Job Number: 131026
Date 14th November 2013

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Basement Structural Method Statement

51 Fitzjohn's Ave
Camden NW3 6PH

John Hough
Oakley Hough Limited
The Barn, Stebbing Farm,
Fishers Green, Stevenage,
Hertfordshire SG1 2JB

Revision	Date	Comment
-	15/11/2013	Issued for comments
1	15/05/2014	Minor amendments to report following comments. Preliminary BH logs added
2	24/09/2014	Network Rail Tunnels location & settlement/heave/uplift calculations added. Alterations to drawings

Table of Contents

1. Design Information - Structural

Structural Summary

51 Fitzjohn's Avenue is a multi occupancy, 6 storey high property with load bearing external masonry walls, internal load bearing masonry walls. The floors on each floor appear to be timber, spanning from left to right between the walls and the roof is of a timber structure. The lower ground floor is already present to the right front side and the left rear side of the building. The floor above the rear lower ground part is a thick concrete/precast slab and to the front there are timber joists.

Figure 1:Front View

Proposed works

The proposed work constitutes amendments to the internal walls layout and a new lower ground development under the part of the property which at present stops at the ground level. This will be constructed in reinforced concrete retaining walls underpinning the existing external walls. Light wells will be created to the front of the property. The light wells will have a grille over them.

Croft Structural Engineers Ltd has extensive knowledge of the design and construction of new basements. Over the last 4 years we have completed over 150 basements in and around the local area. The method developed is:

1. Excavate front to allow conveyor to be inserted.
2. Form 'front of basement' with cantilevered retaining walls
3. Slowly work from the front to the rear inserting 1200 long cantilevered retaining walls sequentially.
4. Cast ground slab

5. Waterproof internal space with a drained cavity system.

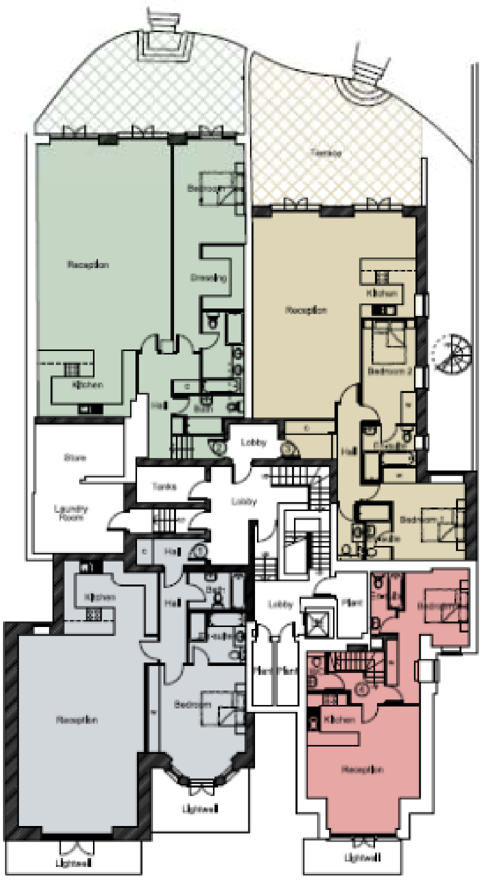


Figure 2: Proposed Lower Ground Floor Plan

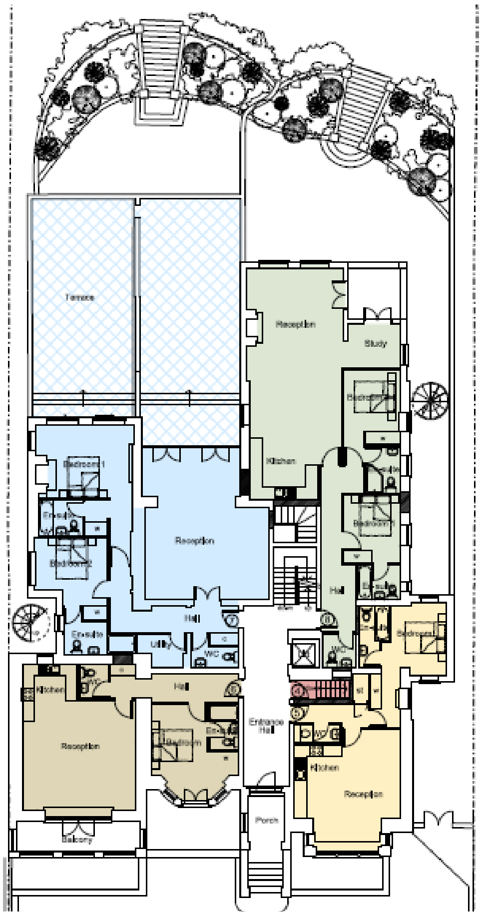


Figure 3: Proposed Ground Floor Plan

Structural Defects Noted

No defects were noted during the Chartered Engineers first visit.

Intended use of structure and user requirements

Family/domestic use

Loading Requirements

	UDL kN/m^2	Concentrated Loads kN
Domestic Single Dwellings	1.5	2.0

The basement does not lie within a 45 angle of the highway and is more than 5m from the road.

$5kN/m^2$ if within 45° of Pavement

Garden Surcharge $2.5kN/m^2$

<table>
<tr><td></td><td>Surcharge for adjacent property 1.5kN/m² + 4kN/m² for concrete ground bearing slab

<u>Adjacent Properties:</u>
All adjacent property's footings within 45° to have additional geotechnical engineers input</td></tr>
<tr><td>Number of storeys</td><td>6

Is Live Load Reduction included in design No</td></tr>
<tr><td></td><td><h2>Progressive Collapse</h2>Design for consequences of localized failure in building from an unspecified cause</td></tr>
<tr><td>Is the Building Multi Occupancy?</td><td>No</td></tr>
<tr><td>Part A3 Progressive collapse</td><td>EN 1991-1-7:1996 Table A1
<table><tr><th></th><th></th></tr><tr><td>Class 2B</td><td>Hotels, flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys</td></tr></table></td></tr>
<tr><td>Progressive collapse Change of use</td><td>To NHBC guidance compliance is only required to other floors if a material change of use occurs to the property.
<table><tr><td>Initial Building Class</td><td>2B</td></tr><tr><td>Proposed Building Class</td><td>2B</td></tr><tr><td>If class has changed material change has occurred</td><td>No</td></tr></table></td></tr>
<tr><td>Additional Design Requirements to Comply with Progressive Collapse</td><td><u>Class 2B – Design provision of effective horizontal and vertical ties to all areas increased in class.</u></td></tr>
<tr><td>Exposure and wind loading conditions</td><td><h2>Lateral Stability</h2>0.6 kN/m²</td></tr>
</table>

Stability design	The main existing masonry stability walls are not being altered. The reinforced concrete retaining walls are designed to carry the lateral loading applied from above. The lateral earth pressure exerts a horizontal force on the retaining walls. The retaining walls will be checked for resistance to the overturning force this produces.
Lateral Actions	Lateral Forces applied from; Soil loads Hydrostatic pressure Surcharge loading These produce retaining wall thrust; this is restrained by the opposing retaining wall.

DP27	
A	**Maintain Structural Stability of the building & Neighbouring Properties.** The attached drawing shows the reinforcement and construction required by maintain stability of the property, the neighbouring buildings and the road. Calculations results are shown in the Impact Assessment Part.
B	**Avoid Adversely Affecting drainage and Run off.** The area of hard standing remains unchanged and run off will not be altered. The property will not affect the main aquifer as the site does not lay above an aquifer. See Screening Stage information
C	Avoid Cumulative Impact upon Structural Stability or the water environment. See Scoping stage that indicates location in relations to water course and Hampstead heath catchment. See Impact Assessment and drawings. Additional drainage layer has been placed under the building. The structure is designed to take account of Hydrostatic head on the basement.
D	**Harm the Amenity of Neighbours** Noise and nuisance have been considered in Impact Assessment stage.
E	**Loss of Open Space or Trees** There is no loss of open space. Trees are unaffected. The current roots will be above the existing foundations and therefore the new foundations will not cut through significant roots.

2. Basement Impact: Screening

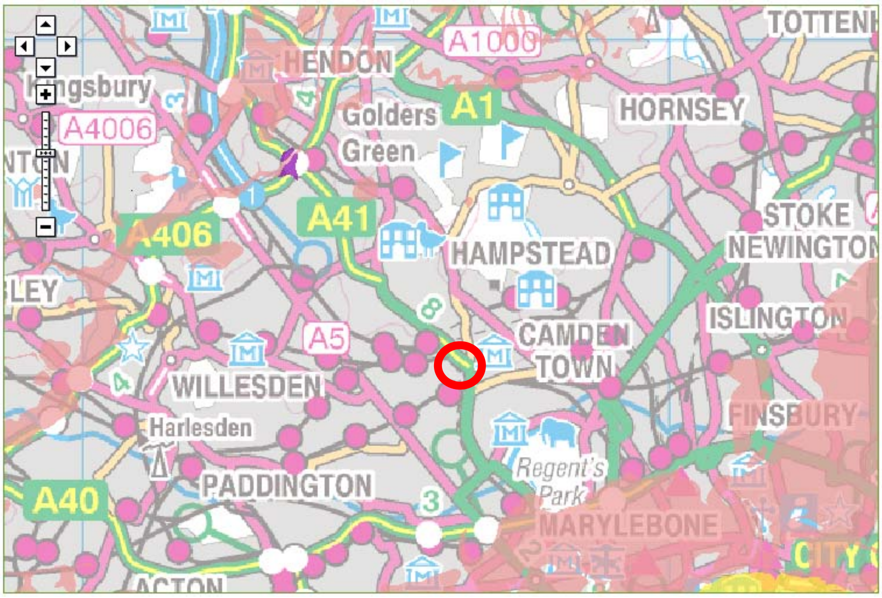
The questions below are taken from the Camden CPG 4 – Basements and Lightwells.

Questions have been taken from Appendix E of the Arup Hydrology report

Groundwater flow

1a. Is the site located directly above an aquifer?

No. The Environment Agency maps do not show the site to lie above an aquifer.



1b. Will the proposed basement extend beneath the water table surface?

Unknown. Proposed basement will extend to approximately 3.2meters.
Requires scoping assessment and investigation.

2. Is the site within 100m is a watercourse, well used/disused or potential spring line?

Unknown. OS maps and local walkover survey show no wells, watercourses or potential spring lines within 100m of the site, although a western tributary to the River Tyburn formerly ran along the current line of Fitzjohn’s Avenue.

Requires scoping and investigation

3. Is the site within the catchment of the pond chains on Hampstead Heath?

No. The site lies outside the areas of the pond chains on Hampstead Heath.

4. Will the proposals basement development result in a change in the proportion of hard surfaced/ paved areas?

No. The surfaces to the front & rear are to remain unchanged.

Slope Stability

5. As part of the site drainage will more surface water (e.g. rainfall and run-off) than at present be discharged to the ground (e.g. via. Soakaways and or SUDS)?
No. Existing roof Drainage will run into the existing drainage system. Surface water will still discharge to ground.

6. Is the lowest point of the proposed excavation (allowing for any drainage and foundation space under the basement floor) close to or lower than, the mean water level in and local pond (not just the pond chains on Hampstead Heath) or spring line?

From walkover and OS maps, there are no local ponds or springs of significance, although the river Tyburn arose at the northern end of Fitzjohn's Avenue and the boundary between the Claygate and the London Clay could generate springs.

Requires scoping and investigation.

Figure 2 – Slope Stability screening flowchart

1. Does the existing site include slopes, natural or man made greater than 7º (approximately 1 in 8)?
No. Difference in height between the rear garden and front is less than 1 in 8 slope (approx flat)

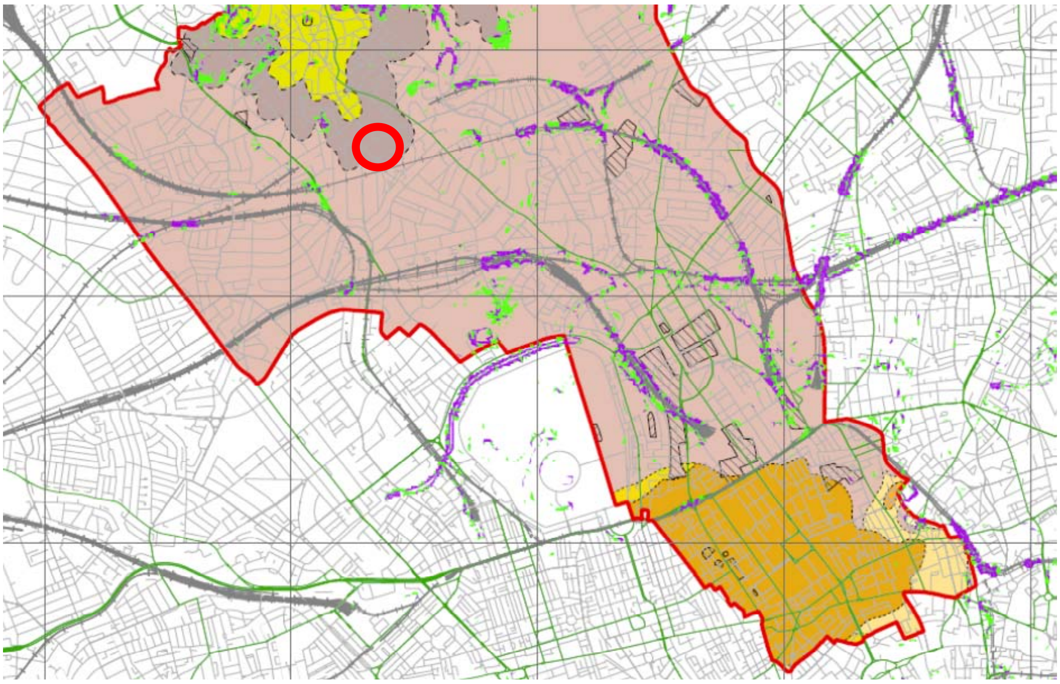


Figure 4: Arup Report Figure 16

2. Will the proposed re profiling of landscaping at site change slopes at the property boundary to more than 7º (approximately 1in 8)?

