8781/L/GS
25th October 2017

Campbell Reith Hill LLP
Friars Bridge Court
41-45 Blackfriars Road
London
SE1 8NZ

To whom it may concern,

Re: 51-52 Tottenham Court Road, London, W1T (Basement Impact Assessment Audit; Project Number: 12336-87 Revision D2)

Many thanks for your report dated October 2017 'Basement Impact Assessment Audit' with regards to the works proposed at the above.

With reference to the comments raised in the report, please be advised that:

1. Ground movement and damage impact assessment.
    1.1. A revised ground movement assessment is being undertaken to ensure that the construction methodology is consistent. The ground movement analysis previously carried out was based on a conservative approach. The inclusion of the piled raft within the assessment is likely to improve ground movement; however, this will be verified in the revised assessment. The inclusion of the 6m deep underpins along the Party Wall to No.53 Tottenham Court Road is likely to improve ground movements; however, this will also be verified in the revised assessment.
    1.2. It has not been possible to confirm the depth of the foundation to the neighbouring building due to access restrictions. This will be verified on site prior to works commencing on site, however a worst-case level has been assumed for the prediction of ground movements.

2. Construction methodology, temporary and permanent works information, retaining wall calculations, foundations assessment.
    2.1. As noted above the construction methodology adopted is in the process of being updated to be consistent throughout the documentation.
    2.2. Permanent works are illustrated on the previously issued structural drawings; initial temporary works proposals are illustrated within the previously issued documentation and will need to be developed by the Contractor prior to commencement on site.
    2.3. Calculations for the retaining walls forming the basement box structure are attached.

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3. Use of resin grouting.
   3.1. Attached is a product data sheet for the resin to be used. We have used this successfully in numerous similar projects around London. The resin is injected at low pressures to prevent the possibility of damage to adjoining structures.

4. Structural monitoring.
   4.1. Attached is our specification for movement monitoring during construction of the underpin walls. The proposals include trigger values to limit movement to Category 1 as required.

The updated ground movement analysis will be submitted upon completion. We trust the enclosed information provides clarity to the comments raised in your report.

Should you require anything further or wish to discuss anything in greater detail, please do not hesitate to get in touch.

Yours faithfully
FOR SINCLAIR JOHNSTON & PARTNERS LIMITED

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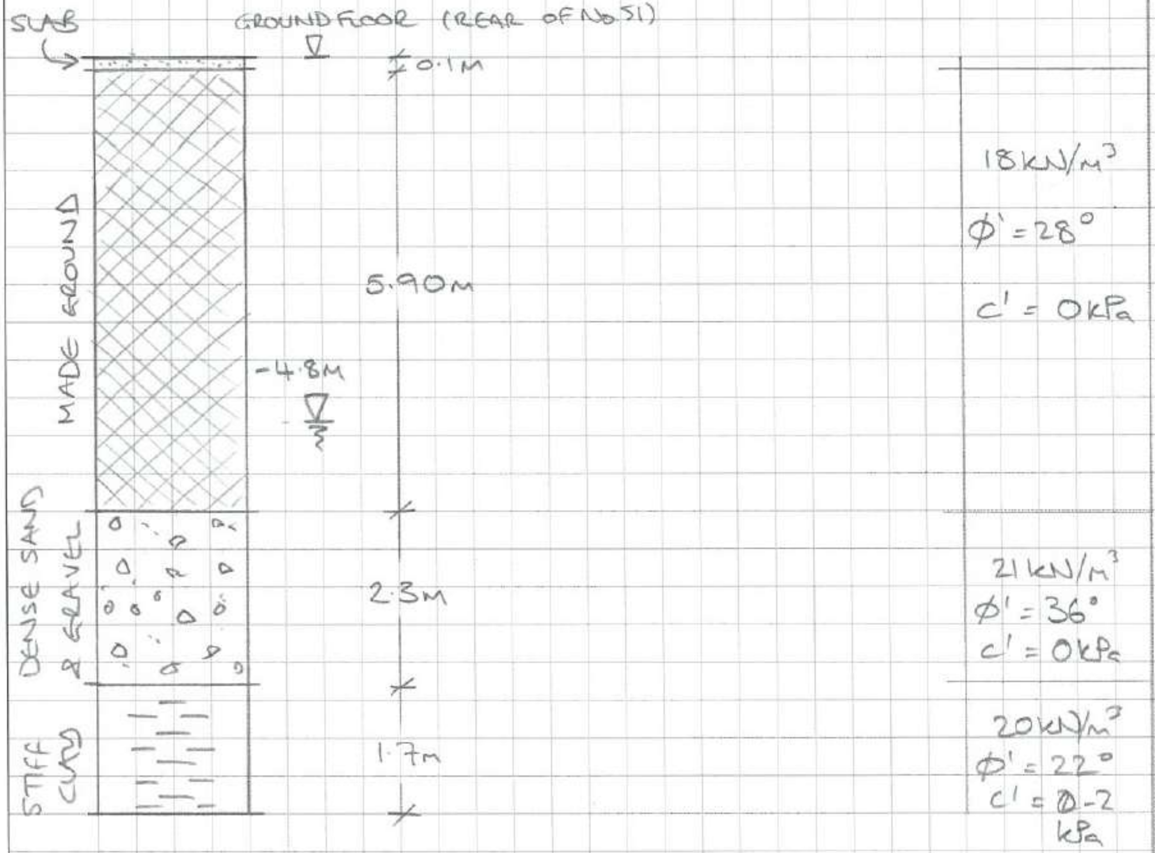
Date 01/11/17	Job No.	Sheet No.
Eng. GS	8781	
Review		
Project 51-52 TOTTENHAM COURT RD		

RETAINING WALL DESIGN

SOIL PROFILE

REFER TO GROUND ENGINEERING REPORT; REF NO. C13604 JAN '16 FOR DETAILS



SJ 2010

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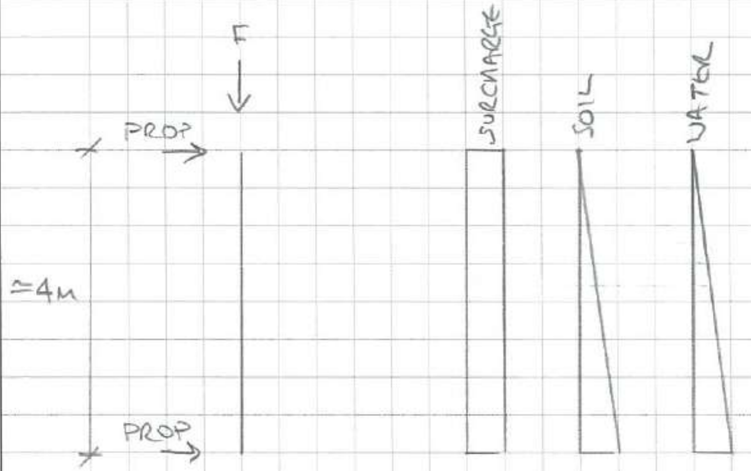
Date 01/11/17	Job No.	Sheet No.
Eng. GS	8781	
Review		
Project 51-52 TOTTENHAM COURT ROAD		

RC UNDERPIN - PW WITH No 53 T.C.R

- NOTE VERTICAL LOAD FROM P/W OVER TAKEN TO GRAVEL VIA MASS CONCRETE UNDERPINNING



F = LOAD FROM FLOORS OVER

SURCHARGE = 5kN/m²
SOIL = 18 kN/m³
WATER = 10 kN/m³

PROP @ HEAD DUE TO METAL DECK }
PROP @ BASE DUE TO RAFT SLAB } ∴ WALL CHECKS NOT REQ. FOR OVERTURNING / SLIDING

FOR LOAD F:
ASSUME 2.5M WIDTH OF LOAD @ EACH FLOOR LEVEL

DEAD LOAD = 160 THK SLAB + FINISHES + SERVICES etc.
$(0.16 \times 25) + (0.075 \times 20) + 1.0 = 6.5 kN/m^2$

LIVE LOAD = $3.0 kN/m^2$

No. FLOORS = 5No. FLOORS + ROOF ∴ SAY 6No.

∴ LOAD = $6.5 \times 6 \times 2.5 + 3 \times 6 \times 2.5$
= 98 kN/m DEAD + 45 kN/m LIVE

VERTICAL LOADS WILL BE TAKEN DOWN TO SUITABLE GROUND VIA RAFT SLAB & PILES ∴ BEARING PRESSURE UNDER FOOTING IS NOT CONSIDERED WITHIN THE CALCS

SJ 2010

RETAINING WALL ANALYSIS

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

Tedds calculation version 2.6.06

Retaining wall details

Stem type	Propped cantilever
Stem height	h_{stem} = **3500** mm
Prop height	h_{prop} = **3500** mm
Stem thickness	t_{stem} = **350** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **2000** mm
Heel length	l_{heel} = **0** mm
Base thickness	t_{base} = **500** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **3500** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **3500** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

Moist density	γ_{mr} = **18** kN/m^3
Saturated density	γ_{sr} = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **28** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **14** deg

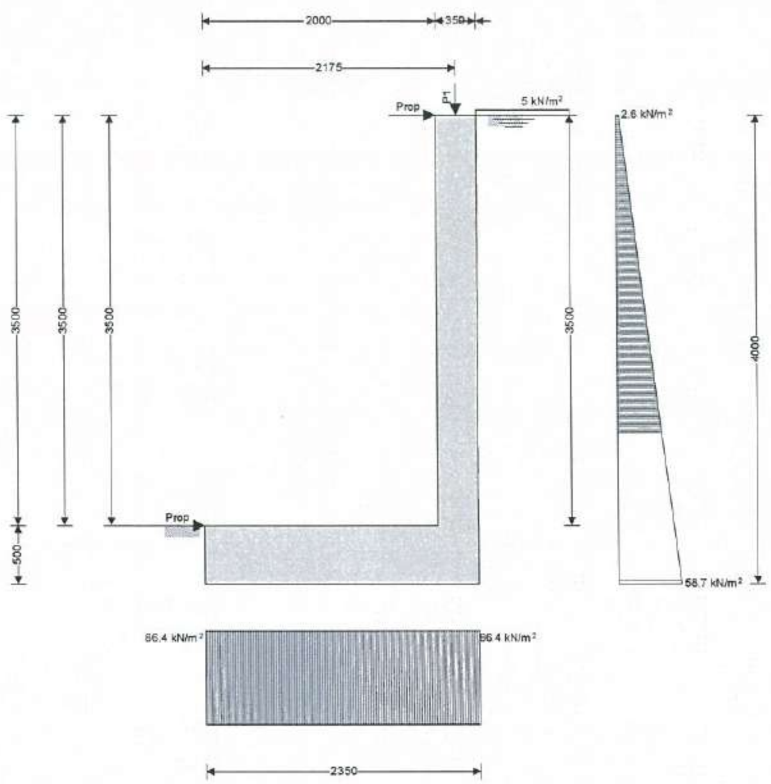
Base soil properties

Soil density	γ_b = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **28** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **14** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **18** deg
Presumed bearing capacity	$P_{bearing}$ = **50** kN/m^2

Loading details

Variable surcharge load	$Surcharge_Q$ = **5** kN/m^2
Vertical line load at 2175 mm	P_{G1} = **98** kN/m
	P_{Q1} = **45** kN/m

Tekla Tedds SinclairJohnston Partners	Project 51-52 Tottenham Court Road			Job no. 8781	
	Calcs for Retaining Wall 1 - PW to No.53			Start page no./Revision 2	
	Calcs by GS	Calcs date 03/11/2017	Checked by	Checked date	Approved by	Approved date



General arrangement

Calculate retaining wall geometry

Base length $l_{base} = l_{toe} + t_{stem} + l_{heel} = \mathbf{2350}$ mm

Saturated soil height $h_{sat} = h_{water} + d_{cover} = \mathbf{3500}$ mm

Moist soil height $h_{moist} = h_{ret} - h_{water} = \mathbf{0}$ mm

Length of surcharge load $l_{sur} = l_{heel} = \mathbf{0}$ mm

- Distance to vertical component $x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{2350}$ mm

Effective height of wall $h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{4000}$ mm

