CROFT STRUCTURAL ENGINEERS

Appendix B

Structural Scheme Drawings

This information is provided for Planning use only and is not to be used for Building control submissions

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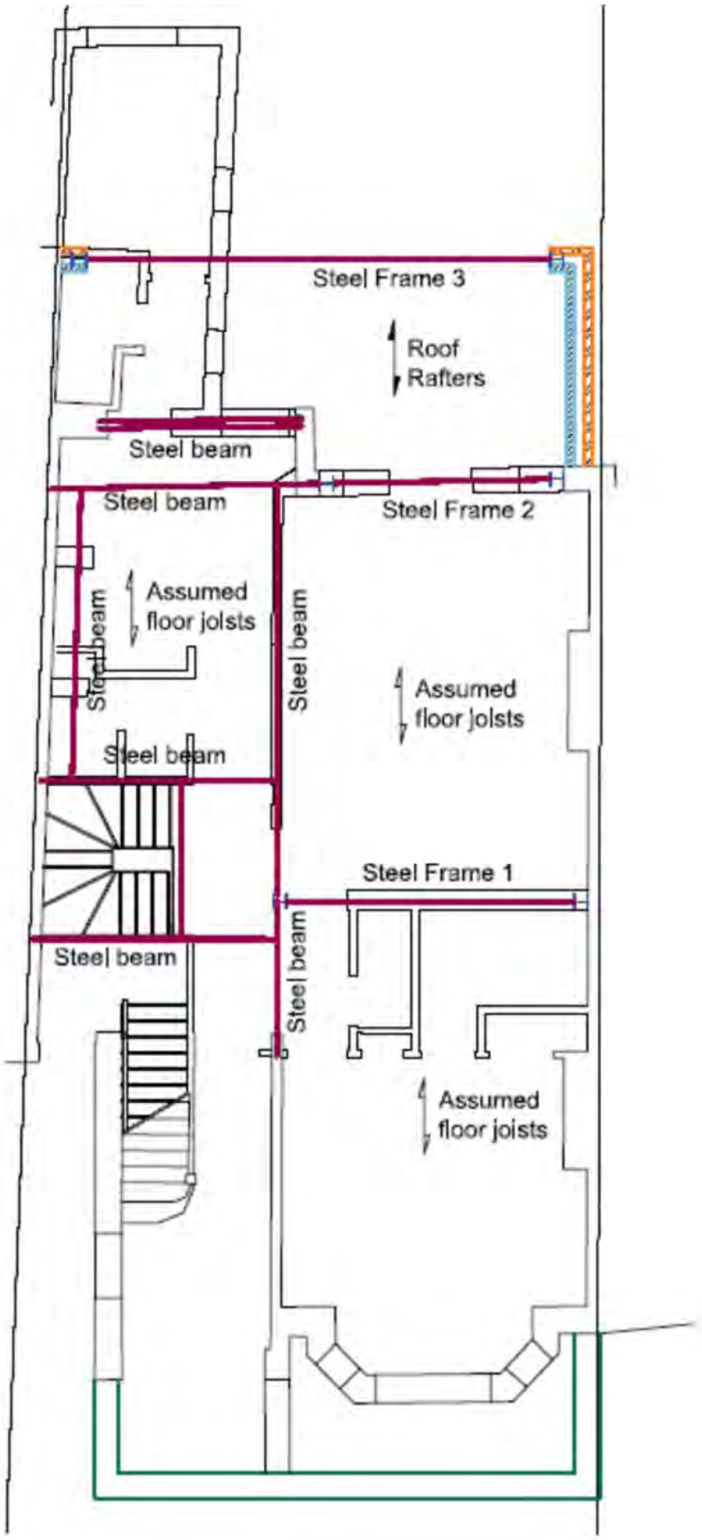

1st floor structure on the reflected Ground Floor Plan
scale 1:100

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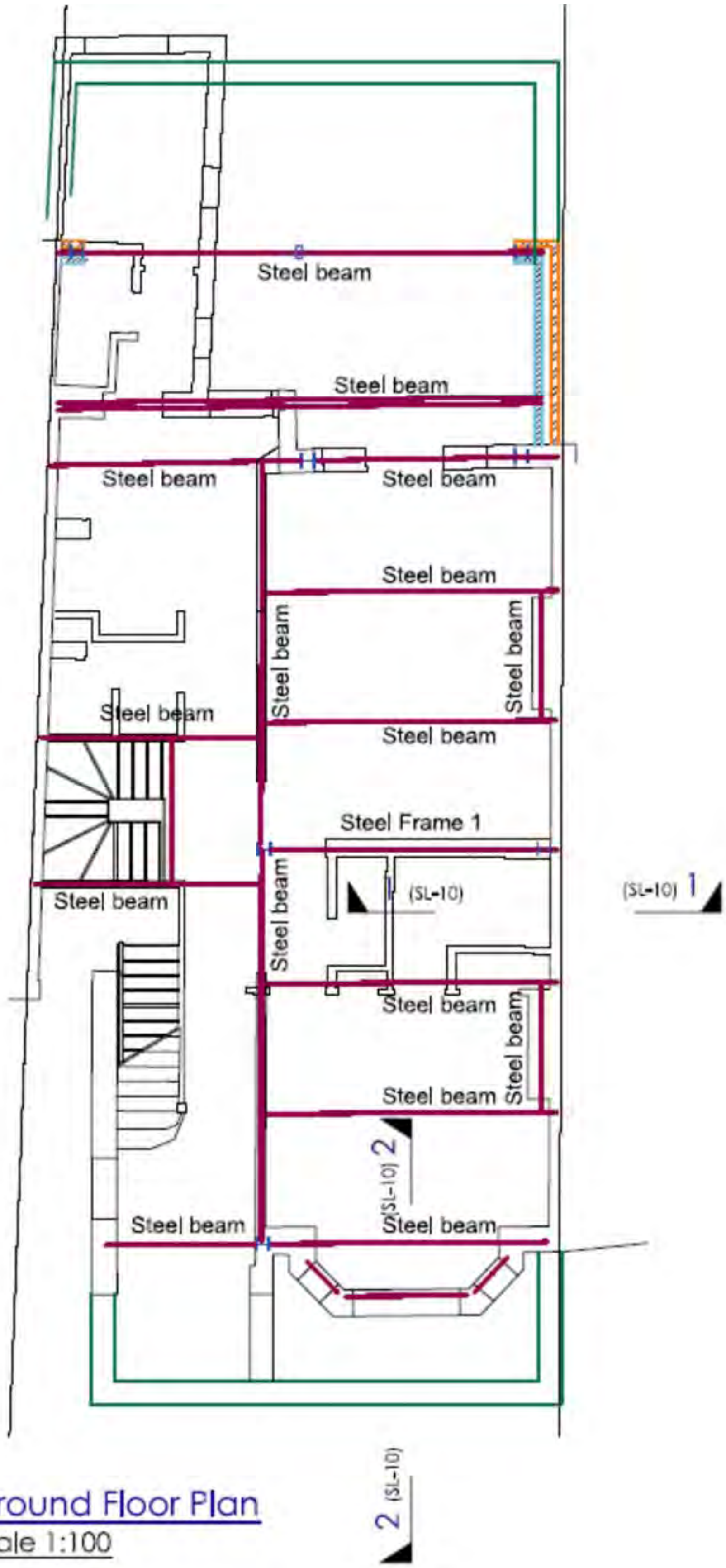

Ground Floor Plan
scale 1:100

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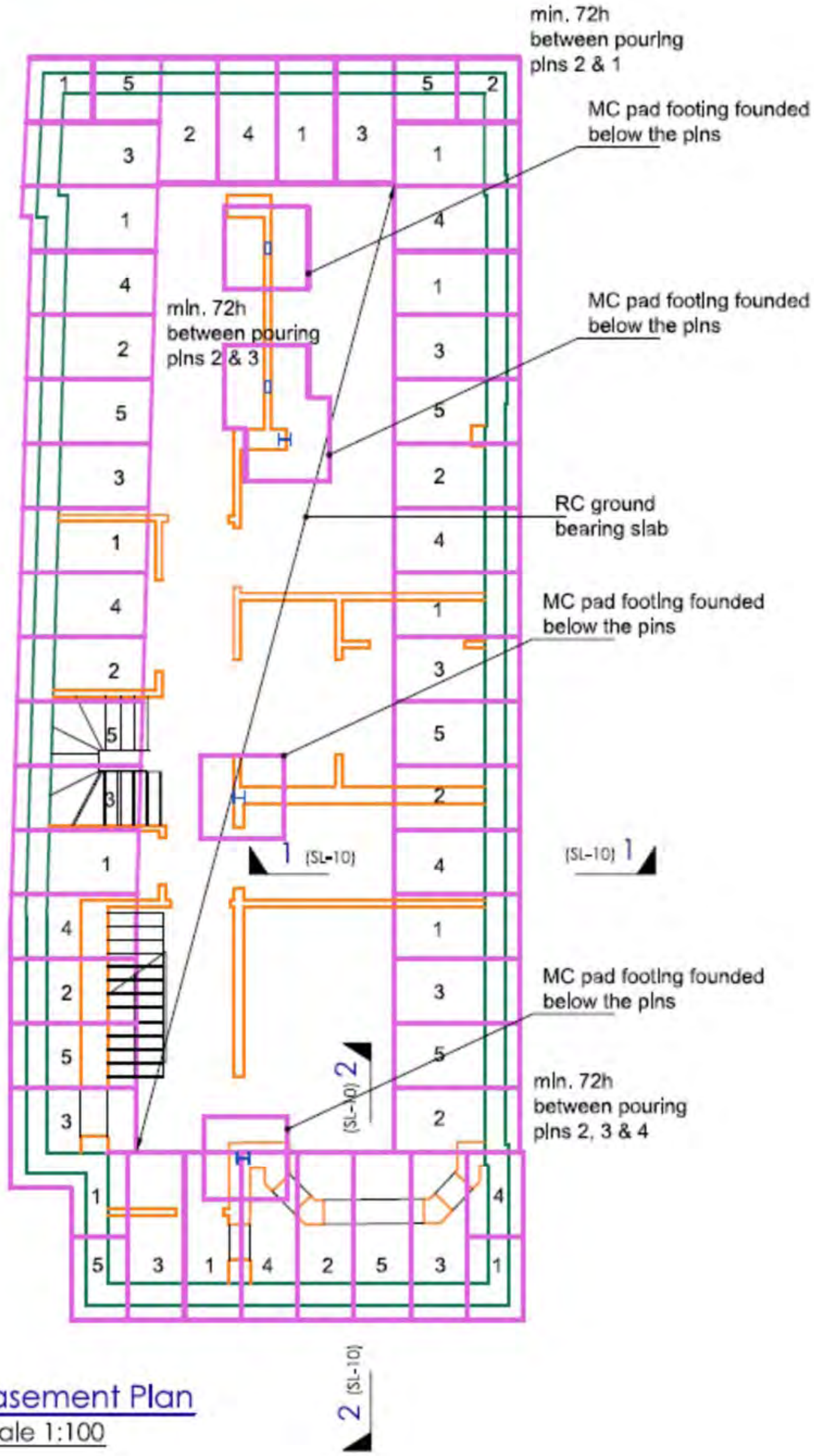
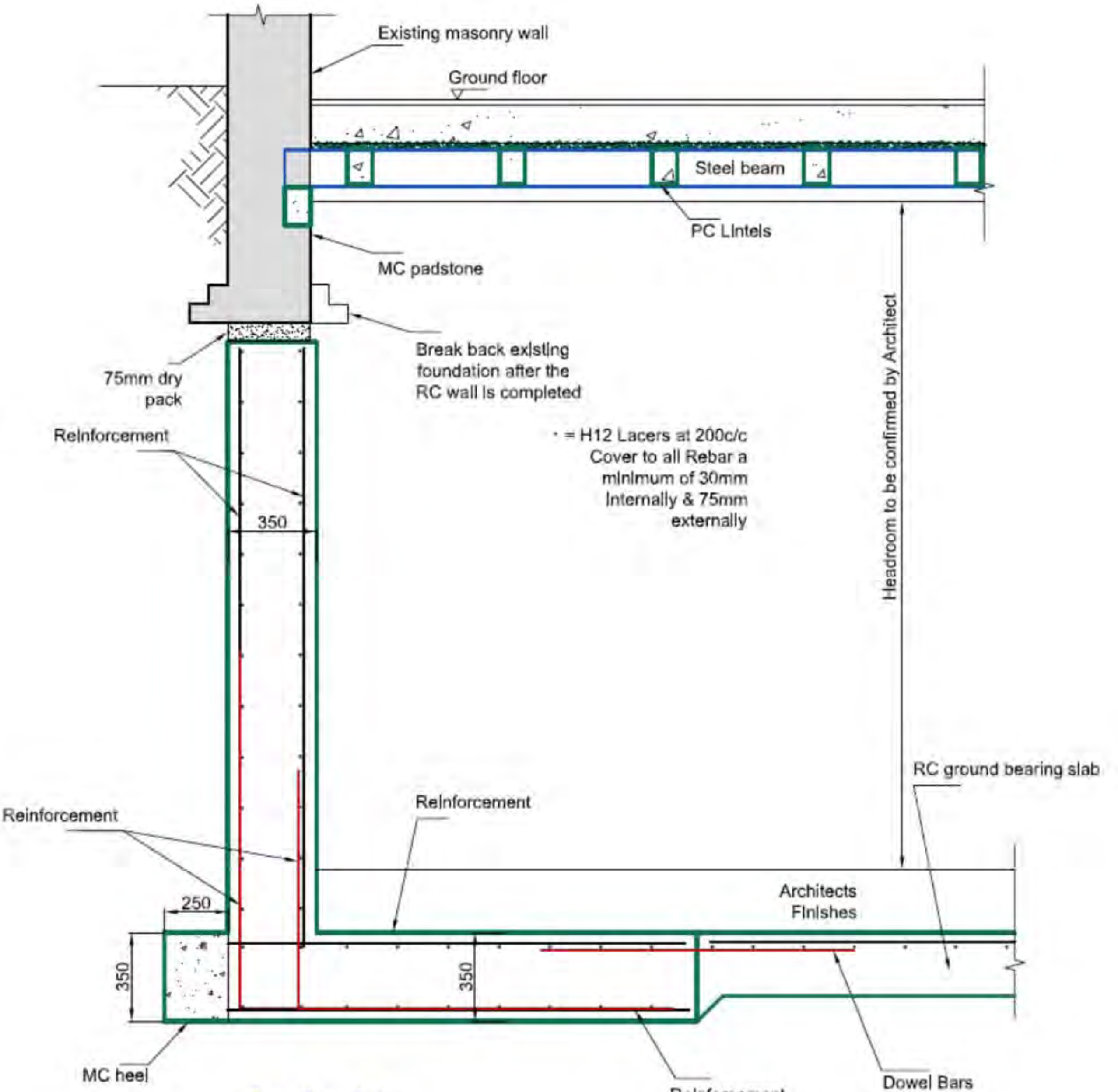
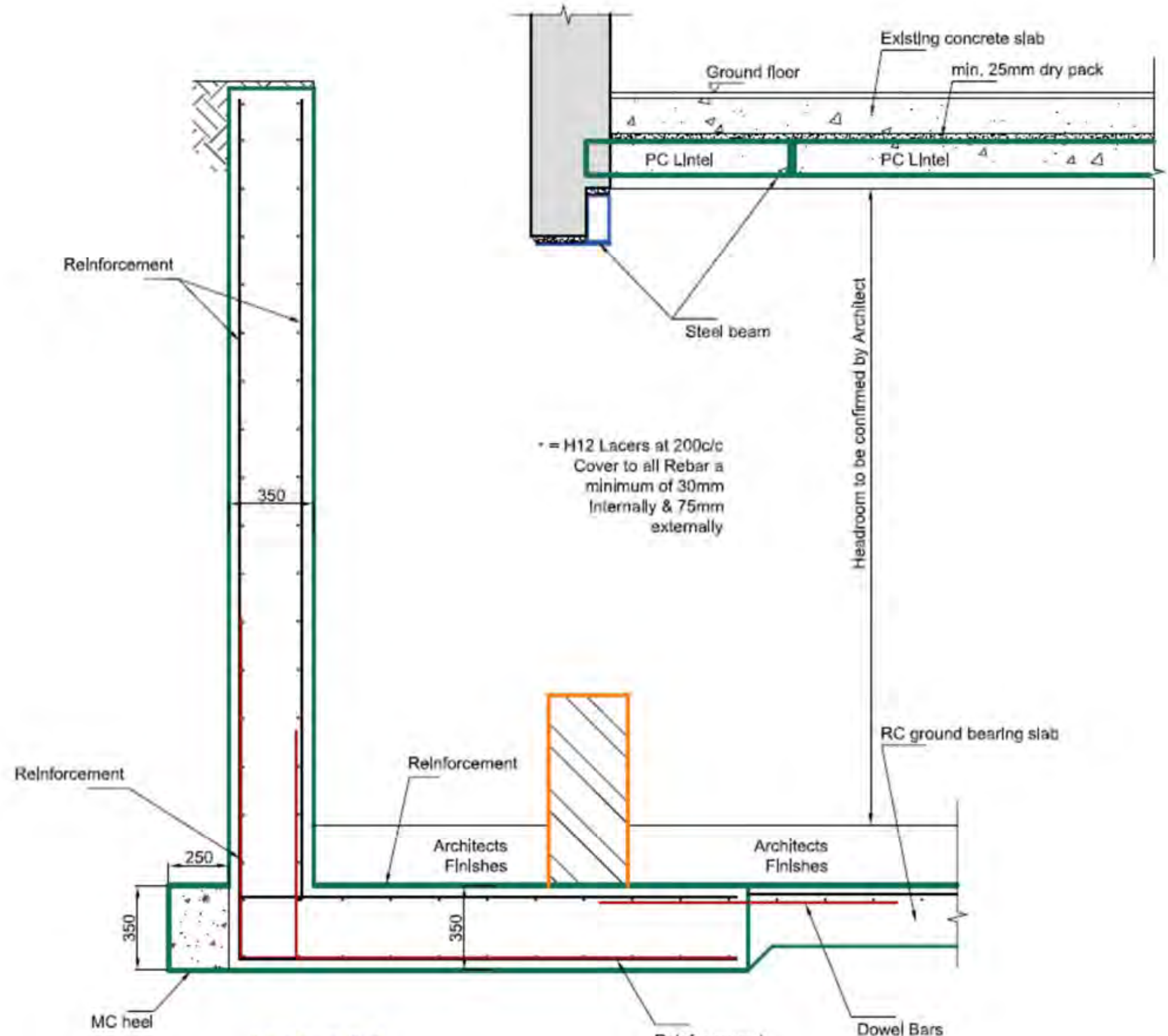


Basement Plan
scale 1:100



Section 1-1
scale 1:20



Section 2-2
scale 1:20

Appendix C

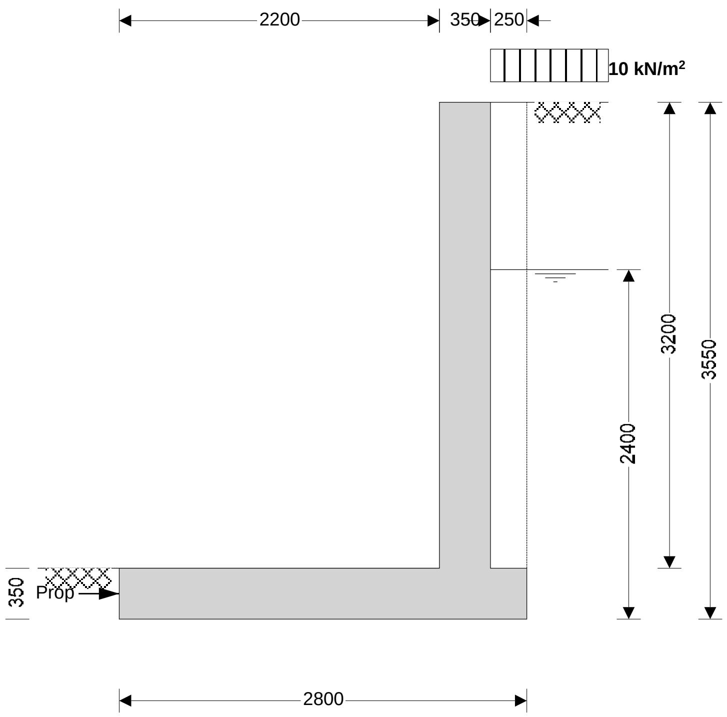
Structural Basement Calculations

This information is provided for Planning use only and is not to be used for Building control submissions

rc retaining wall 1 design

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type	**Cantilever propped at base**
Height of retaining wall stem	h_{stem} = **3200** mm
Thickness of wall stem	t_{wall} = **350** mm
Length of toe	l_{toe} = **2200** mm
Length of heel	l_{heel} = **250** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **2800** mm
Thickness of base	t_{base} = **350** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **1900** mm
Thickness of downstand	t_{ds} = **350** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **3550** mm
Depth of cover in front of wall	d_{cover} = **0** mm
Depth of unplanned excavation	d_{exc} = **0** mm
Height of ground water behind wall	h_{water} = **2400** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0\text{ mm})$ = **2050** mm
Density of wall construction	γ_{wall} = **23.6** kN/m³
Density of base construction	γ_{base} = **23.6** kN/m³
Angle of rear face of wall	α = **90.0** deg
Angle of soil surface behind wall	β = **0.0** deg
Effective height at virtual back of wall	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **3550** mm

Retained material details

Mobilisation factor	M = **1.5**
Moist density of retained material	γ_m = **18.0** kN/m³

Saturated density of retained material — γ_s = **21.0** kN/m^3
Design shear strength — ϕ' = **24.2** deg
Angle of wall friction — δ = **0.0** deg

Base material details

Moist density — γ_{mb} = **18.0** kN/m^3
Design shear strength — ϕ'_b = **24.2** deg
Design base friction — δ_b = **18.6** deg
Allowable bearing pressure — $P_{bearing}$ = **100** kN/m^2

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.419}$$

Passive pressure coefficient for base material

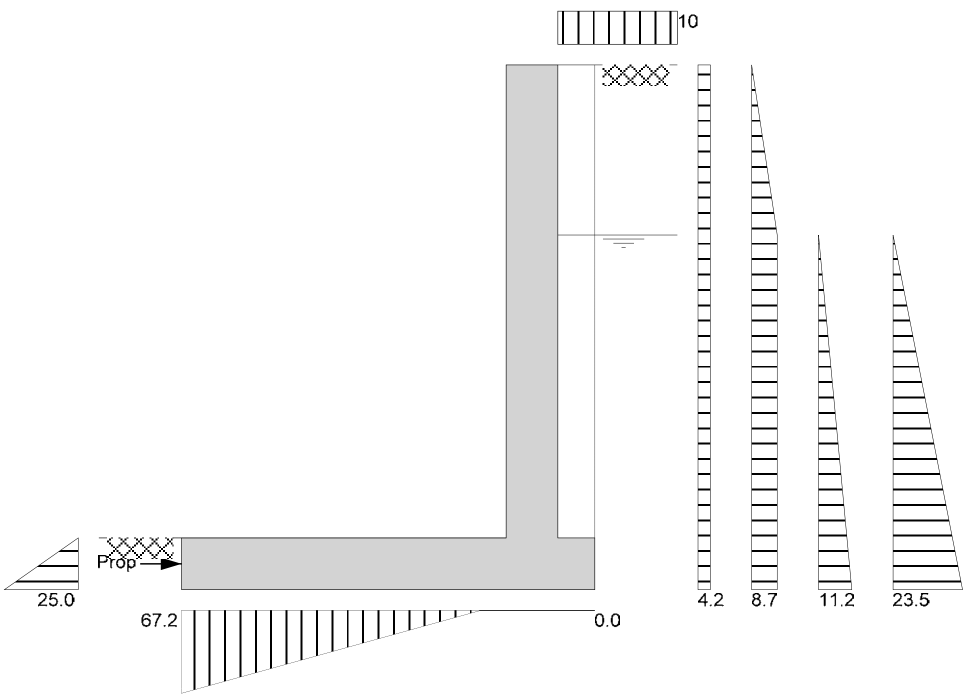
$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{4.187}$$

At-rest pressure

At-rest pressure for retained material — $K_0 = 1 - \sin(\phi') = \mathbf{0.590}$

Loading details

Surcharge load on plan — Surcharge = **10.0** kN/m^2
Applied vertical dead load on wall — W_{dead} = **0.0** kN/m
Applied vertical live load on wall — W_{live} = **0.0** kN/m
Position of applied vertical load on wall — l_{load} = **0** mm
Applied horizontal dead load on wall — F_{dead} = **0.0** kN/m
Applied horizontal live load on wall — F_{live} = **0.0** kN/m
Height of applied horizontal load on wall — h_{load} = **0** mm



Loads shown in kN/m, pressures shown in kN/m^2

Vertical forces on wall

Wall stem — $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **26.4** kN/m
Wall base — $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **23.1** kN/m
Surcharge — w_{sur} = Surcharge $\times\ l_{heel}$ = **2.5** kN/m

Moist backfill to top of wall	$w_{m\_w} = l_{heel} \times (h_{stem} - h_{sat}) \times \gamma_m$ = **5.2** kN/m
Saturated backfill	$w_s = l_{heel} \times h_{sat} \times \gamma_s$ = **10.8** kN/m
Total vertical load	$W_{total} = w_{wall} + w_{base} + w_{sur} + w_{m\_w} + w_s$ = **68** kN/m

Horizontal forces on wall

Surcharge	$F_{sur} = K_a \times$ Surcharge $\times h_{eff}$ = **14.9** kN/m
Moist backfill above water table	$F_{m\_a} = 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **5** kN/m
Moist backfill below water table	$F_{m\_b} = K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **20.8** kN/m
Saturated backfill	$F_s = 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **13.5** kN/m
Water	$F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **28.3** kN/m
Total horizontal load	$F_{total} = F_{sur} + F_{m\_a} + F_{m\_b} + F_s + F_{water}$ = **82.4** kN/m

Calculate propping force

Passive resistance of soil in front of wall	$F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **4.4** kN/m
Propping force	$F_{prop} = \max(F_{total} - F_p - (W_{total} - w_{sur}) \times \tan(\delta_b), 0 \text{ kN/m})$ F_{prop} = **56.0** kN/m