No. Proposed landscaping does not affect the slope.

3. Does the development neighbour land including railway cuttings and the like with a slope greater than 7° (approximately 1 in 8)?
No. Proposed landscaping does not affect the slope.

4. Is the site within a wider hillside setting in which the general slope is greater than 7° (approximately 1 in 8)?
No. The slope of the wider hillside setting is as per the property, less than 7°

5. Is the London Clay the shallowest strata on site?
Yes. The site sits on the Claygate beds part of the London Clay formation.

Requires scoping and investigation.

6. Will any tree/s be felled as part of the proposed development and/or are any of the works proposed within any tree protection zones where trees are to be retained?
No. No local trees are to be felled. The impact of the basement on these trees should be considered
Carry forward to scoping stage.

7. Is there a history of seasonal shrink-swell subsidence in the local area, and/ or evidence of such effects at the site?
No. From the walk over survey Subsidence was not considered as an issue on this site.

The site is on Shrinkable ground and as such has an increased risk to subsidence. The basement and all foundations will be designed to take account of the ground conditions. The basement construction places the loads of the property on to deep ground. The depth further protects the building from the seasonal changes in the ground.

8. Is the site within 100m of a watercourse or a potential spring line?

Unknown. OS maps and local walkover survey show no wells, watercourses or potential spring lines within 100m of the site, although a western tributary to the River Tyburn formerly ran along the current line of Fitzjohn's Avenue.

Requires scoping and investigation.

9. Is the site within an area of previously worked ground?
No. From the historical maps, the site has been residential for the past 150 years.

10. Is the site within an aquifer? If so will the proposed basement extend beneath the water table such that dewatering may be required during construction?

No. The Environment Agency maps do not show the site to lie above an aquifer.

Arups report shows the site to be am unproductive strata.

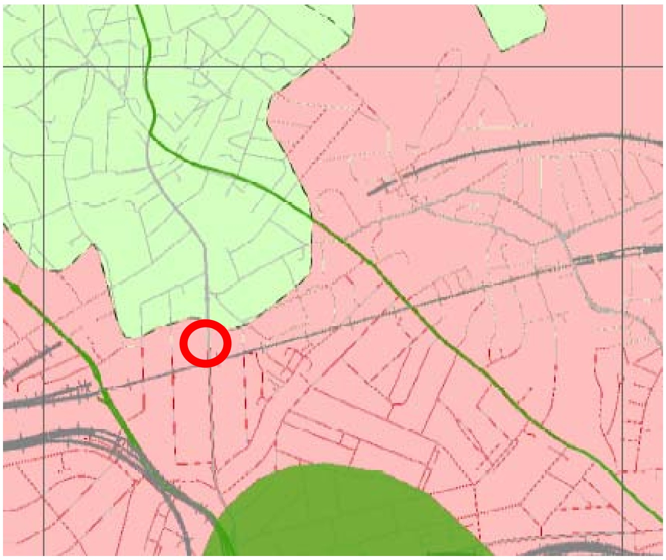


Figure 5: Arup Report Figure 8

11. Is the site within 50m of the Hampstead Heath ponds?
No.

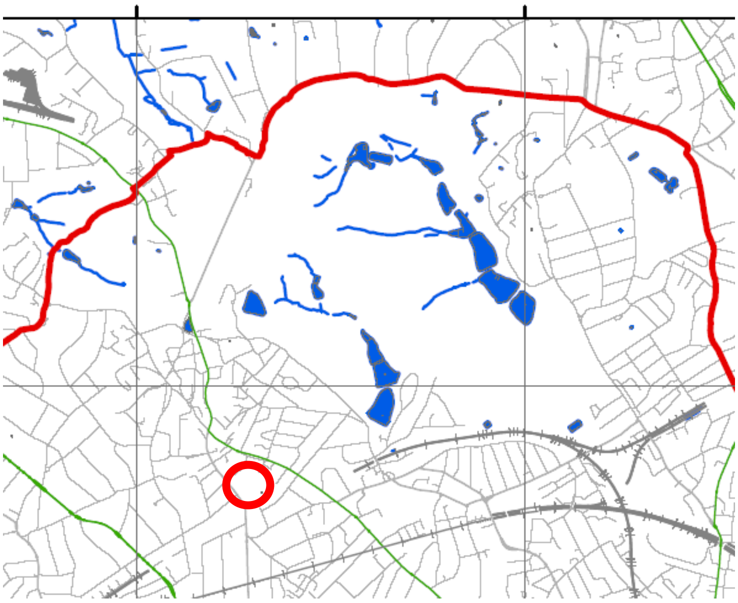


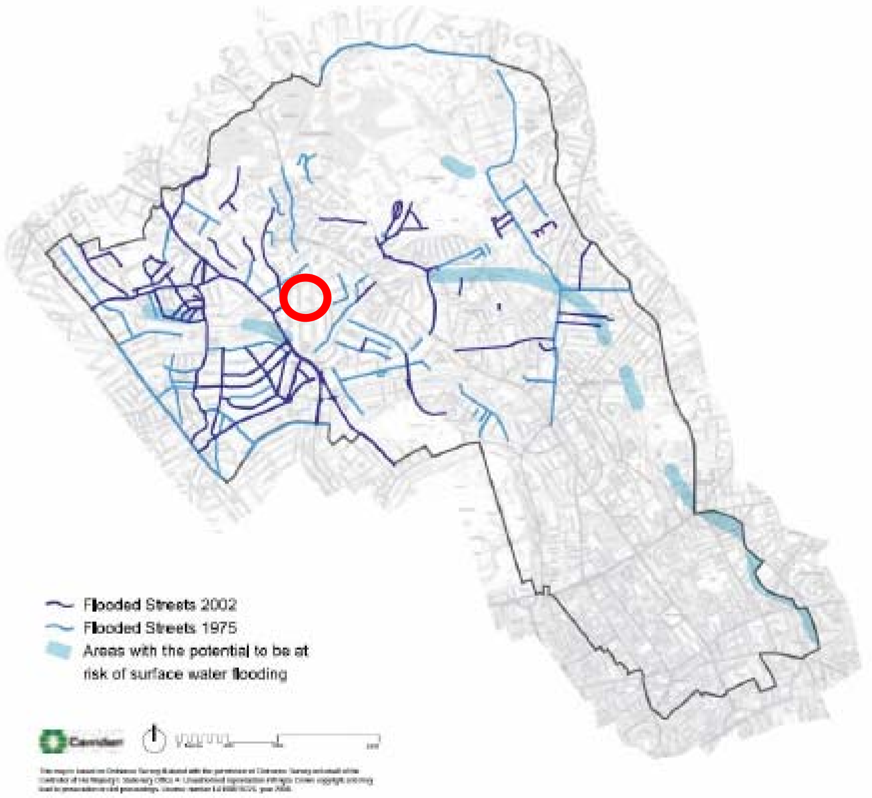
Figure 6: Arup Report Figure 12

12. Is the site within 5m of a highway or pedestrian footway?
Yes. Site is within 5m of the footpath/alleyway.

Carry forward to scoping stage. The design will need to take account of the highway loading.

13. Will the proposed basement significantly increase the differential depth of

	foundations relative to the neighbouring properties? **No**. Existing building already has a lower ground level, and proposed development is to extend the lower ground floor under the full footprint of the building. Party wall will not be underpinned as the building is free standing. Existing footings are expected to be corbelled masonry approx. 1000mm below ground level. **Carry forward to scoping stage: Overall design to be considered.** **14. Is the site over (or within the exclusion zone) of any tunnels, e.g. railway lines?** **No**. Nearest is the LUL Line, approximately 100m from site. Confirmation at design stage from LUL is required to confirm their assets are not affected. **Requires scoping and investigation**
Surface flow and flooding	**1. Is the site within a catchment of the pond chains on Hampstead Heath?** The site lies outside the catchment areas of the Hampstead heath ponds as shown on figure 14 of the Camden Hydrological Study **2. As part of the proposed site drainage, will surface water flows (e.g. volume of rainfall and peak run-off) be materially changed from the existing route?** **No**. The area of hard standing remains unchanged by the development. **3. Will the proposed basement development result in a change to the hard surfaced /paved external areas?** **No.** The amount of hard standing will remain unchanged **4. Will the proposed basement result in changes to the inflows (instantaneous and long term of surface water being received by adjacent properties or downstream watercourses?** **No**. The proposed development will enter the current drainage system. **5. Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream watercourses?** **No**. The quality of water is unlikely to be altered. **6. Is the site in an area known to be at risk from surface water flooding, such as South Hampstead, West Hampstead Gospel Oak and King's Cross or is it at risk form flooding, for example because the proposed basement is below the static water lever of a nearby surface water feature?**



Fitzjohn’s Ave is not noted on the previously flooded streets in Camden’s fig 5 core Strategy.

3. Basement Impact: Screening Maps

Attached maps support Screening information

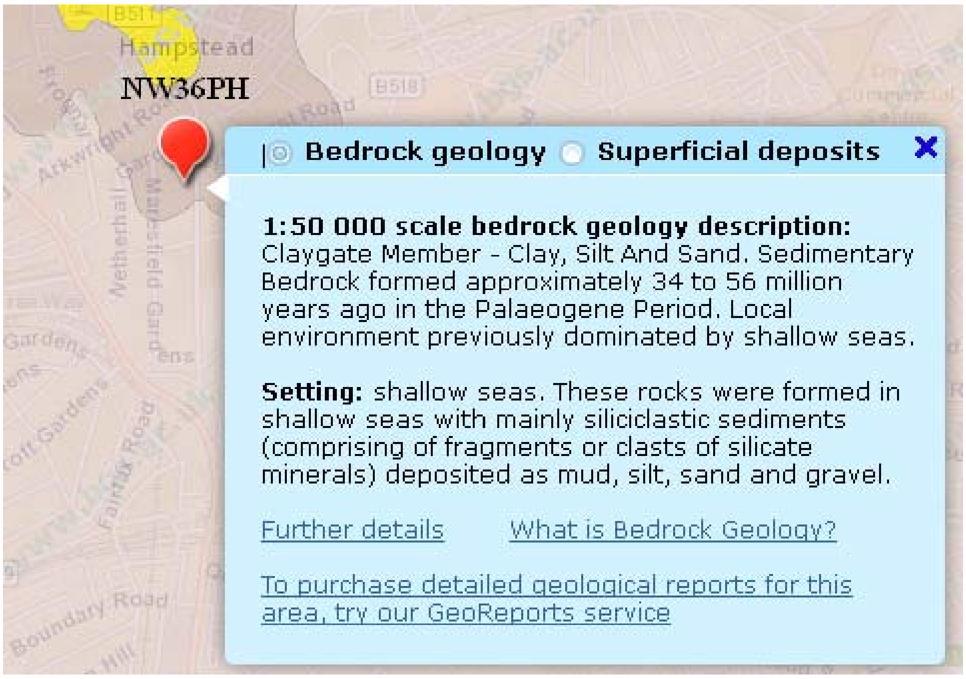
4. Basement Impact: Scoping

Groundwater flow	**Subterranean flow** There is an existing basement already present underneath half of the property, the refurbishment will drop the level only to the existing lower ground floor. The soil investigation report attached in appendix D shows that the water strikes at 5.5m below ground level, which is approximately 2m below new development.
Slope Stability	The Claygate beds are expected to be the top layer. The slope stability of theses beds is in the region of 40˚. The design of the RC retaining walls will take this into account. The basement is within 5m of the footpath, and will therefore be designed with a 5kN/m² surcharge.
Surface flow and flooding	This proposal is not considered to be in an area a risk of flooding. The flow of surface water above the basement (top 1m of soil) will need to be considered.

5. Desk Study and Walkover Survey

Subsoil conditions

The Geology of Britain viewer Map Indicates the site is underlain by Claygate member. This is as expected in the area and was confirmed during soil investigation.