- Distance to horizontal component $x_{sur\_h} = h_{eff} / 2 = \mathbf{2000}$ mm

Area of wall stem $A_{stem} = h_{stem} \times t_{stem} = \mathbf{1.225}$ m^2

- Distance to vertical component $x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{2175}$ mm

Area of wall base $A_{base} = l_{base} \times t_{base} = \mathbf{1.175}$ m^2

- Distance to vertical component $x_{base} = l_{base} / 2 = \mathbf{1175}$ mm

Area of saturated soil $A_{sat} = h_{sat} \times l_{heel} = \mathbf{0}$ m^2

- Distance to vertical component $x_{sat\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} = \mathbf{2350}$ mm

- Distance to horizontal component $x_{sat\_h} = (h_{sat} + h_{base}) / 3 = \mathbf{1333}$ mm

Area of water $A_{water} = h_{sat} \times l_{heel} = \mathbf{0}$ m^2

- Distance to vertical component $x_{water\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} = \mathbf{2350}$ mm

- Distance to horizontal component $x_{water\_h} = (h_{sat} + h_{base}) / 3 = \mathbf{1333}$ mm

Using Coulomb theory

At rest pressure coefficient $K_0 = 1 - \sin(\phi'_{r.k}) = \mathbf{0.531}$

Passive pressure coefficient $K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) = \mathbf{4.325}$

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{30.6}$ kN/m
Wall base	$F_{base} = A_{base} \times \gamma_{base} = \mathbf{29.4}$ kN/m
Surcharge load	$F_{sur\_v} = Surcharge_Q \times l_{heel} = \mathbf{0}$ kN/m
Line loads	$F_{P\_v} = P_{G1} + P_{Q1} = \mathbf{143}$ kN/m
Saturated retained soil	$F_{sat\_v} = A_{sat} \times (\gamma_{sr}' - \gamma_w') = \mathbf{0}$ kN/m
Water	$F_{water\_v} = A_{water} \times \gamma_w' = \mathbf{0}$ kN/m
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{sat\_v} + F_{water\_v} + F_{sur\_v} + F_{P\_v} = \mathbf{203}$ kN/m

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_0 \times \cos(\delta_{r.d}) \times Surcharge_Q \times h_{eff} = \mathbf{10.3}$ kN/m
Saturated retained soil	$F_{sat\_h} = K_0 \times \cos(\delta_{r.d}) \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2 = \mathbf{33.7}$ kN/m
Water	$F_{water\_h} = \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{78.5}$ kN/m
Base soil	$F_{pass\_h} = -K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-9.4}$ kN/m
Total	$F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h} = \mathbf{113.1}$ kN/m

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem} = \mathbf{66.6}$ kNm/m
Wall base	$M_{base} = F_{base} \times x_{base} = \mathbf{34.5}$ kNm/m
Surcharge load	$M_{sur} = F_{sur\_v} \times x_{sur\_v} - F_{sur\_h} \times x_{sur\_h} = \mathbf{-20.6}$ kNm/m
Line loads	$M_P = (P_{G1} + P_{Q1}) \times p_1 = \mathbf{311}$ kNm/m
Saturated retained soil	$M_{sat} = F_{sat\_v} \times x_{sat\_v} - F_{sat\_h} \times x_{sat\_h} = \mathbf{-45}$ kNm/m
Water	$M_{water} = F_{water\_v} \times x_{water\_v} - F_{water\_h} \times x_{water\_h} = \mathbf{-104.6}$ kNm/m
Total	$M_{total} = M_{stem} + M_{base} + M_{sat} + M_{water} + M_{sur} + M_P = \mathbf{242}$ kNm/m

Check bearing pressure

Propping force to stem	$F_{prop\_stem} = (F_{total\_v} \times l_{base} / 2 - M_{total}) / (h_{prop} + t_{base}) = \mathbf{-0.9}$ kN/m
Propping force to base	$F_{prop\_base} = F_{total\_h} - F_{prop\_stem} = \mathbf{113.9}$ kN/m
Moment from propping force	$M_{prop} = F_{prop\_stem} \times (h_{prop} + t_{base}) = \mathbf{-3.4}$ kNm/m
Distance to reaction	$\bar{x} = (M_{total} + M_{prop}) / F_{total\_v} = \mathbf{1175}$ mm
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2 = \mathbf{0}$ mm
Loaded length of base	$l_{load} = l_{base} = \mathbf{2350}$ mm
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) = \mathbf{86.4}$ kN/m^2
Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) = \mathbf{86.4}$ kN/m^2
Factor of safety	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{0.579}$

FAIL - Maximum applied bearing pressure exceeds allowable bearing pressure

ALL LOADS TAKEN TO SUITABLE BEARING STRATA VIA PILED RAFT

RETAINING WALL DESIGN

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

Tedds calculation version 2.6.06

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

Concrete strength class	C32/40
Characteristic compressive cylinder strength	$f_{ck} = \mathbf{32}$ N/mm^2
Characteristic compressive cube strength	$f_{ck,cube} = \mathbf{40}$ N/mm^2

Mean value of compressive cylinder strength $f_{cm} = f_{ck} + 8\ \text{N/mm}^2 = \mathbf{40}\ \text{N/mm}^2$

Mean value of axial tensile strength $f_{ctm} = 0.3\ \text{N/mm}^2 \times (f_{ck} / 1\ \text{N/mm}^2)^{2/3} = \mathbf{3.0}\ \text{N/mm}^2$

5% fractile of axial tensile strength $f_{ctk,0.05} = 0.7 \times f_{ctm} = \mathbf{2.1}\ \text{N/mm}^2$

Secant modulus of elasticity of concrete $E_{cm} = 22\ \text{kN/mm}^2 \times (f_{cm} / 10\ \text{N/mm}^2)^{0.3} = \mathbf{33346}\ \text{N/mm}^2$

Partial factor for concrete - Table 2.1N $\gamma_C = \mathbf{1.50}$

Compressive strength coefficient - cl.3.1.6(1) $\alpha_{cc} = \mathbf{0.85}$

Design compressive concrete strength - exp.3.15 $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{18.1}\ \text{N/mm}^2$

Maximum aggregate size $h_{agg} = \mathbf{20}\ \text{mm}$

Reinforcement details

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

Modulus of elasticity of reinforcement $E_s = \mathbf{200000}\ \text{N/mm}^2$

Partial factor for reinforcing steel - Table 2.1N $\gamma_S = \mathbf{1.15}$

Design yield strength of reinforcement $f_{yd} = f_{yk} / \gamma_S = \mathbf{435}\ \text{N/mm}^2$

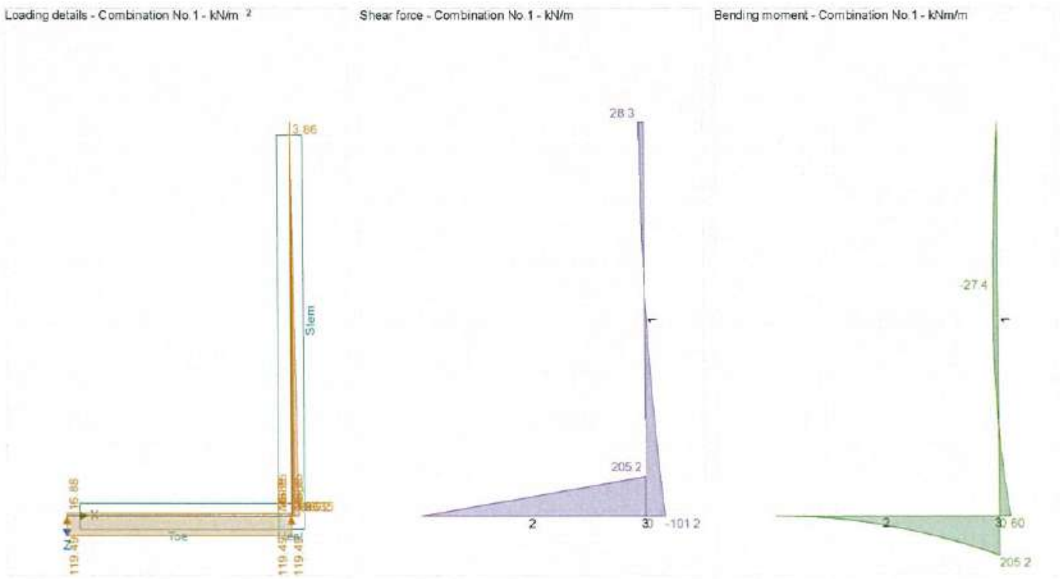
Cover to reinforcement

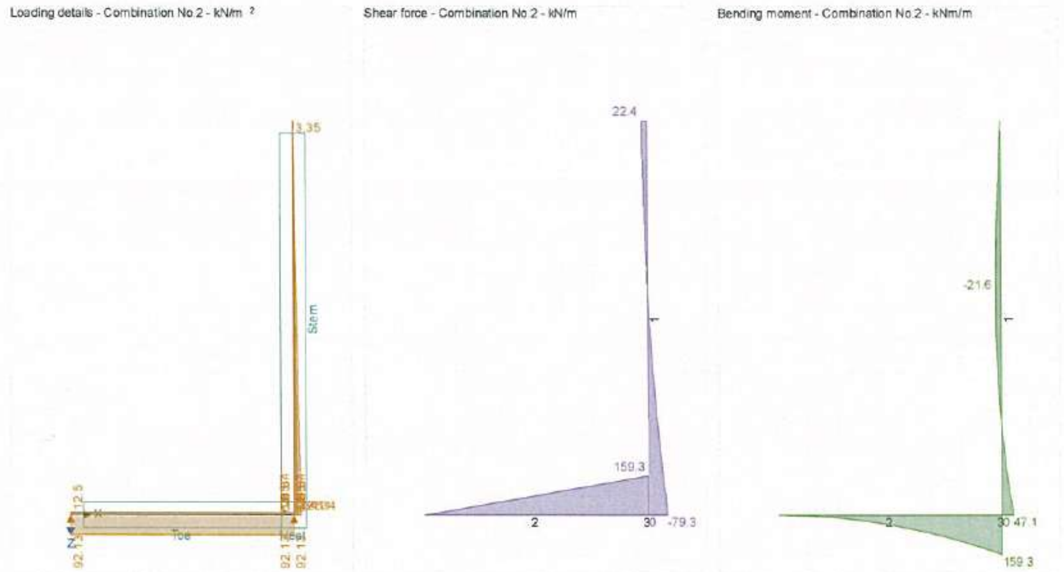
Front face of stem $c_{sf} = \mathbf{35}\ \text{mm}$

Rear face of stem $c_{sr} = \mathbf{75}\ \text{mm}$

Top face of base $c_{bt} = \mathbf{50}\ \text{mm}$

Bottom face of base $c_{bb} = \mathbf{75}\ \text{mm}$





Check stem design at 1964 mm

Depth of section — $h = \mathbf{350}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — $M = \mathbf{27.4}$ kNm/m

Depth to tension reinforcement — $d = h - c_{sf} - \phi_{sx} - \phi_{sfM} / 2 = \mathbf{295}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.010}$

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d = \mathbf{280}$ mm

Depth of neutral axis — $x = 2.5 \times (d - z) = \mathbf{37}$ mm

Area of tension reinforcement required — $A_{sfM.req} = M / (f_{yd} \times z) = \mathbf{225}$ mm²/m

Tension reinforcement provided — 16 dia.bars @ 200 c/c

Area of tension reinforcement provided — $A_{sfM.prov} = \pi \times \phi_{sfM}^2 / (4 \times s_{sfM}) = \mathbf{1005}$ mm²/m

Minimum area of reinforcement - exp.9.1N — $A_{sfM.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{464}$ mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{sfM.max} = 0.04 \times h = \mathbf{14000}$ mm²/m

$\max(A_{sfM.req}, A_{sfM.min}) / A_{sfM.prov} = \mathbf{0.461}$

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

Deflection control - Section 7.4

Reference reinforcement ratio — $\rho_0 = \sqrt{(f_{ck} / 1\ N/mm^2)} / 1000 = \mathbf{0.006}$