Overturning moments

Surcharge	$M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **26.4** kNm/m
Moist backfill above water table	$M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **13.9** kNm/m
Moist backfill below water table	$M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2$ = **25** kNm/m
Saturated backfill	$M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **10.8** kNm/m
Water	$M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **22.6** kNm/m
Total overturning moment	$M_{ot} = M_{sur} + M_{m\_a} + M_{m\_b} + M_s + M_{water}$ = **98.6** kNm/m

Restoring moments

Wall stem	$M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **62.8** kNm/m
Wall base	$M_{base} = w_{base} \times l_{base} / 2$ = **32.4** kNm/m
Moist backfill	$M_{m\_r} = (w_{m\_w} \times (l_{base} - l_{heel} / 2) + w_{m\_s} \times (l_{base} - l_{heel} / 3))$ = **13.8** kNm/m
Saturated backfill	$M_{s\_r} = w_s \times (l_{base} - l_{heel} / 2)$ = **28.8** kNm/m
Total restoring moment	$M_{rest} = M_{wall} + M_{base} + M_{m\_r} + M_{s\_r}$ = **137.8** kNm/m

Check bearing pressure

Surcharge	$M_{sur\_r} = w_{sur} \times (l_{base} - l_{heel} / 2)$ = **6.7** kNm/m
Total moment for bearing	$M_{total} = M_{rest} - M_{ot} + M_{sur\_r}$ = **45.9** kNm/m
Total vertical reaction	$R = W_{total}$ = **68.0** kN/m
Distance to reaction	$x_{bar} = M_{total} / R$ = **675** mm
Eccentricity of reaction	$e = \text{abs}((l_{base} / 2) - x_{bar})$ = **725** mm
	Reaction acts outside middle third of base
Bearing pressure at toe	$p_{toe} = R / (1.5 \times x_{bar})$ = **67.2** kN/m^2
Bearing pressure at heel	$p_{heel} = 0$ kN/m^2 = **0** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

Ultimate limit state load factors

Dead load factor	γ_{f_d} = **1.4**
Live load factor	γ_{f_l} = **1.6**
Earth and water pressure factor	γ_{f_e} = **1.4**

Factored vertical forces on wall

Wall stem	$w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **37** kN/m

Wall base $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **32.4** kN/m
Surcharge $w_{sur\_f} = \gamma_{f\_l} \times Surcharge \times l_{heel}$ = **4** kN/m
Moist backfill to top of wall $w_{m\_w\_f} = \gamma_{f\_d} \times l_{heel} \times (h_{stem} - h_{sat}) \times \gamma_m$ = **7.2** kN/m
Saturated backfill $w_{s\_f} = \gamma_{f\_d} \times l_{heel} \times h_{sat} \times \gamma_s$ = **15.1** kN/m
Total vertical load $W_{total\_f} = w_{wall\_f} + w_{base\_f} + w_{sur\_f} + w_{m\_w\_f} + w_{s\_f}$ = **95.7** kN/m

Factored horizontal at-rest forces on wall

Surcharge $F_{sur\_f} = \gamma_{f\_l} \times K_0 \times Surcharge \times h_{eff}$ = **33.5** kN/m
Moist backfill above water table $F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times \gamma_m \times (h_{eff} - h_{water})^2$ = **9.8** kN/m
Moist backfill below water table $F_{m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **41** kN/m
Saturated backfill $F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **26.6** kN/m
Water $F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water}$ = **39.6** kN/m
Total horizontal load $F_{total\_f} = F_{sur\_f} + F_{m\_a\_f} + F_{m\_b\_f} + F_{s\_f} + F_{water\_f}$ = **150.6** kN/m

Calculate propping force

Passive resistance of soil in front of wall $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **6.1** kN/m
Propping force $F_{prop\_f} = \max(F_{total\_f} - F_{p\_f} - (W_{total\_f} - w_{sur\_f}) \times \tan(\delta_b), 0 \text{ kN/m})$
F_{prop_f} = **113.6** kN/m

Factored overturning moments

Surcharge $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **59.5** kNm/m
Moist backfill above water table $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **27.4** kNm/m
Moist backfill below water table $M_{m\_b\_f} = F_{m\_b\_f} \times (h_{water} - 2 \times d_{ds}) / 2$ = **49.2** kNm/m
Saturated backfill $M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **21.3** kNm/m
Water $M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **31.6** kNm/m
Total overturning moment $M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f} + M_{m\_b\_f} + M_{s\_f} + M_{water\_f}$ = **189.1** kNm/m

Restoring moments

Wall stem $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **87.9** kNm/m
Wall base $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **45.3** kNm/m
Surcharge $M_{sur\_r\_f} = w_{sur\_f} \times (l_{base} - l_{heel} / 2)$ = **10.7** kNm/m
Moist backfill $M_{m\_r\_f} = (w_{m\_w\_f} \times (l_{base} - l_{heel} / 2) + w_{m\_s\_f} \times (l_{base} - l_{heel} / 3))$ = **19.4** kNm/m
Saturated backfill $M_{s\_r\_f} = w_{s\_f} \times (l_{base} - l_{heel} / 2)$ = **40.3** kNm/m
Total restoring moment $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{sur\_r\_f} + M_{m\_r\_f} + M_{s\_r\_f}$ = **203.6** kNm/m

Factored bearing pressure

Total moment for bearing $M_{total\_f} = M_{rest\_f} - M_{ot\_f}$ = **14.6** kNm/m
Total vertical reaction $R_f = W_{total\_f}$ = **95.7** kN/m
Distance to reaction $x_{bar\_f} = M_{total\_f} / R_f$ = **152** mm
Eccentricity of reaction $e_f = \text{abs}((l_{base} / 2) - x_{bar\_f})$ = **1248** mm

Reaction acts outside middle third of base

Bearing pressure at toe $p_{toe\_f} = R_f / (1.5 \times x_{bar\_f})$ = **419.5** kN/m^2
Bearing pressure at heel p_{heel_f} = 0 kN/m^2 = **0** kN/m^2
Rate of change of base reaction rate = $p_{toe\_f} / (3 \times x_{bar\_f})$ = **919.55** kN/m^2/m
Bearing pressure at stem / toe $p_{stem\_toe\_f} = \max(p_{toe\_f} - (rate \times l_{toe}), 0 \text{ kN/m}^2)$ = **0** kN/m^2
Bearing pressure at mid stem $p_{stem\_mid\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall} / 2)), 0 \text{ kN/m}^2)$ = **0** kN/m^2
Bearing pressure at stem / heel $p_{stem\_heel\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall})), 0 \text{ kN/m}^2)$ = **0** kN/m^2

Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

Characteristic strength of concrete — f_{cu} = **35** N/mm^2

Characteristic strength of reinforcement — f_y = **500** N/mm^2

Base details

Minimum area of reinforcement — k = **0.13** %

Cover to reinforcement in toe — c_{toe} = **50** mm

Calculate shear for toe design

Shear from bearing pressure — $V_{toe\_bear} = 3 \times p_{toe\_f} \times x_{bar\_f} / 2$ = **95.7** kN/m

Shear from weight of base — $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base}$ = **25.4** kN/m

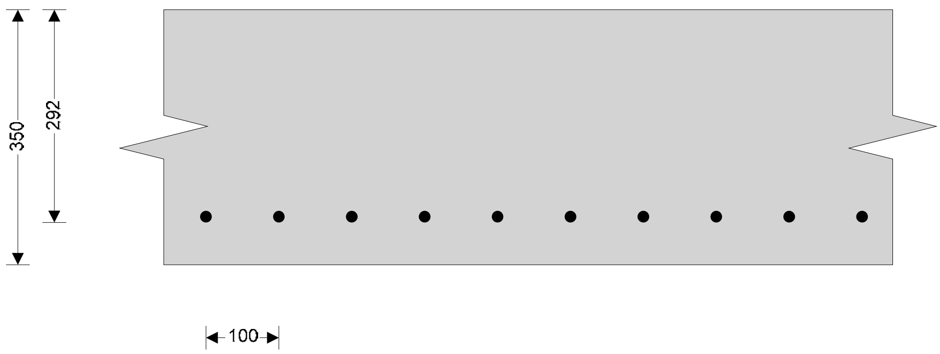
Total shear for toe design — $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base}$ = **70.3** kN/m

Calculate moment for toe design

Moment from bearing pressure — $M_{toe\_bear} = 3 \times p_{toe\_f} \times x_{bar\_f} \times (l_{toe} - x_{bar\_f} + t_{wall} / 2) / 2$ = **212.7** kNm/m

Moment from weight of base — $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2)$ = **32.6** kNm/m

Total moment for toe design — $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base}$ = **180.1** kNm/m



Check toe in bending

Width of toe — b = **1000** mm/m

Depth of reinforcement — $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2)$ = **292.0** mm

Constant — $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu})$ = **0.060**

Compression reinforcement is not required

Lever arm — $z_{toe} = \min(0.5 + \sqrt{(0.25 - (\min(K_{toe}, 0.225) / 0.9))}, 0.95) \times d_{toe}$

z_{toe} = **271** mm

Area of tension reinforcement required — $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe})$ = **1528** mm^2/m

Minimum area of tension reinforcement — $A_{s\_toe\_min} = k \times b \times t_{base}$ = **455** mm^2/m

Area of tension reinforcement required — $A_{s\_toe\_req} = \mathrm{Max}(A_{s\_toe\_des}, A_{s\_toe\_min})$ = **1528** mm^2/m

Reinforcement provided — **16 mm dia.bars @ 100 mm centres**

Area of reinforcement provided — $A_{s\_toe\_prov}$ = **2011** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress — $v_{toe} = V_{toe} / (b \times d_{toe})$ = **0.241** N/mm^2

Allowable shear stress — $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1 \text{ N/mm}^2)}, 5) \times 1 \text{ N/mm}^2$ = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress — v_{c_toe} = **0.675** N/mm^2

$v_{toe} < v_{c\_toe}$ - No shear reinforcement required

Design of reinforced concrete retaining wall heel (BS 8002:1994)

Material properties

Characteristic strength of concrete	f_{cu} = **35** N/mm^2
Characteristic strength of reinforcement	f_y = **500** N/mm^2

Base details

Minimum area of reinforcement	k = **0.13** %
Cover to reinforcement in heel	c_{heel} = **50** mm

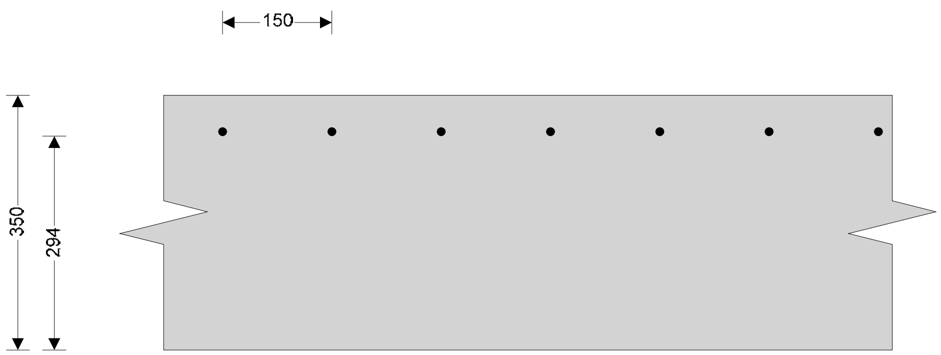
Calculate shear for heel design

Shear from weight of base	$V_{heel\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{heel} \times t_{base}$ = **2.9** kN/m
Shear from weight of moist backfill	$V_{heel\_wt\_m} = w_{m\_w\_f}$ = **7.2** kN/m
Shear from weight of saturated backfill	$V_{heel\_wt\_s} = w_{s\_f}$ = **15.1** kN/m
Shear from surcharge	$V_{heel\_sur} = w_{sur\_f}$ = **4** kN/m
Total shear for heel design	$V_{heel} = V_{heel\_wt\_base} + V_{heel\_wt\_m} + V_{heel\_wt\_s} + V_{heel\_sur}$ = **29.2** kN/m

Calculate moment for heel design

Moment from weight of base	$M_{heel\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{heel} + t_{wall} / 2)^2 / 2)$ = **1** kNm/m
Moment from weight of moist backfill	$M_{heel\_wt\_m} = w_{m\_w\_f} \times (l_{heel} + t_{wall}) / 2$ = **2.2** kNm/m
Moment from weight of saturated backfill	$M_{heel\_wt\_s} = w_{s\_f} \times (l_{heel} + t_{wall}) / 2$ = **4.5** kNm/m
Moment from surcharge	$M_{heel\_sur} = w_{sur\_f} \times (l_{heel} + t_{wall}) / 2$ = **1.2** kNm/m
Total moment for heel design	$M_{heel} = M_{heel\_wt\_base} + M_{heel\_wt\_m} + M_{heel\_wt\_s} + M_{heel\_sur}$ = **8.9**

kNm/m



Check heel in bending

Width of heel	b = **1000** mm/m
Depth of reinforcement	$d_{heel} = t_{base} - c_{heel} - (\phi_{heel} / 2)$ = **294.0** mm
Constant	$K_{heel} = M_{heel} / (b \times d_{heel}^2 \times f_{cu})$ = **0.003**

Compression reinforcement is not required

Lever arm	$z_{heel} = \min(0.5 + \sqrt{(0.25 - (\min(K_{heel}, 0.225) / 0.9))}, 0.95) \times d_{heel}$
	z_{heel} = **279** mm
Area of tension reinforcement required	$As_{heel\_des} = M_{heel} / (0.87 \times f_y \times z_{heel})$ = **74** mm^2/m
Minimum area of tension reinforcement	$As_{heel\_min} = k \times b \times t_{base}$ = **455** mm^2/m
Area of tension reinforcement required	$As_{heel\_req} = \max(As_{heel\_des}, As_{heel\_min})$ = **455** mm^2/m
Reinforcement provided	**12 mm dia.bars @ 150 mm centres**
Area of reinforcement provided	As_{heel_prov} = **754** mm^2/m

PASS - Reinforcement provided at the retaining wall heel is adequate

Check shear resistance at heel

Design shear stress	$v_{heel} = V_{heel} / (b \times d_{heel})$ = **0.099** N/mm^2
Allowable shear stress	$v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1 \text{ N/mm}^2)}, 5) \times 1 \text{ N/mm}^2$ = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress v_{c_heel} = **0.485** N/mm^2

$v_{heel} < v_{c\_heel}$ - No shear reinforcement required

Design of reinforced concrete retaining wall stem (BS 8002:1994)

Material properties

Characteristic strength of concrete f_{cu} = **35** N/mm^2

Characteristic strength of reinforcement f_y = **500** N/mm^2

Wall details

Minimum area of reinforcement k = **0.13** %

Cover to reinforcement in stem c_{stem} = **50** mm

Cover to reinforcement in wall c_{wall} = **30** mm

Factored horizontal at-rest forces on stem

Surcharge $F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds})$ = **30.2** kN/m

Moist backfill above water table $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2$ = **9.8** kN/m

Moist backfill below water table $F_{s\_m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat}$ = **35.1** kN/m

Saturated backfill $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{sat}^2$ = **19.4** kN/m

Water $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2$ = **28.9** kN/m

Calculate shear for stem design

Shear at base of stem $V_{stem} = F_{s\_sur\_f} + F_{s\_m\_a\_f} + F_{s\_m\_b\_f} + F_{s\_s\_f} + F_{s\_water\_f} - F_{prop\_f}$ =
9.8 kN/m

Calculate moment for stem design

Surcharge $M_{s\_sur} = F_{s\_sur\_f} \times (h_{stem} + t_{base}) / 2$ = **53.6** kNm/m

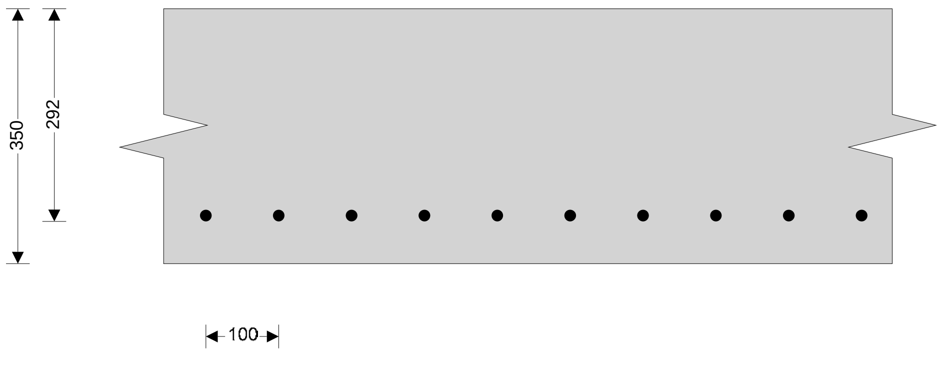
Moist backfill above water table $M_{s\_m\_a} = F_{s\_m\_a\_f} \times (2 \times h_{sat} + h_{eff} - d_{ds} + t_{base} / 2) / 3$ = **25.6**
kNm/m

Moist backfill below water table $M_{s\_m\_b} = F_{s\_m\_b\_f} \times h_{sat} / 2$ = **35.9** kNm/m

Saturated backfill $M_{s\_s} = F_{s\_s\_f} \times h_{sat} / 3$ = **13.3** kNm/m

Water $M_{s\_water} = F_{s\_water\_f} \times h_{sat} / 3$ = **19.7** kNm/m

Total moment for stem design $M_{stem} = M_{s\_sur} + M_{s\_m\_a} + M_{s\_m\_b} + M_{s\_s} + M_{s\_water}$ = **148.2** kNm/m



Check wall stem in bending

Width of wall stem b = **1000** mm/m

Depth of reinforcement $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2)$ = **292.0** mm

Constant $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu})$ = **0.050**

Compression reinforcement is not required

Lever arm $z_{stem} = \min(0.5 + \sqrt{(0.25 - (\min(K_{stem}, 0.225) / 0.9))}, 0.95) \times d_{stem}$

z_{stem} = **275** mm

Area of tension reinforcement required $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem})$ = **1239** mm^2/m

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Minimum area of tension reinforcement $A_{s\_stem\_min} = k \times b \times t_{wall}$ = **455** mm²/m

Area of tension reinforcement required $A_{s\_stem\_req} = Max(A_{s\_stem\_des}, A_{s\_stem\_min})$ = **1239** mm²/m

Reinforcement provided **16 mm dia.bars @ 100 mm centres**

Area of reinforcement provided $A_{s\_stem\_prov}$ = **2011** mm²/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress $v_{stem} = V_{stem} / (b \times d_{stem})$ = **0.034** N/mm²

Allowable shear stress $v_{adm} = min(0.8 \times \sqrt{(f_{cu} / 1 N/mm^2)}, 5) \times 1 N/mm^2$ = **4.733** N/mm²

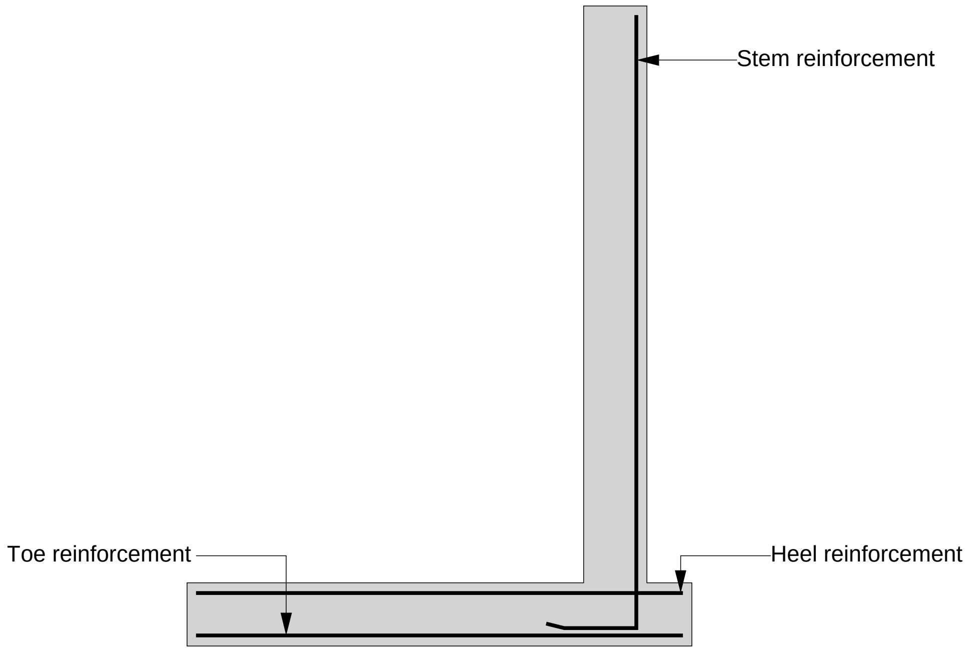
PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress v_{c_stem} = **0.675** N/mm²

$v_{stem} < v_{c\_stem}$ - No shear reinforcement required

Indicative retaining wall reinforcement diagram



Toe bars - 16 mm dia.@ 100 mm centres - (2011 mm^2/m)
Heel bars - 12 mm dia.@ 150 mm centres - (754 mm^2/m)
Stem bars - 16 mm dia.@ 100 mm centres - (2011 mm^2/m)

rc retaining wall 2 design

Floor, ceiling and roof loads doubled to allow for neighbouring load.

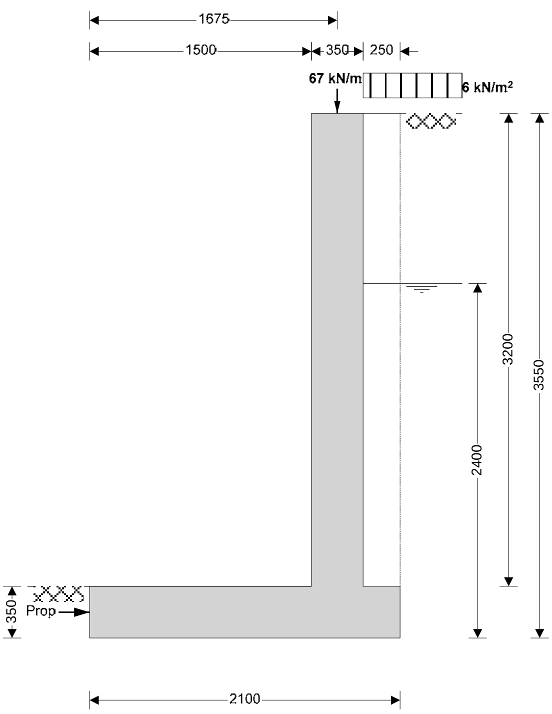
Loading:

325mm masonry wall	DL325 = 7kN/m^2 $\times$ 3m = **21.000**kN/m
225mm masonry wall	DL225 = 5kN/m^2 $\times$ 3m = **15.000**kN/m
Dormer wall	DLdor = 1.1kN/m^2 $\times$ 3m = **3.300**kN/m
Ground bearing slab DL	DLground = 24kN/m^3 $\times$ 0.15m $\times$ 4.4m / 2 = **7.920**kN/m
Floor load (1st, 2nd, loft) DL	DLfloor = 2 $\times$ 0.7kN/m^2 $\times$ 4.4m / 2 = **3.080**kN/m
Ceiling DL	DLceil = 2 $\times$ 0.325kN/m^2 $\times$ 4.4m / 3 = **0.953**kN/m
Roof DL	DLroof = 2 $\times$ 1.1kN/m^2 $\times$ 4.4m / 3 = **3.227**kN/m
Total Dead Load	DL=DL325+DL225+DLdor+DLground+DLfloor+DLceil+DLroof= **54.480**kN/m

Ground bearing slab DL	LLground = 1.5kN/m^2 $\times$ 4.4m / 2 = **3.300**kN/m
Floor load (1st, 2nd, loft) DL	LLfloor = 2 $\times$ 1.5kN/m^2 $\times$ 4.4m / 2 = **6.600**kN/m
Ceiling DL	LLceil = 2 $\times$ 0.25kN/m^2 $\times$ 4.4m / 3 = **0.733**kN/m
Roof DL	LLroof = 2 $\times$ 0.6kN/m^2 $\times$ 4.4m / 3 = **1.760**kN/m
Total Dead Load	LL= LLground+LLfloor+LLceil+LLroof= **12.393**kN/m

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type	**Cantilever propped at base**
Height of retaining wall stem	h_{stem} = **3200** mm
Thickness of wall stem	t_{wall} = **350** mm
Length of toe	l_{toe} = **1500** mm

Length of heel	l_{heel} = **250** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **2100** mm
Thickness of base	t_{base} = **350** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **1650** mm
Thickness of downstand	t_{ds} = **350** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **3550** mm
Depth of cover in front of wall	d_{cover} = **0** mm
Depth of unplanned excavation	d_{exc} = **0** mm
Height of ground water behind wall	h_{water} = **2400** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0\text{ mm})$ = **2050** mm
Density of wall construction	γ_{wall} = **23.6** kN/m^3
Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of rear face of wall	α = **90.0** deg
Angle of soil surface behind wall	β = **0.0** deg
Effective height at virtual back of wall	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **3550** mm

Retained material details

Mobilisation factor	M = **1.5**
Moist density of retained material	γ_m = **18.0** kN/m^3
Saturated density of retained material	γ_s = **21.0** kN/m^3
Design shear strength	ϕ' = **24.2** deg
Angle of wall friction	δ = **0.0** deg

Base material details

Moist density	γ_{mb} = **18.0** kN/m^3
Design shear strength	ϕ'_b = **24.2** deg
Design base friction	δ_b = **18.6** deg
Allowable bearing pressure	$P_{bearing}$ = **100** kN/m^2