1:50 000 scale bedrock geology description: Claygate Member - Clay, Silt And Sand. Sedimentary Bedrock formed approximately 34 to 56 million years ago in the Palaeogene Period. Local environment previously dominated by shallow seas.

Setting: shallow seas. These rocks were formed in shallow seas with mainly siliciclastic sediments (comprising of fragments or clasts of silicate minerals) deposited as mud, silt, sand and gravel.

Further details What is Bedrock Geology?

To purchase detailed geological reports for this area, try our GeoReports service

Figure 7 Extract From North London Drift Sheet

Walk over Survey

Figure 8: Adjacent property on left side

Figure 9: Adjacent property on right side

The existing building did not exhibit any signs of subsidence nor movement. The building is free standing, with adjacent buildings approximately 1.5m away. The effects of the development on the

	adjacent properties will need to be considered.
Drainage effects on Structure	No build over agreements known of.
Under ground	Underground line is approximately 100m away.
Sources of Contaminates	From the Historic Maps it can be seen that the ground use has not been conducive to activities leading to poor ground. During the walk over survey no items were noted that may lead to contamination.
Water Course	No wells were noted on site The site is not shown within the areas of recent local flooding in the Arup's report. The site is not within the Hampstead pond catchment area as shown in the Arup's report. The site is not within any local water course noted in the Arup's report.

6. Impact Assessment

Slope Stability	From the walk over survey, the OS map and the Arups report the slopes around the site are less than 7º. Land slip is not a problem due to any circular failure patterns. The retaining walls must be designed to accommodate the lateral pressures from the soils.
Foundation type	Reinforced concrete cantilevered retaining walls The designs for the retaining walls have been calculated using TEDDS software. The software is specifically designed for retaining walls and ensures the design is kept to a limit to prevent damage to the adjacent property. Attached printout of Calculations can be found in Appendix B. The overall stability of the walls are design using K_a & K_p values, while the design of the wall uses K_o values. This approach minimise the level of movement from the concrete affecting the adjacent properties. The walls are designed to cope with the hydrostatic pressure. It is possible that a water main may break causing local high water table. To account for this the wall is designed for water to the full height fo the retaining wall. The Design also considers floatation as a risk. The design of has considered the weight of the building and the uplift forces from the water. The weight of the building is greater than the uplift resulting in a stable structure. Below are the design pressures and loadings.

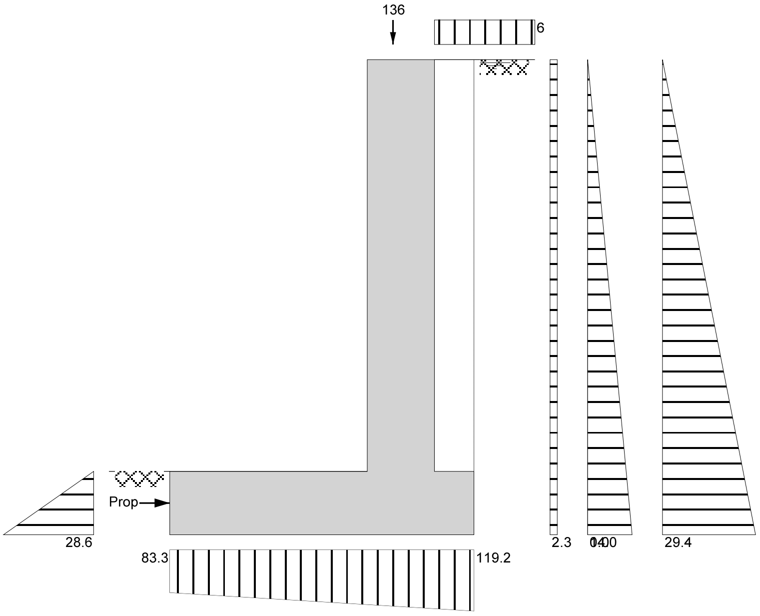


Figure 10: RC Retaining wall 1 design

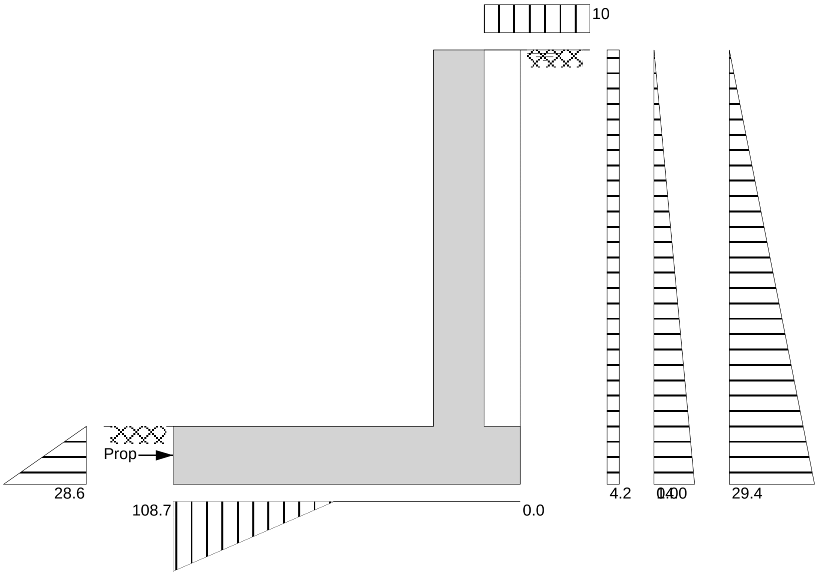


Figure 11: RC Retaining wall 2 design

Full calculations for RC retaining walls 1 & 2 can be found in appendix C.

Special precautions due to trees	Design using NHBC guidance Basement depth will allow for footings to be placed outside the effects of the trees. The current trees roots will be limited by the existing foundations. The new basements excavations will not significantly/ adversely affect the root protection zones of the neighbouring trees.
Drainage effects on Structure	No build over agreements known of. Flooding. The site is not in an area of high risk flooding.
Roads	The building does not undermine the highway, but car parking is present to the front of the property. It is possible for heavier goods vehicles to reverse on to the property to allow for this risk loadings are to be taken from the Highways loading code. 10kN/m^2 to front light well Garden Surcharge 2.5kN/m^2 Surcharge for adjacent property 1.5kN/m^2 + 4kN/m^2 for concrete ground bearing slab

Adjacent Properties	***Any ground works pose an elevated risk to adjacent properties. The proposed works undermines the adjacent property along the party wall line:*** The party wall is to be underpinned. Underpinning the party wall will remove the risk of the movement to the adjacent property. The works must be carried out in accordance with the party wall act and condition surveys will be necessary at the beginning and end of the works. The method statement provided at the end of this report has been formulated with our experience of over 150 basements completed without error. The design of the retaining walls is completed to K_O lateral design stress values. This increases the design stresses on the concrete retaining walls and limits the overall deflection of the retaining wall. It is not expected that any cracking will occurring during the works. However our experience informs us that there is a risk of movement to the neighbours. See settlement, uplift & heave calculations in appendix E. To reduce the risk the development: • Employ a reputable firm for extensive knowledge of basement works. • Employ suitably qualified consultants. Croft Structural engineer has completed over 150 basements in the last 4 years. • Design the underpins to the stable without the need for elaborate temporary propping or needing the floor slab to be present. • Provide method statements for the contractors to follow • Investigate the ground, now completed. • Record and monitor the external properties. This is completed by a condition survey on under the Party Wall Act before and after the works are completed. See end of method statement. • Allow for unforeseen ground conditions: Loose ground is always a concern. The method statement and drawings show the use of precast lintels to areas of soft ground; this follows the guidance by the underpinning association.

With the above the maximum level of cracking anticipated is Hairline cracking which can be repaired with decorative cracking and can be repaired with decorative repairs. Under the party wall Act damage is allowed (although unwanted) to occur to a neighbouring property as long as repairs are suitability undertaken to rectify this. To mitigate this risk The Party Wall Act is to be followed and a Party Wall Surveyor will be appointed.

Extract from The Institution of Structural Engineers "Subsidence of Low-Rise Buildings"

Table 6.2 Classification of visible damage to walls with particular reference to type of repair, and rectification consideration

Category of Damage	Approximate crack width	Definitions of cracks and repair types/considerations
0	**Up to 0.1**	**HAIRLINE – Internally cracks can be filled or covered by wall covering, and redecorated. Externally, cracks rarely visible and remedial works rarely justified.**
1	0.2 to 2	FINE – Internally cracks can be filled or covered by wall covering, and redecorated. Externally, cracks may be visible, sometimes repairs required for weather tightness or aesthetics. NOTE: Plaster cracks may, in time, become visible again if not covered by a wall covering.
2	2 to 5	MODERATE – Internal cracks are likely to need raking out and repairing to a recognised specification. May need to be chopped back, and repaired with expanded metal/plaster, then redecorated. The crack will inevitably become visible again in time if these measures are not carried out. External cracks will require raking out and repointing, cracked bricks may require replacement.
3	5 to 15	SERIOUS – Internal cracks repaired as for MODERATE, plus perhaps reconstruction if seriously cracked. Rebonding will be required. External cracks may require reconstruction perhaps of panels of brickwork. Alternatively, specialist resin bonding techniques may need to be employed and/or joint reinforcement.
4	15 to 25	SEVERE Major reconstruction works to both internal and external wall skins are likely to be required. Realignment of windows and doors may be necessary.
5	Greater than 25	VERY SEVERE –Major reconstruction works, plus possibly structural lifting or sectional demolition

		and rebuild may need to be considered. Replacement of windows and doors, plus other structural elements, possibly necessary. NOTE – Building & CDM Regulations will probably apply to this category of work, see sections 10.4, 10.6 and Appendix F.

Monitoring and Predicted Category of Damage

Monitoring - In order to safeguard the existing structures during underpinning and new basement construction movement monitoring is to be undertaken. Surveying studs are to be attached to the adjacent structures at ground, first, seconds, third, fourth & fifth floor levels at front and rear.

The surveying points on the adjacent structures are to be set up using an EDM prior to commencement of the works and to be read daily and reported against the following control values.

Limits on ground and adjacent structures movement during underpinning and throughout the construction works.

Movement of survey points must not exceed:

Settlement:
Action values: 5mm (stop work)
Trigger values: 65% of action values (submit proposals for ensuring action values are not exceeded)

Lateral displacement:
Action values: 6mm (stop work)
Trigger values: 65% of action values (submit proposals for ensuring action values are not exceeded)

Movement approaching critical values:
Trigger: Submit proposals for ensuring action values are not exceeded
Action: Stop work

The reporting format will be in the form of a table as attached.

Predicted Category of Damage

The predicted category of damage is likely to be within BRE Category Slight, with possible localised crack widths 2mm to 5mm Classification Aesthetic.

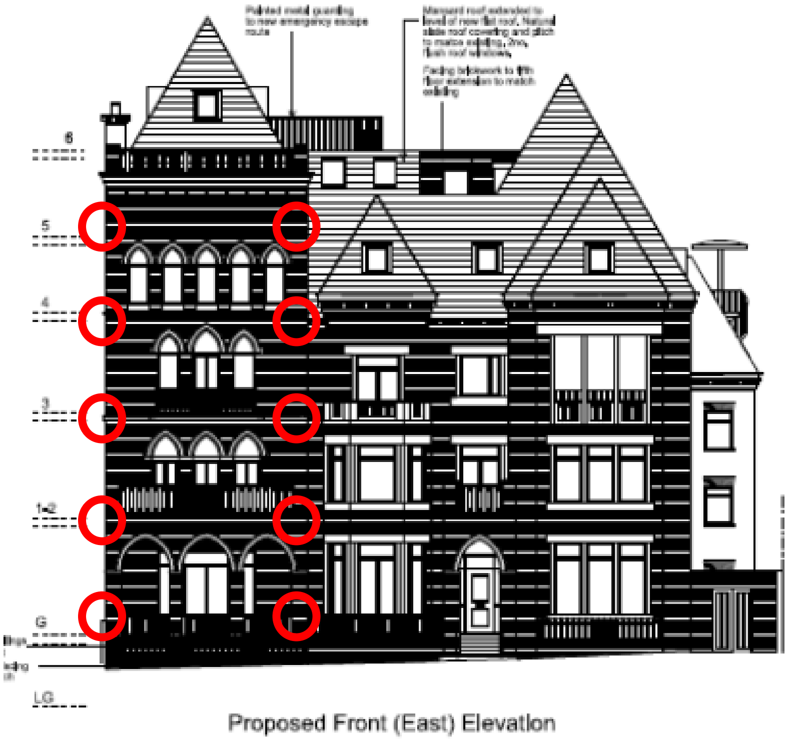


Figure 12: Monitoring Points to the front elevation

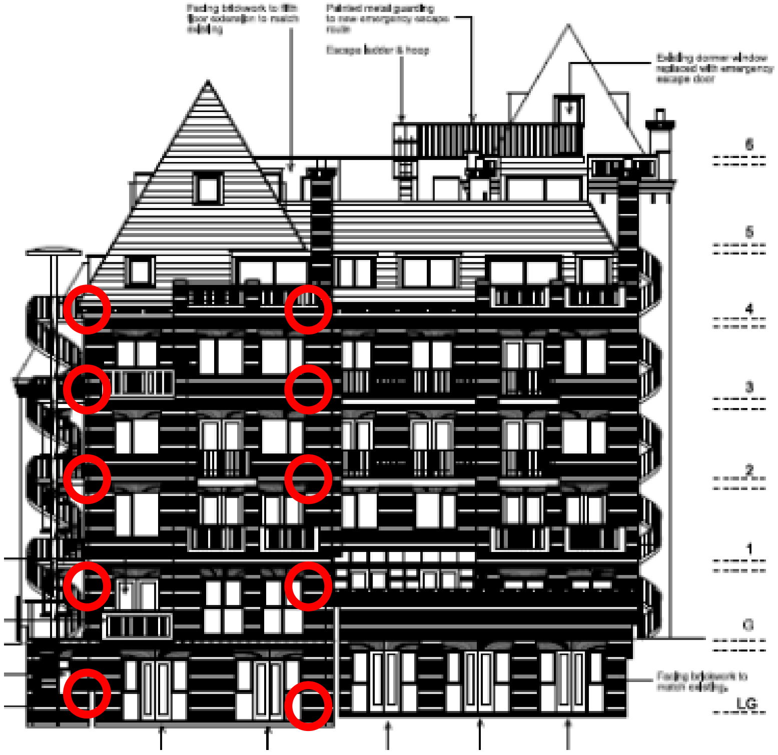


Figure 13: Monitoring Points to the Front Elevation

Drainage and Damp proofing	Assumed that drainage and damp proofing is by others: Details are not provided within our brief. Our recommendation is that drained cavity systems are used to habitable basements with pumped sumps. This is a specialist contractor design item. Concrete is not designed BS 8007. But where possible BS 8007 detailing is observed to help limit crack widths of concrete