Required tension reinforcement ratio — $\rho = A_{sfM.req} / d = \mathbf{0.001}$

Required compression reinforcement ratio — $\rho' = A_{sfM.2.req} / d_2 = \mathbf{0.000}$

Structural system factor - Table 7.4N — $K_b = \mathbf{1}$

Reinforcement factor - exp.7.17 — $K_s = \min(500\ N/mm^2 / (f_{yk} \times A_{sfM.req} / A_{sfM.prov}), 1.5) = \mathbf{1.5}$

Limiting span to depth ratio - exp.7.16.a — $K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}] = \mathbf{552.1}$

Actual span to depth ratio — $h_{prop} / d = \mathbf{11.9}$

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width — $w_{max} = \mathbf{0.3}$ mm

Variable load factor - EN1990 – Table A1.1	ψ_2 = **0.6**
Serviceability bending moment	M_{sls} = **19.2** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sfM.prov} \times z)$ = **68.2** N/mm^2
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h – x) / 3, h / 2)$ = **104375** mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **3.0** N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{sfM.prov} / A_{c.eff}$ = **0.010**
Modular ratio	$\alpha_e = E_s / E_{cm}$ = **5.998**
Bond property coefficient	k_1 = **0.8**
Strain distribution coefficient	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{sf} + k_1 \times k_2 \times k_4 \times \phi_{sfM} / \rho_{p.eff}$ = **401** mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	w_k = **0.082** mm
	w_k / w_{max} = **0.274**

PASS - Maximum crack width is less than limiting crack width

Check stem design at base of stem

Depth of section	h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	M = **60** kNm/m
Depth to tension reinforcement	$d = h - c_{sr} - \phi_{sr} / 2$ = **267** mm
	$K = M / (d^2 \times f_{ck})$ = **0.026**
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **254** mm
Depth of neutral axis	$x = 2.5 \times (d – z)$ = **33** mm
Area of tension reinforcement required	$A_{sr.req} = M / (f_{yd} \times z)$ = **544** mm^2/m
Tension reinforcement provided	16 dia.bars @ 200 c/c
Area of tension reinforcement provided	$A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr})$ = **1005** mm^2/m
Minimum area of reinforcement - exp.9.1N	$A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **420** mm^2/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{sr.max} = 0.04 \times h$ = **14000** mm^2/m
	$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov}$ = **0.541**

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

Deflection control - Section 7.4

Reference reinforcement ratio	$\rho_0 = \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} / 1000$ = **0.006**
Required tension reinforcement ratio	$\rho = A_{sr.req} / d$ = **0.002**
Required compression reinforcement ratio	$\rho' = A_{sr.2.req} / d_2$ = **0.000**
Structural system factor - Table 7.4N	K_b = **1**
Reinforcement factor - exp.7.17	$K_s = \min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5)$ = **1.5**
Limiting span to depth ratio - exp.7.16.a	$K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times (\rho_0 / \rho - 1)^{3/2}]$ = **116**
Actual span to depth ratio	h_{prop} / d = **13.1**

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width: w_{max} = **0.3** mm

Variable load factor - EN1990 – Table A1.1: ψ_2 = **0.6**

Serviceability bending moment: M_{sls} = **42.5** kNm/m

Tensile stress in reinforcement: $\sigma_s = M_{sls} / (A_{sr.prov} \times z)$ = **166.5** N/mm²

Load duration: Long term

Load duration factor: k_t = **0.4**

Effective area of concrete in tension: $A_{c.eff} = \min(2.5 \times (h - d), (h – x) / 3, h / 2)$ = **105542** mm²/m

Mean value of concrete tensile strength: $f_{ct.eff} = f_{ctm}$ = **3.0** N/mm²

Reinforcement ratio: $\rho_{p.eff} = A_{sr.prov} / A_{c.eff}$ = **0.010**

Modular ratio: $\alpha_e = E_s / E_{cm}$ = **5.998**

Bond property coefficient: k_1 = **0.8**

Strain distribution coefficient: k_2 = **0.5**

k_3 = **3.4**

k_4 = **0.425**

Maximum crack spacing - exp.7.11: $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff}$ = **541** mm

Maximum crack width - exp.7.8: $w_k = s_{r.max} \times \max(\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

w_k = **0.27** mm

w_k / w_{max} = **0.9**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force: V = **101.2** kN/m

$C_{Rd.c} = 0.18 / \gamma_C$ = **0.120**

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.865**

Longitudinal reinforcement ratio: $\rho_l = \min(A_{sr.prov} / d, 0.02)$ = **0.004**

$v_{min} = 0.035 \text{ N}^{1/2}\text{/mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.504** N/mm²

Design shear resistance - exp.6.2a & 6.2b: $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2\text{/mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c}$ = **137** kN/m

$V / V_{Rd.c}$ = **0.739**

PASS - Design shear resistance exceeds design shear force

Check stem design at prop

Depth of section: h = **350** mm

Rectangular section in shear - Section 6.2

Design shear force: V = **28.3** kN/m

$C_{Rd.c} = 0.18 / \gamma_C$ = **0.120**

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.865**

Longitudinal reinforcement ratio: $\rho_l = \min(A_{sr1.prov} / d, 0.02)$ = **0.004**

$v_{min} = 0.035 \text{ N}^{1/2}\text{/mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.504** N/mm²

Design shear resistance - exp.6.2a & 6.2b: $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2\text{/mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c}$ = **137** kN/m

$V / V_{Rd.c}$ = **0.206**

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1): $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **350** mm²/m

Maximum spacing of reinforcement – cl.9.6.3(2): s_{sx_max} = **400** mm

Transverse reinforcement provided: 12 dia.bars @ 200 c/c

Area of transverse reinforcement provided $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx}) = \mathbf{565}$ mm²/m

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

Check base design at toe

Depth of section $h = \mathbf{500}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 $M = \mathbf{205.2}$ kNm/m

Depth to tension reinforcement $d = h - c_{bb} - \phi_{bb} / 2 = \mathbf{412}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.038}$

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d = \mathbf{392}$ mm

Depth of neutral axis $x = 2.5 \times (d - z) = \mathbf{52}$ mm

Area of tension reinforcement required $A_{bb.req} = M / (f_{yd} \times z) = \mathbf{1205}$ mm²/m

Tension reinforcement provided 25 dia.bars @ 200 c/c

Area of tension reinforcement provided $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = \mathbf{2454}$ mm²/m

Minimum area of reinforcement - exp.9.1N $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{649}$ mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{bb.max} = 0.04 \times h = \mathbf{20000}$ mm²/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov} = \mathbf{0.491}$

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

Crack control - Section 7.3

Limiting crack width $w_{max} = \mathbf{0.3}$ mm

Variable load factor - EN1990 – Table A1.1 $\psi_2 = \mathbf{0.6}$

Serviceability bending moment $M_{sls} = \mathbf{147.8}$ kNm/m

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{bb.prov} \times z) = \mathbf{153.6}$ N/mm²

Load duration Long term

Load duration factor $k_t = \mathbf{0.4}$

Effective area of concrete in tension $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2) = \mathbf{149479}$ mm²/m

Mean value of concrete tensile strength $f_{ct.eff} = f_{ctm} = \mathbf{3.0}$ N/mm²

Reinforcement ratio $\rho_{p.eff} = A_{bb.prov} / A_{c.eff} = \mathbf{0.016}$

Modular ratio $\alpha_e = E_s / E_{cm} = \mathbf{5.998}$

Bond property coefficient $k_1 = \mathbf{0.8}$

Strain distribution coefficient $k_2 = \mathbf{0.5}$

$k_3 = \mathbf{3.4}$

$k_4 = \mathbf{0.425}$

Maximum crack spacing - exp.7.11 $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff} = \mathbf{514}$ mm

Maximum crack width - exp.7.8 $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

$w_k = \mathbf{0.237}$ mm

$w_k / w_{max} = \mathbf{0.789}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force $V = \mathbf{205.2}$ kN/m

$C_{Rd,c} = 0.18 / \gamma_C = \mathbf{0.120}$

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{1.696}$

Longitudinal reinforcement ratio $\rho_l = \min(A_{bb.prov} / d, 0.02) = \mathbf{0.006}$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.437}$ N/mm²

Design shear resistance - exp.6.2a & 6.2b $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c}$ = **224.2** kN/m

$V / V_{Rd.c}$ = **0.915**

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

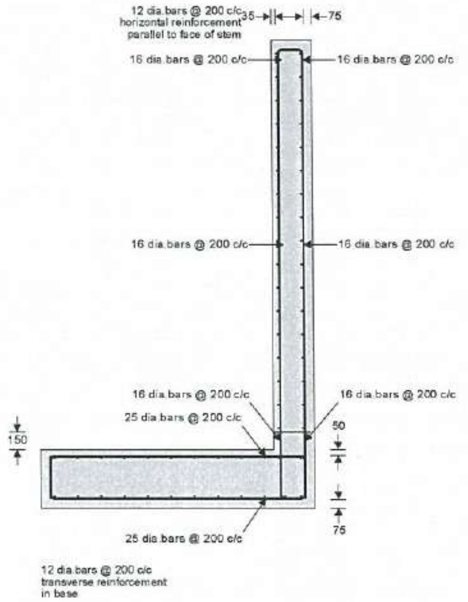
Minimum area of reinforcement – cl.9.3.1.1(2) $A_{bx.req} = 0.2 \times A_{bb.prov}$ = **491** mm²/m

Maximum spacing of reinforcement – cl.9.3.1.1(3) s_{bx_max} = **450** mm

Transverse reinforcement provided 12 dia.bars @ 200 c/c

Area of transverse reinforcement provided $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **565** mm²/m

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



Reinforcement details

SINCLAIR JOHNSTON & PARTNERS LIMITED	Date 01/11/17	Job No.	Sheet No.
	Eng. GS	8781	
	Review		
	Project 51-52 TOTTENHAM COURT ROAD		

<u>RC UNDERPIN – FLANK WALL TO NO 49 TCR.</u>

No 49 = GROUND → 3RD + ROOF

∴ ≈ 3.2m x 4 STOREYS
= 12.8m BRICK

ASSUME 330 THK

$= 12.8 \times 4.6 \times \frac{330}{215} = 90\,kN/m$

ASSUME FOUNDATIONS OF NO. 49 = 0.5m BEL

No. 49 IS APPROX 2.5m FROM REAR FACE OF WALL

∴ SURCHARGE ON BACK OF WALL =

$5\,kN/m^2 + \frac{90\,kN/m^2}{(2.5 \times 2)} = 23\,kN/m^2$

(ASSUMING 45° SPREAD)

VERTICAL LOAD = ~~3~~ AS APPLIED TO B1
+ BRICK WALL

BRICKWALL = 215THK (6NO STOREYS)

$= 4.6 \times 6 \times 3.2 = 90\,kN/m$

HEEL ON WALL TO MATCH EXISTING. ASSUME = 80mm
(MINIMUM)

REFER TO TEDDS OUTPUT

SJ 2010

RETAINING WALL ANALYSIS

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

Tedds calculation version 2.6.06

Retaining wall details

Stem type	Propped cantilever
Stem height	h_{stem} = **3500** mm
Prop height	h_{prop} = **3500** mm
Stem thickness	t_{stem} = **450** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **2000** mm
Heel length	l_{heel} = **80** mm
Base thickness	t_{base} = **500** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **3500** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **3500** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