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.419}$$

Passive pressure coefficient for base material

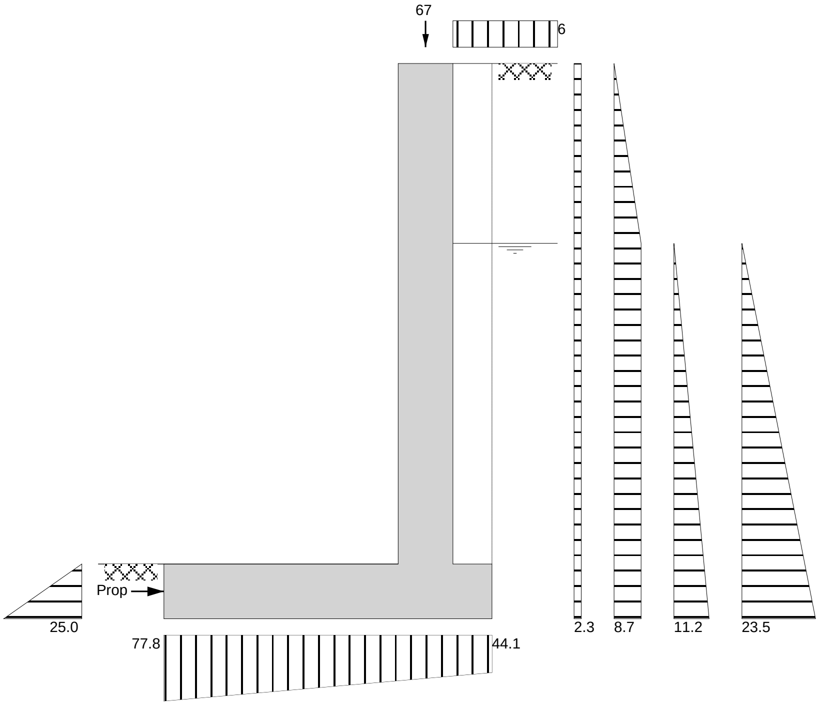
$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{4.187}$$

At-rest pressure

At-rest pressure for retained material	$K_0 = 1 - \sin(\phi') = \mathbf{0.590}$

Loading details

Surcharge load on plan	Surcharge = **5.5** kN/m^2
Applied vertical dead load on wall	W_{dead} = **54.5** kN/m
Applied vertical live load on wall	W_{live} = **12.4** kN/m
Position of applied vertical load on wall	l_{load} = **1675** mm
Applied horizontal dead load on wall	F_{dead} = **0.0** kN/m
Applied horizontal live load on wall	F_{live} = **0.0** kN/m
Height of applied horizontal load on wall	h_{load} = **0** mm



Loads shown in kN/m, pressures shown in kN/m^2

Vertical forces on wall

Wall stem	$w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **26.4** kN/m
Wall base	$w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **17.3** kN/m
Surcharge	w_{sur} = Surcharge $\times\ l_{heel}$ = **1.4** kN/m
Moist backfill to top of wall	$w_{m\_w} = l_{heel} \times (h_{stem} - h_{sat}) \times \gamma_m$ = **5.2** kN/m
Saturated backfill	$w_s = l_{heel} \times h_{sat} \times \gamma_s$ = **10.8** kN/m
Applied vertical load	$W_v = W_{dead} + W_{live}$ = **66.9** kN/m
Total vertical load	$W_{total} = w_{wall} + w_{base} + w_{sur} + w_{m\_w} + w_s + W_v$ = **128** kN/m

Horizontal forces on wall

Surcharge	$F_{sur} = K_a \times$ Surcharge $\times\ h_{eff}$ = **8.2** kN/m
Moist backfill above water table	$F_{m\_a} = 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **5** kN/m
Moist backfill below water table	$F_{m\_b} = K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **20.8** kN/m
Saturated backfill	$F_s = 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **13.5** kN/m
Water	$F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **28.3** kN/m
Total horizontal load	$F_{total} = F_{sur} + F_{m\_a} + F_{m\_b} + F_s + F_{water}$ = **75.7** kN/m

Calculate propping force

Passive resistance of soil in front of wall	$F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **4.4**
kN/m	
Propping force	$F_{prop} = \max(F_{total} - F_p - (W_{total} - w_{sur} - W_{live}) \times \tan(\delta_b), 0\text{ kN/m})$
	F_{prop} = **32.9** kN/m

Overturning moments

Surcharge	$M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **14.5** kNm/m
Moist backfill above water table	$M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **13.9** kNm/m
Moist backfill below water table	$M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2$ = **25** kNm/m
Saturated backfill	$M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **10.8** kNm/m
Water	$M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **22.6** kNm/m
Total overturning moment	$M_{ot} = M_{sur} + M_{m\_a} + M_{m\_b} + M_s + M_{water}$ = **86.7** kNm/m

Restoring moments

Wall stem	$M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2) =$ **44.3** kNm/m
Wall base	$M_{base} = w_{base} \times l_{base} / 2 =$ **18.2** kNm/m
Moist backfill	$M_{m\_r} = (w_{m\_w} \times (l_{base} - l_{heel} / 2) + w_{m\_s} \times (l_{base} - l_{heel} / 3)) =$ **10.2** kNm/m
Saturated backfill	$M_{s\_r} = w_s \times (l_{base} - l_{heel} / 2) =$ **21.3** kNm/m
Design vertical dead load	$M_{dead} = W_{dead} \times l_{load} =$ **91.3** kNm/m
Total restoring moment	$M_{rest} = M_{wall} + M_{base} + M_{m\_r} + M_{s\_r} + M_{dead} =$ **185.2** kNm/m

Check bearing pressure

Surcharge	$M_{sur\_r} = w_{sur} \times (l_{base} - l_{heel} / 2) =$ **2.7** kNm/m
Design vertical live load	$M_{live} = W_{live} \times l_{load} =$ **20.8** kNm/m
Total moment for bearing	$M_{total} = M_{rest} - M_{ot} + M_{sur\_r} + M_{live} =$ **122** kNm/m
Total vertical reaction	$R = W_{total} =$ **128.0** kN/m
Distance to reaction	$x_{bar} = M_{total} / R =$ **953** mm
Eccentricity of reaction	$e = abs((l_{base} / 2) - x_{bar}) =$ **97** mm

Reaction acts within middle third of base

Bearing pressure at toe	$p_{toe} = (R / l_{base}) + (6 \times R \times e / l_{base}^2) =$ **77.8** kN/m^2
Bearing pressure at heel	$p_{heel} = (R / l_{base}) - (6 \times R \times e / l_{base}^2) =$ **44.1** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

Ultimate limit state load factors

Dead load factor	$\gamma_{f\_d} =$ **1.4**
Live load factor	$\gamma_{f\_l} =$ **1.6**
Earth and water pressure factor	$\gamma_{f\_e} =$ **1.4**

Factored vertical forces on wall

Wall stem	$w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall} =$ **37** kN/m
Wall base	$w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base} =$ **24.3** kN/m
Surcharge	$w_{sur\_f} = \gamma_{f\_l} \times$ Surcharge $\times l_{heel} =$ **2.2** kN/m
Moist backfill to top of wall	$w_{m\_w\_f} = \gamma_{f\_d} \times l_{heel} \times (h_{stem} - h_{sat}) \times \gamma_m =$ **7.2** kN/m
Saturated backfill	$w_{s\_f} = \gamma_{f\_d} \times l_{heel} \times h_{sat} \times \gamma_s =$ **15.1** kN/m
Applied vertical load	$W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live} =$ **96.1** kN/m
Total vertical load	$W_{total\_f} = w_{wall\_f} + w_{base\_f} + w_{sur\_f} + w_{m\_w\_f} + w_{s\_f} + W_{v\_f} =$ **181.9** kN/m

Factored horizontal at-rest forces on wall

Surcharge	$F_{sur\_f} = \gamma_{f\_l} \times K_0 \times$ Surcharge $\times h_{eff} =$ **18.4** kN/m
Moist backfill above water table	$F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times \gamma_m \times (h_{eff} - h_{water})^2 =$ **9.8** kN/m
Moist backfill below water table	$F_{m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water} =$ **41** kN/m
Saturated backfill	$F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{water}^2 =$ **26.6** kN/m
Water	$F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water} =$ **39.6** kN/m
Total horizontal load	$F_{total\_f} = F_{sur\_f} + F_{m\_a\_f} + F_{m\_b\_f} + F_{s\_f} + F_{water\_f} =$ **135.5** kN/m

Calculate propping force

Passive resistance of soil in front of wall	$F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb} =$ **6.1** kN/m
Propping force	$F_{prop\_f} = \max(F_{total\_f} - F_{p\_f} - (W_{total\_f} - w_{sur\_f} - \gamma_{f\_l} \times W_{live}) \times \tan(\delta_b), 0$ kN/m)
	$F_{prop\_f} =$ **75.6** kN/m

Factored overturning moments

Surcharge $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2 =$ **32.7** kNm/m

Moist backfill above water table $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3 =$ **27.4** kNm/m

Moist backfill below water table $M_{m\_b\_f} = F_{m\_b\_f} \times (h_{water} - 2 \times d_{ds}) / 2 =$ **49.2** kNm/m

Saturated backfill $M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3 =$ **21.3** kNm/m

Water $M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3 =$ **31.6** kNm/m

Total overturning moment $M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f} + M_{m\_b\_f} + M_{s\_f} + M_{water\_f} =$ **162.3** kNm/m

Restoring moments

Wall stem $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2) =$ **62** kNm/m

Wall base $M_{base\_f} = w_{base\_f} \times l_{base} / 2 =$ **25.5** kNm/m

Surcharge $M_{sur\_r\_f} = w_{sur\_f} \times (l_{base} - l_{heel} / 2) =$ **4.3** kNm/m

Moist backfill $M_{m\_r\_f} = (w_{m\_w\_f} \times (l_{base} - l_{heel} / 2) + w_{m\_s\_f} \times (l_{base} - l_{heel} / 3)) =$ **14.3** kNm/m

Saturated backfill $M_{s\_r\_f} = w_{s\_f} \times (l_{base} - l_{heel} / 2) =$ **29.8** kNm/m

Design vertical load $M_{v\_f} = W_{v\_f} \times l_{load} =$ **161** kNm/m

Total restoring moment $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{sur\_r\_f} + M_{m\_r\_f} + M_{s\_r\_f} + M_{v\_f} =$ **296.9** kNm/m

Factored bearing pressure

Total moment for bearing $M_{total\_f} = M_{rest\_f} - M_{ot\_f} =$ **134.6** kNm/m

Total vertical reaction $R_f = W_{total\_f} =$ **181.9** kN/m

Distance to reaction $x_{bar\_f} = M_{total\_f} / R_f =$ **740** mm

Eccentricity of reaction $e_f = abs((l_{base} / 2) - x_{bar\_f}) =$ **310** mm

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2) =$ **163.4** kN/m²

Bearing pressure at heel $p_{heel\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2) =$ **9.9** kN/m²

Rate of change of base reaction $rate = (p_{toe\_f} - p_{heel\_f}) / l_{base} =$ **73.10** kN/m²/m

Bearing pressure at stem / toe $p_{stem\_toe\_f} = \max(p_{toe\_f} - (rate \times l_{toe}), 0\ kN/m^2) =$ **53.7** kN/m²

Bearing pressure at mid stem $p_{stem\_mid\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall} / 2)), 0\ kN/m^2) =$ **40.9** kN/m²

Bearing pressure at stem / heel $p_{stem\_heel\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall})), 0\ kN/m^2) =$ **28.1** kN/m²

Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

Characteristic strength of concrete $f_{cu} =$ **35** N/mm²

Characteristic strength of reinforcement $f_y =$ **500** N/mm²

Base details

Minimum area of reinforcement $k =$ **0.13** %

Cover to reinforcement in toe $c_{toe} =$ **50** mm

Calculate shear for toe design

Shear from bearing pressure $V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2 =$ **162.8** kN/m

Shear from weight of base $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} =$ **17.3** kN/m

Total shear for toe design $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} =$ **145.5** kN/m

Calculate moment for toe design

Moment from bearing pressure $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6 =$ **171.9** kNm/m

Moment from weight of base $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) =$ **16.2** kNm/m

Total moment for toe design $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} =$ **155.7** kNm/m

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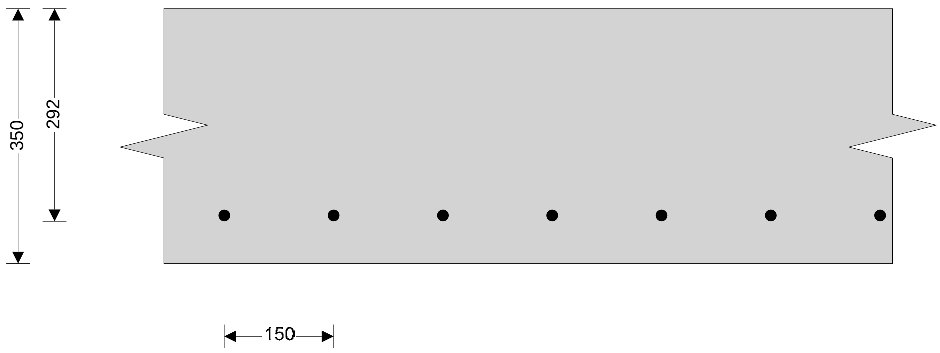


Check toe in bending

Width of toe — b = **1000** mm/m

Depth of reinforcement — $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2)$ = **292.0** mm

Constant — $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu})$ = **0.052**

Compression reinforcement is not required

Lever arm — $z_{toe} = \min(0.5 + \sqrt{(0.25 - (\min(K_{toe}, 0.225) / 0.9))}, 0.95) \times d_{toe}$

z_{toe} = **274** mm

Area of tension reinforcement required — $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe})$ = **1307** mm²/m

Minimum area of tension reinforcement — $A_{s\_toe\_min} = k \times b \times t_{base}$ = **455** mm²/m

Area of tension reinforcement required — $A_{s\_toe\_req} = \mathrm{Max}(A_{s\_toe\_des}, A_{s\_toe\_min})$ = **1307** mm²/m

Reinforcement provided — **16 mm dia.bars @ 150 mm centres**

Area of reinforcement provided — $A_{s\_toe\_prov}$ = **1340** mm²/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress — $v_{toe} = V_{toe} / (b \times d_{toe})$ = **0.498** N/mm²

Allowable shear stress — $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1\ \mathrm{N/mm^2})}, 5) \times 1\ \mathrm{N/mm^2}$ = **4.733** N/mm²

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress — v_{c_toe} = **0.590** N/mm²

$v_{toe} < v_{c\_toe}$ - No shear reinforcement required

Design of reinforced concrete retaining wall heel (BS 8002:1994)

Material properties

Characteristic strength of concrete — f_{cu} = **35** N/mm²

Characteristic strength of reinforcement — f_y = **500** N/mm²

Base details

Minimum area of reinforcement — k = **0.13** %

Cover to reinforcement in heel — c_{heel} = **50** mm

Calculate shear for heel design

Shear from bearing pressure — $V_{heel\_bear} = (p_{heel\_f} + p_{stem\_heel\_f}) \times l_{heel} / 2$ = **4.8** kN/m

Shear from weight of base — $V_{heel\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{heel} \times t_{base}$ = **2.9** kN/m

Shear from weight of moist backfill — $V_{heel\_wt\_m} = w_{m\_w\_f}$ = **7.2** kN/m

Shear from weight of saturated backfill — $V_{heel\_wt\_s} = w_{s\_f}$ = **15.1** kN/m

Shear from surcharge — $V_{heel\_sur} = w_{sur\_f}$ = **2.2** kN/m

Total shear for heel design — $V_{heel} = - V_{heel\_bear} + V_{heel\_wt\_base} + V_{heel\_wt\_m} + V_{heel\_wt\_s} + V_{heel\_sur}$
= **22.7** kN/m

Calculate moment for heel design

Moment from bearing pressure — $M_{heel\_bear} = (2 \times p_{heel\_f} + p_{stem\_mid\_f}) \times (l_{heel} + t_{wall} / 2)^2 / 6$ = **1.8** kNm/m

Moment from weight of base — $M_{heel\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{heel} + t_{wall} / 2)^2 / 2) =$ **1** kNm/m

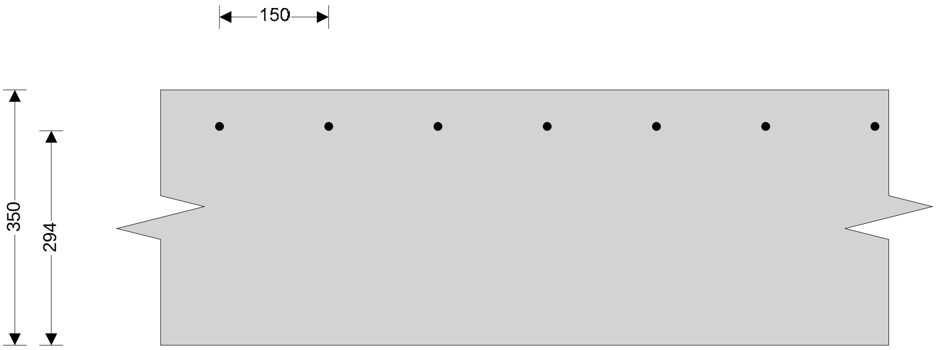
Moment from weight of moist backfill — $M_{heel\_wt\_m} = w_{m\_w\_f} \times (l_{heel} + t_{wall}) / 2 =$ **2.2** kNm/m

Moment from weight of saturated backfill — $M_{heel\_wt\_s} = w_{s\_f} \times (l_{heel} + t_{wall}) / 2 =$ **4.5** kNm/m

Moment from surcharge — $M_{heel\_sur} = w_{sur\_f} \times (l_{heel} + t_{wall}) / 2 =$ **0.7** kNm/m

Total moment for heel design — $M_{heel} = - M_{heel\_bear} + M_{heel\_wt\_base} + M_{heel\_wt\_m} + M_{heel\_wt\_s} +$

$M_{heel\_sur} =$ **6.6** kNm/m



Check heel in bending

Width of heel — b = **1000** mm/m

Depth of reinforcement — $d_{heel} = t_{base} - c_{heel} - (\phi_{heel} / 2) =$ **294.0** mm

Constant — $K_{heel} = M_{heel} / (b \times d_{heel}^2 \times f_{cu}) =$ **0.002**

Compression reinforcement is not required

Lever arm — $z_{heel} = \min(0.5 + \sqrt{(0.25 - (\min(K_{heel}, 0.225) / 0.9))},0.95) \times d_{heel}$

$z_{heel} =$ **279** mm

Area of tension reinforcement required — $A_{s\_heel\_des} = M_{heel} / (0.87 \times f_y \times z_{heel}) =$ **54** mm²/m

Minimum area of tension reinforcement — $A_{s\_heel\_min} = k \times b \times t_{base} =$ **455** mm²/m

Area of tension reinforcement required — $A_{s\_heel\_req} = \text{Max}(A_{s\_heel\_des}, A_{s\_heel\_min}) =$ **455** mm²/m

Reinforcement provided — **12 mm dia.bars @ 150 mm centres**

Area of reinforcement provided — $A_{s\_heel\_prov} =$ **754** mm²/m

PASS - Reinforcement provided at the retaining wall heel is adequate

Check shear resistance at heel

Design shear stress — $v_{heel} = V_{heel} / (b \times d_{heel}) =$ **0.077** N/mm²

Allowable shear stress — $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1 \text{ N/mm}^2)}, 5) \times 1 \text{ N/mm}^2 =$ **4.733** N/mm²

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress — $v_{c\_heel} =$ **0.485** N/mm²

$v_{heel} < v_{c\_heel}$ - No shear reinforcement required

Design of reinforced concrete retaining wall stem (BS 8002:1994)

Material properties

Characteristic strength of concrete — $f_{cu} =$ **35** N/mm²

Characteristic strength of reinforcement — $f_y =$ **500** N/mm²

Wall details

Minimum area of reinforcement — k = **0.13** %

Cover to reinforcement in stem — $c_{stem} =$ **50** mm

Cover to reinforcement in wall — $c_{wall} =$ **30** mm

Factored horizontal at-rest forces on stem

Surcharge — $F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds}) =$ **16.6** kN/m

Moist backfill above water table — $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2 =$ **9.8** kN/m

Moist backfill below water table — $F_{s\_m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat} =$ **35.1** kN/m

Saturated backfill — $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{sat}^2$ = **19.4** kN/m

Water — $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2$ = **28.9** kN/m

Calculate shear for stem design

Shear at base of stem — $V_{stem} = F_{s\_sur\_f} + F_{s\_m\_a\_f} + F_{s\_m\_b\_f} + F_{s\_s\_f} + F_{s\_water\_f} - F_{prop\_f}$ = **34.2** kN/m

Calculate moment for stem design

Surcharge — $M_{s\_sur} = F_{s\_sur\_f} \times (h_{stem} + t_{base}) / 2$ = **29.5** kNm/m

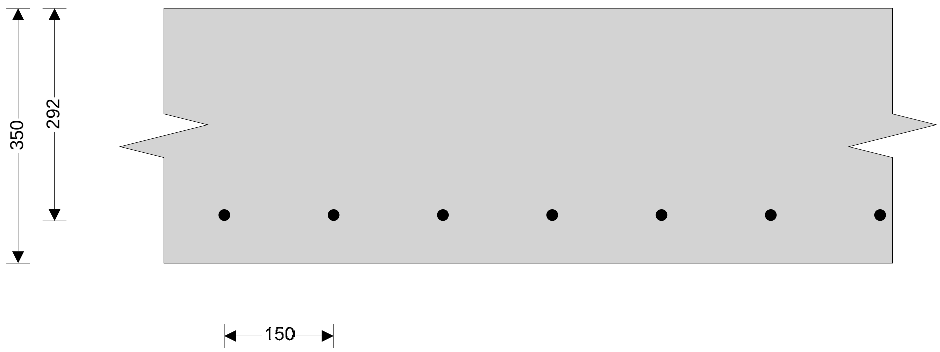
Moist backfill above water table — $M_{s\_m\_a} = F_{s\_m\_a\_f} \times (2 \times h_{sat} + h_{eff} - d_{ds} + t_{base} / 2) / 3$ = **25.6** kNm/m

Moist backfill below water table — $M_{s\_m\_b} = F_{s\_m\_b\_f} \times h_{sat} / 2$ = **35.9** kNm/m

Saturated backfill — $M_{s\_s} = F_{s\_s\_f} \times h_{sat} / 3$ = **13.3** kNm/m

Water — $M_{s\_water} = F_{s\_water\_f} \times h_{sat} / 3$ = **19.7** kNm/m

Total moment for stem design — $M_{stem} = M_{s\_sur} + M_{s\_m\_a} + M_{s\_m\_b} + M_{s\_s} + M_{s\_water}$ = **124.1** kNm/m



Check wall stem in bending

Width of wall stem — b = **1000** mm/m

Depth of reinforcement — $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2)$ = **292.0** mm

Constant — $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu})$ = **0.042**

Compression reinforcement is not required

Lever arm — $z_{stem} = \min(0.5 + \sqrt{(0.25 - (\min(K_{stem}, 0.225) / 0.9))}, 0.95) \times d_{stem}$

z_{stem} = **277** mm

Area of tension reinforcement required — $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem})$ = **1028** mm²/m

Minimum area of tension reinforcement — $A_{s\_stem\_min} = k \times b \times t_{wall}$ = **455** mm²/m

Area of tension reinforcement required — $A_{s\_stem\_req} = \max(A_{s\_stem\_des}, A_{s\_stem\_min})$ = **1028** mm²/m

Reinforcement provided — **16 mm dia.bars @ 150 mm centres**

Area of reinforcement provided — $A_{s\_stem\_prov}$ = **1340** mm²/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress — $v_{stem} = V_{stem} / (b \times d_{stem})$ = **0.117** N/mm²

Allowable shear stress — $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1\ N/mm^2)}, 5) \times 1\ N/mm^2$ = **4.733** N/mm²

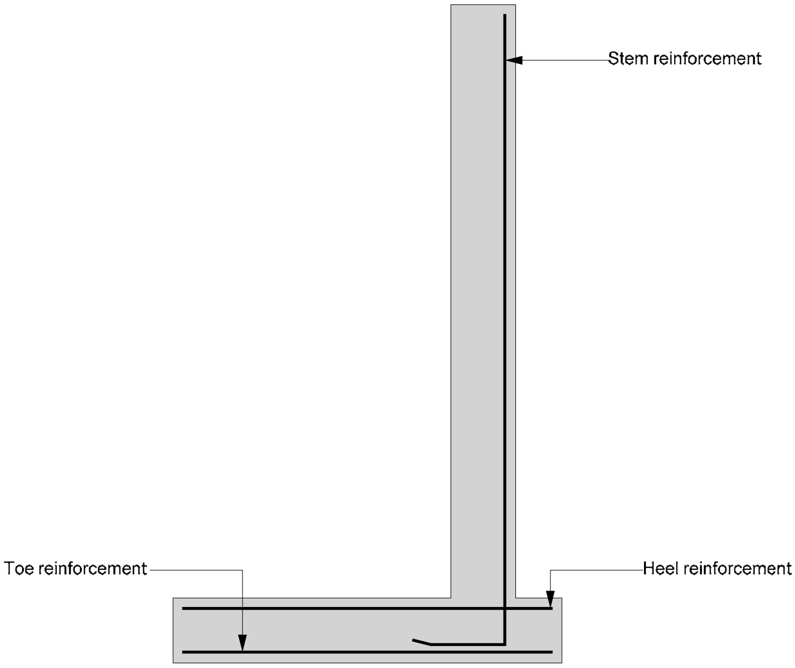
PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress — v_{c_stem} = **0.590** N/mm²

$v_{stem} < v_{c\_stem}$ - No shear reinforcement required

Indicative retaining wall reinforcement diagram



Toe bars - 16 mm dia.@ 150 mm centres - (1340 mm^2/m)
Heel bars - 12 mm dia.@ 150 mm centres - (754 mm^2/m)
Stem bars - 16 mm dia.@ 150 mm centres - (1340 mm^2/m)

Appendix D

Method Statement

156 Goldhurst Terrace

1. Basement Formation Suggested Method Statement.

1.1. This method statement provides an approach which will allow the basement design to be correctly considered during construction, and the temporary support to be provided during the works. The Contractor is responsible for the works on site and the final temporary works methodology and design on this site and any adjacent sites.

1.2. This method statement 156 Goldhurst Terrace has been written by a Chartered Engineer. The sequencing has been developed considering guidance from ASUC.

1.3. This method has been produced to allow for improved costings and for inclusion in the party wall Award. Should the contractor provide alternative methodology the changes shall be at their own costs, and an Addendum to the Party Wall Award will be required.

1.4. Contact party wall surveyors to inform them of any changes to this method statement.

1.5. The approach followed in this design is; to remove load from above and place loads onto supporting steelwork, then to cast cantilever retaining walls in underpin sections at the new basement level.

1.6. The cantilever pins are designed to be inherently stable during the construction stage without temporary propping to the head. The base benefits from propping, this is provided in the final condition by the ground slab. In the temporary condition the edge of the slab is buttressed against the soil in the middle of the property, also the skin friction between the concrete base and the soil provides further resistance. The central slab is to be poured in a maximum of a 1/3 of the floor area.