Party Wall	Underpinning basement works has a risk associated to it. To mitigate these risks a Party wall surveyor must be appointed

Temporary Works	Temporary works are the contractor's responsibility. Loads can be provided on request. Foundations; All trenches deeper than 1.0m must be shored. Where works undermine existing foundations contractor must allow for additional support. The Method statement lays out the process for constructing the basement

Noise and Nuisance	The contractor is to follow the good working practices and guidance laid down in the "Considerate Constructors Scheme". The hours of working will be limited to those allowed; 8am to 5pm Monday to Friday and Saturday Morning 8am to 1pm. None of the practices cause undue noise that one would typically expect from a construction site. The conveyor belt typically runs at around 70dB. The site has car parking to the front to which the skip will be stored. The site will be hoarded with soil 8' site hoarding to prevent access. The hours of working will further be defined within the Party Wall Act. The site is to be hoarded to minimise the level of direct noise from the

	site. Ground floor slab is not being removed minimising the vibration and sound to adjacent properties. While working in the basement the work generally requires hand tools to be used. The level of noise generally will be no greater than that of digging of soil. The noise is reduced and muffled by the works being undertaken underground. A level of noise from a basement is lower than typical ground level construction due to this.

7. OS Map extract showing location of Railway

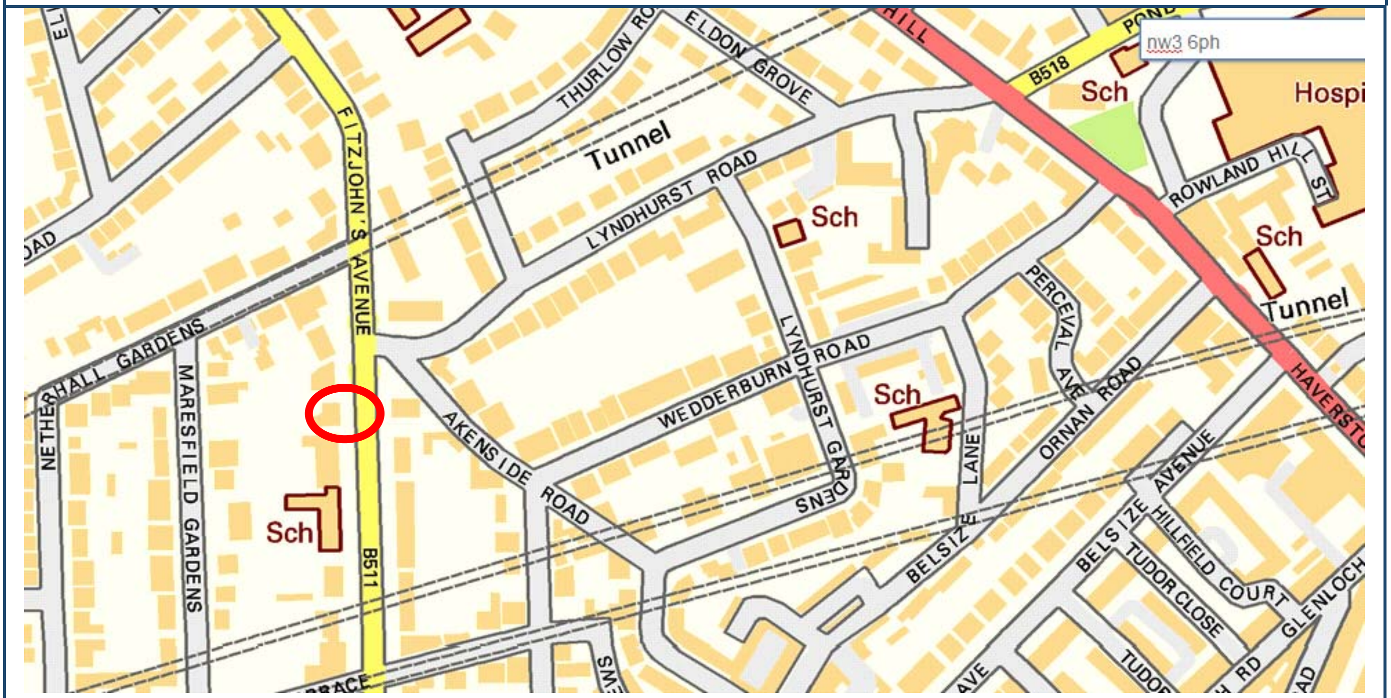


Figure 14: 51 Fitzjohn's Ave - Railway tunnel location

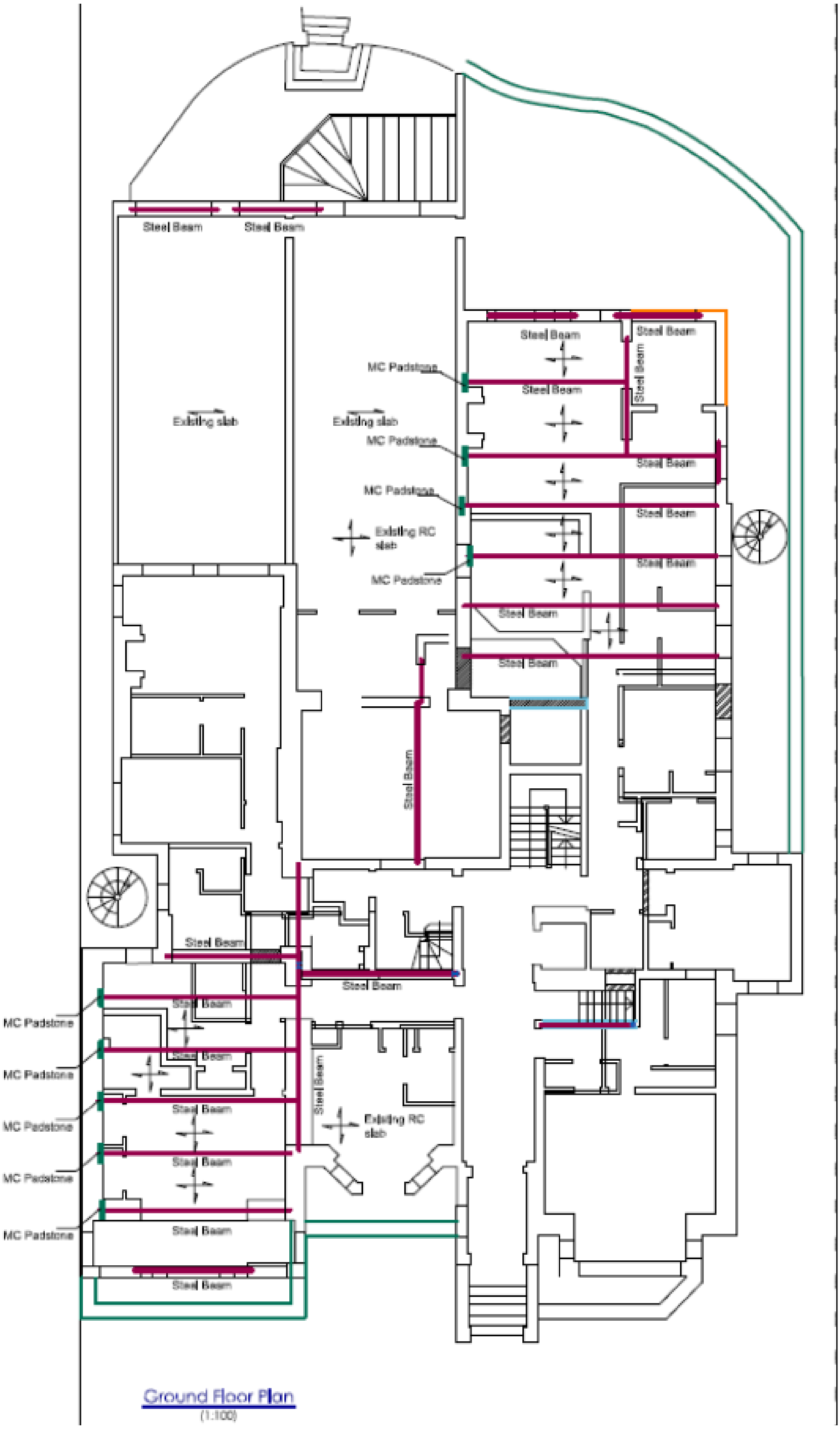
Location of 51 Fitzjohn's Avenue shown on the OS map with the location of the Network Rail Tunnels. The closest tunnel lies approximately 100m away from the site.