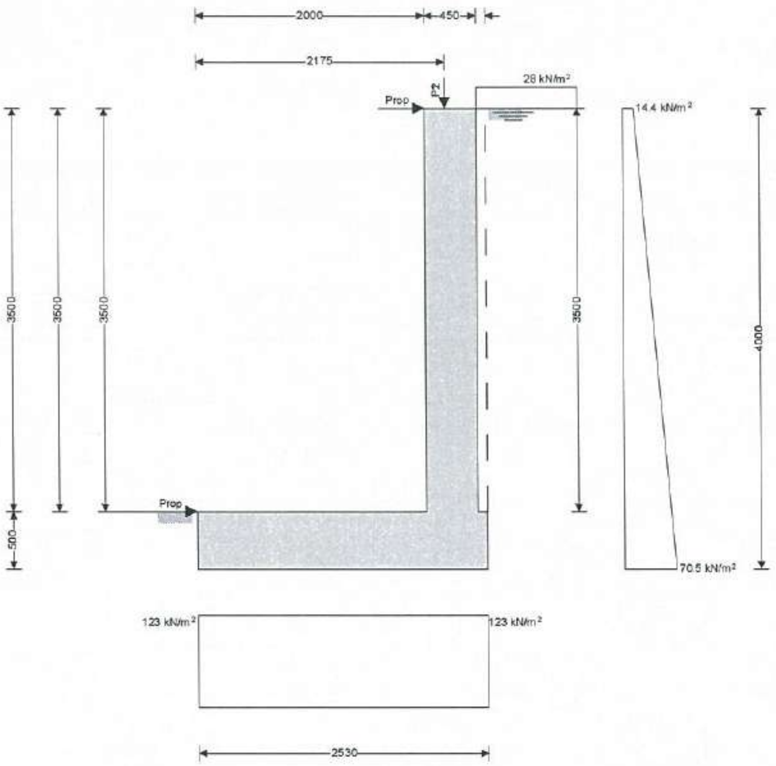
Moist density	γ_{mr} = **18** kN/m^3
Saturated density	γ_{sr} = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **28** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **14** deg

Base soil properties

Soil density	γ_b = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **28** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **14** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **18** deg
Presumed bearing capacity	$P_{bearing}$ = **50** kN/m^2

Loading details

Permanent surcharge load	$Surcharge_G$ = **23** kN/m^2
Variable surcharge load	$Surcharge_Q$ = **5** kN/m^2
Vertical line load at 2175 mm	P_{G1} = **98** kN/m
	P_{Q1} = **45** kN/m
Vertical line load at 2175 mm	P_{G2} = **90** kN/m



General arrangement

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} + l_{heel} = \mathbf{2530}$ mm
Saturated soil height	$h_{sat} = h_{water} + d_{cover} = \mathbf{3500}$ mm
Moist soil height	$h_{moist} = h_{ret} - h_{water} = \mathbf{0}$ mm
Length of surcharge load	$l_{sur} = l_{heel} = \mathbf{80}$ **mm**
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{2490}$ mm
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{4000}$ mm
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 = \mathbf{2000}$ **mm**
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{1.575}$ m^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{2225}$ mm
Area of wall base	$A_{base} = l_{base} \times t_{base} = \mathbf{1.265}$ m^2
- Distance to vertical component	$x_{base} = l_{base} / 2 = \mathbf{1265}$ mm
Area of saturated soil	$A_{sat} = h_{sat} \times l_{heel} = \mathbf{0.28}$ **m^2**
- Distance to vertical component	$x_{sat\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} = \mathbf{2490}$ **mm**
- Distance to horizontal component	$x_{sat\_h} = (h_{sat} + h_{base}) / 3 = \mathbf{1333}$ mm
Area of water	$A_{water} = h_{sat} \times l_{heel} = \mathbf{0.28}$ m^2
- Distance to vertical component	$x_{water\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} = \mathbf{2490}$ mm
- Distance to horizontal component	$x_{water\_h} = (h_{sat} + h_{base}) / 3 = \mathbf{1333}$ mm

Using Coulomb theory

At rest pressure coefficient	$K_0 = 1 - \sin(\phi'_{r.k}) = \mathbf{0.531}$
Passive pressure coefficient	$K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) = \mathbf{4.325}$

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem}$ = **39.4** kN/m

Wall base $F_{base} = A_{base} \times \gamma_{base}$ = **31.6** kN/m

Surcharge load $F_{sur\_v} = (Surcharge_G + Surcharge_Q) \times l_{heel}$ = **2.2** kN/m

Line loads $F_{P\_v} = P_{G1} + P_{Q1} + P_{G2}$ = **233** kN/m

Saturated retained soil $F_{sat\_v} = A_{sat} \times (\gamma_{sr}' - \gamma_w')$ = **2.3** kN/m

Water $F_{water\_v} = A_{water} \times \gamma_w'$ = **2.7** kN/m

Total $F_{total\_v} = F_{stem} + F_{base} + F_{sat\_v} + F_{water\_v} + F_{sur\_v} + F_{P\_v}$ = **311.3** kN/m

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_0 \times \cos(\delta_{r.d}) \times (Surcharge_G + Surcharge_Q) \times h_{eff}$ = **57.7** kN/m

Saturated retained soil $F_{sat\_h} = K_0 \times \cos(\delta_{r.d}) \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2$ = **33.7** kN/m

Water $F_{water\_h} = \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2$ = **78.5** kN/m

Base soil $F_{pass\_h} = -K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2$ = **-9.4** kN/m

Total $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h}$ = **160.4** kN/m

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem}$ = **87.6** kNm/m

Wall base $M_{base} = F_{base} \times x_{base}$ = **40** kNm/m

Surcharge load $M_{sur} = F_{sur\_v} \times x_{sur\_v} - F_{sur\_h} \times x_{sur\_h}$ = **-109.7** kNm/m

Line loads $M_P = (P_{G1} + P_{Q1}) \times p_1 + (P_{G2}) \times p_2$ = **506.8** kNm/m

Saturated retained soil $M_{sat} = F_{sat\_v} \times x_{sat\_v} - F_{sat\_h} \times x_{sat\_h}$ = **-39.3** kNm/m

Water $M_{water} = F_{water\_v} \times x_{water\_v} - F_{water\_h} \times x_{water\_h}$ = **-97.8** kNm/m

Total $M_{total} = M_{stem} + M_{base} + M_{sat} + M_{water} + M_{sur} + M_P$ = **387.6** kNm/m

Check bearing pressure

Propping force to stem $F_{prop\_stem} = (F_{total\_v} \times l_{base} / 2 - M_{total}) / (h_{prop} + t_{base})$ = **1.5** kN/m

Propping force to base $F_{prop\_base} = F_{total\_h} - F_{prop\_stem}$ = **158.9** kN/m

Moment from propping force $M_{prop} = F_{prop\_stem} \times (h_{prop} + t_{base})$ = **6.2** kNm/m

Distance to reaction $\bar{x} = (M_{total} + M_{prop}) / F_{total\_v}$ = **1265** mm

Eccentricity of reaction $e = \bar{x} - l_{base} / 2$ = **0** mm

Loaded length of base $l_{load} = l_{base}$ = **2530** mm

Bearing pressure at toe $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base})$ = **123** kN/m²

Bearing pressure at heel $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base})$ = **123** kN/m²

Factor of safety $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel})$ = **0.406**

FAIL - Maximum applied bearing pressure exceeds allowable bearing pressure

ALL LOADS TAKEN TO SUITABLE BEARING STRATA VIA PILED RAFT

RETAINING WALL DESIGN

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

Tedds calculation version 2.6.06

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

Concrete strength class C32/40

Characteristic compressive cylinder strength f_{ck} = **32** N/mm²

Characteristic compressive cube strength $f_{ck,cube}$ = **40** N/mm²

Mean value of compressive cylinder strength $f_{cm} = f_{ck} + 8$ N/mm² = **40** N/mm²

Mean value of axial tensile strength $f_{ctm} = 0.3$ N/mm² $\times (f_{ck} / 1$ N/mm²$)^{2/3}$ = **3.0** N/mm²

5% fractile of axial tensile strength	$f_{ctk,0.05} = 0.7 \times f_{ctm} = \mathbf{2.1}$ N/mm^2
Secant modulus of elasticity of concrete	$E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3} = \mathbf{33346}$ N/mm^2
Partial factor for concrete - Table 2.1N	$\gamma_C = \mathbf{1.50}$
Compressive strength coefficient - cl.3.1.6(1)	$\alpha_{cc} = \mathbf{0.85}$
Design compressive concrete strength - exp.3.15	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{18.1}$ N/mm^2
Maximum aggregate size	$h_{agg} = \mathbf{20}$ mm

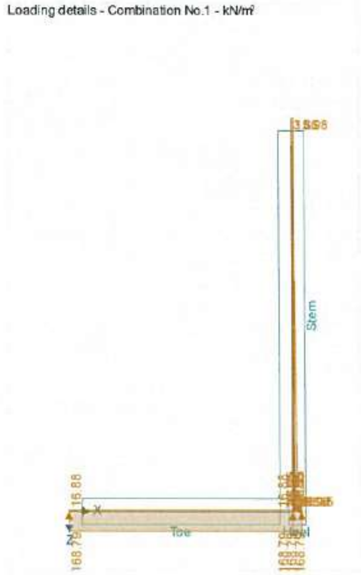
Reinforcement details

Characteristic yield strength of reinforcement	$f_{yk} = \mathbf{500}$ N/mm^2
Modulus of elasticity of reinforcement	$E_s = \mathbf{200000}$ N/mm^2
Partial factor for reinforcing steel - Table 2.1N	$\gamma_S = \mathbf{1.15}$
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_S = \mathbf{435}$ N/mm^2

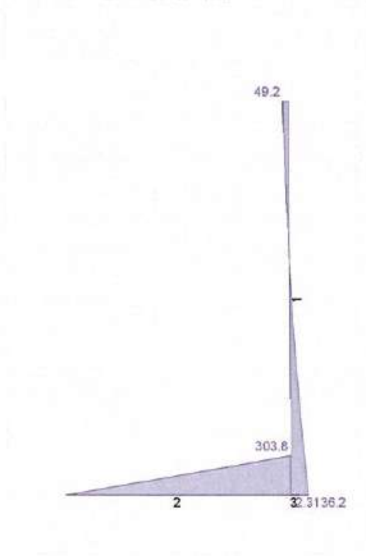
Cover to reinforcement

Front face of stem	$c_{sf} = \mathbf{35}$ mm
Rear face of stem	$c_{sr} = \mathbf{75}$ mm
Top face of base	$c_{bt} = \mathbf{50}$ mm
Bottom face of base	$c_{bb} = \mathbf{75}$ mm

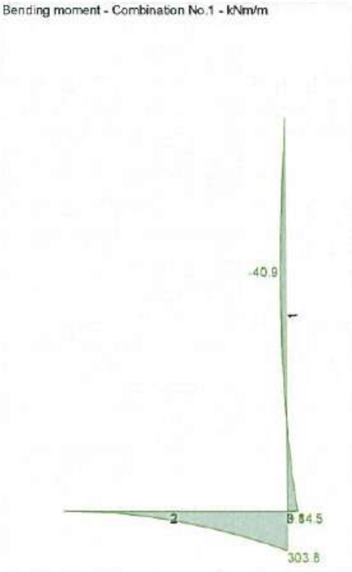
Loading details - Combination No.1 - kN/m²



Shear force - Combination No.1 - kN/m



Bending moment - Combination No.1 - kNm/m



Check stem design at 2038 mm

Depth of section	$h = \mathbf{450}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	$M = \mathbf{40.9}$ kNm/m
Depth to tension reinforcement	$d = h - c_{sf} - \phi_{sx} - \phi_{sfM} / 2 = \mathbf{393}$ mm
	$K = M / (d^2 \times f_{ck}) = \mathbf{0.008}$
	$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d = \mathbf{373}$ mm
Depth of neutral axis	$x = 2.5 \times (d - z) = \mathbf{49}$ mm
Area of tension reinforcement required	$A_{sfM.req} = M / (f_{yd} \times z) = \mathbf{252}$ mm^2/m
Tension reinforcement provided	20 dia.bars @ 200 c/c

Area of tension reinforcement provided	$A_{sfM.prov} = \pi \times \phi_{sfM}^2 / (4 \times s_{sfM})$ = **1571** mm²/m
Minimum area of reinforcement - exp.9.1N	$A_{sfM.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **618** mm²/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{sfM.max} = 0.04 \times h$ = **18000** mm²/m
	$\max(A_{sfM.req}, A_{sfM.min}) / A_{sfM.prov}$ = **0.393**