1.7. A soil investigation has been undertaken. The soil conditions are London clays.

1.8. The bearing pressures have been limited to 100kN/m^2. This is standard loadings for local ground conditions and acceptable to building control and their approvals.

2. Enabling Works

2.1. The site is to be hoarded with ply sheet to 2.2m to prevent unauthorised public access.

2.2. Licenses for Skips and conveyors to be posted on hoarding

2.3. Provide protection to public where conveyor extends over footpath. Depending on the requirements of the local authority, construct a plywood bulkhead onto the pavement. Hoarding to have a plywood roof covering, night-lights and safety notices.

2.4. On commencement of construction the contractor will determine the foundation type, width and depth. Any discrepancies will be reported to the structural engineer in order that the detailed design may be modified as necessary.

3. Basement Sequencing

3.1. Begin by placing cantilevered walls 1 2 noted on plans. (Cantilevered walls to be placed in accordance with section 4.)

3.2. Needle & prop the walls over.

3.3. Insert steel over and sit on cantilevered walls.

3.3.1. Beams over 6m to be jacked on site to reduce deflections of floors.

3.3.2. Dry pack to steelwork. Ensure a minimum of 72 hours from casting cantilevered walls to dry packing. Grout column bases

3.4. Excavate Light well to front of property down to 600mm below external ground level.

3.5. Excavate first front corner of light well. (Follow methodology in section 4)

3.6. Excavate second front corner of light well. (Follow methodology in section 4)

3.7. Continue excavating section pins to form front light well. (Follow methodology in section 4)

3.8. Place cantilevered retaining wall to the left side of front opening. After 72 hours place cantilevered retaining wall to the right side of front opening.

3.9. Needle and prop bay. Insert support

3.10. Excavate out first 1.2m around front opening prop floor and erect conveyor.

3.11. Continue cantilevered wall formation around perimeter of basement following the numbering sequence on the drawings.

3.12. Excavation for the next numbered sections of underpinning shall not commence until at least 8 hours after drypacking of previous works. Excavation of adjacent pin to not commence until 24 hours after drypacking. (24hours possible due to inclusion of Conbextra 100 cement accelerator to dry pack mix)

3.13. Excavate a maximum of a 1/3 of the middle section of basement floor. Place reinforcement to central section of ground bearing slab and pour concrete. Excavate next third and cast slab. Excavate and cast final third and cast.

3.14. Provide structure to ground floor and water proofing to retaining walls as required.

4. Underpinning and Cantilevered Walls

4.1. Prior to installation of new structural beams in the superstructure, the contractor may undertake the local exploration of specific areas in the superstructure. This will confirm the exact form and location of the temporary works that are required. The permanent structural work can then be undertaken whilst ensuring that the full integrity of the structure above is maintained.

4.2. Provide propping to floor where necessary.

4.3. Excavate first section of retaining wall (no more than 1000mm wide). Where excavation is greater than 1.2m deep provide temporary propping to sides of excavation to prevent earth collapse (Health and Safety). A 1000mm width wall has a lower risk of collapse to the heel face.

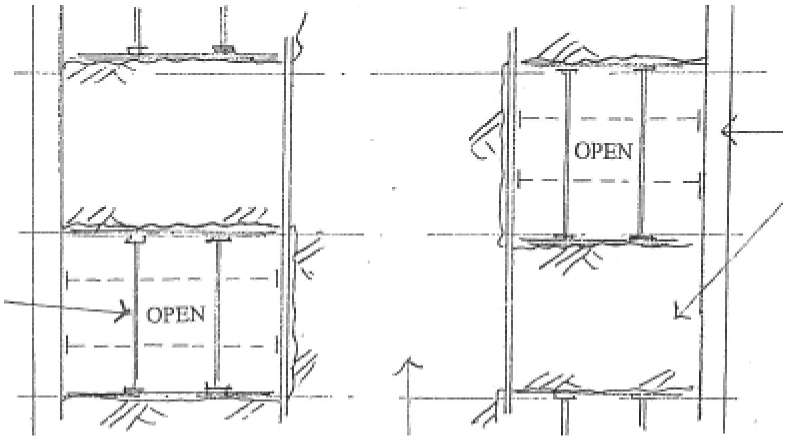


Figure 25 – Schematic Plan view of Soil Propping

Figure 26 Propping

4.4. Backpropping of rear face. Rear face to be propped in the temporary conditions with a minimum of 2 Trench sheets. Trench sheets are to extend over entire height of excavation. Trench sheets can be placed in short sections are the excavation progresses.

4.4.1. If the ground is stable, trench sheets can be removed as the wall reinforcement is placed and the shuttering is constructed.

4.4.2. Where soft spots are encountered leave in trench sheets or alternatively back prop with Precast lintels or trench sheeting. (If the soil support to the ends of the lintels is insufficient then brace the ends of the PC lintels with 150x150 C24 Timbers and prop with Acrows diagonally back to the floor.)

4.4.3. Where voids are present behind the lintels or trench sheeting. Grout voids behind sacrificial propping; Grout to be 3:1 sand cement packed into voids.

4.4.4. Prior to casting place layer of DPM between trench sheeting (or PC lintels) and new concrete. The lintels are to be cut into the soil by 150mm either side of the pin. A site stock of a minimum of 10 lintels to be present for to prevent delays due to ordering.

4.5. If cut face is not straight, or sacrificial boards noted have been used, place a 15mm cement particle board between sacrificial sheets and or soil prior to casting. Cement particle board is to line up with the adjacent owners face of wall. The method adopted to prevent localised collapse of the soil is to install these progressively one at a time. Cement particle board must be used to in any condition where overspill onto the adjacent owners land is possible.

4.6. Excavate base. Mass concrete heels to be excavated. If soil over unstable prop top with PC lintel and sacrificial prop.

4.7. Visually inspect the footings and provide propping to local brickwork, if necessary sacrificial acrow, or pit props, to be sacrificial and cast into the retaining wall.

4.8. Clear underside of existing footing.

4.9. Local authority inspection to be carried for approval of excavation base.

4.10. Place reinforcement for retaining wall base & toe. Site supervisor to inspect and sign off works for proceeding to next stage.

4.11. Cast base. (on short stems it is possible to cast base and wall at same time)

4.12. Take 2 cubes of concrete and store for testing. Test one at 28 days if result is low test second cube. Provide results to client and design team on request or if values are below those required.

4.13. Horizontal temporary prop to base of wall to be inserted. Alternatively cast base against soil.

4.14. Place reinforcement for retaining wall stem. Site supervisor to inspect and sign off works for proceeding to next stage.

4.15. Drive H16 Bars U Bars into soil along centre line of stem to act as shear ties to adjacent wall.

4.16. Place shuttering & pour concrete for retaining wall. Stop a minimum of 75mm from the underside of existing footing.

4.17. Ram in drypack between retaining wall and existing masonry. (24 hours after pouring the concrete pin the gap shall be filled using a dry pack mortar.)

4.18. After 24 hours the temporary wall shutters are removed.

4.19. Trim back existing masonry corbel and concrete on internal face.

4.20. Site supervisor to inspect and sign off for proceeding to the next stage. A record will be kept of the sequence of construction, which will be in strict accordance with recognised industry procedures.

5. Floor Support

Timber Floor

5.1. The timber floor will remain in situ, and be supported by a series of steel beams that will support the floors, to provide the open areas in the basement.

5.2. Position 100 x 100mm temporary timber beam lightly packed to underside of joists either side of existing sleeper wall and support with vertical acrow props @ 750 centres. Remove sleeper walls and insert steel beam as a replacement. Beams to bear onto concrete padstones built into the masonry walls (refer to Structural Engineer's details for padstone & beam sizes)

5.3. Dismantle props and remove timber plates on completion of installation of permanent steel beams.

6. Supporting existing walls above basement excavation

6.1. Where steel beams need to be installed directly under load bearing walls, temporary works will be required to enable this work. Support comprises the installation of steel needle beams at high level, supported on vertical props, to enable safe removal of brickwork below, and installation of the new beams and columns.

6.1.1. The condition of the brickworks must be inspected by the foreman to determine its condition and to assess the centres of needles. The foreman must inspect upstairs to consider where loads are greatest. Point loads and between windows should be given greater consideration.

6.1.2. Needles are to be spaced to prevent the brickwork above “saw toothing”. Where brickwork is good needles must be placed at a maximum of 1100mm centers. Lighter needles or strong boys should be placed at tighter centres under door thresholds

6.2. Props are to be placed on Sleepers of firm ground or if necessary temporary footings will be cast.

6.3. Once the props are fully tightened, the brickwork will be broken out carefully by hand. All necessary platforms and crash decks will be provided during this operation.

6.4. Decking and support platforms to enable handling of steel beams and columns will be provided as required.

6.5. Once full structural bearing is provided via beams and columns down to the new basement floor level. The temporary works will be redundant and can be safely removed.

6.6. Any voids between the top of the permanent steel beams and the underside of the existing walls will be packed out as necessary. Voids will be drypacked with a 1:3 (cement: sharp sand) drypack layer, between the top of the steel and underside of brickwork above.

6.7. Any voids in the brickwork left after removal of needle beams can at this point be repaired by bricking up and/or drypacking, to ensure continuity of the structural fabric.

7. Approval

7.1. Building control officer/approved inspector to inspect pin bases and reinforcement prior to casting concrete.

7.2. Contractor to keep list of dates pins inspected & cast

7.3. One month after work completed the contractor is to contact adjacent party wall surveyor to attend site and complete final condition survey and to sign off works.

CROFT STRUCTURAL ENGINEERS

8. Trench sheet design and temporary prop Calculations

This calculation has been provided for the trench sheet and prop design of standard underpins in the temporary condition. There are gaps left between the sheeting and as such no water pressure will occur. Any water present will flow through the gaps between the sheeting and will be required to pump out.

Trench sheets should be placed at centers to deal with the ground. It is expected that the soil between the trench sheeting will arch. Looser soil will required tighter centers. It is typical for underpins to be placed at 1200c/c, in this condition the highest load on a trench sheet is when 2 trench sheets are used. It is for this design that these calculations have been provided.

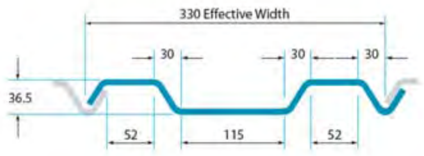
Soil and ground conditions are variable. Typically one finds that in the temporary condition clays are more stable and the C_u (cohesive) values in clay reduce the risk of collapse. It is this cohesive nature that allows clays to be cut into a vertical slope. For these calculations weak sand and gravels have been assumed The soil properties are:

Surcharge	sur = **10.** kN/m^2	
Soil density	δ = **20** kN/m^3	
Angle of friction	ϕ = **25** °	
Soil depth	Dsoil = **3000.000** mm	
	$k_a = (1 - \sin(\phi)) / (1 + \sin(\phi))$	= **0.406**
	$k_p = 1 / k_a$	= **2.464**
Soil Pressure bottom	soil = $k_a * \delta *$ Dsoil	= **21.916**kN/m^2
Surcharge pressure	surcharge = sur * k_a	= **4.059** kN/m^2

Standard Lap Trench Sheeting

STANDARD LAP

The overlapping trench sheeting profile is designed primarily for construction work and also temporary deployment.



Technical Information

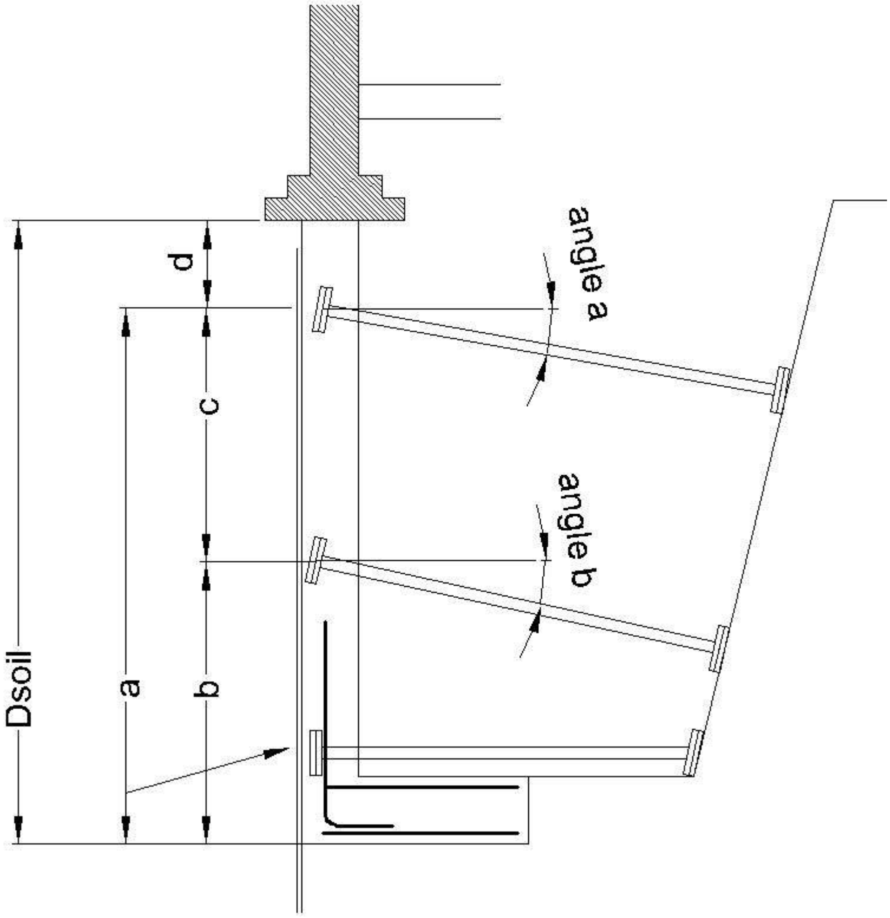
Effective width per sheet (mm)	330
Thickness (mm)	3.4
Depth (mm)	35
Weight per linear metre (kg/m)	10.8
Weight per m² (kg)	32.9
Section modulus per metre width (cm³)	48.3
Section modulus per sheet (cm³)	15.9
I value per metre width (cm⁴)	81.7
I value per sheet (cm⁴)	26.9
Total rolled metres per tonne	92.1

Sxx = 15.9 cm^3

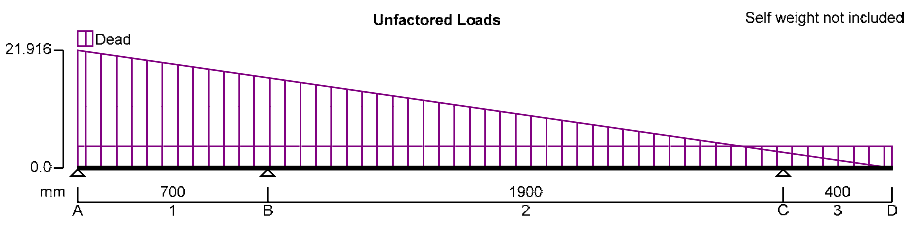
py = 275N/mm^2

Ixx = 26.9cm^4

A = ($1m^2$ * 32.9kg/m^2) / (330mm * 7750kg/m^3) = **12864.125**mm^2



Length a	a = **2.600** m
Length b bottom	b = **0.700** m
Length c Middle	c = a – b = **1.900**m
Length d top	d = Dsoil – a = **0.400**m



CONTINUOUS BEAM ANALYSIS - INPUT

BEAM DETAILS

Number of spans = **3**

Material Properties:

Modulus of elasticity = **205** kN/mm^2

Material density = **7860** kg/m^3

Support Conditions:

Support A	Vertically **"Restrained"**	Rotationally **"Free"**
Support B	Vertically **"Restrained"**	Rotationally **"Free"**
Support C	Vertically **"Restrained"**	Rotationally **"Free"**

Support D Vertically **"Free"** Rotationally **"Free"**

Span Definitions:

Span 1	Length = **700** mm	Cross-sectional area = **12864** mm^2	Moment of inertia = **269.×10^3** mm^4
Span 2	Length = **1900** mm	Cross-sectional area = **12864** mm^2	Moment of inertia = **269.×10^3** mm^4
Span 3	Length = **400** mm	Cross-sectional area = **12864** mm^2	Moment of inertia = **269.×10^3** mm^4

LOADING DETAILS

Beam Loads:

Load 1 UDL Dead load **4.1** kN/m

Load 2 VDL Dead load **21.9** kN/m to **0.0** kN/m

LOAD COMBINATIONS

Load combination 1

Span 1 1×Dead

Span 2 1×Dead

Span 3 1×Dead

CONTINUOUS BEAM ANALYSIS - RESULTS

Unfactored support reactions

	Dead (kN)							
Support A	**-1.4**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
Support B	**-32.8**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
Support C	**-10.8**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**
Support D	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**	**0.0**

Support Reactions - Combination Summary

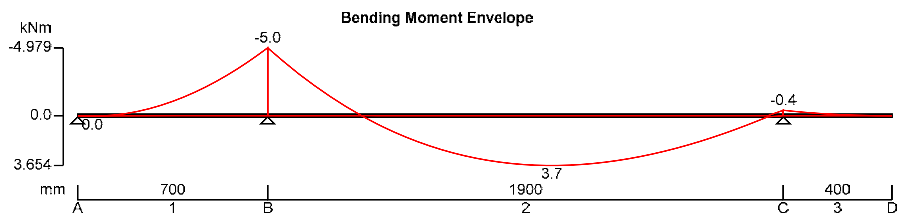
Support A	Max react = **-1.4** kN	Min react = **-1.4** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support B	Max react = **-32.8** kN	Min react = **-32.8** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support C	Max react = **-10.8** kN	Min react = **-10.8** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support D	Max react = **0.0** kN	Min react = **0.0** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm

Beam Max/Min results - Combination Summary

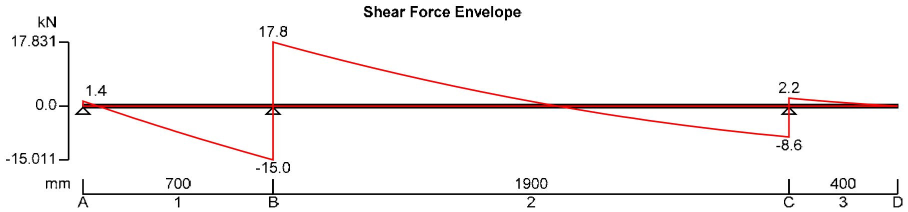
Maximum shear = **17.8** kN Minimum shearF_{min} = **-15.0** kN

Maximum moment = **3.7** kNm Minimum moment = **-5.0** kNm

Maximum deflection = **21.0** mm Minimum deflection = **-14.3** mm



CROFT STRUCTURAL ENGINEERS



Number of sheets Nos = 2

Mallowable = Sxx * py * Nos = **8.745**kNm

SRU 4.0

Safe working loads for Acrow Props — loads given in kN

For normal purposes 1 kilo Newton (kN) = 100 kg	Height m / ft	2.0 / 6.6	2.25 / 7.4	2.5 / 8.2	2.75 / 9.0	3.0 / 9.8	3.25 / 10.7	3.5 / 11.5	3.75 / 12.3	4.0 / 13.1	4.25 / 13.9	4.5 / 14.8	4.75 / 15.6
TABLE A Props loaded concentrically and erected vertically	Prop size 1 or 2	35	35	35	34	27	23						-
	Prop size 3				34	27	23	21	19	17			
	Prop size 4						32	25	21	18	16	14	12
TABLE B Props loaded concentrically and erected 1½° max. out of vertical	Prop size 1 or 2 or 3	35	32	26	23	19	17	15	13	12			
	Prop size 4						24	19	15	12	11	10	9
TABLE C Props loaded 25 mm eccentricity and erected 1½° max. out of vertical	Prop size 1 or 2 or 3	17	17	17	17	15	13	11	10	9			
	Prop size 4						17	14	11	10	9	8	7
TABLE D Props loaded concentrically and erected 1½° out of vertical and laced with scaffold tubes and fittings	Prop size 3				35	33	32	28	24	20			
	Prop size 4						35	35	35	35	27	25	21

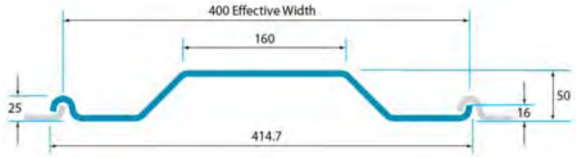
Shear V = (14.6kN + 13.4kN) /2 = 14.000kN

Any Acro Prop is accetpable

KD4 sheets

KD4

The overlapping trench sheeting profile is a heavier version of the Standard Lap, with a wider gauge and width coverage, designed in large for construction work.



Technical Information

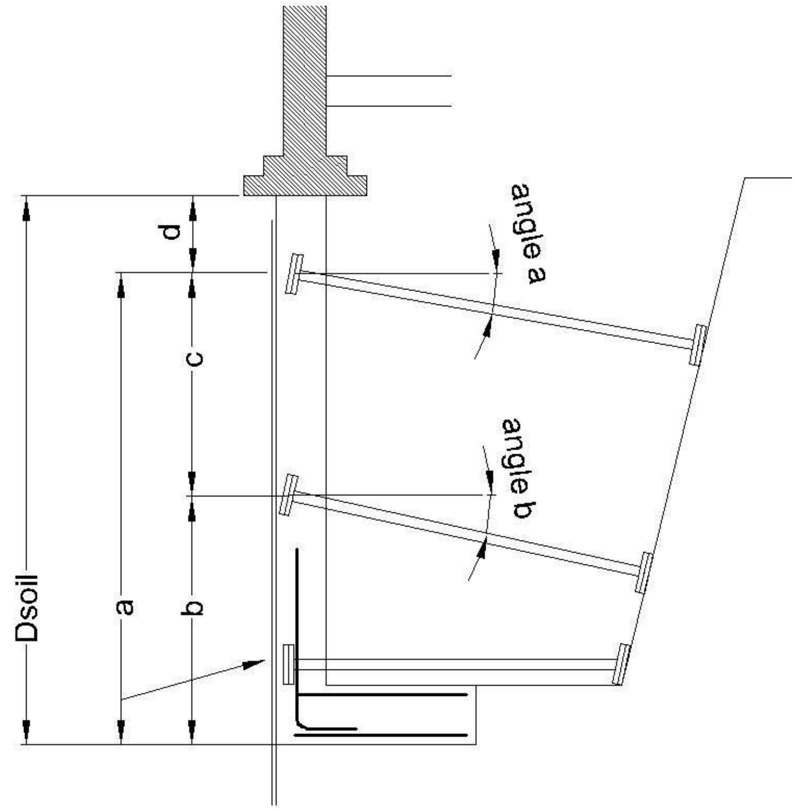
Effective width per sheet (mm)	400
Thickness (mm)	6.0
Depth (mm)	50
Weight per linear metre (kg/m)	21.90
Weight per m² (kg)	55.2
Section modulus per metre width (cm^3)	101
Section modulus per sheet (cm^3)	40.34
I value per metre width (cm^4)	250
I value per sheet (cm^4)	101
Total rolled metres per tonne	45.659

$Sxx = 48.3cm^3$

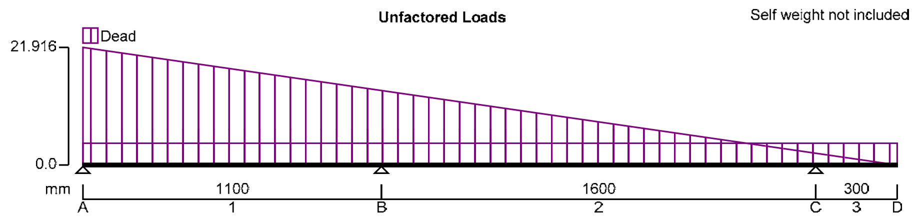
$py = 275N/mm^2$

$Ixx = 26.9cm^4$

$A = (1m^2 * 55.2kg/m^2) / (400mm * 7750kg/m^3) = \mathbf{17806.452}mm^2$



Length a	a = **2.700** m
Length b bottom	b = **1.100** m
Length c Middle	c = a – b = **1.600**m
Length d top	d = Dsoil – a = **0.300**m



CONTINUOUS BEAM ANALYSIS - INPUT

BEAM DETAILS

Number of spans = **3**

Material Properties:

Modulus of elasticity = **205** kN/mm^2 Material density = **7860** kg/m^3

Support Conditions:

Support A	Vertically **"Restrained"**	Rotationally **"Free"**
Support B	Vertically **"Restrained"**	Rotationally **"Free"**
Support C	Vertically **"Restrained"**	Rotationally **"Free"**
Support D	Vertically **"Free"**	Rotationally **"Free"**

Span Definitions:

Span 1	Length = **1100** mm	Cross-sectional area = **17806** mm^2	Moment of inertia = **269.×10^3** mm^4
Span 2	Length = **1600** mm	Cross-sectional area = **17806** mm^2	Moment of inertia = **269.×10^3** mm^4

Span 3 Length = **300** mm Cross-sectional area = **17806** mm^2 Moment of inertia = **269.**$\times$**10**^{3} mm^4

LOADING DETAILS

Beam Loads:

Load 1 VDL Dead load **21.9** kN/m to **0.0** kN/m

Load 2 UDL Dead load **4.1** kN/m

LOAD COMBINATIONS

Load combination 1

Span 1 1×Dead

Span 2 1×Dead

Span 3 1×Dead

CONTINUOUS BEAM ANALYSIS - RESULTS

Support Reactions - Combination Summary

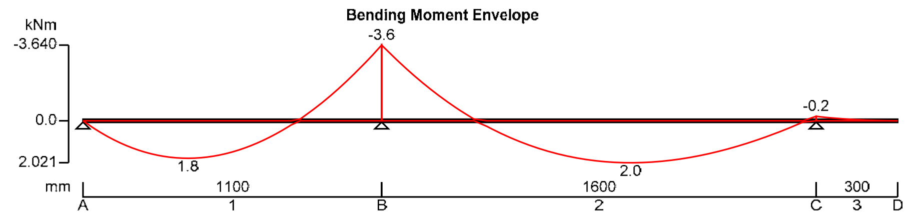
Support A	Max react = **-9.5** kN	Min react = **-9.5** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support B	Max react = **-28.0** kN	Min react = **-28.0** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support C	Max react = **-7.5** kN	Min react = **-7.5** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support D	Max react = **0.0** kN	Min react = **0.0** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm

