reinforcement	$A_{s,min} = 0.0013 \times b \times h$ = **325** mm^2
Maximum area of reinforcement	$A_{s,max} = 0.04 \times b \times h$ = **10000** mm^2

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

Rectangular section in shear

Shear reinforcement provided	5 × 6ϕ legs at 50 c/c
Area of shear reinforcement provided	$A_{sv,prov}$ = **2827** mm^2/m
Minimum area of shear reinforcement (Table 3.7)	$A_{sv,min} = 0.4\text{N/mm}^2 \times b / (0.87 \times f_{yv})$ = **920** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (cl. 3.4.5.5)	$s_{vl,max} = 0.75 \times d$ = **158** mm

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

Design concrete shear stress	$v_c = 0.79\text{N/mm}^2 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1, (400\text{mm} / d)^{1/4}) \times (\min(f_{cu}, 40\text{N/mm}^2) / 25\text{N/mm}^2)^{1/3} / \gamma_m$ = **0.746** N/mm^2
Design shear resistance provided	$v_{s,prov} = A_{sv,prov} \times 0.87 \times f_{yv} / b$ = **1.230** N/mm^2
Design shear stress provided	$v_{prov} = v_{s,prov} + v_c$ = **1.976** N/mm^2
Design shear resistance	$V_{prov} = v_{prov} \times (b \times d)$ = **416.8** kN

Shear links provided valid between 0 mm and 3725 mm with tension reinforcement of 1340 mm^2

Spacing of reinforcement (cl 3.12.11)

Actual distance between bars in tension	$s = (b - 2 \times (c_{nom\_s} + \phi_v + \phi_{bot}/2)) /(N_{bot} - 1) - \phi_{bot}$ = **148** mm

Minimum distance between bars in tension (cl 3.12.11.1)

Minimum distance between bars in tension	$s_{min} = h_{agg} + 5\text{ mm}$ = **25** mm

PASS - Satisfies the minimum spacing criteria

Maximum distance between bars in tension (cl 3.12.11.2)

Design service stress	$f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b)$ = **258.2** N/mm^2

Maximum distance between bars in tension s_{max} = min(47000 N/mm / f_s, 300 mm) = **182** mm

PASS - Satisfies the maximum spacing criteria

Span to depth ratio (cl. 3.4.6)

Basic span to depth ratio (Table 3.9) $span\_to\_depth_{basic}$ = **20.0**

Design service stress in tension reinforcement $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b)$ = **258.2** N/mm²

Modification for tension reinforcement

f_{tens} = min(2.0, 0.55 + (477N/mm² - f_s) / (120 × (0.9N/mm² + (M / (b × d²))))) = **1.176**

Modification for compression reinforcement

f_{comp} = min(1.5, 1 + (100 × $A_{s2,prov}$ / (b × d)) / (3 + (100 × $A_{s2,prov}$ / (b × d)))) = **1.076**

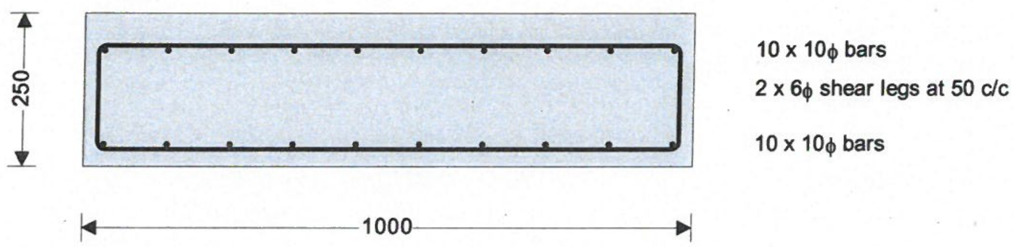
Modification for span length f_{long} = 1.000

Allowable span to depth ratio $span\_to\_depth_{allow} = span\_to\_depth_{basic} \times f_{tens} \times f_{comp}$ = **25.3**

Actual span to depth ratio $span\_to\_depth_{actual} = L_{s1} / d$ = **17.7**

PASS - Actual span to depth ratio is within the allowable limit

<u>Support B</u>



Rectangular section in shear

Design shear force span 1 at 3511 mm V = abs(min($V_{B\_s1\_max}$, $V_{B\_s1\_red}$)) = **102** kN

Design shear stress v = V / (b × d) = **0.476** N/mm²

Design concrete shear stress v_c = 0.79 × min(3,[100 × $A_{s,prov}$ / (b × d)]$^{1/3}$) × max(1, (400 /d)$^{1/4}$) × (min(f_{cu}, 40) / 25)$^{1/3}$ / γ_m

v_c = **0.619** N/mm²

Allowable design shear stress v_{max} = min(0.8 N/mm² × (f_{cu}/1 N/mm²)$^{0.5}$, 5 N/mm²) = **5.000** N/mm²

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7 0.5 × v_c < v < (v_c + 0.4 N/mm²)

Design shear resistance required v_s = max(v - v_c, 0.4 N/mm²) = **0.400** N/mm²

Area of shear reinforcement required $A_{sv,req} = v_s \times b / (0.87 \times f_{yv})$ = **920** mm²/m

Shear reinforcement provided 2 × 6ϕ legs at 50 c/c

Area of shear reinforcement provided $A_{sv,prov}$ = **1131** mm²/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing $s_{vl,max}$ = 0.75 × d = **161** mm

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