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

Deflection control - Section 7.4

Reference reinforcement ratio	$\rho_0 = \sqrt{(f_{ck} / 1 N/mm^2)} / 1000$ = **0.006**
Required tension reinforcement ratio	$\rho = A_{sfM.req} / d$ = **0.001**
Required compression reinforcement ratio	$\rho' = A_{sfM.2.req} / d_2$ = **0.000**
Structural system factor - Table 7.4N	K_b = **1**
Reinforcement factor - exp.7.17	$K_s = \min(500 N/mm^2 / (f_{yk} \times A_{sfM.req} / A_{sfM.prov}), 1.5)$ = **1.5**
Limiting span to depth ratio - exp.7.16.a	$K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1 N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1 N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}]$ = **722.4**
Actual span to depth ratio	h_{prop} / d = **8.9**

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width	w_{max} = **0.3** mm
Variable load factor - EN1990 – Table A1.1	ψ_2 = **0.6**
Serviceability bending moment	M_{sls} = **29.2** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sfM.prov} \times z)$ = **49.8** N/mm²
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **133625** mm²/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **3.0** N/mm²
Reinforcement ratio	$\rho_{p.eff} = A_{sfM.prov} / A_{c.eff}$ = **0.012**
Modular ratio	$\alpha_e = E_s / E_{cm}$ = **5.998**
Bond property coefficient	k_1 = **0.8**
Strain distribution coefficient	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{sf} + k_1 \times k_2 \times k_4 \times \phi_{sfM} / \rho_{p.eff}$ = **408** mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	w_k = **0.061** mm
	w_k / w_{max} = **0.203**

PASS - Maximum crack width is less than limiting crack width

Check stem design at base of stem

Depth of section	h = **450** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	M = **84.5** kNm/m
Depth to tension reinforcement	$d = h - c_{sr} - \phi_{sr} / 2$ = **365** mm
	$K = M / (d^2 \times f_{ck})$ = **0.020**
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **347** mm
Depth of neutral axis	$x = 2.5 \times (d - z)$ = **46** mm

Area of tension reinforcement required — $A_{sr.req} = M / (f_{yd} \times z) = \mathbf{561}$ mm²/m

Tension reinforcement provided — 20 dia.bars @ 200 c/c

Area of tension reinforcement provided — $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = \mathbf{1571}$ mm²/m

Minimum area of reinforcement - exp.9.1N — $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{574}$ mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{sr.max} = 0.04 \times h = \mathbf{18000}$ mm²/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} = \mathbf{0.365}$

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

Deflection control - Section 7.4

Reference reinforcement ratio — $\rho_0 = \sqrt{(f_{ck} / 1\ N/mm^2)} / 1000 = \mathbf{0.006}$

Required tension reinforcement ratio — $\rho = A_{sr.req} / d = \mathbf{0.002}$

Required compression reinforcement ratio — $\rho' = A_{sr.2.req} / d_2 = \mathbf{0.000}$

Structural system factor - Table 7.4N — $K_b = \mathbf{1}$

Reinforcement factor - exp.7.17 — $K_s = \min(500\ N/mm^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5) = \mathbf{1.5}$

Limiting span to depth ratio - exp.7.16.a — $K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}] = \mathbf{182.7}$

Actual span to depth ratio — $h_{prop} / d = \mathbf{9.6}$

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width — $w_{max} = \mathbf{0.3}$ mm

Variable load factor - EN1990 – Table A1.1 — $\psi_2 = \mathbf{0.6}$

Serviceability bending moment — $M_{sls} = \mathbf{60.6}$ kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{sr.prov} \times z) = \mathbf{111.2}$ N/mm²

Load duration — Long term

Load duration factor — $k_t = \mathbf{0.4}$

Effective area of concrete in tension — $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2) = \mathbf{134792}$ mm²/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm} = \mathbf{3.0}$ N/mm²

Reinforcement ratio — $\rho_{p.eff} = A_{sr.prov} / A_{c.eff} = \mathbf{0.012}$

Modular ratio — $\alpha_e = E_s / E_{cm} = \mathbf{5.998}$

Bond property coefficient — $k_1 = \mathbf{0.8}$

Strain distribution coefficient — $k_2 = \mathbf{0.5}$

$k_3 = \mathbf{3.4}$

$k_4 = \mathbf{0.425}$

Maximum crack spacing - exp.7.11 — $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} = \mathbf{547}$ mm

Maximum crack width - exp.7.8 — $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

$w_k = \mathbf{0.182}$ mm

$w_k / w_{max} = \mathbf{0.608}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force — $V = \mathbf{136.2}$ kN/m

$C_{Rd,c} = 0.18 / \gamma_C = \mathbf{0.120}$

$k = \min(1 + \sqrt{(200\ mm / d)}, 2) = \mathbf{1.740}$

Longitudinal reinforcement ratio — $\rho_l = \min(A_{sr.prov} / d, 0.02) = \mathbf{0.004}$

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.455}$ N/mm²

Design shear resistance - exp.6.2a & 6.2b — $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c} = \mathbf{182.7}$ kN/m

$V / V_{Rd.c} = \mathbf{0.745}$

PASS - Design shear resistance exceeds design shear force

Check stem design at prop

Depth of section — h = **450** mm

Rectangular section in shear - Section 6.2

Design shear force — V = **49.2** kN/m

$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**

$k = \min(1 + \sqrt{(200\text{ mm} / d)}, 2)$ = **1.740**

Longitudinal reinforcement ratio — $\rho_l = \min(A_{sr1.prov} / d, 0.02)$ = **0.004**

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5}$ = **0.455** N/mm²

Design shear resistance - exp.6.2a & 6.2b — $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c}$ = **182.7** kN/m

$V / V_{Rd.c}$ = **0.270**

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1) — $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **450** mm²/m

Maximum spacing of reinforcement – cl.9.6.3(2) — s_{sx_max} = **400** mm

Transverse reinforcement provided — 12 dia.bars @ 200 c/c

Area of transverse reinforcement provided — $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **565** mm²/m

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

Check base design at toe

Depth of section — h = **500** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — M = **303.8** kNm/m

Depth to tension reinforcement — $d = h - c_{bb} - \phi_{bb} / 2$ = **409** mm

$K = M / (d^2 \times f_{ck})$ = **0.057**

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **387** mm

Depth of neutral axis — $x = 2.5 \times (d - z)$ = **54** mm

Area of tension reinforcement required — $A_{bb.req} = M / (f_{yd} \times z)$ = **1804** mm²/m

Tension reinforcement provided — 32 dia.bars @ 100 c/c

Area of tension reinforcement provided — $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **8042** mm²/m

Minimum area of reinforcement - exp.9.1N — $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **643** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{bb.max} = 0.04 \times h$ = **20000** mm²/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov}$ = **0.224**

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

Crack control - Section 7.3

Limiting crack width — w_{max} = **0.3** mm

Variable load factor - EN1990 – Table A1.1 — ψ_2 = **0.6**

Serviceability bending moment — M_{sls} = **221.1** kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **71** N/mm²

Load duration — Long term

Load duration factor — k_t = **0.4**

Effective area of concrete in tension — $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **148641** mm²/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm}$ = **3.0** N/mm²

Reinforcement ratio $\rho_{p.eff} = A_{bb.prov} / A_{c.eff} = \mathbf{0.054}$

Modular ratio $\alpha_e = E_s / E_{cm} = \mathbf{5.998}$

Bond property coefficient $k_1 = \mathbf{0.8}$

Strain distribution coefficient $k_2 = \mathbf{0.5}$

$k_3 = \mathbf{3.4}$

$k_4 = \mathbf{0.425}$

Maximum crack spacing - exp.7.11 $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff} = \mathbf{356}$ mm

Maximum crack width - exp.7.8 $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

$w_k = \mathbf{0.076}$ mm

$w_k / w_{max} = \mathbf{0.252}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force $V = \mathbf{303.8}$ kN/m

$C_{Rd,c} = 0.18 / \gamma_C = \mathbf{0.120}$

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{1.699}$

Longitudinal reinforcement ratio $\rho_l = \min(A_{bb.prov} / d, 0.02) = \mathbf{0.020}$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.439}$ N/mm²

Design shear resistance - exp.6.2a & 6.2b $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c} = \mathbf{331.7}$ kN/m

$V / V_{Rd.c} = \mathbf{0.916}$

PASS - Design shear resistance exceeds design shear force

Check base design at heel

Depth of section $h = \mathbf{500}$ mm

Rectangular section in shear - Section 6.2

Design shear force $V = \mathbf{2.3}$ kN/m

$C_{Rd,c} = 0.18 / \gamma_C = \mathbf{0.120}$

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{1.699}$

Longitudinal reinforcement ratio $\rho_l = \min(A_{bt.prov} / d, 0.02) = \mathbf{0.008}$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.439}$ N/mm²

Design shear resistance - exp.6.2a & 6.2b $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c} = \mathbf{242.5}$ kN/m

$V / V_{Rd.c} = \mathbf{0.009}$

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

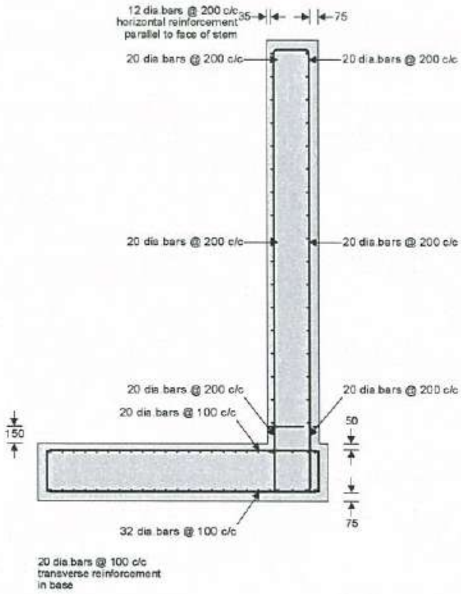
Minimum area of reinforcement – cl.9.3.1.1(2) $A_{bx.req} = 0.2 \times A_{bb.prov} = \mathbf{1608}$ mm²/m

Maximum spacing of reinforcement – cl.9.3.1.1(3) $s_{bx\_max} = \mathbf{450}$ mm

Transverse reinforcement provided 20 dia.bars @ 100 c/c

Area of transverse reinforcement provided $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = \mathbf{3142}$ mm²/m

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



Reinforcement details

SINCLAIR JOHNSTON
& PARTNERS LIMITED

Date 01/11/17	Job No.	Sheet No.
Eng. ES	8781	
Review		
Project 51-52 TOTTENHAM COURT ROAD		

RC UNDERPIN - REAR WALL

BUILDING TO THE REAR OF THE SITE

ASSUME 5No. STOREYS - TBC ON SITE.