Appendix A

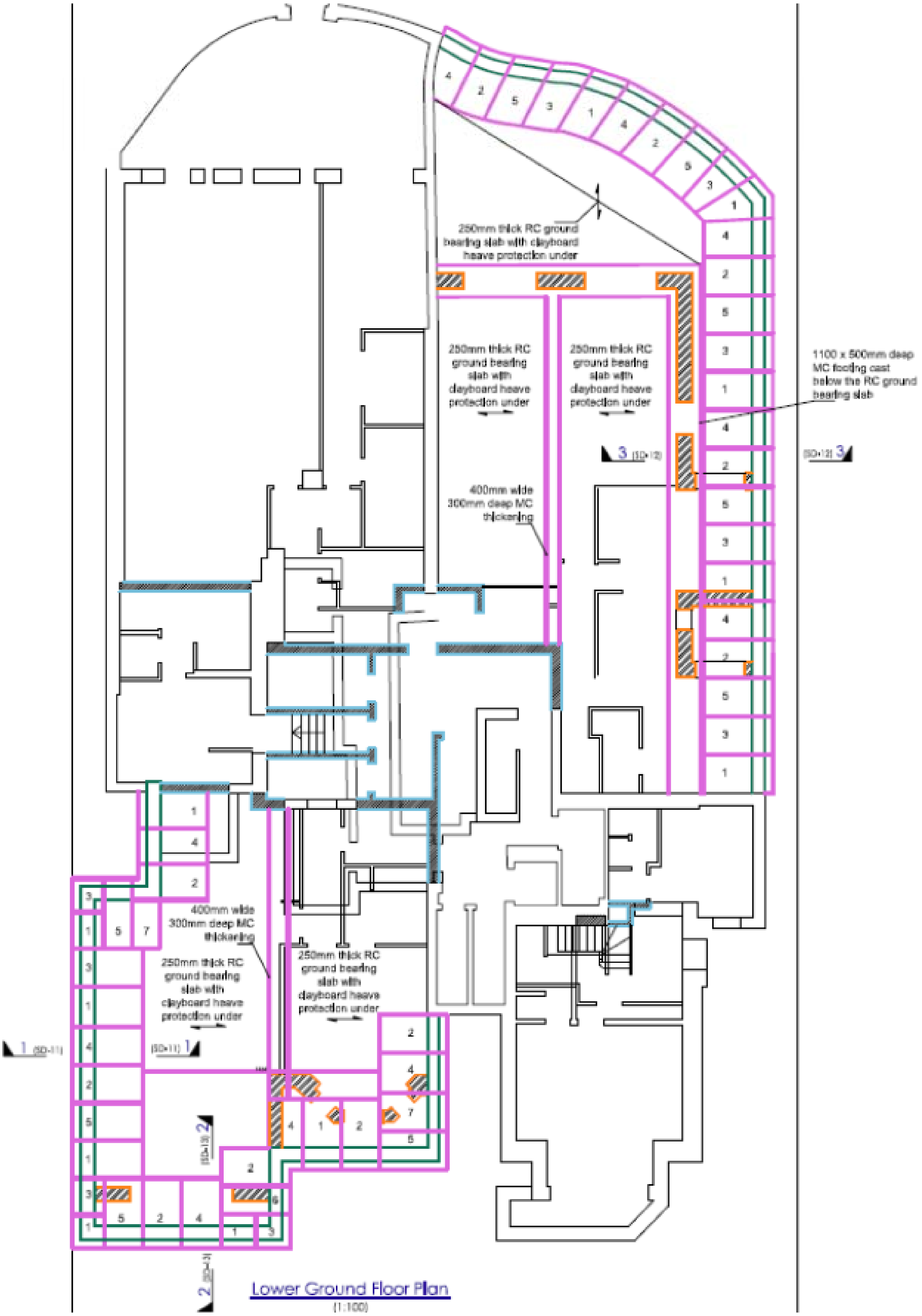
Structural Scheme Drawings

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

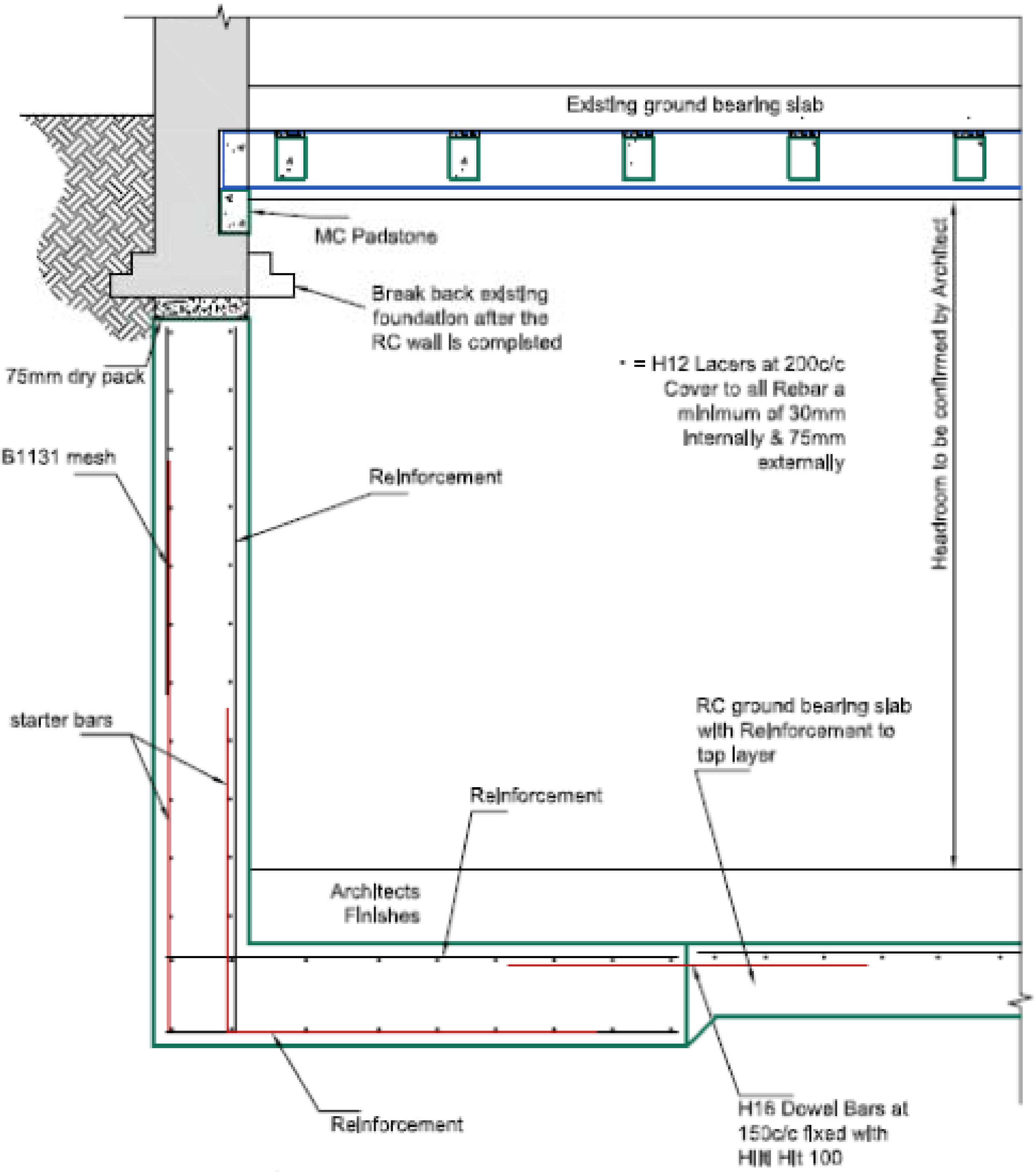
Ground Floor Plan



Basement Plan

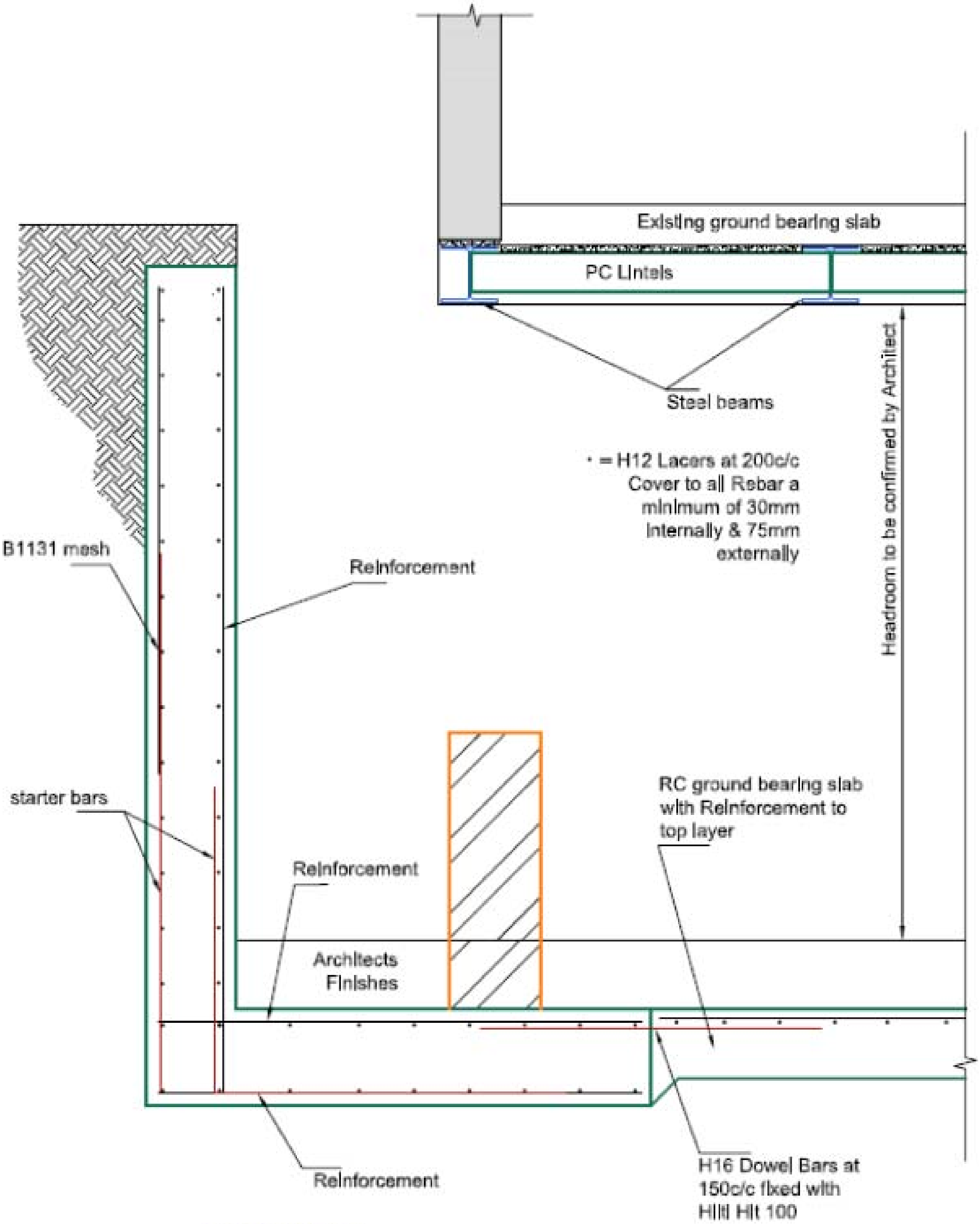


Section 1-1



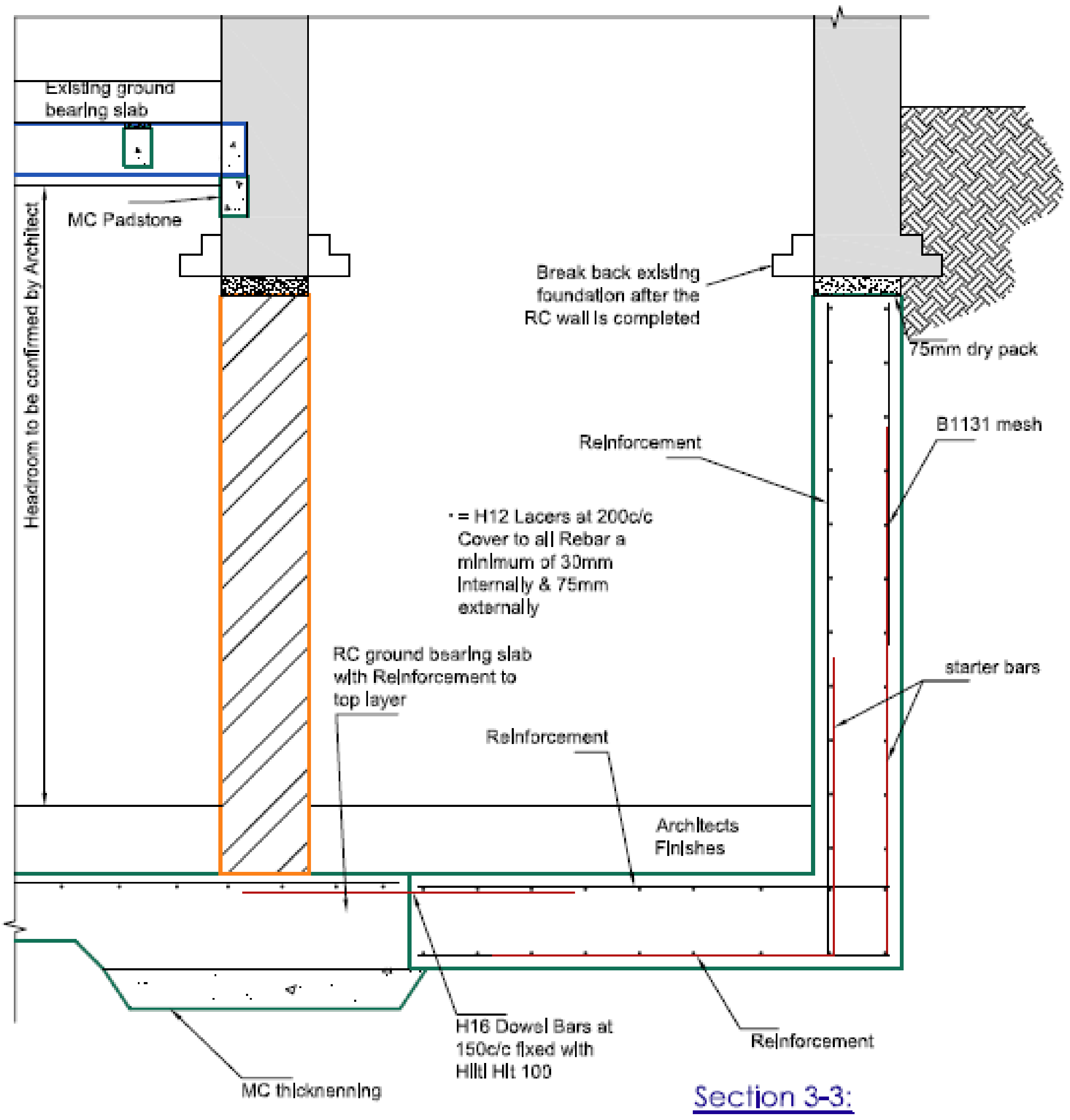
Section 1-1:
Typical RC retaining wall section
scale 1:20