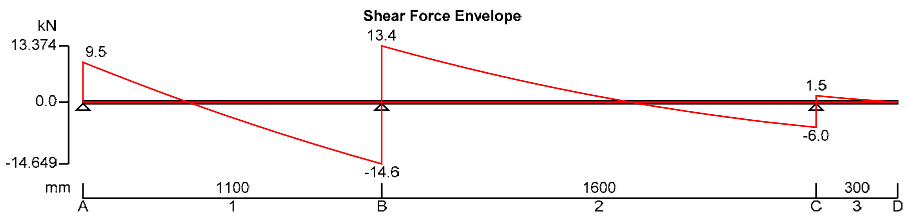
Beam Max/Min results - Combination Summary

Maximum shear = **13.4** kN Minimum shearF_{min} = **-14.6** kN

Maximum moment = **2.0** kNm Minimum moment = **-3.6** kNm

Maximum deflection = **7.7** mm Minimum deflection = **-4.9** mm





Number of sheets Nos = 2

Mallowable = Sxx * py * Nos = **26.565**kNm

CROFT STRUCTURAL ENGINEERS

Safe working loads for Acrow Props — loads given in kN SRU 4.0

For normal purposes 1 kilo Newton (kN) = 100 kg	Height m / ft	2.0 / 6.6	2.25 / 7.4	2.5 / 8.2	2.75 / 9.0	3.0 / 9.8	3.25 / 10.7	3.5 / 11.5	3.75 / 12.3	4.0 / 13.1	4.25 / 13.9	4.5 / 14.8	4.75 / 15.6
TABLE A Props loaded concentrically and erected vertically	Prop size 1 or 2	35	35	35	34	27	23						-
	Prop size 3				34	27	23	21	19	17			
	Prop size 4						32	25	21	18	16	14	12
TABLE B Props loaded concentrically and erected 1½° max. out of vertical	Prop size 1 or 2 or 3	35	32	26	23	19	17	15	13	12			
	Prop size 4						24	19	15	12	11	10	9
TABLE C Props loaded 25 mm eccentricity and erected 1½° max. out of vertical	Prop size 1 or 2 or 3	17	17	17	17	15	13	11	10	9			
	Prop size 4						17	14	11	10	9	8	7
TABLE D Props loaded concentrically and erected 1½° out of vertical and laced with scaffold tubes and fittings	Prop size 3				35	33	32	28	24	20			
	Prop size 4						35	35	35	35	27	25	21

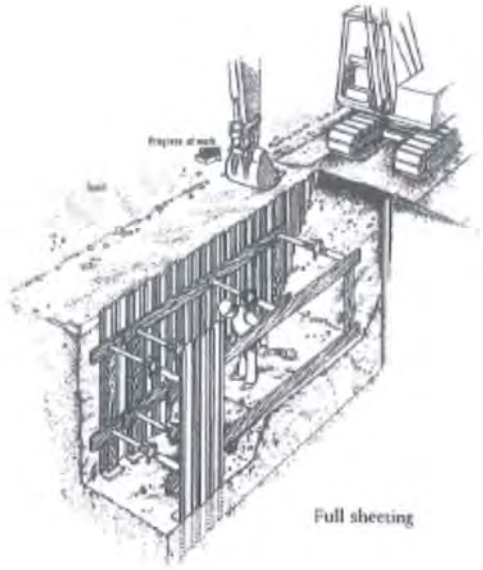
Shear V = (14.6kN + 13.4kN) /2 = 14.000kN

Any Acro Prop is accetpable

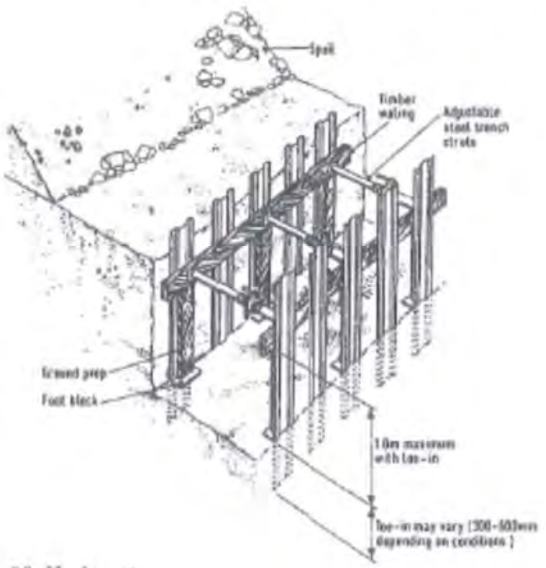
Sheeting requirements

Ground Type	Trench Depth, D: less than 1.2m(1)	1.2 to 3m	3 to 4.5m	4.5 to 6m
Sands and gravels Silt Soft Clay High compressibility Peat	Close, ½, ¼, ⅛ or nil	Close	Close	Close
Firm/stiff Clay Low compressibility Peat	¼, ⅛ or nil	½ or ¼	½ or ¼	Close or ½
Rock(2)	From ½ for incompetent rock to nil for competent rock(3)			

Sheeting requirements

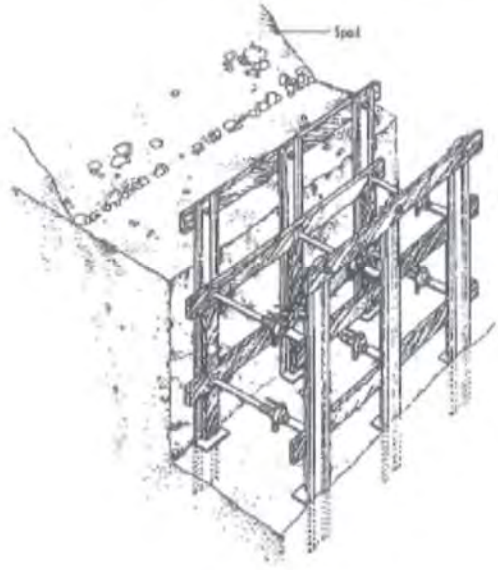


Sheeting requirements



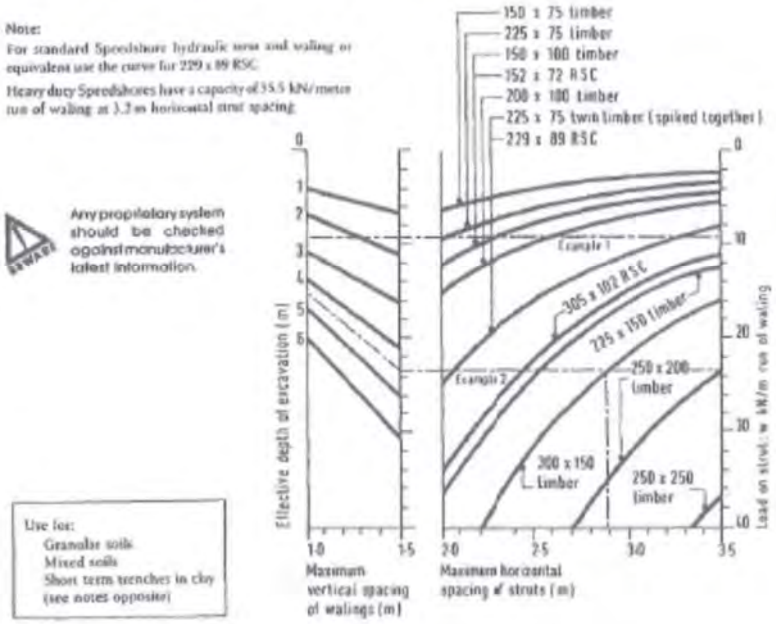
Half sheeting
shown for 1.5 m deep trench

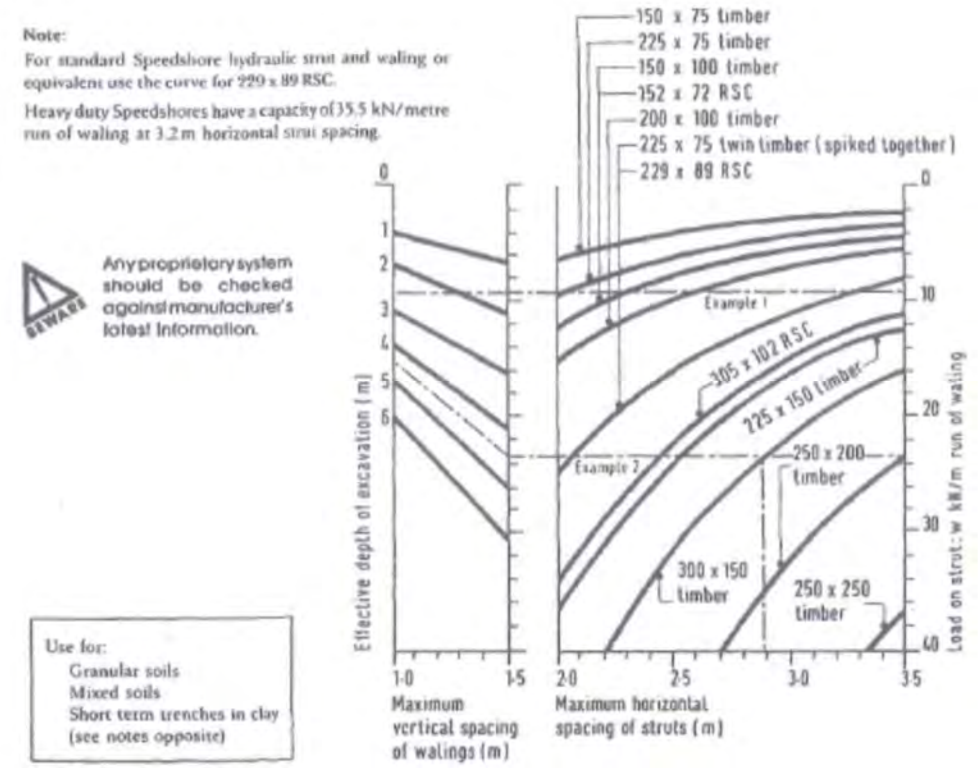
Sheeting requirements



Quarter sheeting

Design to CIRIA 97




Note:
For standard Speedshore hydraulic strut and waling or equivalent use the curve for 229 x 89 RSC.
Heavy duty Speedshores have a capacity of 35.5 kN/metre run of waling at 3.2 m horizontal strut spacing.
Any proprietory system should be checked against manufacturer's latest information.
BEWARE
Use for:
Granular soils
Mixed soils
Short term trenches in clay
(see notes opposite)
150 x 75 timber
225 x 75 timber
150 x 100 timber
152 x 72 RSC
200 x 100 timber
225 x 75 twin timber (spiked together)
229 x 89 RSC
305 x 102 RSC
225 x 150 timber
250 x 200 timber
300 x 150 timber
250 x 250 timber
Example 1
Example 2
Effective depth of excavation (m)
Load on strut: w kN/m run of waling
Maximum vertical spacing of walings (m)
Maximum horizontal spacing of struts (m)
1·0
1·5
2·0
2·5
3·0
3·5
0
10
20
30
40

Appendix E

Soil Investigation Report

GROUND INVESTIGATION REPORT

for the site at

156 GOLDHURST TERRACE, SOUTH HAMPSTEAD, LONDON NW6 3HP

on behalf of

GUY SHANI c/o CROFT STRUCTURAL ENGINEERS LIMITED

<table>
<tr><td colspan="2">Report Reference: GWPR910/GIR/May 2014</td><td>Status: FINAL</td></tr>
<tr><td>Issue:</td><td>Prepared By:</td><td>Verified By:</td></tr>
<tr><td rowspan="2">V1.01 May 2014</td><td></td><td></td></tr>
<tr><td>Roger Foord BA (Hons) MSc DIC
FGS MSoBRA
Senior Geotechnical Engineer</td><td>Francis Williams M.Geol. (Hons) FGS
CEnv AGS MSoBRA
Director</td></tr>
<tr><td colspan="3">File Reference: Ground and Water/Project Files/
GWPR910 156 Goldhurst Terrace, London NW6 3HP</td></tr>
</table>

CONTENTS

FIGURES

APPENDICES

1.0 INTRODUCTION

1.1 General

Ground and Water Limited were instructed by Guy Shani c/o Croft Structural Engineers Limited, on the 16th April 2014, to undertake a Ground Investigation on a site at 156 Goldhurst Terrace, South Hampstead, London NW6 3HP. The scope of the investigation was detailed within the Ground and Water Limited fee proposal ref: GWQ2101, dated 11th April 2014.

1.2 Aims of the Investigation

The aim of the investigation was understood to be to supply the client and their designers with information regarding the ground conditions underlying the site to assist them in preparing an appropriate scheme for development.

The investigation was to be undertaken to provide parameters for the design of foundations by means of in-situ and laboratory geotechnical testing undertaken on soil samples recovered from trial holes.

The requirements of the London Borough of Camden, Camden Geological, Hydrogeological and Hydrological Study, Guidance for Subterranean Development (November 2010) was reviewed with respect to this report.

A Desk Study and full scale contamination assessment were not part of the remit of this report.

The techniques adopted for the investigation were chosen considering the anticipated ground conditions and development proposals on-site, and bearing in mind the nature of the site, limitations to site access and other logistical limitations.

1.3 Conditions and Limitations

This report has been prepared based on the terms, conditions and limitations outlined within Appendix A.

2.0 SITE SETTING

2.1 Site Location

The site comprised an approximately rectangular shaped plot of land, totalling ~350m^2 in area and orientated in a north by north-east to south by south-west direction, located on the northern side of Goldhurst Terrace. The site was located in South Hampstead in the London Borough of Camden.

The national grid reference for the centre of the site was approximately TQ 25901 84190. A site location plan is given within Figure 1 and a plan. A plan showing the site area is given within Figure 2.

2.2 Site Description

The site was occupied by a terraced three storey brick built residential house with existing cellar fronting the property. A centrally located paved front pathway was flanked by soft landscaping and accessed via a <0.80m wide gate. The rear garden of the property was only accessible through the existing building.

Goldhurst Terrace, located adjacent to the southern boundary of the site, was noted to be at ~39m AOD.

2.3 Proposed Development

At the time of reporting, May 2014, the proposed redevelopment will comprise the extension of the existing basement beneath the entire footprint of the house. The basement is anticipated to be founded at ~3.0 – 3.5m below existing ground level (bgl) and be ~23m by 8m in area.

The proposed development fell within Geotechnical Design Category 2 in accordance with Eurocode 7. The proposed foundation loads were not known to Ground and Water Limited at the time of reporting but are likely to range from 75 – 150kN/m^2.

The proposed development was understood not to involve any re-profiling of the site and its immediate environs. It is understood that no trees will be removed to facilitate the construction of the basement.

2.4 Geology

The geology map of the British Geological Survey of Great Britain of the South Hampstead area (Sheet No. 256 North London) revealed the site to be situated on the London Clay Formation.

Figure 3 of the Camden Geological, Hydrogeological and Hydrological Study indicated that no Made Ground or Worked Ground was noted within a close proximity of the site.

London Clay Formation

The London Clay Formation comprises stiff grey fissured clay, weathering to brown near surface. Concretions of argillaceous limestone in nodular form (Claystones) occur throughout the formation. Crystals of gypsum (Selenite) are often found within the weathered part of the London Clay Formation, and precautions against sulphate attack to concrete are sometimes required.

The lowest part of the formation is a sandy bed with black rounded gravel and occasional layers of sandstone and is known as the Basement Bed.

There were no BGS boreholes records within a close proximity of the site.

2.5 Slope Stability and Subterranean Developments

The site was not situated within an area where a natural or man-made slope of greater than 7^{o} was present (Figure 16 Camden Geological, Hydrogeological and Hydrological Study).

Figure 17 of the Camden Geological, Hydrogeological and Hydrological Study indicated the site was not situated within an area prone to landslides.

Figure 18 of the Camden Geological, Hydrogeological and Hydrological Study indicated that no major subterranean infrastructure (including existing and proposed tunnels) was noted within close proximity to the site. The map showed that an over ground train line was present ~125m south of the site.

2.6 Hydrogeology and Hydrology

A study of the aquifer maps on the Environment Agency website, and Figure 8 of the Camden Geological, Hydrogeological and Hydrological Study, revealed the site to be located on **Unproductive Strata** comprising the bedrock of the London Clay Formation. No designation was given for any superficial deposits due to their likely absence.

Unproductive strata are rock layers with low permeability that have negligible significance for water supply or river base flow. These were formerly classified as non-aquifers.

Superficial (Drift) deposits are permeable unconsolidated (loose) deposits, for example, sands and gravels. The bedrock is described as solid permeable formations e.g. sandstone, chalk and limestone.

Examination of the Environment Agency records showed that the site did not fall within a Groundwater Source Protection Zone as classified in the Policy and Practice for the Protection of Groundwater.

A surface water feature comprising a pond was noted ~750m east of the site in accordance with Figure 12 of the Camden Geological, Hydrogeological and Hydrological Study. Figure 11 revealed the site was located close to where a southerly flowing tributary of the “Lost” Westbourne River was present.

Figure 14 of the Camden Geological, Hydrogeological and Hydrological Study revealed the site was not located within the catchment of Hampstead Ponds.

From analysis of hydrogeological and topographical maps groundwater was anticipated to be encountered at moderate to deep depth (4-6m below existing ground level (bgl)) and it was considered that the groundwater was flowing in a south-easterly direction in accordance with the local topography and towards a groundwater source protection borehole ~1.7km south-east of the site.

Examination of the Environment Agency records showed that the site was not situated within a floodplain or flood warning area. Figure 15 the Camden Geological, Hydrogeological and Hydrological Study revealed that Goldhurst Terrace suffered surface water flooding in 2002.

2.7 Radon

BRE 211 (2007) Map 5 of London, Sussex and West Kent revealed the site **was not** located within an area where mandatory protection measures against the ingress of Radon were required. The site

was not located within an area where a risk assessment was required.

3.0 FIELDWORK

3.1 Scope of Works

Fieldwork was undertaken on the 24th April 2014 and comprised the drilling of two window sampler boreholes (WS1 and WS2) to a depth of 6.00m bgl and the hand excavation of two trial pit foundation exposures (TP/FE1 and TP/FE2). A Heavy Dynamic Probe (HDP) (DP1) was undertaken adjacent to WS1 to 10.10m bgl.

A groundwater monitoring standpipe was installed in WS1 to a depth of 5.00m bgl to enable the measurement of standing groundwater levels.

The construction of the well installed can be seen tabulated below.

Combined Bio-gas and Groundwater Monitoring Well Construction				
Trial Hole	**Depth of Installation (m bgl)**	**Thickness of slotted piping with gravel filter pack (m)**	**Depth of plain piping with bentonite seal (m bgl)**	**Piping external diameter (mm)**
WS1	5.00	4.00	1.00	63

The approximate locations of the trial holes can be seen within Figure 4.

Prior to commencing the ground investigation, a walkover survey was carried out to identify the presence of underground services and drainage. Where underground services/drainage were suspected and/or positively identified, exploratory positions were relocated away from these areas.

Upon completion of the site works, the trial holes were backfilled and made good/reinstated in relation to the surrounding area.

3.2 Sampling Procedures

Small disturbed samples were recovered from the trial holes at the depths shown on the trial hole records. Soil samples were generally retrieved from each change of strata and/or at specific areas of concern. Samples were also taken at approximately 0.5m intervals during broad homogenous soil horizons.

A selection of samples were despatched for geotechnical testing purposes.

4.0 ENCOUNTERED GROUND CONDITIONS

4.1 Soil Conditions

All exploratory holes were logged by David McMillan of Ground and Water Limited generally in accordance with BS EN 14688 'Geotechnical Investigation and Testing – Identification and Classification of Soil'.

The ground conditions encountered within the trial holes constructed on the site generally conformed to that anticipated from examination of the geology map. A capping of Made Ground and Head Deposits was noted to overlie the London Clay Formation.

The ground conditions encountered during the investigation are described in this section. For more complete information about the Made Ground, Head Deposits and the London Clay Formation at particular points, reference must be made to the individual trial hole logs within Appendix B.

The trial hole location plan can be viewed in Figure 4.

For the purposes of discussion the succession of conditions encountered in the trial holes in descending order can be summarised as follows:

Made Ground
Head Deposits
London Clay Formation

Made Ground
Made Ground was encountered from ground surface in WS1, and beneath a 0.07m thick paving slab in WS2, to a depth of 1.10m bgl.

In WS1 the Made Ground comprised a dark brown to black gravelly sandy clay to 0.30m bgl overlying a brown to dark brown gravelly sandy clay to 1.10m bgl. The sand was fine to medium grained and the gravel was rare, fine to coarse, sub-rounded to sub-angular flint and brick, with carbonaceous material (clinker) noted between 0.30-1.10m bgl

In WS2 the Made Ground comprised a 0.07m thick paving slab over a dark brown sandy gravel to 0.35m bgl and a brown to dark brown sandy silty gravelly clay to 1.10m bgl. The sand was fine to coarse grained and the gravel was rare, fine to coarse, sub-rounded to sub-angular flint and brick, with slate fragments noted between 0.35-1.10m bgl.

Head Deposits
Soils described as Head Deposits and comprising an orange brown to light brown, locally sandy (WS2), gravelly silty clay to 2.20m bgl in WS1 and 2.30m bgl in WS2. The sand where encountered was fine grained and the gravel was rare to occasional, fine to coarse, sub-rounded to sub-angular flint.

London Clay Formation
Soils of the London Clay Formation, generally comprising a brown to grey silty clay, were encountered underlying the Head Deposits for the remaining depth of each of the boreholes, a depth of 6.00m bgl in WS1 and WS2. In WS1 an orange brown to brown sandy silty clay was encountered between 2.20-2.60m bgl. The sand was fine grained.

4.2 Foundation Exposures

A description of the foundation layout and ground conditions encountered within the hand dug trial pit/foundation exposures are given within this section of the report.

TP/FE1

Trial pit foundation exposure TP/FE1 was hand excavated from ground level at the front of the existing property. The exact location of the trial hole can be seen in Figure 4 with a section drawing of the foundation encountered in Figure 5.

The foundation exposure was measured from ground level.

The foundation layout encountered consisted of a brick wall to ground level. From ground level to a depth of 0.77m bgl a brick wall was noted. A step was then noted 0.13m out from the property and 0.17m in thickness. The brick step was noted to rest upon a brick footing that stepped out by 0.20m from the property and was 0.07m in thickness. The foundation was noted to rest upon soils described as Head Deposits and comprising an orange to light brown silty gravelly clay at 1.01m bgl. The ground conditions encountered directly surrounding the foundation are shown in Figure 5.

TP/FE2

Trial pit foundation exposure, TP/FE2, was hand excavated from ground level at the rear of the existing property. The exact location of the trial hole can be seen in Figure 4 and a section drawing of the foundation encountered during TP/FE2 can be seen in Figure 6.

The foundation exposure was measured from ground level.

The foundation layout encountered consisted of a brick wall to ground level. From ground level to a depth of 0.75m bgl a brick wall was noted. Two brick steps out (both 0.06m in width) from the property were then noted comprising a single course of bricks (0.07m in thickness) and two courses of bricks (0.23m in thickness) which were noted to rest upon a 0.10m thick layer of crushed brick. The foundation was noted to rest upon soils described as Head Deposits and comprising an orange brown and light brown silty sandy gravelly clay at 1.05m bgl. The ground conditions encountered directly surrounding the foundation are shown in Figure 6.

4.3 Roots Encountered

The depth of root penetration observed within each trial hole is tabulated below.

Depth of Root Penetrated Soils Observed Within Trial Holes		
Trial Hole	**Depth of Fresh Root Penetration (m bgl)**	**Depth of Dark Brown/Black Friable Rootlets (m bgl)**
WS1	Roots to 1.50m bgl	None
WS2	Roots to 4.00m bgl	None
TP/FE1	None	None
TP/FE2	None	None

It must be noted that the chance of determining actual depth of root penetration through a narrow diameter borehole is low. Roots may be found to greater depths at other locations on the site, particularly close to trees and/or trees that have been removed both within the site and its close environs.

4.4 Groundwater Conditions

Groundwater was not encountered in the trial holes. A standing water level of 2.11m bgl was recorded in the standpipe installed in WS1 on the 30th May 2014.

The standing water level in WS1 is likely to represent surface water or perched groundwater, migrating through the Made Ground or Head Deposits, collecting within a standpipe installed within the impermeable soils of the London Clay Formation.

Changes in groundwater level occur for a number of reasons including seasonal effects and variations in drainage. Exact groundwater levels may only be determined through long term measurements from monitoring wells installed on-site. The investigation was undertaken in April and May 2014, when groundwater levels are falling from their annual maximum (highest elevation).

Isolated pockets of groundwater may be perched within any Made Ground found at other locations around the site.

4.5 Obstructions

No artificial or natural sub-surface obstructions were noted during construction of the trial holes.

5.0 INSITU AND LABORATORY GEOTECHNICAL TESTING

5.1 In-Situ Geotechnical Testing

A Heavy Dynamic Probe (HDP) (DP1) was undertaken adjacent to WS1 to 10.10m bgl. The test results are presented on the borehole log within Appendix B.

Window Sampler Boreholes provide samples of the ground for assessment but they do not give any engineering data. Dynamic Probing involves the driving of a metal cone into the ground via a series of steel rods. These rods are driven from the surface by a hammer system that lifts and drops a 50.0kg hammer onto the top of the rods through a set height, thus ensuring a consistent energy input. The number of hammer blows that are required to drive the cone down by each 100mm increment are recorded. These blow counts then provide a comparative assessment from which correlations have been published, based on dynamic energy, which permits engineering parameters to be generated. *(The Dynamic Probe 'Heavy' (HDP) Tests were conducted in accordance with BS 1377; 1990; Part 9, Clause 3.2).*

The cohesive soils of the Head Deposits and London Clay Formation were classified based on the table below.

Undrained Shear Strength from Field Inspection/equivalent SPT derived from HDP results Cohesive Soils (EN ISO 14688-2:2004 & Stroud (1974))		
Classification	**Undrained Shear Strength (kPa)**	**Field Indications**
Extremely High	>300	-
Very High	150 – 300	Brittle or very tough
High	75 – 150	Cannot be moulded in the fingers
Medium	40 – 75	Can be moulded in the fingers by strong pressure
Low	20 – 40	Easily moulded in the fingers
Very Low	10 – 20	Exudes between fingers when squeezed in the fist
Extremely Low	<10	-

An interpretation of the in-situ geotechnical testing results is given in the table below.