330THK BRICK GROUND → 3RD
215THK BRICK 3RD → 4TH
TIMBER 4TH → ROOF

$\therefore \left(3.2 \times 3 \times 4.6 \times \frac{330}{215}\right) + (3.2 \times 1 \times 4.6) + (3.2 \times 1 \times 1)$

$= 68 + 15 + 3.2$
$= 86\ kN/m^2$

- ASSUME DIRECTLY BEHIND OUR RC. STRUCTURE (CONSERVATIVE)

VERTICAL LOAD = GROUND → 3RD FLOOR (215 BRICK)
$= 4.6 \times 3.2 \times 3 = 44\ kN/m$

+ FLOOR LOAD (= 1m WIDE STRIP)
$= (6.5 \times 4 \times 1) + (3 \times 4 \times 1)$
$= 26\ kN/m + 12\ kN/m$

REFER TO TEDDS OUTPUT

RETAINING WALL ANALYSIS

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

Tedds calculation version 2.6.06

Retaining wall details

Stem type	Propped cantilever
Stem height	h_{stem} = **3500** mm
Prop height	h_{prop} = **3500** mm
Stem thickness	t_{stem} = **450** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **2000** mm
Heel length	l_{heel} = **80** mm
Base thickness	t_{base} = **500** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **3500** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **3500** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

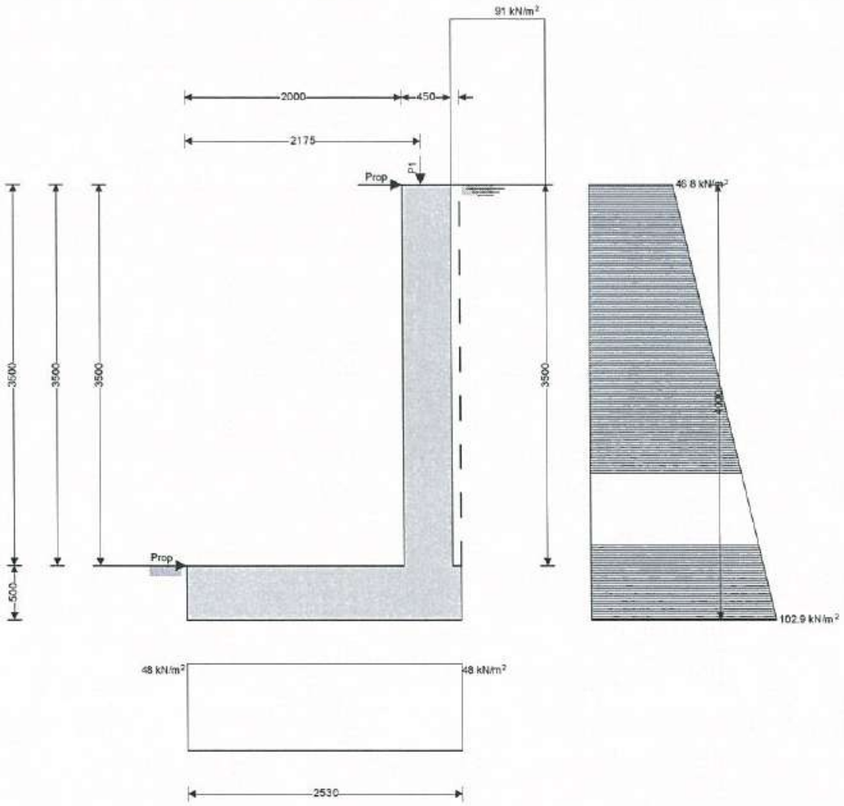
Moist density	γ_{mr} = **18** kN/m^3
Saturated density	γ_{sr} = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **28** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **14** deg

Base soil properties

Soil density	γ_b = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **28** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **14** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **18** deg
Presumed bearing capacity	$P_{bearing}$ = **50** kN/m^2

Loading details

Permanent surcharge load	$Surcharge_G$ = **86** kN/m^2
Variable surcharge load	$Surcharge_Q$ = **5** kN/m^2
Vertical line load at 2175 mm	P_{G1} = **26** kN/m
	P_{Q1} = **12** kN/m



General arrangement

Calculate retaining wall geometry

Base length $l_{base} = l_{toe} + t_{stem} + l_{heel} = \mathbf{2530}$ mm

Saturated soil height $h_{sat} = h_{water} + d_{cover} = \mathbf{3500}$ mm

Moist soil height $h_{moist} = h_{ret} - h_{water} = \mathbf{0}$ mm

Length of surcharge load $l_{sur} = l_{heel} = \mathbf{80}$ mm

- Distance to vertical component $x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{2490}$ mm

Effective height of wall $h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{4000}$ mm

- Distance to horizontal component $x_{sur\_h} = h_{eff} / 2 = \mathbf{2000}$ mm

Area of wall stem $A_{stem} = h_{stem} \times t_{stem} = \mathbf{1.575}$ m^2

- Distance to vertical component $x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{2225}$ mm

Area of wall base $A_{base} = l_{base} \times t_{base} = \mathbf{1.265}$ m^2

- Distance to vertical component $x_{base} = l_{base} / 2 = \mathbf{1265}$ mm

Area of saturated soil $A_{sat} = h_{sat} \times l_{heel} = \mathbf{0.28}$ m^2

- Distance to vertical component $x_{sat\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} = \mathbf{2490}$ mm

- Distance to horizontal component $x_{sat\_h} = (h_{sat} + h_{base}) / 3 = \mathbf{1333}$ mm

Area of water $A_{water} = h_{sat} \times l_{heel} = \mathbf{0.28}$ m^2

- Distance to vertical component $x_{water\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} = \mathbf{2490}$ mm

- Distance to horizontal component $x_{water\_h} = (h_{sat} + h_{base}) / 3 = \mathbf{1333}$ mm

Using Coulomb theory

At rest pressure coefficient $K_0 = 1 - \sin(\phi'_{r.k}) = \mathbf{0.531}$

Passive pressure coefficient $K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) = \mathbf{4.325}$

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem}$ = **39.4 kN/m**
Wall base	$F_{base} = A_{base} \times \gamma_{base}$ = **31.6 kN/m**
Surcharge load	$F_{sur\_v} = (Surcharge_G + Surcharge_Q) \times l_{heel}$ = **7.3 kN/m**
Line loads	$F_{P\_v} = P_{G1} + P_{Q1}$ = **38 kN/m**
Saturated retained soil	$F_{sat\_v} = A_{sat} \times (\gamma_{sr}' - \gamma_w')$ = **2.3 kN/m**
Water	$F_{water\_v} = A_{water} \times \gamma_w'$ = **2.7 kN/m**
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{sat\_v} + F_{water\_v} + F_{sur\_v} + F_{P\_v}$ = **121.3 kN/m**

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_0 \times \cos(\delta_{r.d}) \times (Surcharge_G + Surcharge_Q) \times h_{eff}$ = **187.4 kN/m**
Saturated retained soil	$F_{sat\_h} = K_0 \times \cos(\delta_{r.d}) \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2$ = **33.7 kN/m**
Water	$F_{water\_h} = \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2$ = **78.5 kN/m**
Base soil	$F_{pass\_h} = -K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2$ = **-9.4 kN/m**
Total	$F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h}$ = **290.1 kN/m**

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem}$ = **87.6 kNm/m**
Wall base	$M_{base} = F_{base} \times x_{base}$ = **40 kNm/m**
Surcharge load	$M_{sur} = F_{sur\_v} \times x_{sur\_v} - F_{sur\_h} \times x_{sur\_h}$ = **-356.6 kNm/m**
Line loads	$M_P = (P_{G1} + P_{Q1}) \times p_1$ = **82.7 kNm/m**
Saturated retained soil	$M_{sat} = F_{sat\_v} \times x_{sat\_v} - F_{sat\_h} \times x_{sat\_h}$ = **-39.3 kNm/m**
Water	$M_{water} = F_{water\_v} \times x_{water\_v} - F_{water\_h} \times x_{water\_h}$ = **-97.8 kNm/m**
Total	$M_{total} = M_{stem} + M_{base} + M_{sat} + M_{water} + M_{sur} + M_P$ = **-283.4 kNm/m**

Check bearing pressure

Propping force to stem	$F_{prop\_stem} = (F_{total\_v} \times l_{base} / 2 - M_{total}) / (h_{prop} + t_{base})$ = **109.2 kN/m**
Propping force to base	$F_{prop\_base} = F_{total\_h} - F_{prop\_stem}$ = **180.9 kN/m**
Moment from propping force	$M_{prop} = F_{prop\_stem} \times (h_{prop} + t_{base})$ = **436.9 kNm/m**
Distance to reaction	$\bar{x} = (M_{total} + M_{prop}) / F_{total\_v}$ = **1265 mm**
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2$ = **0 mm**
Loaded length of base	$l_{load} = l_{base}$ = **2530 mm**
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base})$ = **48 kN/m²**
Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base})$ = **48 kN/m²**
Factor of safety	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel})$ = **1.043**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

ALL LOADS TAKEN TO SUITABLE BEARING STRATA VIA PILED RAFT

RETAINING WALL DESIGN

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

Tedds calculation version 2.6.06

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

Concrete strength class	C32/40
Characteristic compressive cylinder strength	f_{ck} = **32 N/mm²**
Characteristic compressive cube strength	$f_{ck,cube}$ = **40 N/mm²**
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8\ N/mm^2$ = **40 N/mm²**
Mean value of axial tensile strength	$f_{ctm} = 0.3\ N/mm^2 \times (f_{ck} / 1\ N/mm^2)^{2/3}$ = **3.0 N/mm²**

5% fractile of axial tensile strength $f_{ctk,0.05} = 0.7 \times f_{ctm} = \mathbf{2.1}$ N/mm²

Secant modulus of elasticity of concrete $E_{cm} = 22$ kN/mm² $\times (f_{cm} / 10$ N/mm²$)^{0.3} = \mathbf{33346}$ N/mm²

Partial factor for concrete - Table 2.1N $\gamma_C = \mathbf{1.50}$

Compressive strength coefficient - cl.3.1.6(1) $\alpha_{cc} = \mathbf{0.85}$

Design compressive concrete strength - exp.3.15 $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{18.1}$ N/mm²

Maximum aggregate size $h_{agg} = \mathbf{20}$ mm

Reinforcement details

Characteristic yield strength of reinforcement $f_{yk} = \mathbf{500}$ N/mm²

Modulus of elasticity of reinforcement $E_s = \mathbf{200000}$ N/mm²

Partial factor for reinforcing steel - Table 2.1N $\gamma_S = \mathbf{1.15}$

Design yield strength of reinforcement $f_{yd} = f_{yk} / \gamma_S = \mathbf{435}$ N/mm²

Cover to reinforcement

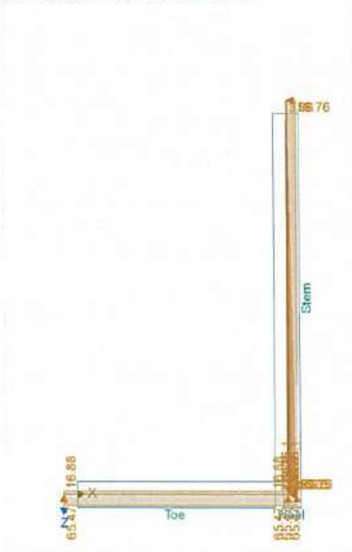
Front face of stem $c_{sf} = \mathbf{35}$ mm

Rear face of stem $c_{sr} = \mathbf{75}$ mm

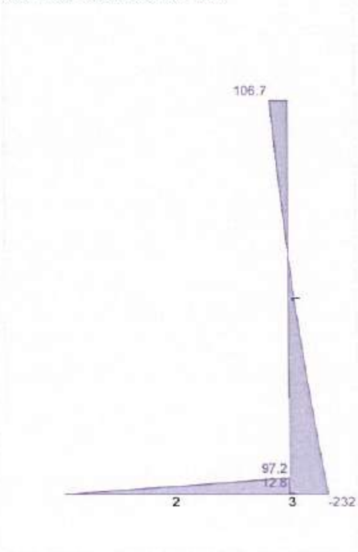
Top face of base $c_{bt} = \mathbf{50}$ mm

Bottom face of base $c_{bb} = \mathbf{75}$ mm

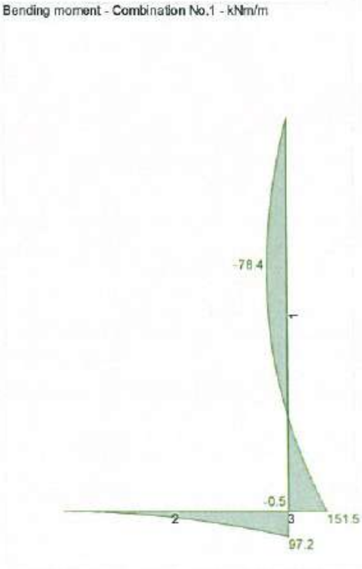
Loading details - Combination No.1 - kN/m²