Section 2-2



Section 2-2:
Typical RC retaining wall section
scale 1:20

Section 3-3



Section 3-3:
Typical RC retaining wall section
scale 1:20

Appendix B

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

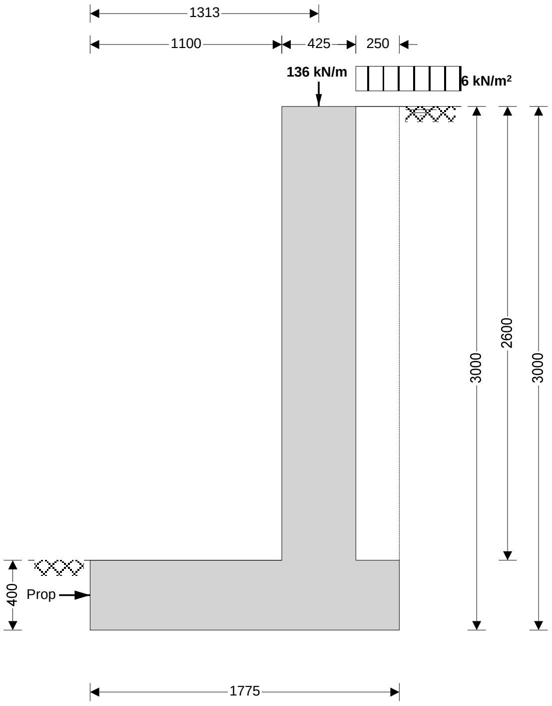
Loading:

325mm thick masonry wall	DL325 = 7kN/m^2 × 5.3m = **37.100**kN/m
225mm thick masonry wall	DL225 = 5kN/m^2 × 10m = **50.000**kN/m
Floor DL (1st, 2nd, 3rd , 4th, 5th floors)	DLfloor = 0.7kN/m^2 × 5.5m / 2 × 5 = **9.625**kN/m
Ground floor	DLground = 24kN/m^3 × 0.2m × 5.5m / 2 = **13.200**kN/m
Roof DL	DLroof = 1.1kN/m^2 × 3.3m = **3.630**kN/m
Total Dead Load	DL = DL325+DL225+DLfloor+DLground+DLroof=**113.555**kN/m

Floor LL (1st, 2nd, 3rd , 4th, 5th floors)	LLfloor = 1.5kN/m^2 × 5.5m / 2 × 5 = **20.625**kN/m
Roof LL	LLroof = 0.6kN/m^2 × 3.3m = **1.980**kN/m
Total Live Load	LL = LLfloor + LLroof = **22.605**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} = **2600** mm
Thickness of wall stem	t_{wall} = **425** mm
Length of toe	l_{toe} = **1100** mm
Length of heel	l_{heel} = **250** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1775** mm
Thickness of base	t_{base} = **400** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **1225** mm
Thickness of downstand	t_{ds} = **400** mm

Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds} = \mathbf{3000}$ mm
Depth of cover in front of wall	$d_{cover} = \mathbf{0}$ mm
Depth of unplanned excavation	$d_{exc} = \mathbf{0}$ mm
Height of ground water behind wall	$h_{water} = \mathbf{3000}$ mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0 \text{ mm}) = \mathbf{2600}$ mm
Density of wall construction	$\gamma_{wall} = \mathbf{23.6}$ kN/m^3
Density of base construction	$\gamma_{base} = \mathbf{23.6}$ kN/m^3
Angle of rear face of wall	$\alpha = \mathbf{90.0}$ deg
Angle of soil surface behind wall	$\beta = \mathbf{0.0}$ deg
Effective height at virtual back of wall	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta) = \mathbf{3000}$ mm

Retained material details

Mobilisation factor	$M = \mathbf{1.5}$
Moist density of retained material	$\gamma_m = \mathbf{18.0}$ kN/m^3
Saturated density of retained material	$\gamma_s = \mathbf{21.0}$ kN/m^3
Design shear strength	$\phi' = \mathbf{24.2}$ deg
Angle of wall friction	$\delta = \mathbf{0.0}$ deg

Base material details

Moist density	$\gamma_{mb} = \mathbf{18.0}$ kN/m^3
Design shear strength	$\phi'_b = \mathbf{24.2}$ deg
Design base friction	$\delta_b = \mathbf{18.6}$ deg
Allowable bearing pressure	$P_{bearing} = \mathbf{125}$ kN/m^2

Using Coulomb theory

Active pressure coefficient for retained material

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

Passive pressure coefficient for base material

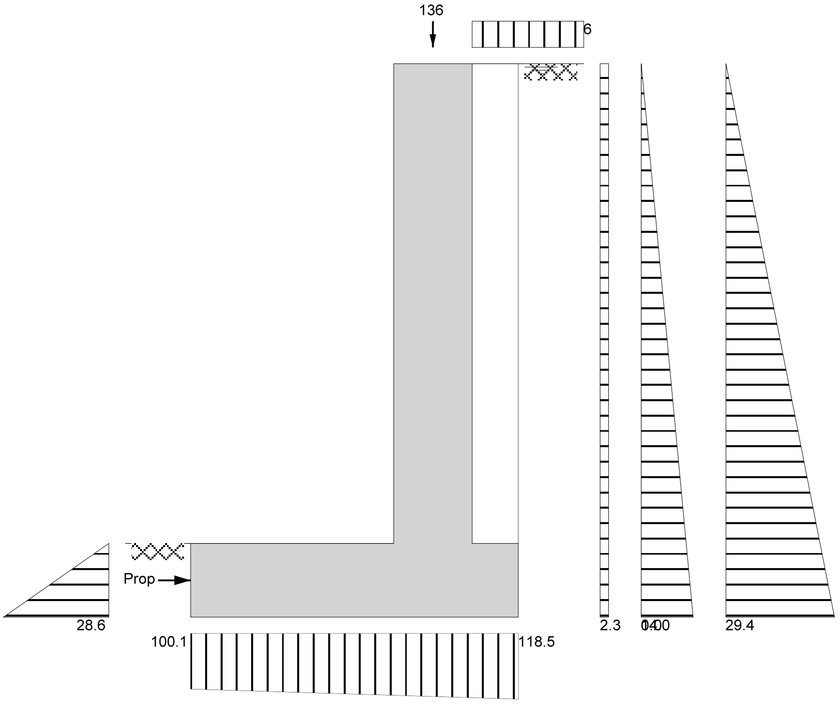
$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \surd(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))]^2) = \mathbf{4.187}$$

At-rest pressure

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

Loading details

Surcharge load on plan	Surcharge = $\mathbf{5.5}$ kN/m^2
Applied vertical dead load on wall	$W_{dead} = \mathbf{113.6}$ kN/m
Applied vertical live load on wall	$W_{live} = \mathbf{22.6}$ kN/m
Position of applied vertical load on wall	$l_{load} = \mathbf{1313}$ mm
Applied horizontal dead load on wall	$F_{dead} = \mathbf{0.0}$ kN/m
Applied horizontal live load on wall	$F_{live} = \mathbf{0.0}$ kN/m
Height of applied horizontal load on wall	$h_{load} = \mathbf{0}$ mm



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

Vertical forces on wall

Wall stem $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall} = \mathbf{26.1}$ kN/m

Wall base $w_{base} = l_{base} \times t_{base} \times \gamma_{base} = \mathbf{16.8}$ kN/m

Surcharge $w_{sur} = \text{Surcharge} \times l_{heel} = \mathbf{1.4}$ kN/m

Saturated backfill $w_s = l_{heel} \times h_{sat} \times \gamma_s = \mathbf{13.7}$ kN/m

Applied vertical load $W_v = W_{dead} + W_{live} = \mathbf{136.2}$ kN/m

Total vertical load $W_{total} = w_{wall} + w_{base} + w_{sur} + w_s + W_v = \mathbf{194}$ kN/m

Horizontal forces on wall

Surcharge $F_{sur} = K_a \times \text{Surcharge} \times h_{eff} = \mathbf{6.9}$ kN/m

Saturated backfill $F_s = 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2 = \mathbf{21.1}$ kN/m

Water $F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water} = \mathbf{44.1}$ kN/m

Total horizontal load $F_{total} = F_{sur} + F_s + F_{water} = \mathbf{72.1}$ kN/m

Calculate propping force

Passive resistance of soil in front of wall $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb} = \mathbf{5.7}$ kN/m

Propping force $F_{prop} = \max(F_{total} - F_p - (W_{total} - w_{sur} - W_{live}) \times \tan(\delta_b), 0 \text{ kN/m})$

$F_{prop} = \mathbf{9.2}$ kN/m

Overturning moments

Surcharge $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2 = \mathbf{10.4}$ kNm/m

Saturated backfill $M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3 = \mathbf{21.1}$ kNm/m

Water $M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3 = \mathbf{44.1}$ kNm/m

Total overturning moment $M_{ot} = M_{sur} + M_s + M_{water} = \mathbf{75.6}$ kNm/m

Restoring moments

Wall stem $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2) = \mathbf{34.2}$ kNm/m

Wall base $M_{base} = w_{base} \times l_{base} / 2 = \mathbf{14.9}$ kNm/m

Saturated backfill $M_{s\_r} = w_s \times (l_{base} - l_{heel} / 2) = \mathbf{22.5}$ kNm/m

Design vertical dead load $M_{dead} = W_{dead} \times l_{load} = \mathbf{149}$ kNm/m

Total restoring moment $M_{rest} = M_{wall} + M_{base} + M_{s\_r} + M_{dead} = \mathbf{220.7}$ kNm/m

Check bearing pressure

Surcharge	$M_{sur\_r} = w_{sur} \times (l_{base} - l_{heel} / 2) =$ **2.3** kNm/m
Design vertical live load	$M_{live} = W_{live} \times l_{load} =$ **29.7** kNm/m
Total moment for bearing	$M_{total} = M_{rest} - M_{ot} + M_{sur\_r} + M_{live} =$ **177** kNm/m
Total vertical reaction	$R = W_{total} =$ **194.0** kN/m
Distance to reaction	$x_{bar} = M_{total} / R =$ **912** mm
Eccentricity of reaction	$e = abs((l_{base} / 2) - x_{bar}) =$ **25** mm
	Reaction acts within middle third of base
Bearing pressure at toe	$p_{toe} = (R / l_{base}) - (6 \times R \times e / l_{base}^2) =$ **100.1** kN/m^2
Bearing pressure at heel	$p_{heel} = (R / l_{base}) + (6 \times R \times e / l_{base}^2) =$ **118.5** 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	$w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **36.5** kN/m
Wall base	$w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **23.5** kN/m
Surcharge	$w_{sur\_f} = \gamma_{f\_l} \times \text{Surcharge} \times l_{heel}$ = **2.2** kN/m
Saturated backfill	$w_{s\_f} = \gamma_{f\_d} \times l_{heel} \times h_{sat} \times \gamma_{s}$ = **19.1** kN/m
Applied vertical load	$W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **195.1** kN/m
Total vertical load	$W_{total\_f} = w_{wall\_f} + w_{base\_f} + w_{sur\_f} + w_{s\_f} + W_{v\_f}$ = **276.4** kN/m

Factored horizontal at-rest forces on wall

Surcharge	$F_{sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times h_{eff}$ = **15.6** kN/m
Saturated backfill	$F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times (\gamma_{s} - \gamma_{water}) \times h_{water}^2$ = **41.6** kN/m
Water	$F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water}$ = **61.8** kN/m
Total horizontal load	$F_{total\_f} = F_{sur\_f} + F_{s\_f} + F_{water\_f}$ = **119** kN/m

Calculate propping force

Passive resistance of soil in front of wall	$F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **8** 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} = **30.9** kN/m

Factored overturning moments

Surcharge	$M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **23.4** kNm/m
Saturated backfill	$M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **41.6** kNm/m
Water	$M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **61.8** kNm/m
Total overturning moment	$M_{ot\_f} = M_{sur\_f} + M_{s\_f} + M_{water\_f}$ = **126.8** kNm/m

Restoring moments

Wall stem	$M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **47.9** kNm/m
Wall base	$M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **20.8** kNm/m
Surcharge	$M_{sur\_r\_f} = w_{sur\_f} \times (l_{base} - l_{heel} / 2)$ = **3.6** kNm/m
Saturated backfill	$M_{s\_r\_f} = w_{s\_f} \times (l_{base} - l_{heel} / 2)$ = **31.5** kNm/m
Design vertical load	$M_{v\_f} = W_{v\_f} \times l_{load}$ = **256.1** kNm/m
Total restoring moment	$M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{sur\_r\_f} + M_{s\_r\_f} + M_{v\_f}$ = **360** kNm/m

Factored bearing pressure

Total moment for bearing	$M_{total\_f} = M_{rest\_f} - M_{ot\_f}$ = **233.3** kNm/m
Total vertical reaction	$R_f = W_{total\_f}$ = **276.4** kN/m
Distance to reaction	$x_{bar\_f} = M_{total\_f} / R_f$ = **844** mm
Eccentricity of reaction	$e_f = \text{abs}((l_{base} / 2) - x_{bar\_f})$ = **44** mm
	Reaction acts within middle third of base
Bearing pressure at toe	$p_{toe\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2)$ = **178.7** kN/m^2
Bearing pressure at heel	$p_{heel\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2)$ = **132.8** kN/m^2
Rate of change of base reaction	rate = $(p_{toe\_f} - p_{heel\_f}) / l_{base}$ = **25.89** kN/m^2/m
Bearing pressure at stem / toe	$p_{stem\_toe\_f} = \max(p_{toe\_f} - (\text{rate} \times l_{toe}), 0$ kN/m^2) = **150.2** kN/m^2
Bearing pressure at mid stem	$p_{stem\_mid\_f} = \max(p_{toe\_f} - (\text{rate} \times (l_{toe} + t_{wall} / 2)), 0$ kN/m^2) = **144.7** kN/m^2