In-Situ Geotechnical Testing Results Summary					
Strata	**Equivalent SPT "N" Blow Counts derived from HDP**	**Undrained Shear Strength kPa (based on Stroud, 1974)**	**Soil Type**		**Trial Hole**
			Cohesive	**Granular**	
Head Deposits	2 – 6	10 – 30	Ext. Low/Low - Low	-	WS/DP1 (1.30 – 2.20m bgl)
London Clay Formation	4 – 10	20 – 50	V Low/Low – Medium	-	WS/DP1 (2.20 – 6.00m bgl)
Assumed London Clay Formation*	8 – 46	40 – 230	Low/Medium – V High	-	WS/DP1 (6.00 – 10.10m bgl)

*assumed London Clay formation based on the results of the dynamic probing.

It must be noted that field measurements of undrained shear strength are dependent on a number of variables including disturbance of sample, method of investigation and also the size of specimen or test zone etc.

The dynamic probe indicated a lens of high to very high undrained shear strength soils between 7.9 – 8.9m bgl likely associated with the presence of claystones within the London Clay Formation.

The test results are presented on the trial hole logs within Appendix B.

5.2 Laboratory Geotechnical Testing

A programme of geotechnical laboratory testing, scheduled by Ground and Water Limited and carried out by K4 Soils Laboratory and QTS Environmental Limited, was undertaken on samples recovered from the Head Deposits and the London Clay Formation. The results of the tests are presented in Appendix C.

The test procedures used were generally in accordance with the methods described in BS1377:1990.

Details of the specific tests used in each case are given below:

Standard Methodology for Laboratory Geotechnical Testing		
Test	**Standard**	**Number of Tests**
Atterberg Limit Tests	BS1377:1990:Part 2:Clauses 3.2, 4.3 & 5	7
Moisture Content	BS1377:1990:Part 2:Clause 3.2	13
Water Soluble Sulphate & pH	BS1377:1990:Part 3:Clause 5	2
BRE Special Digest 1 (incl. Ph, Electrical Conductivity, Total Sulphate, W/S Sulphate, Total Chlorine, W/S Chlorine, Total Sulphur, Ammonium as NH4, W/S Nitrate, W/S Magnesium)	BRE Special Digest 1 “Concrete in Aggressive Ground (BRE, 2005).	1

5.2.1 Atterberg Limit Tests

A précis of Atterberg Limit Tests undertaken on three samples of the Head Deposits and four samples of the London Clay Formation can be seen tabulated below.

Atterberg Limit Tests Results Summary							
Stratum/Depth	**Moisture Content (%)**	**Passing 425 µm sieve (%)**	**Modified PI (%)**	**Soil Class**	**Consistency Index (Ic)**	**Volume Change Potential**	
						NHBC	**BRE**
Head Deposits	21 – 30	90 – 98	27.9 – 35.3	CH	Stiff – V Stiff	Medium	Medium
London Clay Formation	32 - 34	99 – 100	43.0 – 46.0	CH – CV	Stiff	High	High

NB: NP – Non-plastic
BRE Volume Change Potential refers to BRE Digest 240 (based on Atterberg results)
Soil Classification based on British Soil Classification System.
Consistency Index (Ic) based on BS EN IS0 14688-2:2004.

5.2.2 Comparison of Soil's Moisture Content with Index Properties

5.2.2.1 Liquidity Index Analyses

The results of the Atterberg Limit tests undertaken on three samples of the Head Deposits and four samples of the London Clay Formation were analysed to determine the Liquidity Index of the samples. This gives an indication as to whether the samples recovered showed a moisture deficit and their degree of consolidation. The results are tabulated below.

The test results are presented within Appendix C.

Liquidity Index Calculations Summary					
Stratum/Trial Hole/Depth	**Moisture Content (%)**	**Plastic Limit (%)**	**Modified Plasticity Index (%)**	**Liquidity Index**	**Result**
Head Deposits **WS1/1.50m bgl** (Brown, orange and occasional grey slightly gravelly silty CLAY (gravel is fine to medium and angular))	30	26	35.3	0.133	Heavily Overconsolidated.
London Clay Formation **WS1/3.50m bgl** (Brown and occasional blue grey silty CLAY with occasional fine siltstone fragments)	32	30	43.6	0.046	Heavily Overconsolidated
London Clay Formation **WS1/4.50m bgl** (Brown slightly mottled blue grey silty CLAY with traces of selenite crystals)	34	32	46.0	0.043	Heavily Overconsolidated.
Head Deposits **WS2/1.50m bgl** (Brown, orange and grey slightly gravelly slightly sandy silty CLAY (gravel is fine to medium and sub-angular to angular))	25	25	30.6	0.000	Heavily Overconsolidated.
Head Deposits **WS2/2.00m bgl** (Orange brown slightly gravelly slightly sandy silty CLAY (gravel is fine to medium and sub-angular to angular))	21	29	27.9	-0.287	**Potential Moisture Deficit**
London Clay Formation **WS2/3.50m bgl** (Brown and occasional blue grey silty CLAY)	34	32	43.0	0.047	Heavily Overconsolidated.
London Clay Formation **WS2/4.00m bgl** (Brown slightly mottled blue grey silty CLAY with traces of selenite crystals)	32	31	43.0	0.023	Heavily Overconsolidated.

The results in the table above indicate that a potential moisture deficit is present within one sample of the Head Deposits tested (WS2/2.00m). The sample was described as an orange brown slightly gravelly slightly sandy silty clay. The gravel was fine to medium and sub-angular to angular. Roots were noted to a depth of 4.00m bgl in WS2. Consequently, the apparent moisture deficit could be related to a combination of the lithology of the soil (heavily overconsolidated soils and 10% coarse fraction) and the water demand from the roots.

Liquidity Index testing revealed no evidence for moisture deficit within the remaining overconsolidated to heavily overconsolidated samples of the Head Deposits and the London Clay Formation tested.

5.2.2.2 Liquid Limit

A comparison of the soil moisture content and the liquid limit can be seen tabulated below.

Moisture Content vs. Liquid Limit				
Strata/Trial Hole/Depth/Soil Description	**Moisture Content (MC) (%)**	**Liquid Limit (LL) (%)**	**40% Liquid Limit (LL)**	**Result**
Head Deposits **WS1/1.50m bgl** (Brown, orange and occasional grey slightly gravelly silty CLAY (gravel is fine to medium and angular))	30	62	24.8	MC > 0.4 x LL (No significant moisture deficit)
London Clay Formation **WS1/3.50m bgl** (Brown and occasional blue grey silty CLAY with occasional fine siltstone fragments)	32	74	29.6	MC > 0.4 x LL (No significant moisture deficit)
London Clay Formation **WS1/4.50m bgl** (Brown slightly mottled blue grey silty CLAY with traces of selenite crystals)	34	78	31.2	MC > 0.4 x LL (No significant moisture deficit)
Head Deposits **WS2/1.50m bgl** (Brown, orange and grey slightly gravelly slightly sandy silty CLAY (gravel is fine to medium and sub-angular to angular))	25	59	23.6	MC > 0.4 x LL (No significant moisture deficit)
Head Deposits **WS2/2.00m bgl** (Orange brown slightly gravelly slightly sandy silty CLAY (gravel is fine to medium and sub-angular to angular))	21	60	24.0	**MC < 0.4 x LL** **(Potentially significant moisture deficit)**
London Clay Formation **WS2/3.50m bgl** (Brown and occasional blue grey silty CLAY)	34	75	30.0	MC > 0.4 x LL (No significant moisture deficit)
London Clay Formation **WS2/4.00m bgl** (Brown slightly mottled blue grey silty CLAY with traces of selenite crystals)	32	74	29.6	MC > 0.4 x LL (No significant moisture deficit)

The results in the table above indicate that a potential significant moisture deficit was present within one sample of the Head Deposits tested (WS2/2.00m). The moisture content value was below 40% of the liquid limit. .

The sample was described as an orange brown slightly gravelly slightly sandy silty clay. The gravel was fine to medium and sub-angular to angular. Roots were noted to a depth of 4.00m bgl in WS2. Geotechnical testing on a shallower sample (WS2/1.50m bgl) showed no potential moisture deficit. The apparent moisture deficit could be related to a combination of the lithology of the soil (heavily overconsolidated soils and 10% coarse fraction) and the water demand from the roots.

The results in the table above indicate that the remaining samples of the Head Deposits and the London Clay Formation tested showed no evidence of a significant moisture deficit.

5.2.3 Moisture Content Profiling

Moisture content versus depth plots for WS1 and WS2 can be seen within Figures 7 and 8.

Figure 7 & 8 show a possible moisture deficit in both at 2.00m bgl due to a lowering of the moisture content of the sample of the Head Deposits from that depth. Roots were noted to a depth of 1.50m bgl in WS1 and to 4.00m bgl within WS2. The deposits were described as a gravelly silty clay.

Given the absence of roots within WS1 the lower moisture content was likely due to the coarse fraction (gravel content) rather than the moisture demand from nearby trees.

No other significant areas of very low moisture content were noted, with the profile showing variations in moisture content that would be as expected based on variations in lithology, rather than the moisture demand from nearby trees. However given the presence of roots to 4.00m bgl the affect of nearby trees on the moisture content of the London Clay Formation within WS2 cannot be discounted.

5.2.4 Sulphate and pH Tests

Sulphate and pH tests were undertaken on two samples from the Head Deposits (WS1/1.50m and WS2/2.0m bgl). The sulphate concentration ranged from 70-190mg/l with a pH range of 7.6-7.7.

5.2.5 BRE Special Digest 1

In accordance with BRE Special Digest 1 ‘Concrete in Aggressive Ground’ (BRE, 2005) one sample of the London Clay Formation (WS1/4.00m) were scheduled for laboratory analysis to determine parameters for concrete specification.

The results are given within Appendix C and a summary is tabulated overpage.

Summary of Results of BRE Special Digest Testing			
Determinand	**Unit**	**Minimum**	**Maximum**
pH	-	7.7	-
Ammonium as NH_4	mg/kg	6.6	-
Sulphur	mg/kg	1802	-
Chloride (water soluble)	mg/kg	104	-
Magnesium (water soluble)	g/l	0.2490	-
Nitrate (water soluble)	mg/kg	<3	-
Sulphate (water soluble)	g/l	1.50	-
Sulphate (total)	mg/kg	5341	-

6.0 ENGINEERING CONSIDERATIONS

6.1 Soil Characteristics and Geotechnical Parameters

Based on the results of the intrusive investigation and geotechnical laboratory testing the following interpretations have been made with respect to engineering considerations.

- Made Ground was encountered to a depth of 1.10m bgl in both boreholes.

  As a result of the inherent variability of Made Ground, it is usually unpredictable in terms of bearing capacity and settlement characteristics. Foundations should, therefore, be taken through any Made Ground and either into, or onto a suitable underlying natural stratum of adequate bearing characteristics.

- Soils described as Head Deposits and comprising an orange brown to light brown, locally sandy (WS2), gravelly silty clay to 2.20m bgl in WS1 and 2.30m bgl in WS2. The sand where encountered was fine grained and the gravel was rare to occasional, fine to coarse, sub-rounded to sub-angular flint.

  The cohesive soils of the Head Deposits comprised extremely low/low to low undrained shear strength (10-30kPa) soils between 1.30-2.20m bgl in WS1.

  The soils of the Head Deposits were shown to have a **medium** potential for volume change in accordance both BRE240 and NHBC Standards Chapter 4.2.

  Consistency Index calculations indicated the cohesive Head Deposits to be stiff to very stiff. Liquidity Index testing revealed the soils to be heavily overconsolidated.

  Geotechnical analysis revealed a potential significant moisture deficit was present within one sample of the Head Deposits tested (WS2/2.00m bgl) that was considered likely to be due to the lithology of the soil (heavily overconsolidated soils and 10% coarse fraction). However given roots were noted within the Head Deposits the moisture demand from nearby trees could not be discounted.

  Whilst the soils of the Head Deposits are heavily overconsolidated cohesive soils, given their limited depth (2.20-2.30m bgl), they will be by-passed by the basement foundation and therefore not considered to be a suitable bearing stratum.

- Soils of the London Clay Formation, generally comprising a brown to grey silty clay, were encountered underlying the Head Deposits for the remaining depth of each of the boreholes, a depth of 6.00m bgl in WS1 and WS2. In WS1 an orange brown to brown sandy silty clay was encountered between 2.20-2.60m bgl. The sand was fine grained.

  The cohesive soils of the London Clay Formation comprised very low/low to medium undrained shear strength (20-50kPa) soils from 2.20-6.00m bgl and with an assumed low/medium to very high undrained shear strength (40-230 kPa) between 6.0-10.10m bgl. The dynamic probe indicated a lense of high to very high undrained shear strength soils between 7.9 – 8.9m bgl likely associated with the presence of claystones within the London Clay Formation.

The soils of the London Clay Formation were shown to have a **high** potential for volume change in accordance both BRE240 and NHBC Standards Chapter 4.2.

Consistency Index calculations indicated the cohesive London Clay Formation to be stiff. Liquidity Index testing revealed the soils to be heavily overconsolidated.

Geotechnical analysis revealed no potential significant moisture deficits were present within the samples of the London Clay Formation tested. Moisture content profiling indicate that the moisture profile with depth within the London Clay Formation was as expected with minor variation noted associated with small changes in lithology. However, given the presence of roots to 4.00m bgl within WS2 the potential for moisture variations due to plant uptake cannot be discounted.

The soils of the London Clay Formation are heavily overconsolidated cohesive soils and are therefore likely to be a suitable stratum for the proposed traditional strip, mat or piled foundations associated with the basement. The settlements induced on loading are likely to be low to moderate.

The final design of foundations will need to take into account the volume change potential of the soil, the depth of root penetration and/or moisture deficit and the likely serviceability and settlement requirements of the proposed structure. These parameters for design are discussed in the next section of this report.

- Groundwater was not encountered in the trial holes. A standing water level of 2.11m bgl was recorded in the standpipe installed in WS1 on the 30th May 2014. The standing water level in WS1 is likely to represent surface water or perched groundwater, migrating through the Made Ground or Head Deposits, collecting within a standpipe installed within the impermeable soils of the London Clay Formation.

- Roots were noted to a depth of 1.50m bgl in WS1 and 4.00m bgl in WS2.

6.2 Basement Foundations

At the time of reporting, May 2014, the proposed redevelopment will comprise the extension of the existing basement beneath the entire footprint of the house. The basement is anticipated to be founded at ~3.0 – 3.5m below existing ground level (bgl) and be ~23m by 8m in area.

The proposed development fell within Geotechnical Design Category 2 in accordance with Eurocode 7. The proposed foundation loads were not known to Ground and Water Limited at the time of reporting but are likely to range from 75 – 150kN/m^2.

Foundations should be designed in accordance with soils of **high volume change potential** in accordance with BRE Digest 240 and NHBC Chapter 4.2.

Given the cohesive nature of the shallow deposits foundations must therefore **not** be placed within cohesive root penetrated and/or desiccated soils and the influence of the trees surrounding the site must be taken into account (NHBC Standards Chapter 4.2). It is recommended that foundations are taken at least 300mm into non-root penetrated strata or granular soils of no volume change potential.

Where trees are mentioned in the text this means existing trees, recently removed trees (approximately 15 years to full recovery on cohesive soils) and those planned as part of the site landscaping. Should trees be removed from the footprint of the proposed building then an alternative foundation system, such as piles or isolated pads should be considered.

Roots were observed to a depth of 1.50m bgl in WS1 and 4.00m bgl in WSH2, therefore a minimum foundation depth of ~4.30m bgl would be required.

Further investigation into the depth of root penetration to the rear of the property should be undertaken as the roots at depth may be relic and pose no risk to the proposed structure. No significant changes in moisture content within the London Clay Formation were noted between WS1 and WS2 however aerial views of the site indicate large trees to be close to WS2. Insufficient information is available at present to confirm this.

It is considered likely the proposed basement will be constructed with load bearing concrete retaining walls with semi-ground bearing concrete floors. The following bearing capacities could be adopted for 5.0m long by 0.75m and 1.00m wide footings at a depth of 4.30m bgl. The bearing capacities and settlements were determined based on BH1.

Limit State: Bearing Capacities Calculated		
Depth (m BGL)	**Foundation System**	**Limit Bearing Capacity (kN/m^2)**
4.30m	5.00m by 0.75m Strip	162.16
	5.00m by 1.00m Strip	162.16

Serviceability State: Settlement Parameters Calculated			
Depth (m BGL)	**Foundation System**	**Limit Bearing Capacity (kN/m^2)**	**Settlement (mm)**
4.30m	5.00m by 0.75m Strip	150	<19
	5.00m by 1.00m Strip	140	<20

It must be noted that a bearing capacity of less than 60kN/m^2 at 4.30m bgl may results in heave of the underlying soils.

Site levels may need to be brought up to underside of proposed slab level using with suitable granular soil (Type I or Type II) rolled in thin layers.

It must be mentioned that it was assumed that excavations will be kept dry and either concreted or blinded as soon after excavation as possible. If water were allowed to accumulate on the formation for even a short time not only would an increase in heave occur resulting from the soil increasing in volume by taking up water, but also the shear strength and hence the bearing capacity would also be reduced.

If the construction works take place during the winter months, when the groundwater level is expected to be at its higher elevation, perched water could accumulate thus dewatering could be required to facilitate the construction and prevent the base of the excavation blowing before the slab was cast. The advice of a reputable dewatering contractor, familiar with the type of ground and groundwater conditions encountered on this site, should be sought prior to finalising the design of the excavation for the basement.

The basement must be suitably tanked to prevent ingress of groundwater and also surface water run-off. The basement must also be designed to take into account pressure exerted by the presence of groundwater in and around the basement.

6.3 Piled Foundations

Should the bearing values given above be unsuitable for the proposed development or the potential need for extending the basement to avoid roots increase construction costs, then attention should be given to the adoption of a piled foundation.

The construction of a piled foundation is a specialist job, and the advice of a reputable contractor, familiar with the type of ground and groundwater conditions encountered on this site, should be sought prior to finalising the foundation design, as the actual pile working load will depend on the particular type of pile and method of installation adopted.

The foundation would comprise a piled foundation with reinforced ground beams. For the cumulative pile capacity calculations, shaft friction over the desiccated levels should be ignored and piles should not be terminated within desiccated soils where moisture recovery following tree removal could occur.

Indicative limit loads and settlements for a bored pile have been given within the table below and have been based on the strength profile within WS/DP1.

An allowance for negative skin friction to occur within the top 4.0m of the soil has been included within the calculations where it could pass through any Made Ground, root penetrated soils and soils showing a possible moisture deficit. An adhesion factor of 0.45m has been applied.

The bearing values may be limited by the maximum permissible stress allowable on a concrete pile.
To achieve the full bearing value a pile should penetrate the bearing stratum by at least five times the pile diameter.

Bored Pile – Limit Loads and Settlement Parameters						
Depth (m bgl)	Diameter (m)	Limit States (kN)			Settlement (Poulos Davis (1968))	
		Tip	Lateral	Total	Load (kN)	Total (Elastic + Rigid) (cm)
8	0.30	48.57	120.75	155.19	150	0.14
	0.45	109.29	181.13	258.61	250	0.25
	0.60	194.29	241.51	379.27	370	0.28

The bearing values given in the table above are applicable to single piles. Where piles are to be constructed in groups the bearing value of each individual pile should be reduced by a factor of approximately 0.8 and a calculation made to check the factor of safety against block failure.

The piles will need to be designed in accordance with the volume change potential of the soils encountered, depth of desiccation, root penetration, etc. Temporary casing may be required where the upper portion of the pile passes through the Made Ground, particularly where perched water is encountered, to prevent necking of the concrete.

6.4 Piled Basements

Basement rafts founded on piles have an effect of stiffening the raft and reducing or eliminating reconsolidation of ground heave, thereby reducing differential settlements or tilting.

Where piles are terminated on a yielding stratum such as stiff clay, settlement of the piles as the working load is built up are likely to result in some of the load being carried by the underside of the slab raft or by the pile caps. The soil beneath these relatively shallow structures is likely to then compress, causing the load to transfer back to the piles. The process is continuous with some proportion of the load being carried by the piles and some by the capping structure. Therefore while the piles must be designed to carry the fall of the super structure loading, the slab raft which transfers the load to the piles should have sufficient strength to withstand loading on the underside equivalent to the net load of the superstructure or to some proportionate of the net load which is assessed from a consideration of the likely yielding of the piles, the compressibility of the shallow soil layers and the effects of basement excavation and pile installation.

For piles constructed wholly in compressible clays, in the course of excavation for the basement, heave takes place, with further upwards movement caused by displacement due to pile driving, or if bored piles are used, there may be a small reduction in the amount of heave due to inward movement of the clay around the pile boreholes.

After completion of the piling, we suspect the swelled soils would be trimmed off to the specified level of the underside of the basement. After concreting the basement slab, it was considered that there would be some tendency for pressure to increase due to long term swelling of the soil, but this is likely to be counteracted to some extent if driven piles are used by the soil displaced by the driving settling away from the slab as it reconsolidates around the piles. However, as the load of the basement increases with superstructure loading, the piles themselves are likely to settle due to consolidation of the soil in the region of the piles. It was considered that the soil surrounding the upper part of the piles would follow the downward movement of the underlying soil and thus there is likely to be no appreciable tendency for the full structural loading to come onto the basement slab.

After completion of the building, long-term settlement due to consolidation of the soil beneath the piles would most likely continue, but at all times the overlying soils would be considered to move downwards and are unlikely to develop appreciable pressure on the basement slab.

Thus, it can be stated that the maximum load which is likely to come from the underside of the slab would most likely be that due to the soil swelling in the early days after pile driving together with water pressure if the basement is below groundwater level. If; however, the working loads on the piles were to exceed their ultimate carrying capacity, they would move downwards relative to the surrounding soil. The slab would then carry the full load of the building, until consolidation of the soil throws the load back on the piles with progressive movement continuing until equilibrium is reached.

The net downward movement resulting from the algebraic sum of heave, reconsolidation, and further consolidation will be lower for the piled basement than for an unpiled basement. This is illustrated in the Figure A below.

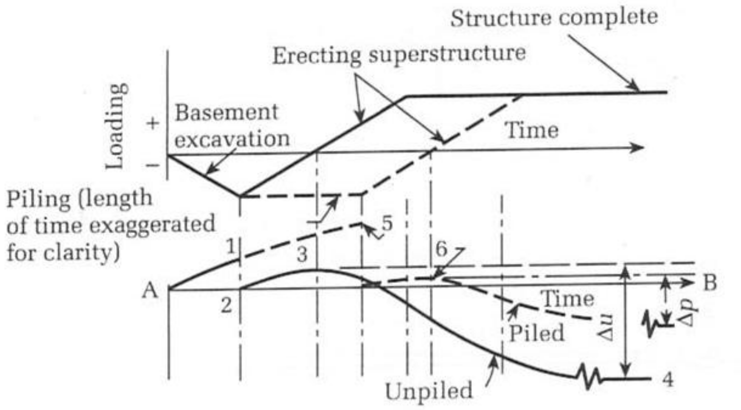


Figure A: Comparison of settlement/heave associated with piled and unpiled basements

In the case of the piled basement, the excavation will generally remain open and unconcreted for a longer period until all piles have been installed. After completion of piling (Point 5) the soil is trimmed off to the specified level and the floor slab is constructed. There will be some continuing upward movement of the basement level as the soil around and beneath the piles continue to swell, but if the piles are long in relation to the width of the building such movement will be very small. When the superstructure loading reaches the original overburden pressure (Point 6) reconsolidation will take place. The net downward movement (Δp) will be less, since the swelling is less and the consolidation due to net additional super-structure loading will also be less since the piles have been terminated in soil of lower compressibility.

If however, the piles are relatively short, it was considered that there would be no appreciable reduction in net settlement as compared to an unpiled basement. The piles would then be wholly within the zone of swelling which may be greater because the excavation would remain open for a longer period. **To be effective in reducing net settlements, piles should be terminated below the zone of swelling.**

Therefore, based on the above, piles which are terminated below the zone of swelling and anchored against uplift by shaft friction or enlarged bases are considered to have considerable tension, and measures should be taken to prevent its occurrence. Reinforcement of the pile shafts in addition to sleeving the piles within the swelling zone could be considered. Uplift on the underside of the basement slab and the consequent transfer of the uplift forces to the piles can be prevented by providing a layer of weak compressible material below the slab.

Piles tend to be installed in groups under each column with the column load transferred to the pile group by the pile cap. These caps may also need some protection by installation of compressible layers below the pile cap. The underside of ground beams, running between pile caps, should also be fitted with these compressible materials in accordance with NHBC requirements for compressible materials on the sides of the pile caps and ground beams (inside edges).

A further risk with piled basements constructed by top-down methods in heaving clay is upward convexity occurring in the ground floor and upper immediate basement slabs where these are connected to the steel columns at an early stage in construction. In some circumstances tension can develop at the junction between the columns and the tops of the piles, and care is necessary to ensure that the holding-down bolts to the column base plates are sufficiently long and not overstressed.

6.5 Basement Excavations & Stability

Shallow excavations in the Made Ground Head Deposits and London Clay Formation are likely to be marginally stable at best. Long, deep excavations, through both of these strata are likely to become unstable.

The excavation of the basement must not affect the integrity of the adjacent structures beyond the boundaries. The excavation must be supported by suitably designed retaining walls. It is considered unlikely that battering the sides of the excavation, casting the retaining walls and then backfilling to the rear of the walls would be suitable given the close proximity of the party walls.

The retaining walls for the basement will need to be constructed based on cohesive soils with an appropriate angle of shear resistance (Φ') for the ground conditions encountered.

Based on the ground conditions encountered within the boreholes the following parameters could be used in the design of retaining walls. These have been designed based on the DPH profile recorded, results of geotechnical classification tests and reference to literature.

Retaining Wall/Basement Design Parameters					
Strata	**Unit Volume Weight (kN/m^3)**	**Cohesion Intercept (c') (kPa)**	**Angle of Shearing Resistance (Ø)**	**Ka**	**Kp**
Made Ground and Head Deposits	~15	0	12	0.66	1.52
London Clay Formation	~20-22	0	24	0.42	2.37

Unsupported earth faces formed during excavation may be liable to collapse without warning and suitable safety precautions should therefore be taken to ensure that such earth faces are adequately supported before excavations are entered by personnel.

Based on the groundwater readings taken during this investigation to date, it was considered likely that perched groundwater would be encountered during basement construction. Dewatering from sumps introduced into the floor of the excavation is likely to be required. Consideration should be given to creating a coffer dam using contiguous piled or sheet piled walls to aid basement construction below the perched water table.