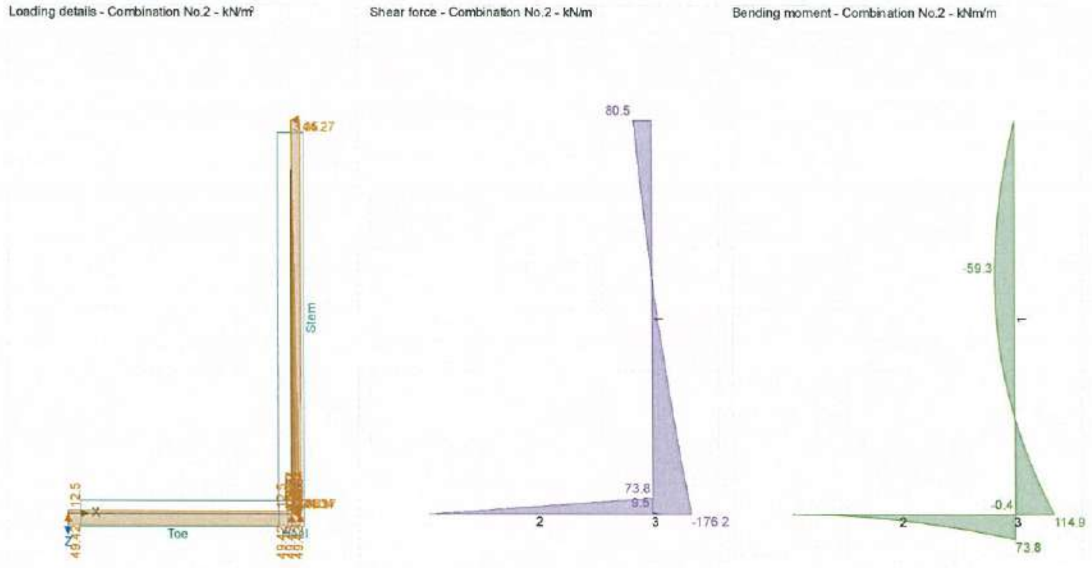


Shear force - Combination No.1 - kN/m



Bending moment - Combination No.1 - kNm/m





Check stem design at 2110 mm

Depth of section — $h = \mathbf{450}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — $M = \mathbf{78.4}$ kNm/m

Depth to tension reinforcement — $d = h - c_{sf} - \phi_{sx} - \phi_{sfM} / 2 = \mathbf{389}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.016}$

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d = \mathbf{370}$ mm

Depth of neutral axis — $x = 2.5 \times (d - z) = \mathbf{49}$ mm

Area of tension reinforcement required — $A_{sfM.req} = M / (f_{yd} \times z) = \mathbf{488}$ mm²/m

Tension reinforcement provided — 20 dia.bars @ 200 c/c

Area of tension reinforcement provided — $A_{sfM.prov} = \pi \times \phi_{sfM}^2 / (4 \times s_{sfM}) = \mathbf{1571}$ mm²/m

Minimum area of reinforcement - exp.9.1N — $A_{sfM.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{612}$ mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{sfM.max} = 0.04 \times h = \mathbf{18000}$ mm²/m

$\max(A_{sfM.req}, A_{sfM.min}) / A_{sfM.prov} = \mathbf{0.389}$

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

Deflection control - Section 7.4

Reference reinforcement ratio — $\rho_0 = \sqrt{(f_{ck} / 1\ N/mm^2)} / 1000 = \mathbf{0.006}$

Required tension reinforcement ratio — $\rho = A_{sfM.req} / d = \mathbf{0.001}$

Required compression reinforcement ratio — $\rho' = A_{sfM.2.req} / d_2 = \mathbf{0.000}$

Structural system factor - Table 7.4N — $K_b = \mathbf{1}$

Reinforcement factor - exp.7.17 — $K_s = \min(500\ N/mm^2 / (f_{yk} \times A_{sfM.req} / A_{sfM.prov}), 1.5) = \mathbf{1.5}$

Limiting span to depth ratio - exp.7.16.a — $K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}] = \mathbf{252.6}$

Actual span to depth ratio — $h_{prop} / d = \mathbf{9}$

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width — $w_{max} = \mathbf{0.3}$ mm

Variable load factor - EN1990 – Table A1.1 — ψ_2 = **0.6**

Serviceability bending moment — M_{sls} = **56.9** kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{sfM.prov} \times z)$ = **98.1** N/mm²

Load duration — Long term

Load duration factor — k_t = **0.4**

Effective area of concrete in tension — $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **133792** mm²/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm}$ = **3.0** N/mm²

Reinforcement ratio — $\rho_{p.eff} = A_{sfM.prov} / A_{c.eff}$ = **0.012**

Modular ratio — $\alpha_e = E_s / E_{cm}$ = **5.998**

Bond property coefficient — k_1 = **0.8**

Strain distribution coefficient — k_2 = **0.5**

k_3 = **3.4**

k_4 = **0.425**

Maximum crack spacing - exp.7.11 — $s_{r.max} = k_3 \times c_{sf} + k_1 \times k_2 \times k_4 \times \phi_{sfM} / \rho_{p.eff}$ = **409** mm

Maximum crack width - exp.7.8 — $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

w_k = **0.12** mm

w_k / w_{max} = **0.401**

PASS - Maximum crack width is less than limiting crack width

Check stem design at base of stem

Depth of section — h = **450** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — M = **151.5** kNm/m

Depth to tension reinforcement — $d = h - c_{sr} - \phi_{sr} / 2$ = **363** mm

$K = M / (d^2 \times f_{ck})$ = **0.036**

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **344** mm

Depth of neutral axis — $x = 2.5 \times (d - z)$ = **45** mm

Area of tension reinforcement required — $A_{sr.req} = M / (f_{yd} \times z)$ = **1012** mm²/m

Tension reinforcement provided — 25 dia.bars @ 100 c/c

Area of tension reinforcement provided — $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr})$ = **4909** mm²/m

Minimum area of reinforcement - exp.9.1N — $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **570** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{sr.max} = 0.04 \times h$ = **18000** mm²/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov}$ = **0.206**

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

Deflection control - Section 7.4

Reference reinforcement ratio — $\rho_0 = \sqrt{(f_{ck} / 1\ N/mm^2)} / 1000$ = **0.006**

Required tension reinforcement ratio — $\rho = A_{sr.req} / d$ = **0.003**

Required compression reinforcement ratio — $\rho' = A_{sr.2.req} / d_2$ = **0.000**

Structural system factor - Table 7.4N — K_b = **1**

Reinforcement factor - exp.7.17 — $K_s = \min(500\ N/mm^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5)$ = **1.5**

Limiting span to depth ratio - exp.7.16.a — $K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}]$ = **70.5**

Actual span to depth ratio — h_{prop} / d = **9.7**

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width	w_{max} = **0.3** mm
Variable load factor - EN1990 – Table A1.1	ψ_2 = **0.6**
Serviceability bending moment	M_{sls} = **110.2** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sr.prov} \times z)$ = **65.2** N/mm^2
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **134896** mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **3.0** N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{sr.prov} / A_{c.eff}$ = **0.036**
Modular ratio	$\alpha_e = E_s / E_{cm}$ = **5.998**
Bond property coefficient	k_1 = **0.8**
Strain distribution coefficient	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff}$ = **372** mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	w_k = **0.073** mm
	w_k / w_{max} = **0.242**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	V = **232** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.743**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{sr.prov} / d, 0.02)$ = **0.014**
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.456** N/mm^2
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **266.3** kN/m
	$V / V_{Rd.c}$ = **0.871**

PASS - Design shear resistance exceeds design shear force

Check stem design at prop

Depth of section	h = **450** mm

Rectangular section in shear - Section 6.2

Design shear force	V = **106.7** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.743**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{sr1.prov} / d, 0.02)$ = **0.004**
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.456** N/mm^2
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **182.1** kN/m
	$V / V_{Rd.c}$ = **0.586**

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1)	$A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **1227** mm^2/m
Maximum spacing of reinforcement – cl.9.6.3(2)	s_{sx_max} = **400** mm
Transverse reinforcement provided	16 dia.bars @ 100 c/c

Area of transverse reinforcement provided $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **2011** mm²/m

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

Check base design at toe

Depth of section h = **500** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 M = **97.2** kNm/m

Depth to tension reinforcement $d = h - c_{bb} - \phi_{bb} / 2$ = **409** mm

$K = M / (d^2 \times f_{ck})$ = **0.018**

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **389** mm

Depth of neutral axis $x = 2.5 \times (d - z)$ = **51** mm

Area of tension reinforcement required $A_{bb.req} = M / (f_{yd} \times z)$ = **575** mm²/m

Tension reinforcement provided 32 dia.bars @ 100 c/c

Area of tension reinforcement provided $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **8042** mm²/m

Minimum area of reinforcement - exp.9.1N $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **643** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{bb.max} = 0.04 \times h$ = **20000** mm²/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov}$ = **0.08**

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

Crack control - Section 7.3

Limiting crack width w_{max} = **0.3** mm

Variable load factor - EN1990 – Table A1.1 ψ_2 = **0.6**

Serviceability bending moment M_{sls} = **70.9** kNm/m

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **22.7** N/mm²

Load duration Long term

Load duration factor k_t = **0.4**

Effective area of concrete in tension $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **149625** mm²/m

Mean value of concrete tensile strength $f_{ct.eff} = f_{ctm}$ = **3.0** N/mm²

Reinforcement ratio $\rho_{p.eff} = A_{bb.prov} / A_{c.eff}$ = **0.054**

Modular ratio $\alpha_e = E_s / E_{cm}$ = **5.998**

Bond property coefficient k_1 = **0.8**

Strain distribution coefficient k_2 = **0.5**

k_3 = **3.4**

k_4 = **0.425**

Maximum crack spacing - exp.7.11 $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **356** mm

Maximum crack width - exp.7.8 $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

w_k = **0.024** mm

w_k / w_{max} = **0.081**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force V = **97.2** kN/m

$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.699**

Longitudinal reinforcement ratio $\rho_l = \min(A_{bb.prov} / d, 0.02)$ = **0.020**

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.439** N/mm²

Design shear resistance - exp.6.2a & 6.2b — $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c}$ = **331.7** kN/m

$V / V_{Rd.c}$ = **0.293**

PASS - Design shear resistance exceeds design shear force

Check base design at heel

Depth of section — h = **500** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — M = **0.5** kNm/m

Depth to tension reinforcement — $d = h - c_{bt} - \phi_{bt} / 2$ = **440** mm

$K = M / (d^2 \times f_{ck})$ = **0.000**

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **418** mm

Depth of neutral axis — $x = 2.5 \times (d - z)$ = **55** mm

Area of tension reinforcement required — $A_{bt.req} = M / (f_{yd} \times z)$ = **3** mm²/m

Tension reinforcement provided — 20 dia.bars @ 100 c/c

Area of tension reinforcement provided — $A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt})$ = **3142** mm²/m

Minimum area of reinforcement - exp.9.1N — $A_{bt.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **692** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{bt.max} = 0.04 \times h$ = **20000** mm²/m

$\max(A_{bt.req}, A_{bt.min}) / A_{bt.prov}$ = **0.22**

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

Crack control - Section 7.3

Limiting crack width — w_{max} = **0.3** mm

Variable load factor - EN1990 – Table A1.1 — ψ_2 = **0.6**

Serviceability bending moment — M_{sls} = **0.4** kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{bt.prov} \times z)$ = **0.3** N/mm²

Load duration — Long term

Load duration factor — k_t = **0.4**

Effective area of concrete in tension — $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **148333** mm²/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm}$ = **3.0** N/mm²

Reinforcement ratio — $\rho_{p.eff} = A_{bt.prov} / A_{c.eff}$ = **0.021**

Modular ratio — $\alpha_e = E_s / E_{cm}$ = **5.998**

Bond property coefficient — k_1 = **0.8**

Strain distribution coefficient — k_2 = **0.5**

k_3 = **3.4**

k_4 = **0.425**

Maximum crack spacing - exp.7.11 — $s_{r.max} = k_3 \times c_{bt} + k_1 \times k_2 \times k_4 \times \phi_{bt} / \rho_{p.eff}$ = **331** mm

Maximum crack width - exp.7.8 — $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

w_k = 0 mm

w_k / w_{max} = **0.001**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force — V = **12.8** kN/m

$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**

$k = \min(1 + \sqrt{(200\ mm / d)}, 2)$ = **1.674**

Longitudinal reinforcement ratio — $\rho_l = \min(A_{bt.prov} / d, 0.02)$ = **0.007**

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.429}\ N/mm^2$

Design shear resistance - exp.6.2a & 6.2b
$V_{Rd.c} = max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c} = \mathbf{250.8}\ kN/m$