Bearing pressure at stem / heel $p_{stem\_heel\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall})), 0 \text{ kN/m}^2)$ = **139.2** 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} = **75** mm

Calculate shear for toe design

Shear from bearing pressure $V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2$ = **180.9** kN/m

Shear from weight of base $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base}$ = **14.5** kN/m

Total shear for toe design $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base}$ = **166.4** kN/m

Calculate moment for toe design

Moment from bearing pressure $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6$ = **144.2** kNm/m

Moment from weight of base $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2)$ = **11.4** kNm/m

Total moment for toe design $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base}$ = **132.8** kNm/m

Check toe in bending

Width of toe b = **1000** mm/m

Depth of reinforcement $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2)$ = **319.0** mm

Constant $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu})$ = **0.037**

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} = **303** mm

Area of tension reinforcement required $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe})$ = **1007** mm^2/m

Minimum area of tension reinforcement $A_{s\_toe\_min} = k \times b \times t_{base}$ = **520** mm^2/m

Area of tension reinforcement required $A_{s\_toe\_req} = \text{Max}(A_{s\_toe\_des}, A_{s\_toe\_min})$ = **1007** mm^2/m

Reinforcement provided **B1131 mesh**

Area of reinforcement provided $A_{s\_toe\_prov}$ = **1131** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress $v_{toe} = V_{toe} / (b \times d_{toe})$ = **0.522** 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} = **0.530** 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} = **75** mm

Calculate shear for heel design

Shear from bearing pressure $V_{\text{heel\_bear}} = (p_{\text{heel\_f}} + p_{\text{stem\_heel\_f}}) \times l_{\text{heel}} / 2 = \mathbf{34}$ kN/m

Shear from weight of base $V_{\text{heel\_wt\_base}} = \gamma_{\text{f\_d}} \times \gamma_{\text{base}} \times l_{\text{heel}} \times t_{\text{base}} = \mathbf{3.3}$ kN/m

Shear from weight of saturated backfill $V_{\text{heel\_wt\_s}} = w_{\text{s\_f}} = \mathbf{19.1}$ kN/m

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

Total shear for heel design $V_{\text{heel}} = - V_{\text{heel\_bear}} + V_{\text{heel\_wt\_base}} + V_{\text{heel\_wt\_s}} + V_{\text{heel\_sur}} = \mathbf{-9.4}$ kN/m

Calculate moment for heel design

Moment from bearing pressure $M_{\text{heel\_bear}} = (2 \times p_{\text{heel\_f}} + p_{\text{stem\_mid\_f}}) \times (l_{\text{heel}} + t_{\text{wall}} / 2)^2 / 6 = \mathbf{14.6}$ kNm/m

Moment from weight of base $M_{\text{heel\_wt\_base}} = (\gamma_{\text{f\_d}} \times \gamma_{\text{base}} \times t_{\text{base}} \times (l_{\text{heel}} + t_{\text{wall}} / 2)^2 / 2) = \mathbf{1.4}$ kNm/m

Moment from weight of saturated backfill $M_{\text{heel\_wt\_s}} = w_{\text{s\_f}} \times (l_{\text{heel}} + t_{\text{wall}}) / 2 = \mathbf{6.4}$ kNm/m

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

Total moment for heel design $M_{\text{heel}} = - M_{\text{heel\_bear}} + M_{\text{heel\_wt\_base}} + M_{\text{heel\_wt\_s}} + M_{\text{heel\_sur}} = \mathbf{-6}$ kNm/m

As the moment is negative the design of the retaining wall heel is beyond the scope of this calculation

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

Material properties

Characteristic strength of concrete $f_{cu} = \mathbf{35}$ N/mm^2

Characteristic strength of reinforcement $f_y = \mathbf{500}$ N/mm^2

Wall details

Minimum area of reinforcement $k = \mathbf{0.13}$ %

Cover to reinforcement in stem $c_{\text{stem}} = \mathbf{75}$ mm

Cover to reinforcement in wall $c_{\text{wall}} = \mathbf{30}$ mm

Factored horizontal at-rest forces on stem

Surcharge $F_{\text{s\_sur\_f}} = \gamma_{\text{f\_l}} \times K_0 \times \text{Surcharge} \times (h_{\text{eff}} - t_{\text{base}} - d_{\text{ds}}) = \mathbf{13.5}$ kN/m

Saturated backfill $F_{\text{s\_s\_f}} = 0.5 \times \gamma_{\text{f\_e}} \times K_0 \times (\gamma_s - \gamma_{\text{water}}) \times h_{\text{sat}}^2 = \mathbf{31.2}$ kN/m

Water $F_{\text{s\_water\_f}} = 0.5 \times \gamma_{\text{f\_e}} \times \gamma_{\text{water}} \times h_{\text{sat}}^2 = \mathbf{46.4}$ kN/m

Calculate shear for stem design

Shear at base of stem $V_{\text{stem}} = F_{\text{s\_sur\_f}} + F_{\text{s\_s\_f}} + F_{\text{s\_water\_f}} - F_{\text{prop\_f}} = \mathbf{60.3}$ kN/m

Calculate moment for stem design

Surcharge $M_{\text{s\_sur}} = F_{\text{s\_sur\_f}} \times (h_{\text{stem}} + t_{\text{base}}) / 2 = \mathbf{20.3}$ kNm/m

Saturated backfill $M_{\text{s\_s}} = F_{\text{s\_s\_f}} \times h_{\text{sat}} / 3 = \mathbf{27.1}$ kNm/m

Water $M_{\text{s\_water}} = F_{\text{s\_water\_f}} \times h_{\text{sat}} / 3 = \mathbf{40.2}$ kNm/m

Total moment for stem design $M_{\text{stem}} = M_{\text{s\_sur}} + M_{\text{s\_s}} + M_{\text{s\_water}} = \mathbf{87.6}$ kNm/m

Check wall stem in bending

Width of wall stem $b = \mathbf{1000}$ mm/m

Depth of reinforcement $d_{\text{stem}} = t_{\text{wall}} - c_{\text{stem}} - (\phi_{\text{stem}} / 2) = \mathbf{342.0}$ mm

Constant $K_{\text{stem}} = M_{\text{stem}} / (b \times d_{\text{stem}}^2 \times f_{cu}) = \mathbf{0.021}$

Compression reinforcement is not required

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

$z_{\text{stem}} = \mathbf{325}$ mm

Area of tension reinforcement required $A_{\text{s\_stem\_des}} = M_{\text{stem}} / (0.87 \times f_y \times z_{\text{stem}}) = \mathbf{620}$ mm^2/m

Minimum area of tension reinforcement $A_{\text{s\_stem\_min}} = k \times b \times t_{\text{wall}} = \mathbf{553}$ mm^2/m

Area of tension reinforcement required $A_{\text{s\_stem\_req}} = \text{Max}(A_{\text{s\_stem\_des}}, A_{\text{s\_stem\_min}}) = \mathbf{620}$ mm^2/m

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

Area of reinforcement provided $A_{\text{s\_stem\_prov}} = \mathbf{1340}$ mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress $v_{stem} = V_{stem} / (b \times d_{stem}) = \mathbf{0.176}$ N/mm^2

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

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

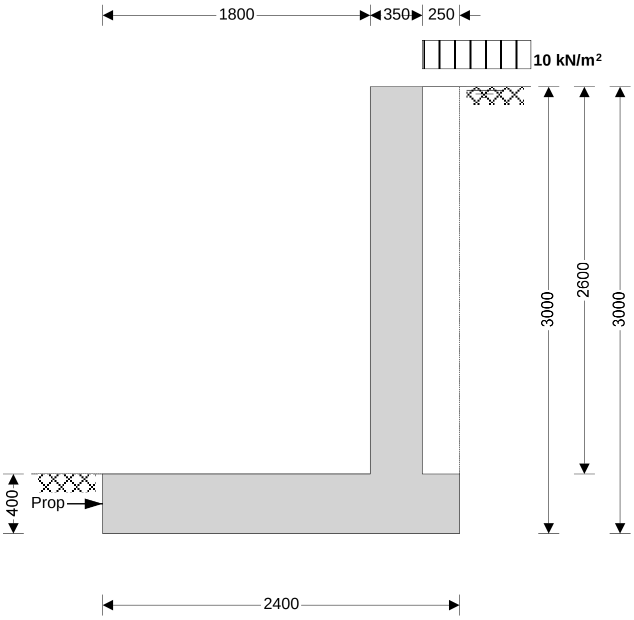
Design concrete shear stress $v_{c\_stem} = \mathbf{0.538}$ N/mm^2

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

RC retaining wall design 2

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} = **2600** mm
Thickness of wall stem	t_{wall} = **350** mm
Length of toe	l_{toe} = **1800** mm
Length of heel	l_{heel} = **250** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **2400** mm
Thickness of base	t_{base} = **400** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **1700** mm
Thickness of downstand	t_{ds} = **400** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **3000** 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} = **3000** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0\text{ mm})$ = **2600** 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	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **3000** 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 $P_{bearing}$ = **125** 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

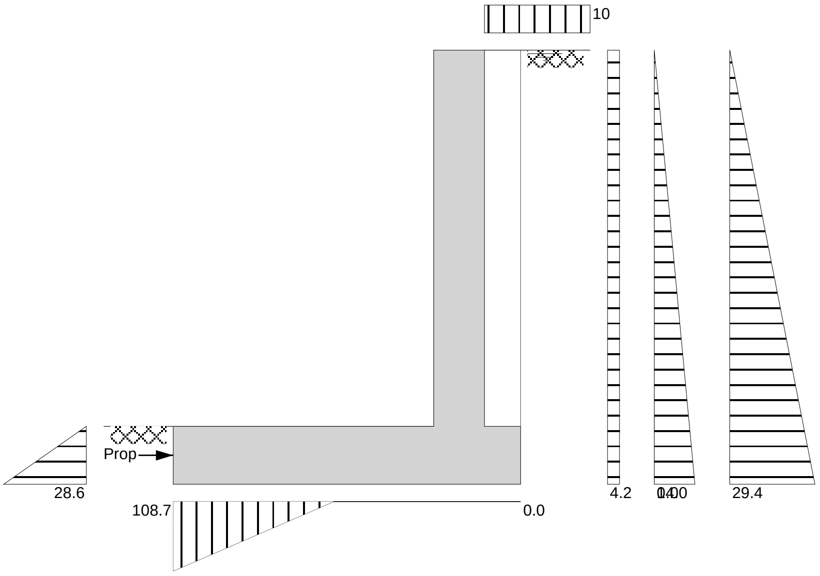
Applied vertical live load on wall W_{live} = **0.0** kN/m

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 $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **21.5** kN/m

Wall base $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **22.7** kN/m

Surcharge w_{sur} = Surcharge $\times\ l_{heel}$ = **2.5** kN/m

Saturated backfill $w_s = l_{heel} \times h_{sat} \times \gamma_s$ = **13.7** kN/m

Total vertical load $W_{total} = w_{wall} + w_{base} + w_{sur} + w_s$ = **60.3** kN/m

Horizontal forces on wall

Surcharge $F_{sur} = K_a \times$ Surcharge $\times\ h_{eff}$ = **12.6** kN/m

Saturated backfill $F_s = 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **21.1** kN/m

Water $F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **44.1** kN/m

Total horizontal load	$F_{total} = F_{sur} + F_s + F_{water} =$ **77.8** kN/m

Calculate propping force

Passive resistance of soil in front of wall	$F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb} =$ **5.7** kN/m
Propping force	$F_{prop} = \max(F_{total} - F_p - (W_{total} - w_{sur}) \times \tan(\delta_b), 0\text{ kN/m})$ $F_{prop} =$ **52.6** kN/m

Overturning moments

Surcharge	$M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2 =$ **18.8** kNm/m
Saturated backfill	$M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3 =$ **21.1** kNm/m
Water	$M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3 =$ **44.1** kNm/m
Total overturning moment	$M_{ot} = M_{sur} + M_s + M_{water} =$ **84.1** kNm/m

Restoring moments

Wall stem	$M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2) =$ **42.4** kNm/m
Wall base	$M_{base} = w_{base} \times l_{base} / 2 =$ **27.2** kNm/m
Saturated backfill	$M_{s\_r} = w_s \times (l_{base} - l_{heel} / 2) =$ **31.1** kNm/m
Total restoring moment	$M_{rest} = M_{wall} + M_{base} + M_{s\_r} =$ **100.7** kNm/m