6.6 Hydrogeological Effects

The proposed development is located on **Unproductive Strata** relating to the London Clay Formation.

The ground conditions encountered generally comprised a capping of cohesive Made Ground and Head Deposits over the cohesive London Clay Formation. Based on a visual appraisal of the soils encountered the permeability of the Head Deposits and the London Clay Formation was likely to be very low to negligible permeability.

Groundwater was not encountered in the trial holes. A standing water level of 2.11m bgl was recorded in the standpipe installed in WS1 on the 30th May 2014.

The standing water level in WS1 is likely to represent surface water or perched groundwater, migrating through the Made Ground or Head Deposits, collecting within a standpipe installed within the impermeable soils of the London Clay Formation.

The Environment Agency records show that the highest recorded tide for the nearest river station on the River Thames at Westminster is 4.50m AOD with high tides generally at ~3.00m AOD. The elevation of the site is ~39.00m AOD. Based on a 3.00-3.50m bgl deep basement slab a formation level of 36.00-35.50m AOD is assumed. This means that the basement will be constructed above general high tide levels of the River Thames.

Based on the above it is considered likely that perched water will be encountered during basement construction, but the basement will not be constructed below the groundwater table. In relation to the basement, once constructed, the Made Ground will act as a slightly porous medium for water to migrate however additional drainage should be considered as the London Clay Formation will act as a barrier for groundwater migration.

6.7 Sub-Surface Concrete

Sulphate concentrations measured in 2:1 water/soil extracts taken from the Made Ground and London Clay Formation, from both the geotechnical and chemical laboratory testing, fell into Class DS-1 and DS-2 of the BRE Special Digest 1, 2005, *'Concrete in Aggressive Ground'.*

Table C1 of the Digest indicated an ACEC (Aggressive Chemical Environment for Concrete) classification of AC-2 for foundations within the Made Ground and Head Deposits. For the classification given, the “mobile” and “natural” case was adopted given the presence of gravel within the formation (permeability likely to exceed 10-7 m/se) and residential use of the site.

Table C1 of the Digest indicated an ACEC (Aggressive Chemical Environment for Concrete) classification of AC-1s for foundations within the London Clay Formation. For the classification given, the “static” and “natural” case was adopted given the cohesive nature of the deposits (permeability unlikely to exceed 10-7 m/se) and residential use of the site.

The sulphate concentration in the samples ranged from 70-1500mg/l with a pH range of 7.6-7.7. The total sulphate concentration recorded was 0.53%.

Concrete to be placed in contact with soil or groundwater must be designed in accordance with the recommendations of Building Research Establishment Special Digest 1, 2005, *'Concrete in Aggressive Ground'* taking into account the pH of the soils.

It is prudent to note that pyrite nodules may be present within the London Clay Formation. Pyrite can oxidise to gypsum and this normally only occurs in the upper weathered layer, but excavation allows faster oxidation and water soluble sulphate values can rapidly increase during construction. Therefore rising sulphate values should be taken into account should ferruginous staining/pyrite nodules be encountered within the London Clay Formation.

6.8 Surface Water Disposal

Infiltration tests were beyond the scope of the investigation.

Soakaway construction within the cohesive soils of the Head Deposits and London Clay Formation are unlikely to prove satisfactory due to negligible to low anticipated infiltration rates. Therefore an alternative method of surface water disposal is required.

Consultation with the Environment Agency must be sought regarding any use that may have an impact on groundwater resources.

The principles of sustainable urban drainage system (SUDS) should be applied to reduce the risk of flooding from surface water ponding and collection associated with the construction of the basement.

6.9 Discovery Strategy

There may be areas of contamination that have not been identified during the course of the intrusive investigation. For example, there may have been underground storage tanks (UST's) not identified during the Ground Investigation for which there is no historical or contemporary evidence.

Such occurrences may be discovered during the demolition and construction phases for the redevelopment of the site.

Groundworkers should be instructed to report to the Site Manager any evidence for such contamination; this may comprise visual indicators, such as fibrous materials within the soil, discolouration, or odours and emission. Upon discovery advice must be taken from a suitably qualified person before proceeding, such that appropriate remedial measures and health and safety protection may be applied.

Should a new source of contamination be suspected or identified then the Local Authority will need to be informed.

6.10 Waste Disposal

The excavation of foundations is likely to produce waste which will require classification and then recycling or removal from site.

Under the Landfill (England and Wales) Regulations 2002 (as amended), prior to disposal all waste must be classified as;

- Inert;
- Non-hazardous, or;
- Hazardous.

The Environment Agency's Hazardous Waste Technical Guidance (WM2) document outlines the methodology for classifying wastes.

Once classified the waste can be removed to the appropriately licensed facilities, with some waste requiring pre-treatments prior to disposal.

INERT waste classification should be undertaken to determine if the proposed waste confirms to INERT or NON-HAZARDOUS Waste Acceptable Criteria (WAC).

6.11 Imported Material

Any soil which is to be imported onto the site must undergo chemical analysis to prove that it is suitable for the purpose for which it is intended.

The Topsoil must be fit for purpose and must either be supplied with traceable chemical laboratory test certificates or be tested, either prior to placing (ideally) or after placing, to ensure that the human receptor cannot come into contact with compounds that could be detrimental to human health.

6.12 Duty of Care

Groundworkers must maintain a good standard of personal hygiene including the wearing of overalls, boots, gloves and eye protectors and the use of dust masks during periods of dry weather.

To prevent exposure to airborne dust by both the general public and construction personnel the site should be kept damp during dry weather and at other times when dust were generated as a result of construction activities.

The site should be securely fenced at all times to prevent unauthorised access. Washing facilities should be provided and eating restricted to mess huts.

N

Approximate Site Boundary

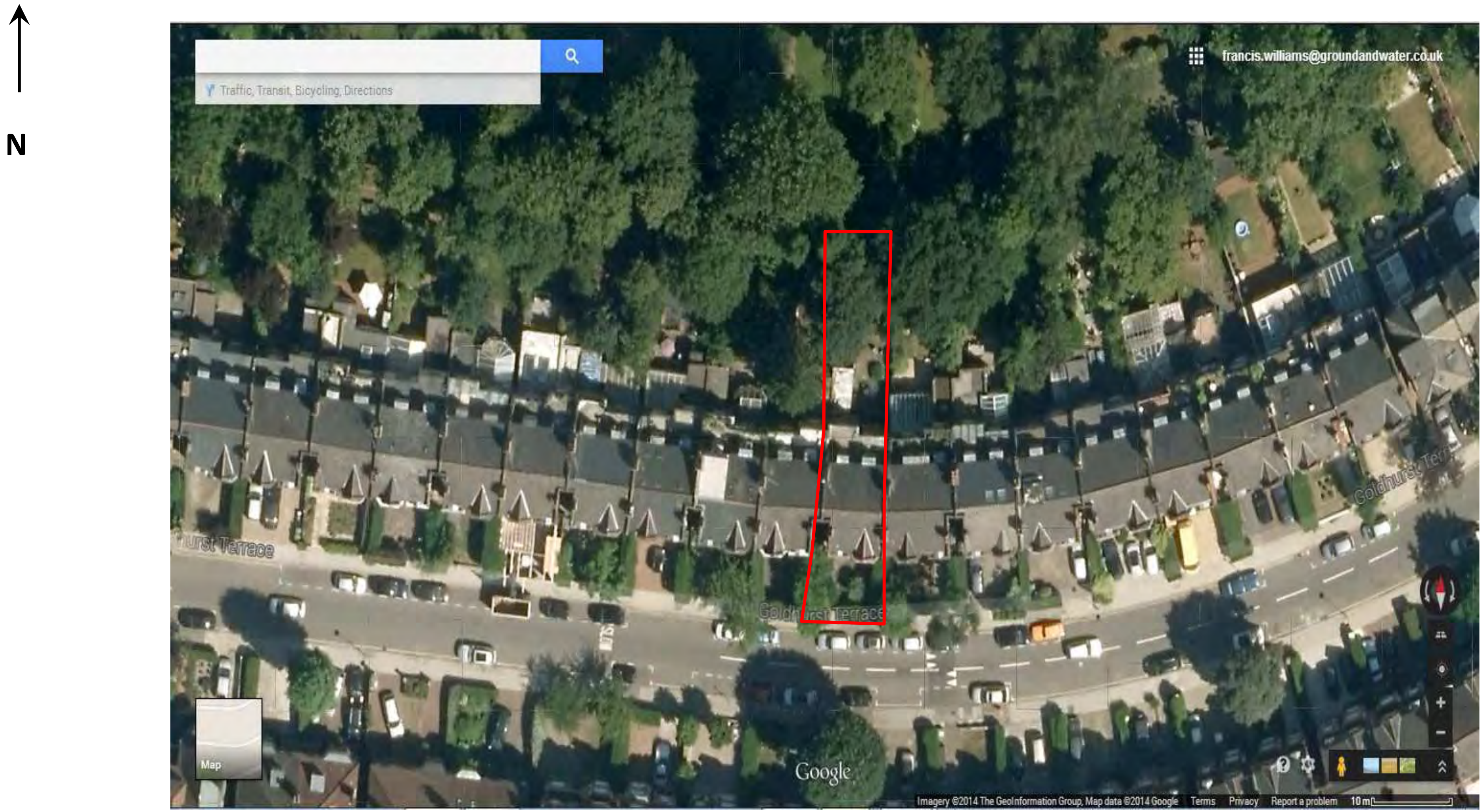
NOT TO SCALE

Project: 156 Goldhurst Terrace, South Hampstead, London NW6 3HP		**Figure 1**
Client: Guy Shani c/o Croft Structural Engineers Limited	**Date:** May 2014	ground&water
Site Location Plan	**Ref:** GWPR910	

Approximate Site Boundary

NOT TO SCALE

Project: 156 Goldhurst Terrace, South Hampstead, London NW6 3HP		Figure 2
Client: Guy Shani c/o Croft Structural Engineers Limited	Date: May 2014	ground&water
Site Development Area	Ref: GWPR910	

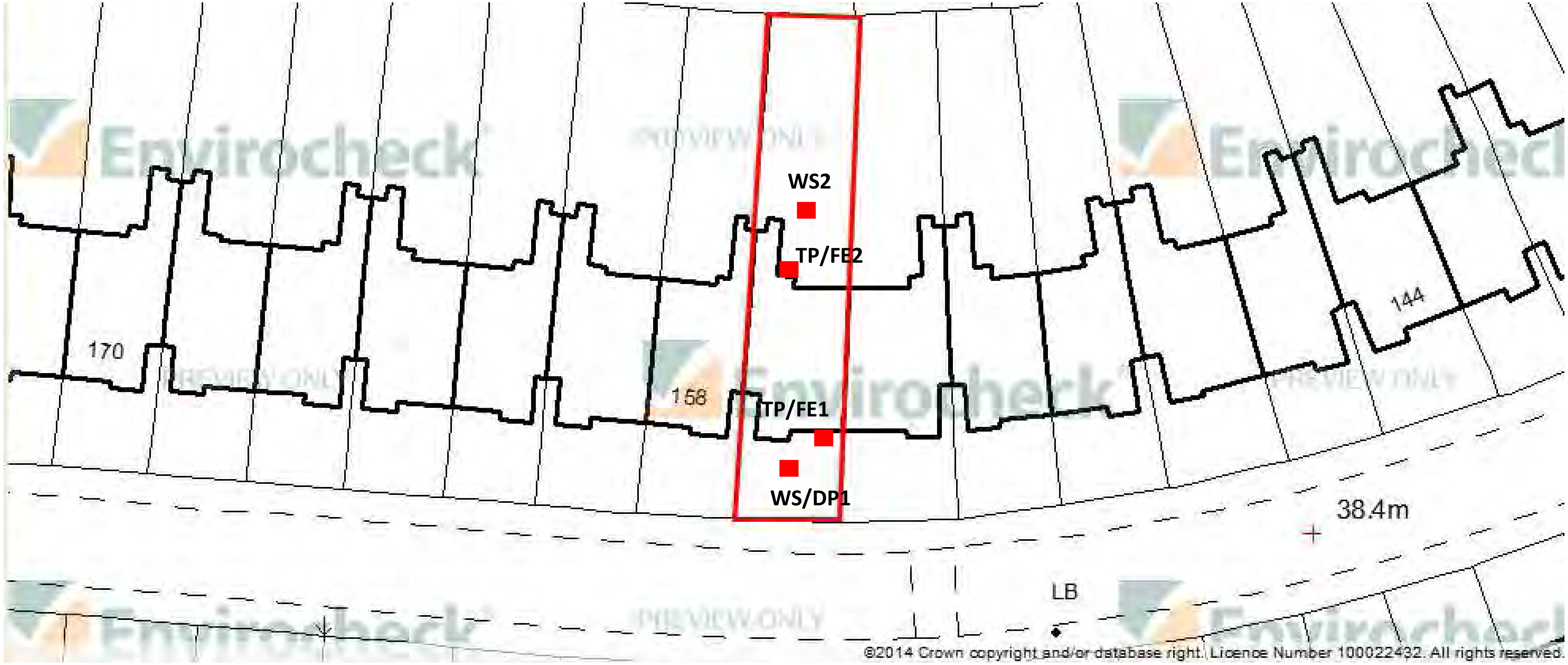


Approximate Site Boundary

NOT TO SCALE

Project: 156 Goldhurst Terrace, South Hampstead, London NW6 3HP		**Figure 3**
Client: Guy Shani c/o Croft Structural Engineers Limited	**Date:** May 2014	ground&water
Aerial View of the Site	**Ref:** GWPR910	

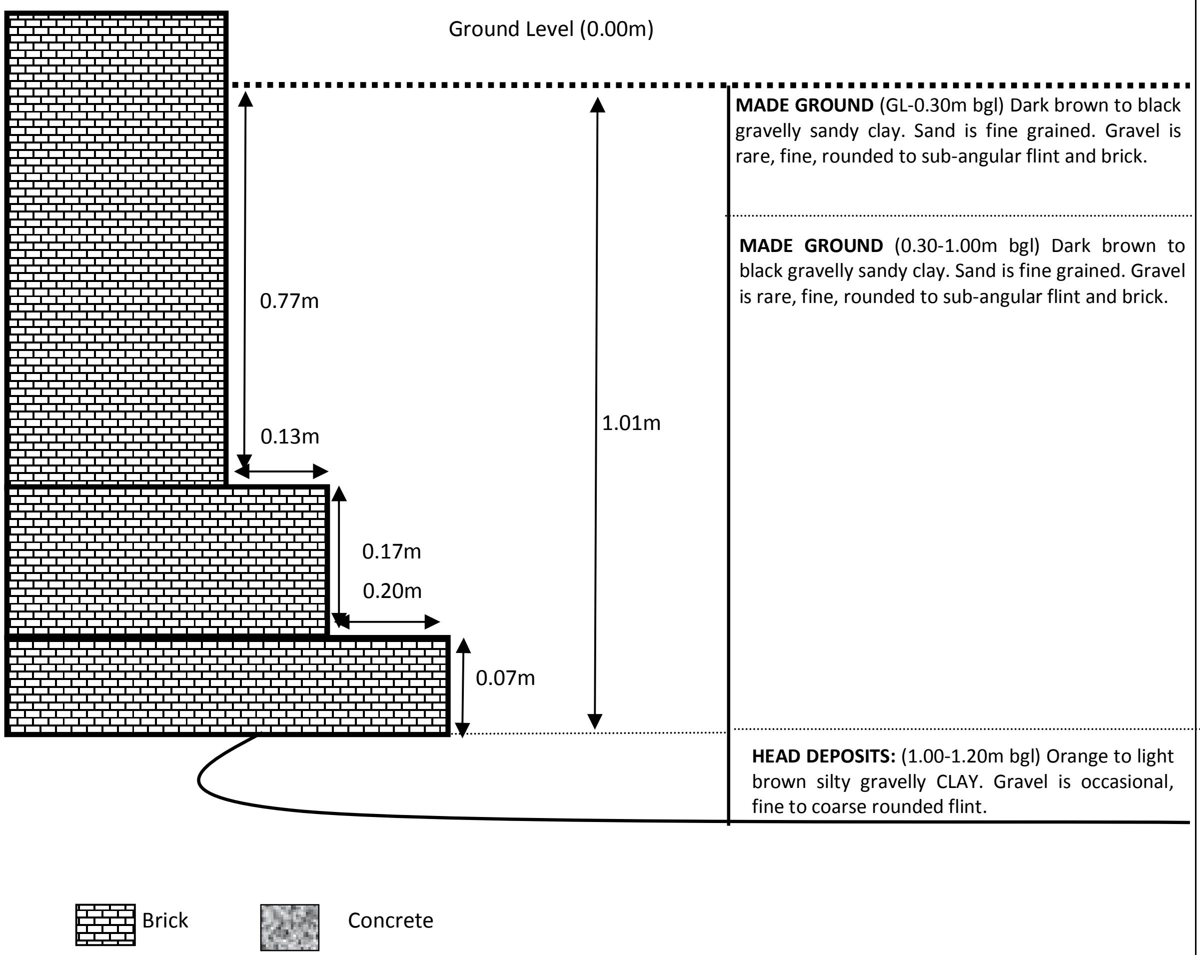
N



Approximate Site Boundary

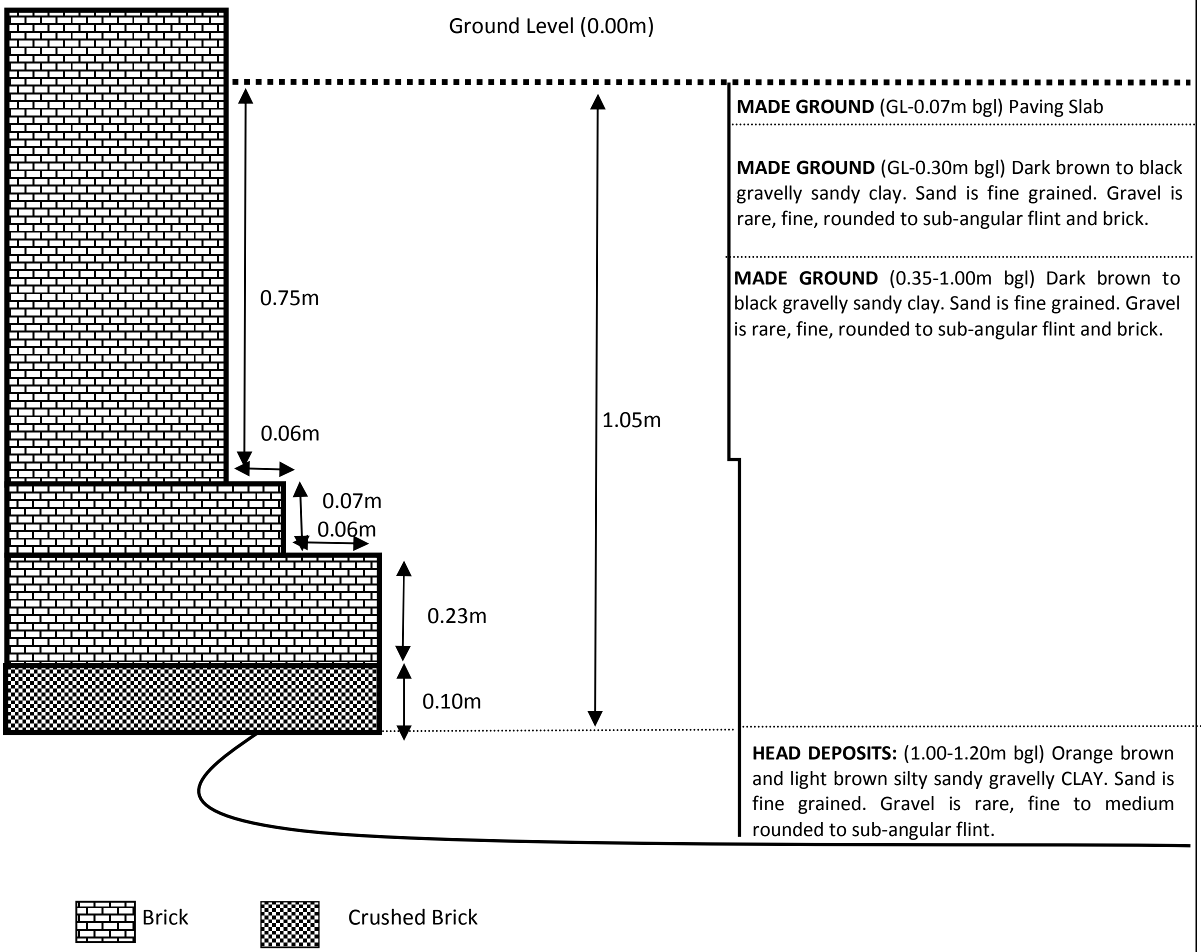
NOT TO SCALE

Project: 156 Goldhurst Terrace, South Hampstead, London NW6 3HP		**Figure 4**
Client: Guy Shani c/o Croft Structural Engineers Limited	**Date:** May 2014	ground&water
Trial Hole Location Plan	**Ref:** GWPR910	



NOTE: NOT TO SCALE

Project: 156 Goldhurst Terrace, South Hampstead, London NW6 3HP		**Figure 5** ground&water
Client: Guy Shani c/o Croft Structural Engineers Limited	**Date:** May 2014	
Section Drawing: Foundation Exposure TP/FE1	**Ref:** GWPR910	



NOTE: NOT TO SCALE

Project:
156 Goldhurst Terrace, South Hampstead, London NW6 3HP

Client:
Guy Shani c/o Croft Structural Engineers Limited

Date:
May 2014

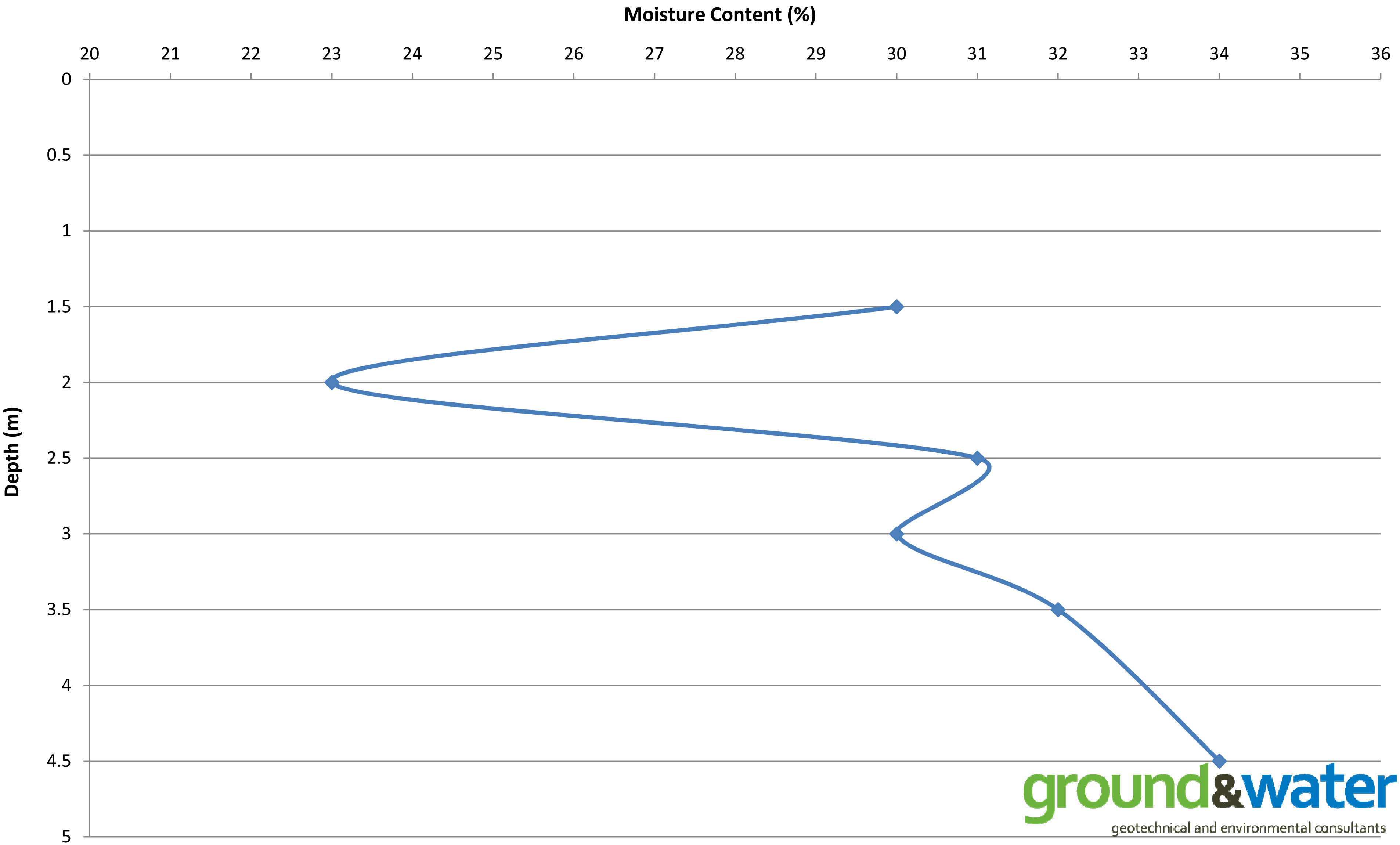
Section Drawing: Foundation Exposure TP/FE2

Ref:
GWPR910

Figure 6

ground&water

Figure 7: Change in Moisture Content With Depth Within WS1



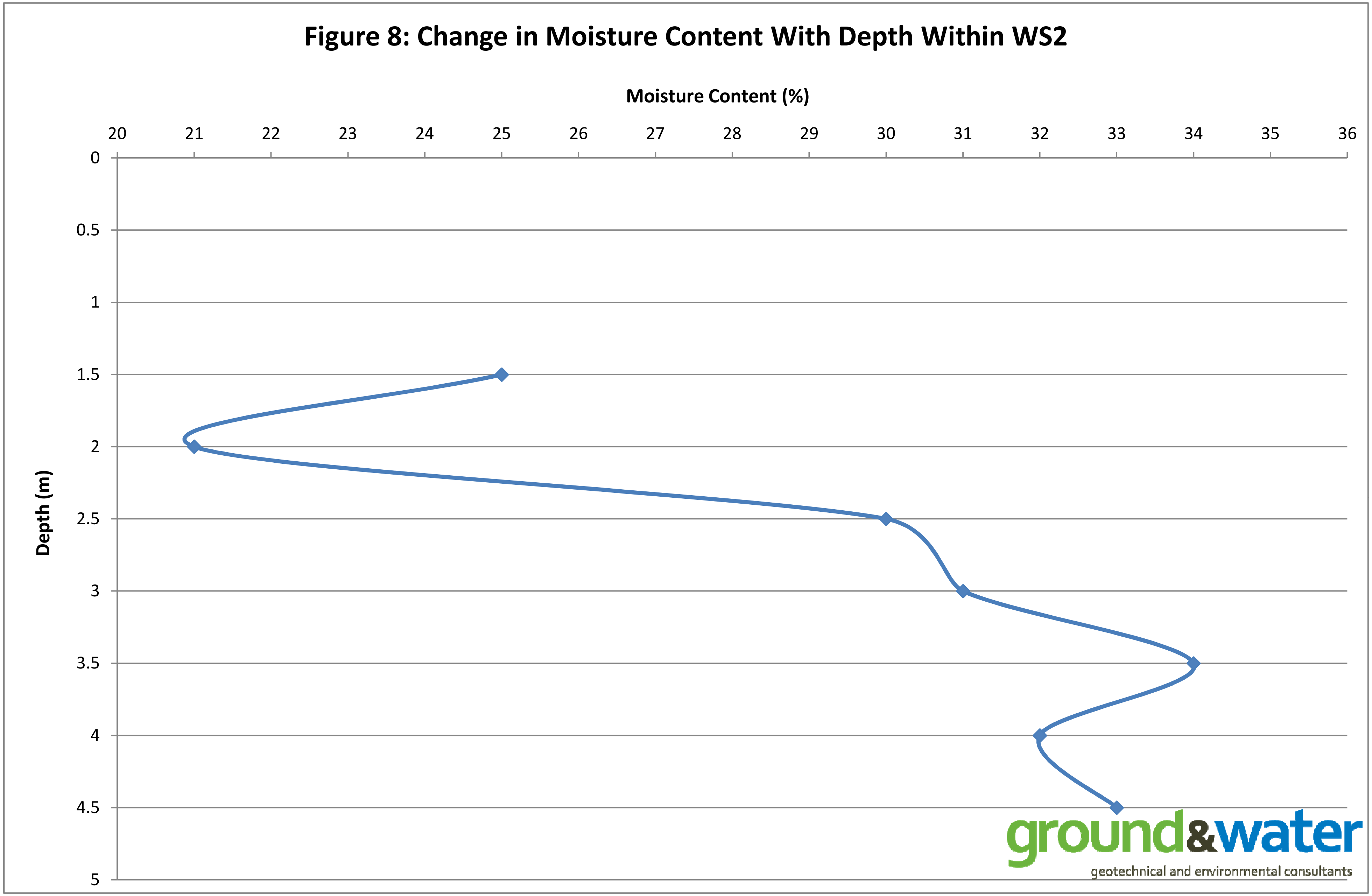


Figure 8: Change in Moisture Content With Depth Within WS2
Moisture Content (%)
20
21
22
23
24
25
26
27
28
29
30
31
32
33
34
35
36
Depth (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
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APPENDIX A
Conditions and Limitations

The ground is a product of continuing natural and artificial processes. As a result, the ground will exhibit a variety of characteristics that vary from place to place across a site, and also with time. Whilst a ground investigation will mitigate to a greater or lesser degree against the resulting risk from variation, the risks cannot be eliminated.