$V / V_{Rd.c} = \mathbf{0.051}$

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

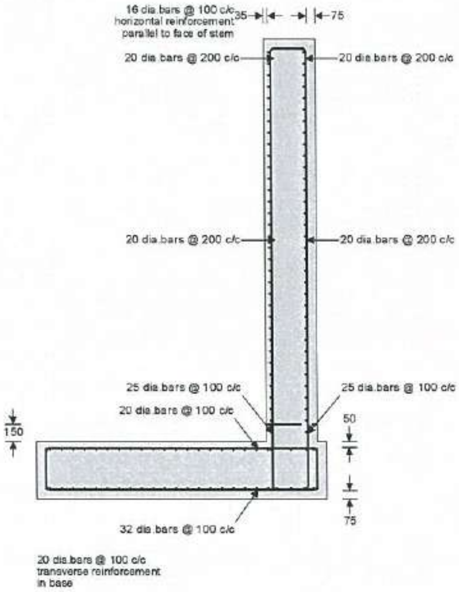
Minimum area of reinforcement – cl.9.3.1.1(2)
$A_{bx.req} = 0.2 \times A_{bb.prov} = \mathbf{1608}\ mm^2/m$

Maximum spacing of reinforcement – cl.9.3.1.1(3)
$s_{bx\_max} = \mathbf{450}\ mm$

Transverse reinforcement provided
20 dia.bars @ 100 c/c

Area of transverse reinforcement provided
$A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = \mathbf{3142}\ mm^2/m$

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



Reinforcement details

Stabila

Product Information

P200 Soil Injection Resin

Aqua-reactive PU Injection Resin

Stabila P200 SOIL is a solvent free, one component aqua-reactive polyurethane injection resin. It is sufficiently fast reacting to cut off active or gushing water flows, even under pressure. Stabila P200 SOIL Accelerator is added to Stabila P200 SOIL to vary the speed of reaction for the application in hand.
When P200 SOIL comes into contact with water, it reacts to produce a rapidly expanding closed cell foam. This rapid expansion acts to close off flow paths and hence arrest movement of free water. The foam is moderately flexible, hydrophobic and chemically resistant. It is harmless to the environment and resists biological attack

Uses

Injecting into cracks and minor open fissures in concrete and masonry structures where water leaks are to be controlled.

Injection into open, granular soils where specialised stabilisation is required.

Advantages

An extremely efficient injection resin for leak sealing use where flowing water is encountered.

Exhibits excellent penetration into voids and porous substrates.

Application

Stabila P200 SOIL injection is by single component pump through injection packers or ports as appropriate.

Speed of reaction is dependent on percentage of accelerator used and water temperature. Cold water will increase reaction times.

Where cold water, or relatively fast water flows occur, higher accelerator dosage is necessary. There is no advantage to be gained by increasing the accelerator dose beyond the recommended maximum.

Stabila P200 SOIL may be used to control fast flowing water or water under pressure, but where such conditions are encountered, please contact our Technical Department before use.

Package & Storage

Stabila P200 SOIL is supplied in 25kg drums. P200 SOIL accelerator is supplied in 2.5kg plastic containers. Store in original containers in a dry area, protect from heat and sunlight. Once opened, use as soon as possible.

Health & Safety

Avoid contact with eyes and skin. Follow advice in separate Health & Safety data sheet.

Technical data

	P200 Soil	P200 Accelerator
Form	Liquid	Liquid
Viscosity (25°C)	190 mPas	9 mPas
Colour	Brown	Pale Yellow
Specific gravity (20°C)	1.10	1.04
Mixing ratio	1% to 10% Accelerator by weight	
Application temp	Not less than 5°C	

Reaction times (15°C)

% Accel dosage	Induction time	Gel time
3% (by weight)	50 sec	8 min 20 secs
6% “	29 sec	2 min 35 secs
9% “	28 sec	2 min 05 secs

Stabila UK. Oxon. OX27 7SR
tel 01869 346010 fax 01869 345455

SINCLAIRJOHNSTON

SPECIFICATION FOR MOVEMENT MONITORING OF PARTY WALLS SHARED WITH PROPERTY FOR WORKS ASSOCIATED WITH CONSTRUCTION OF BASEMENT

AT:

51-52 TOTTENHAM COURT ROAD,
LONDON, W1

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8781/MOVEMENT MONITORING SPEC/RA NOV. 2017

I N D E X

1.0 INTRODUCTION

2.0 MOVEMENT MONITORING SPECIFICATION

APPENDICES

A1 OUTLINE SURVEY TARGET LOCATIONS FOR MOVEMENT MONITORING

1.0 INTRODUCTION

1.1 This specification document outlines the structural requirements for movement monitoring of the Party Walls either side of No. 51-52 Tottenham Court Road, specifically during basement construction works proposed within the curtilage of the site.

1.2 Please refer to '8781 Construction Method Statement 170412 GS RevA' dated April 2017 for further information on the existing building, existing ground conditions, and structural alteration works proposed.

2.0 MOVEMENT MONITORING SPECIFICATION

2.1 Outline locations for survey targets which are to be monitored for movement are shown on plan in Appendix A.

2.2 The Contractor appointed to undertake the basement construction works will be required to develop the movement monitoring proposals in detail, in accordance with the drawings and the specification enclosed.

2.3 The proposals must be submitted to Sinclair Johnston & Partners Ltd for comment, and subsequent agreement with Party Wall Surveyors prior to implementation.

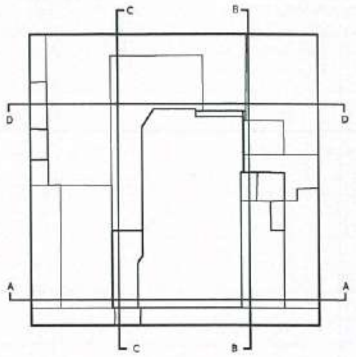
51-52 TOTTENHAM COURT ROAD, W1

APPENDIX A

OUTLINE SURVEY TARGET LOCATIONS FOR MOVEMENT MONITORING

NOTES:

DO NOT SCALE FROM THIS DRAWING.
ALL DIMENSIONS TO BE CHECKED ON SITE.
ALL OMISSIONS AND DISCREPANCIES TO BE REPORTED TO THE ARCHITECT IMMEDIATELY.



X = INDICATIVE LOCATIONS FOR MOVEMENT MONITORING TARGETS – TO BE UNDERTAK AT WEEKLY INTERVALS DUR CONSTRUCTION OF UNDERPIN WALLS UNTIL 2No. WEEKS AFTER UNDERPINNING HAS BEEN COMPLETED OR 2No. CONSECUTIVE WEEKS READINGS WITH NO RESIDU MOVEMENT. REFER TO SJP SPEC. FOR FURTHER INFORMA

⊕ = TARGET STATIONS

Revision description	Date	Check	Rev
Issued for Information	07/03/16	AL	C
Issued for Information	26/03/15	LLo	B
Issued for Information	04/11/14	LLo	A
Issued for Information - RDL	11/09/14	LLo	-

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Project
51-52 Tottenham Court Road,
London. W1T

Drawing
Ground Floor Plan
Proposed

Drawn	Date	Scale
AL	07/03/2016	1:100@A1 1:200@A3
Job number	**Drawing number**	**Revision**
13081	C645_P_00_001	C

SINCLAIRJOHNSTON

51-52 Tottenham Court Road - Movement Monitoring Specification

03 November 2017

B50 General structural requirements

20 INFORMATION TO BE PROVIDED BY CONTRACTOR PRIOR TO COMMENCEMENT OF EXCAVATION WORKS FOR BASEMENT FORMED BY UNDERPINNING.

- Submit the following:
  - Technical information: Proposals for monitoring movement of Party Walls either side of 51-52 Tottenham Court Road; Flank Wall to No.49 Tottenham Court Road; and Rear Wal to No. 10 Whitfield Street during underpinning works, including locations of measuring stations.
  - Proposals: Contractor to prepare drawings showing locations of all proposed survey targets for movement monitoring. These are to be submitted to Sinclair Johnston & Partners Ltd for comment and subsequent agreement with Party Wall Surveyors.

PERFORMANCE

250 LIMITS ON MOVEMENT GENERATED BY CONSTRUCTION

- Definition of critical values:
  - Threshold value: The value beyond which further movement will be of significant concern.
  - Action value: The value at which execution must cease.
- Precautions: Take as follows if movements reach critical values:
  - Threshold: Review situation, assess possible causes and submit proposals to ensure that action values are not exceeded.
  - Action: Stop work, report and revise working procedures to prevent further movements.

260 LATERAL MOVEMENT OF EMBEDDED RETAINING WALLS DURING EXCAVATION ANI CONSTRUCTION

- Action values: 5 mm.
- Trigger values: 3mm.
- Timing: Movement measurements are to be taken at least weekly from commencement of excavation works until permanent support structure has been constructed.

270 LIMITS ON GROUND MOVEMENT AROUND SITE PERIMETER.

- Location of survey points: Contractor to submit proposals for monitoring Party Walls No. 51 -52 Tottenham Court Road; Flank Wall to No.49 Tottenham Court Road; and Rear Wall to No. 10 Whitfield Street around basement excavation.
- Movement of survey points must not exceed:
  - Settlement:
    Action values: 5 mm.
    Threshold values: 3 mm.
  - Lateral displacement:
    Action values: 5 mm.
    Threshold values: 3 mm.
- Timing: Movement measurements are to be taken at least every week from commencement of excavation works until permanent support structure has been constructed.

280 SETTLEMENT OF EXISTING STRUCTURES

- Location: Existing building Party Walls.
- Action values: 5 mm.
- Threshold values: 3 mm.

310 DAMAGE TO EXISTING STRUCTURES AND SERVICES
- Permissible damage criteria:
  - Structures: No damage permitted.
  - Services: No damage permitted.

EXECUTION

740 CONDITION SURVEY OF EXISTING BUILDINGS AND STRUCTURES
- Application: Party Walls both sides to 51-52 Tottenham Court Road; Flank Wall to No.49 Tottenham Court Road; and Rear Wall to No. 10 Whitfield Street : visually, prior to commencement of any excavation works.
- Before starting work: Survey structure. Record and take photographs of damaged or defective areas.
  - Items to be recorded: Location, extent and magnitude of cracks, spalling, indications of movement, previous repairs, modifications and other irregularities of the fabric.
  - Additional investigations: None.
- Information supplied: None.
- Report: Submit for comment.
  - Include recommendations: For repair or monitoring of any defects uncovered.

760 MONITORING OF EXISTING BUILDINGS/ STRUCTURES
- Application: Party Walls to No.51-52 Tottenham Court Road; Flank Wall to No.49 Tottenham Court Road; and Rear Wall to No. 10 Whitfield Street.
- Requirement: Visually inspect buildings/ structures for signs of movement, cracking or other indications of distress.
- Period of inspection: Commence prior to start of excavation works to obtain base readings and continue until permanent support structure has been constructed.
- Frequency of inspection: Weekly.
- Record: Date and time of inspections.
- Action: If movement cracking or other signs of distress are noted stop work, investigate and report .

770 MOVEMENT MONITORING PARTY WALLS TO NO.51-52 TOTTENHAM COURT ROAD; FLANK WALL TO NO.49 TOTTENHAM COURT ROAD; AND REAR WALL TO NO. 10 WHITFIELD STREET.
- Application: Movement monitoring.
- Survey points: Agree number and location of survey points and record initial positions to enable monitoring of:
  - Movements: Settlement.
- Method: Submit method statement.
  - Accuracy of reading: ± 1 mm.
- Special requirements: Visually inspect structure on weekly basis.

790 FREQUENCY OF MONITORING GROUND MOVEMENTS.
- Initial readings: Agree and record as soon as survey points have been established.
- Frequency of readings: Weekly.
- Increase frequency of readings if:
  - Movements accelerate.
  - Trend of movements changes unexpectedly.
- Additional readings:
  - A single set: Immediately following an unexpected event that could have affected the movements.
  - Increase frequency of readings: daily until two consecutive sets of readings are stable and consistent when survey points are first established.
- Period of monitoring: Until permanent basement structure has been constructed.

COMPLETION

910 SUBMISSION OF INFORMATION

- Submit:
  - Details and results of monitoring.
  - Details and purpose of any changes to the monitoring regime.
- Timing: Within 3No. days of monitoring readings.
- Special requirements: Reports to be in graphic and tabular form, relative to base readings for ease of reference.