The investigation, interpretations, and recommendations given in this report were prepared for the sole benefit of the client in accordance with their brief; as such these do not necessarily address all aspects of ground behaviour at the site. No liability is accepted for any reliance placed on it by others unless specifically agreed in writing.

Current regulations and good practice were used in the preparation of this report. An appropriately qualified person must review the recommendations given in this report at the time of preparation of the scheme design to ensure that any recommendations given remain valid in light of changes in regulation and practice, or additional information obtained regarding the site.

This report is based on readily available geological records, the recorded physical investigation, the strata observed in the works, together with the results of completed site and laboratory tests. Whilst skill and care has been taken to interpret these conditions likely between or below investigation points, the possibility of other characteristics not revealed cannot be discounted, for which no liability can be accepted. The impact of our assessment on other aspects of the development required evaluation by other involved parties.

The opinions expressed cannot be absolute due to the limitations of time and resources within the context of the agreed brief and the possibility of unrecorded previous in ground activities. The ground conditions have been samples or monitored in recorded locations and tests for some of the more common chemicals generally expected. Other concentrations of types of chemicals may exist. It was not part of the scope of this report to comment on environment/contaminated land considerations.

The conclusions and recommendations relate to 156 Goldhurst Terrace, South Hampstead, London NW6 3HP.

Trial hole is a generic term used to describe a method of direct investigation. The term trial pit, borehole or window sampler borehole implies the specific technique used to produce a trial hole.

The depth to roots and/or of desiccation may vary from that found during the investigation. The client is responsible for establishing the depth to roots and/or of desiccation on a plot-by-plot basis prior to the construction of foundations. Where trees are mentioned in the text this means existing trees, recently removed trees (approximately 15 years to full recovery on cohesive soils) and those planned as part of the site landscaping.

Ownership of copyright of all printed material including reports, laboratory test results, trial pit and borehole log sheets, including drillers log sheets, remain with Ground and Water Limited. Licence is for the sole use of the client and may not be assigned, transferred or given to a third party.

APPENDIX B
Fieldwork Logs

Ground and Water Ltd
Tel: 0333 600 1221
email: enquiries@groundandwater.co.uk
www.groundandwater.co.uk

Borehole No **WS1**

Project Name	Project No.	Co-ords: -	Hole Type
156 Goldhurst Terrace	GWPR910		WS
Location: London NW6 3HP		Level: -	Scale 1:50
Client: Guy Shani c/o Croft Structural Engineers		Dates: 24/04/2014	Logged By DM

Well	Water Strikes	Samples & In Situ Testing Depth (m)	Type	Results	Depth (m)	Level (m AOD)	Legend	Stratum Description
		0.30	D		0.30			MADE GROUND: Dark brown to black gravelly sandy clay. Sand is fine grained. Gravel is rare, fine, rounded to sub-angular flint and brick.
		0.50	D					MADE GROUND: Mid to dark brown gravelly sandy clay. Sand is fine to medium grained. Gravel is rare, fine to coarse, sub-rounded to angular flint, brick and carbonaceous material (clinker).
		0.80	D					
		1.00	D		1.10			HEAD DEPOSITS: Orange to light brown silty gravelly CLAY. Gravel is occasional fine to coarse rounded flints.
		1.50	D					
		2.00	D		2.20			LONDON CLAY FORMATION: Orange brown to mid brown sandy silty CLAY. Sand is fine grained.
		2.50	D		2.60			LONDON CLAY FORMATION: Mid brown to grey silty CLAY.
		3.00	D					
		3.50	D					
		4.00	D					
		4.50	D					
		5.00	D					
		5.50	D					
		6.00	D		6.00			End of Borehole at 6.00 m

Remarks: Fine roots encountered to 1.50m bgl.
No groundwater encountered.

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email: enquiries@groundandwater.co.uk
www.groundandwater.co.uk

Borehole No
WS2
Sheet 1 of 1

Project Name: 156 Goldhurst Terrace	Project No.: GWPR910	Co-ords: -	Hole Type: WS
Location: London NW6 3HP		Level: -	Scale: 1:50
Client: Guy Shani c/o Croft Structural Engineers		Dates: 24/04/2014	Logged By: DM

Sample Depth (m)	Type	Results
0.30	D	
0.50	D	
0.80	D	
1.00	D	
1.50	D	
2.00	D	
2.50	D	
3.00	D	
3.50	D	
4.00	D	
4.50	D	
5.00	D	
5.50	D	
6.00	D	

Depth (m)	Level (m AOD)	Stratum Description
0.07		PAVING SLABS
0.35		MADE GROUND: Dark brown clayey sandy gravel. Sand is fine to coarse. Gravel is abundant, fine to coarse, rounded to angular flint and brick.
1.10		MADE GROUND: Dark brown to mid brown sandy silty gravelly clay. Sand is fine to coarse. Gravel is abundant, fine to coarse, rounded to angular slate, brick and flint.
2.30		HEAD DEPOSITS: Orange brown and light brown silty sandy gravelly CLAY. Sand is fine grained. Gravel is rare, fine to medium, rounded to sub-angular flint.
6.00		LONDON CLAY FORMATION: Brown and grey mottled silty CLAY.

End of Borehole at 6.00 m

Remarks: Fine roots noted to 4.00m bgl.
No groundwater encountered.

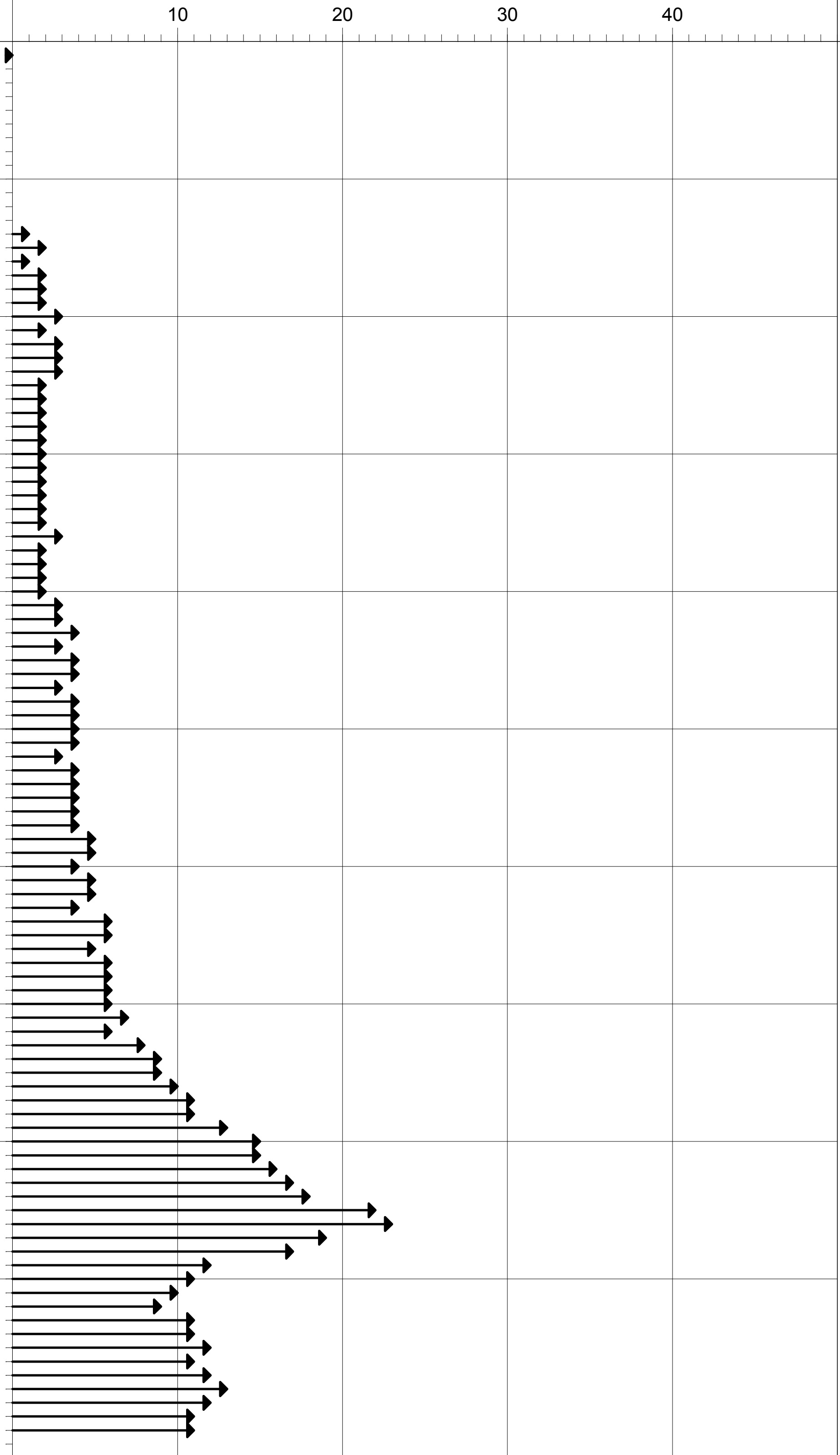
AGS

DYNAMIC PROBING

Probe No	**DP1**
Client **Guy Shani c/o Croft Structural Engineers**	Sheet 1 of 1
Site **156 Goldhurst Terrace**	*Project No* **GWPR910**
E **-** *N* **-** *Level* **-**	*Date* **24/04/2014** *Logged by* **SJM**

Depth (m)	Readings Blows/100mm					Torque (Nm)
	0	-	-	-	-	0
	-	-	-	-	-	
1.0	-	-	-	1	2	
	1	2	2	2	3	
2.0	2	3	3	3	2	
	2	2	2	2	2	
3.0	2	2	2	2	2	
	3	2	2	2	2	
4.0	3	3	4	3	4	
	4	3	4	4	4	
5.0	4	3	4	4	4	
	4	4	5	5	4	
6.0	5	5	4	6	6	
	5	6	6	6	6	
7.0	7	6	8	9	9	
	10	11	11	13	15	
8.0	15	16	17	18	22	
	23	19	17	12	11	
9.0	10	9	11	11	12	
	11	12	13	12	11	
	11					

Diagram (N100 Values)



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email: enquiries@groundandwater.co.uk
www.groundandwater.co.uk

Fall Height	**500**	*Cone Base Diameter*	**43**
Hammer Wt	**50.00**	*Final Depth*	**10.00**
Probe Type	**DPH**	*Log Scale*	1:50

AGS

APPENDIX C
Geotechnical Laboratory Test Results

Project Name:	Goldhurst Terrace, London			**Samples Received:**	07/05/2014	**K4 SOILS**
				Project Started:	08/05/2014	
Client:	Ground and Water Ltd			**Testing Started:**	16/05/2014	
Project No:	GWPR910	**Our job/report no:**	16641	**Date Reported:**	19/05/2014	

Borehole No:	Sample No:	Depth (m)	Description	Moisture content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing 0.425 mm (%)	Remarks
WS1	-	1.50	Brown, orange and ocacsional grey slightly gravelly silty CLAY (gravel is fm and angular)	30	62	26	36	98	
WS1	-	2.00	Orange brown gravelly silty CLAY (gravel is fm and angular to rounded)	23					
WS1	-	2.50	Mauve brown, orange and blue grey silty CLAY with occasional black carbonaceous deposits	31					
WS1	-	3.00	Dark mauve brown and occasional blue grey and orange silty CLAY	30					
WS1	-	3.50	Brown and occasional blue grey silty CLAY with occasional fine siltstone fragments	32	74	30	44	99	
WS1	-	4.50	Brown slightly mottled blue grey silty CLAY with traces of selenite crystals	34	78	32	46	100	
WS2	-	1.50	Brown, orange and grey slightly gravelly slightly sandy silty CLAY (gravel is fm and sub-angular to angular)	25	59	25	34	90	
WS2	-	2.00	Orange brown slightly gravelly slightly sandy silty CLAY (gravel is fm and angular)	21	60	29	31	90	
WS2	-	2.50	Brown slightly mottled blue grey silty CLAY	30					
WS2	-	3.00	Brown slightly mottled blue grey silty CLAY	31					
WS2	-	3.50	Brown and occasional blue grey silty CLAY	34	75	32	43	100	
WS2	-	4.00	Brown slightly mottled blue grey silty CLAY with traces of selenite crystals	32	74	31	43	100	
WS2	-	4.50	Brown slightly mottled blue grey silty CLAY	33					

UKAS TESTING 2519

Summary of Test Results

BS 1377 : Part 2 : Clause 4.4 : 1990 Determination of the liquid limit by the cone penetrometer method.
BS 1377 : Part 2 : Clause 5 : 1990 Determination of the plastic limit and plasticity index.
BS 1377 : Part 2 : Clause 3.2 : 1990 Determination of the moisture content by the oven-drying method.

Checked and Approved
Initials: K.P
Date: 19/05/2014

Test Report by K4 SOILS LABORATORY Unit 8 Olds Close Olds Approach Watford Herts WD18 9RU

Test Results relate only to the sample numbers shown above. Approved Signatories: K.Phaure (Tech.Mgr) J.Phaure (Lab.Mgr)

Project Name:	Goldhurst Terrace, London		
Client:	Ground and Water Ltd	Project no:	GWPR910
		Our job no:	16641

K4 SOILS

Borehole No:	Sample No:	Depth m	Description	pH	Sulphate content (g/l)
WS1	-	1.50	Brown, orange and ocacsional grey slightly gravelly silty CLAY (gravel is fm and angular)	7.6	0.07
WS2	-	2.00	Orange brown slighlty gravelly slightly sandy silty CLAY (gravel is fm and angular)	7.7	0.19

Summary of Test Results

Date
19/05/2014

BS 1377 : Part 3 :Clause 5 : 1990

Determination of sulphate content of soil and ground water : gravimetric method

Checked and Approved
Initials : kp

Francis Williams
Ground & Water Ltd
2 The Long Barn
Norton Farm
Selborne Road
Alton
Hampshire
GU34 3NB

QTS Environmental Ltd
Unit 1
Rose Lane Industrial Estate
Rose Lane
Lenham Heath
Kent
ME17 2JN
t: 01622 850410
russell.jarvis@qtsenvironmental.com

QTS Environmental Report No: 14-21407

Site Reference:	Goldhurst Terrace, London
Project / Job Ref:	GWPR910
Order No:	None Supplied
Sample Receipt Date:	07/05/2014
Sample Scheduled Date:	07/05/2014
Report Issue Number:	1
Reporting Date:	13/05/2014

Authorised by:

Russell Jarvis
Director
On behalf of QTS Environmental Ltd

Authorised by:

Kevin Old
Director
On behalf of QTS Environmental Ltd

QTS Environmental Ltd
Unit 1, Rose Lane Industrial Estate
Rose Lane
Lenham Heath
Maidstone
Kent ME17 2JN
Tel : 01622 850410

mCERTS THE ENVIRONMENT AGENCY'S MONITORING CERTIFICATION SCHEME

UKAS TESTING 4480

Soil Analysis Certificate							
QTS Environmental Report No: 14-21407	**Date Sampled**	24/04/14					
Ground & Water Ltd	**Time Sampled**	None Supplied					
Site Reference: Goldhurst Terrace, London	**TP / BH No**	WS1					
Project / Job Ref: GWPR910	**Additional Refs**	None Supplied					
Order No: None Supplied	**Depth (m)**	4.00					
Reporting Date: 13/05/2014	**QTSE Sample No**	102878					

Determinand	**Unit**	**RL**	**Accreditation**					
pH	pH Units	N/a	**MCERTS**	7.7				
Total Sulphate as SO_4	mg/kg	< 200	NONE	5341				
W/S Sulphate as SO4 (2:1)	g/l	< 0.01	**MCERTS**	1.50				
Total Sulphur	mg/kg	< 200	NONE	1802				
Ammonium as NH_4	mg/kg	< 0.5	NONE	6.6				
W/S Chloride (2:1)	mg/kg	< 1	**MCERTS**	104				
Water Soluble Nitrate (2:1) as NO_3	mg/kg	< 3	**MCERTS**	< 3				
W/S Magnesium	g/l	< 0.0001	NONE	0.2490				

Analytical results are expressed on a dry weight basis where samples are dried at less than 30^{O}C
Analysis carried out on the dried sample is corrected for the stone content
Subcontracted analysis $^{(S)}$

QTS Environmental Ltd
Unit 1, Rose Lane Industrial Estate
Rose Lane
Lenham Heath
Maidstone
Kent ME17 2JN
Tel : 01622 850410

4480

Soil Analysis Certificate - Sample Descriptions	
QTS Environmental Report No: 14-21407	
Ground & Water Ltd	
Site Reference: Goldhurst Terrace, London	
Project / Job Ref: GWPR910	
Order No: None Supplied	
Reporting Date: 13/05/2014	

QTSE Sample No	TP / BH No	Additional Refs	Depth (m)	Moisture Content (%)	Sample Matrix Description
$ 102878	WS1	None Supplied	4.00	19	Light brown clay with chalk

Moisture content is part of procedure E003 & is not an accredited test
Insufficient Sample I/S
Unsuitable Sample U/S

$ samples exceeded recommended holding times

QTS Environmental Ltd
Unit 1, Rose Lane Industrial Estate
Rose Lane
Lenham Heath
Maidstone
Kent ME17 2JN
Tel : 01622 850410

4480

Soil Analysis Certificate - Methodology & Miscellaneous Information
QTS Environmental Report No: 14-21407
Ground & Water Ltd
Site Reference: Goldhurst Terrace, London
Project / Job Ref: GWPR910
Order No: None Supplied
Reporting Date: 13/05/2014

Matrix	Analysed On	Determinand	Brief Method Description	Method No
Soil	D	Boron - Water Soluble	Determination of water soluble boron in soil by 2:1 hot water extract followed by ICP-OES	E012
Soil	AR	BTEX	Determination of BTEX by headspace GC-MS	E001
Soil	D	Cations	Determination of cations in soil by aqua-regia digestion followed by ICP-OES	E002
Soil	D	Chloride - Water Soluble (2:1)	Determination of chloride by extraction with water & analysed by ion chromatography	E009
Soil	AR	Chromium - Hexavalent	Determination of hexavalent chromium in soil by extraction in water then by acidification, addition of 1,5 diphenylcarbazide followed by colorimetry	E016
Soil	AR	Cyanide - Complex	Determination of complex cyanide by distillation followed by colorimetry	E015
Soil	AR	Cyanide - Free	Determination of free cyanide by distillation followed by colorimetry	E015
Soil	AR	Cyanide - Total	Determination of total cyanide by distillation followed by colorimetry	E015
Soil	D	Cyclohexane Extractable Matter (CEM)	Gravimetrically determined through extraction with cyclohexane	E011
Soil	AR	Diesel Range Organics (C10 - C24)	Determination of hexane/acetone extractable hydrocarbons by GC-FID	E004
Soil	AR	Electrical Conductivity	Determination of electrical conductivity by addition of saturated calcium sulphate followed by electrometric measurement	E022
Soil	AR	Electrical Conductivity	Determination of electrical conductivity by addition of water followed by electrometric measurement	E023
Soil	D	Elemental Sulphur	Determination of elemental sulphur by solvent extraction followed by GC-MS	E020
Soil	AR	EPH (C10 – C40)	Determination of acetone/hexane extractable hydrocarbons by GC-FID	E004
Soil	AR	EPH Product ID	Determination of acetone/hexane extractable hydrocarbons by GC-FID	E004
Soil	AR	EPH TEXAS	Determination of acetone/hexane extractable hydrocarbons by GC-FID	E004
Soil	D	Fluoride - Water Soluble	Determination of Fluoride by extraction with water & analysed by ion chromatography	E009
Soil	D	FOC (Fraction Organic Carbon)	Determination of fraction of organic carbon by oxidising with potassium dichromate followed by titration with iron (II) sulphate	E010
Soil	D	Loss on Ignition @ 450oC	Determination of loss on ignition in soil by gravimetrically with the sample being ignited in a muffle furnace	E019
Soil	D	Magnesium - Water Soluble	Determination of water soluble magnesium by extraction with water followed by ICP-OES	E025
Soil	D	Metals	Determination of metals by aqua-regia digestion followed by ICP-OES	E002
Soil	AR	Mineral Oil (C10 - C40)	Determination of hexane/acetone extractable hydrocarbons by GC-FID fractionating with SPE cartridge	E004
Soil	AR	Moisture Content	Moisture content; determined gravimetrically	E003
Soil	D	Nitrate - Water Soluble (2:1)	Determination of nitrate by extraction with water & analysed by ion chromatography	E009
Soil	D	Organic Matter	Determination of organic matter by oxidising with potassium dichromate followed by titration with iron (II) sulphate	E010
Soil	AR	PAH - Speciated (EPA 16)	Determination of PAH compounds by extraction in acetone and hexane followed by GC-MS with the use of surrogate and internal standards	E005
Soil	AR	PCB - 7 Congeners	Determination of PCB by extraction with acetone and hexane followed by GC-MS	E008
Soil	D	Petroleum Ether Extract (PEE)	Gravimetrically determined through extraction with petroleum ether	E011
Soil	AR	pH	Determination of pH by addition of water followed by electrometric measurement	E007
Soil	AR	Phenols - Total (monohydric)	Determination of phenols by distillation followed by colorimetry	E021
Soil	D	Phosphate - Water Soluble (2:1)	Determination of phosphate by extraction with water & analysed by ion chromatography	E009
Soil	D	Sulphate (as SO4) - Total	Determination of total sulphate by extraction with 10% HCl followed by ICP-OES	E013
Soil	D	Sulphate (as SO4) - Water Soluble (2:1)	Determination of sulphate by extraction with water & analysed by ion chromatography	E009
Soil	D	Sulphate (as SO4) - Water Soluble (2:1)	Determination of water soluble sulphate by extraction with water followed by ICP-OES	E014
Soil	AR	Sulphide	Determination of sulphide by distillation followed by colorimetry	E018
Soil	D	Sulphur - Total	Determination of total sulphur by extraction with aqua-regia followed by ICP-OES	E024
Soil	AR	SVOC	Determination of semi-volatile organic compounds by extraction in acetone and hexane followed by GC-MS	E006
Soil	AR	Thiocyanate (as SCN)	Determination of thiocyanate by extraction in caustic soda followed by acidification followed by addition of ferric nitrate followed by colorimetry	E017
Soil	D	Toluene Extractable Matter (TEM)	Gravimetrically determined through extraction with toluene	E011
Soil	D	Total Organic Carbon (TOC)	Determination of organic matter by oxidising with potassium dichromate followed by titration with iron (II) sulphate	E010
Soil	AR	TPH CWG	Determination of hexane/acetone extractable hydrocarbons by GC-FID fractionating with SPE cartridge	E004
Soil	AR	TPH LQM	Determination of hexane/acetone extractable hydrocarbons by GC-FID fractionating with SPE cartridge	E004
Soil	AR	VOCs	Determination of volatile organic compounds by headspace GC-MS	E001
Soil	AR	VPH (C6 - C10)	Determination of hydrocarbons C6-C10 by headspace GC-MS	E001

D Dried
AR As Received