Check bearing pressure

Surcharge	$M_{sur\_r} = w_{sur} \times (l_{base} - l_{heel} / 2) =$ **5.7** kNm/m
Total moment for bearing	$M_{total} = M_{rest} - M_{ot} + M_{sur\_r} =$ **22.3** kNm/m
Total vertical reaction	$R = W_{total} =$ **60.3** kN/m
Distance to reaction	$x_{bar} = M_{total} / R =$ **370** mm
Eccentricity of reaction	$e = \text{abs}((l_{base} / 2) - x_{bar}) =$ **830** mm
	Reaction acts outside middle third of base
Bearing pressure at toe	$p_{toe} = R / (1.5 \times x_{bar}) =$ **108.7** 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 $\gamma_{f\_d} = \mathbf{1.4}$

Live load factor $\gamma_{f\_l} = \mathbf{1.6}$

Earth and water pressure factor $\gamma_{f\_e} = \mathbf{1.4}$

Factored vertical forces on wall

Wall stem $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall} = \mathbf{30.1}\text{ kN/m}$

Wall base $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base} = \mathbf{31.7}\text{ kN/m}$

Surcharge $w_{sur\_f} = \gamma_{f\_l} \times \text{Surcharge} \times l_{heel} = \mathbf{4}\text{ kN/m}$

Saturated backfill $w_{s\_f} = \gamma_{f\_d} \times l_{heel} \times h_{sat} \times \gamma_{s} = \mathbf{19.1}\text{ kN/m}$

Total vertical load $W_{total\_f} = w_{wall\_f} + w_{base\_f} + w_{sur\_f} + w_{s\_f} = \mathbf{84.9}\text{ kN/m}$

Factored horizontal at-rest forces on wall

Surcharge $F_{sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times h_{eff} = \mathbf{28.3}\text{ kN/m}$

Saturated backfill $F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times (\gamma_{s} - \gamma_{water}) \times h_{water}^2 = \mathbf{41.6}\text{ kN/m}$

Water $F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water} = \mathbf{61.8}\text{ kN/m}$

Total horizontal load $F_{total\_f} = F_{sur\_f} + F_{s\_f} + F_{water\_f} = \mathbf{131.7}\text{ kN/m}$

Calculate propping force

Passive resistance of soil in front of wall $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb} = \mathbf{8}\text{ kN/m}$

Propping force $F_{prop\_f} = \max(F_{total\_f} - F_{p\_f} - (W_{total\_f} - w_{sur\_f}) \times \tan(\delta_b), 0\text{ kN/m})$

$F_{prop\_f} = \mathbf{96.5}\text{ kN/m}$

Factored overturning moments

Surcharge $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2 = \mathbf{42.5}\text{ kNm/m}$

Saturated backfill $M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3 = \mathbf{41.6}\text{ kNm/m}$

Water $M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3 = \mathbf{61.8}\text{ kNm/m}$

Total overturning moment $M_{ot\_f} = M_{sur\_f} + M_{s\_f} + M_{water\_f} = \mathbf{145.9}\text{ kNm/m}$

Restoring moments

Wall stem $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2) = \mathbf{59.4}\text{ kNm/m}$

Wall base $M_{base\_f} = w_{base\_f} \times l_{base} / 2 = \mathbf{38.1}\text{ kNm/m}$

Surcharge $M_{sur\_r\_f} = w_{sur\_f} \times (l_{base} - l_{heel} / 2) = \mathbf{9.1}\text{ kNm/m}$

Saturated backfill $M_{s\_r\_f} = w_{s\_f} \times (l_{base} - l_{heel} / 2) = \mathbf{43.5}\text{ kNm/m}$

Total restoring moment $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{sur\_r\_f} + M_{s\_r\_f} = \mathbf{150}\text{ kNm/m}$

Factored bearing pressure

Total moment for bearing $M_{total\_f} = M_{rest\_f} - M_{ot\_f} = \mathbf{4.1}\text{ kNm/m}$

Total vertical reaction $R_f = W_{total\_f} = \mathbf{84.9}\text{ kN/m}$

Distance to reaction $x_{bar\_f} = M_{total\_f} / R_f = \mathbf{49}\text{ mm}$

Eccentricity of reaction $e_f = \text{abs}((l_{base} / 2) - x_{bar\_f}) = \mathbf{1151}\text{ mm}$

Reaction acts outside middle third of base

Bearing pressure at toe $p_{toe\_f} = R_f / (1.5 \times x_{bar\_f}) = \mathbf{1163}\text{ kN/m}^2$

Bearing pressure at heel $p_{heel\_f} = 0\text{ kN/m}^2 = \mathbf{0}\text{ kN/m}^2$

Rate of change of base reaction $\text{rate} = p_{toe\_f} / (3 \times x_{bar\_f}) = \mathbf{7966.42}\text{ kN/m}^2\text{/m}$

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

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

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

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	$c_{toe} =$ **75** mm

Calculate shear for toe design

Shear from bearing pressure	$V_{toe\_bear} = 3 \times p_{toe\_f} \times x_{bar\_f} / 2 =$ **84.9** kN/m
Shear from weight of base	$V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} =$ **23.8** kN/m
Total shear for toe design	$V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} =$ **61.1** kN/m

Calculate moment for toe design

Moment from bearing pressure	$M_{toe\_bear} = 3 \times p_{toe\_f} \times x_{bar\_f} \times (l_{toe} - x_{bar\_f} + t_{wall} / 2) / 2 =$ **163.5** kNm/m
Moment from weight of base	$M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) =$ **25.8** kNm/m
Total moment for toe design	$M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} =$ **137.8** kNm/m

Check toe in bending

Width of toe	b = **1000** mm/m
Depth of reinforcement	$d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) =$ **319.0** mm
Constant	$K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) =$ **0.039**

Compression reinforcement is not required

Lever arm	$z_{toe} = \min(0.5 + \sqrt{(0.25 - (\min(K_{toe}, 0.225) / 0.9))},0.95) \times d_{toe}$
	$z_{toe} =$ **303** mm
Area of tension reinforcement required	$A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) =$ **1045** mm^2/m
Minimum area of tension reinforcement	$A_{s\_toe\_min} = k \times b \times t_{base} =$ **520** mm^2/m
Area of tension reinforcement required	$A_{s\_toe\_req} = \mathrm{Max}(A_{s\_toe\_des}, A_{s\_toe\_min}) =$ **1045** mm^2/m
Reinforcement provided	**B1131 mesh**
Area of reinforcement provided	$A_{s\_toe\_prov} =$ **1131** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress	$v_{toe} = V_{toe} / (b \times d_{toe}) =$ **0.192** 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	$v_{c\_toe} =$ **0.530** 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	$c_{heel} =$ **75** mm

Calculate shear for heel design

Shear from weight of base	$V_{heel\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{heel} \times t_{base} =$ **3.3** kN/m
Shear from weight of saturated backfill	$V_{heel\_wt\_s} = w_{s\_f} =$ **19.1** kN/m
Shear from surcharge	$V_{heel\_sur} = w_{sur\_f} =$ **4** kN/m
Total shear for heel design	$V_{heel} = V_{heel\_wt\_base} + V_{heel\_wt\_s} + V_{heel\_sur} =$ **26.4** kN/m

Calculate moment for heel design

Moment from weight of base	$M_{heel\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{heel} + t_{wall} / 2)^2 / 2)$ = **1.2** kNm/m
Moment from weight of saturated backfill	$M_{heel\_wt\_s} = w_{s\_f} \times (l_{heel} + t_{wall}) / 2$ = **5.7** 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	$M_{heel} = M_{heel\_wt\_base} + M_{heel\_wt\_s} + M_{heel\_sur}$ = **8.1** kNm/m

Check heel in bending

Width of heel	b = **1000** mm/m
Depth of reinforcement	$d_{heel} = t_{base} – c_{heel} – (\phi_{heel} / 2)$ = **319.0** 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} = **303** mm
Area of tension reinforcement required	$A_{s\_heel\_des} = M_{heel} / (0.87 \times f_y \times z_{heel})$ = **62** mm^2/m
Minimum area of tension reinforcement	$A_{s\_heel\_min} = k \times b \times t_{base}$ = **520** mm^2/m
Area of tension reinforcement required	$A_{s\_heel\_req} = \mathrm{Max}(A_{s\_heel\_des}, A_{s\_heel\_min})$ = **520** mm^2/m
Reinforcement provided	**12 mm dia.bars @ 150 mm centres**
Area of reinforcement provided	$A_{s\_heel\_prov}$ = **754** mm^2/m

PASS - Reinforcement provided at the retaining wall heel is adequate

Check shear resistance at heel

Design shear stress	$v_{heel} = V_{heel} / (b \times d_{heel})$ = **0.083** N/mm^2
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/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} = **0.463** 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} = **75** mm
Cover to reinforcement in wall	c_{wall} = **30** mm

Factored horizontal at-rest forces on stem

Surcharge	$F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds})$ = **24.5** kN/m
Saturated backfill	$F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{sat}^2$ = **31.2** kN/m
Water	$F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2$ = **46.4** kN/m

Calculate shear for stem design

Shear at base of stem	$V_{stem} = F_{s\_sur\_f} + F_{s\_s\_f} + F_{s\_water\_f} - F_{prop\_f}$ = **5.7** kN/m

Calculate moment for stem design

Surcharge	$M_{s\_sur} = F_{s\_sur\_f} \times (h_{stem} + t_{base}) / 2$ = **36.8** kNm/m
Saturated backfill	$M_{s\_s} = F_{s\_s\_f} \times h_{sat} / 3$ = **27.1** kNm/m
Water	$M_{s\_water} = F_{s\_water\_f} \times h_{sat} / 3$ = **40.2** kNm/m
Total moment for stem design	$M_{stem} = M_{s\_sur} + M_{s\_s} + M_{s\_water}$ = **104.1** kNm/m

Check wall stem in bending

Width of wall stem	b = **1000** mm/m
Depth of reinforcement	$d_{stem} = t_{wall} – c_{stem} – (\phi_{stem} / 2)$ = **267.0** mm

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

Compression reinforcement is not required

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

$z_{stem} = \mathbf{254}$ mm

Area of tension reinforcement required $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem}) = \mathbf{944}$ mm^2/m

Minimum area of tension reinforcement $A_{s\_stem\_min} = k \times b \times t_{wall} = \mathbf{455}$ mm^2/m

Area of tension reinforcement required $A_{s\_stem\_req} = \mathrm{Max}(A_{s\_stem\_des}, A_{s\_stem\_min}) = \mathbf{944}$ mm^2/m

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

Area of reinforcement provided $A_{s\_stem\_prov} = \mathbf{1340}$ mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress $v_{stem} = V_{stem} / (b \times d_{stem}) = \mathbf{0.021}$ N/mm^2

Allowable shear stress $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1 \text{ N/mm}^2)}, 5) \times 1 \text{ N/mm}^2 = \mathbf{4.733}$ 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 $v_{c\_stem} = \mathbf{0.622}$ N/mm^2

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

Appendix C

Method Statement

Revision	**Date**	**Comments**
-	15.11.13	First Issue for Comment

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 51 Fitzjohn's Ave has been written by a Chartered Engineer and in accordance with the recommendations stated in the Royal Borough of Kensington and Chelsea Town Planning policy on Subterranean Development & Camden New Basement Development Guidance Notes. 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.

The Local geological drift sheets imply the ground to be London Clays

1.7. The bearing pressures have been limited to 125kN/m^2. This is standard loadings for local ground conditions and acceptable to building control and their approvals.

Enabling works

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

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

Basement Sequencing

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

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

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

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

1.14. 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.

1.15. Needle and prop bay. Insert support

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

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

1.17.1. 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)

1.17.2. Floor over to be propped as excavations progress. Steelwork to support Floor to be inserted as works progress.

1.18. 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.

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

Underpinning – Cantilevered Wall Creation

1.20. Excavate first section of retaining wall (no more than 1200mm wide). Where excavation is greater than 1.2m deep provide temporary propping to sides of excavation to prevent earth collapse (Health and Safety). A 1200mm width wall has a lower risk of collapse to the heel face.

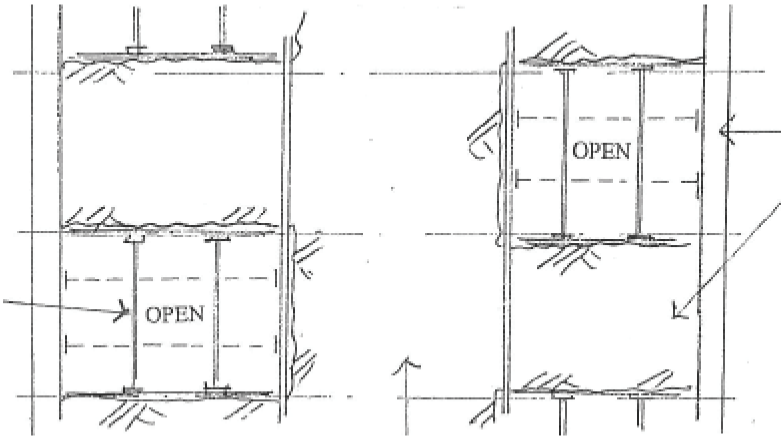
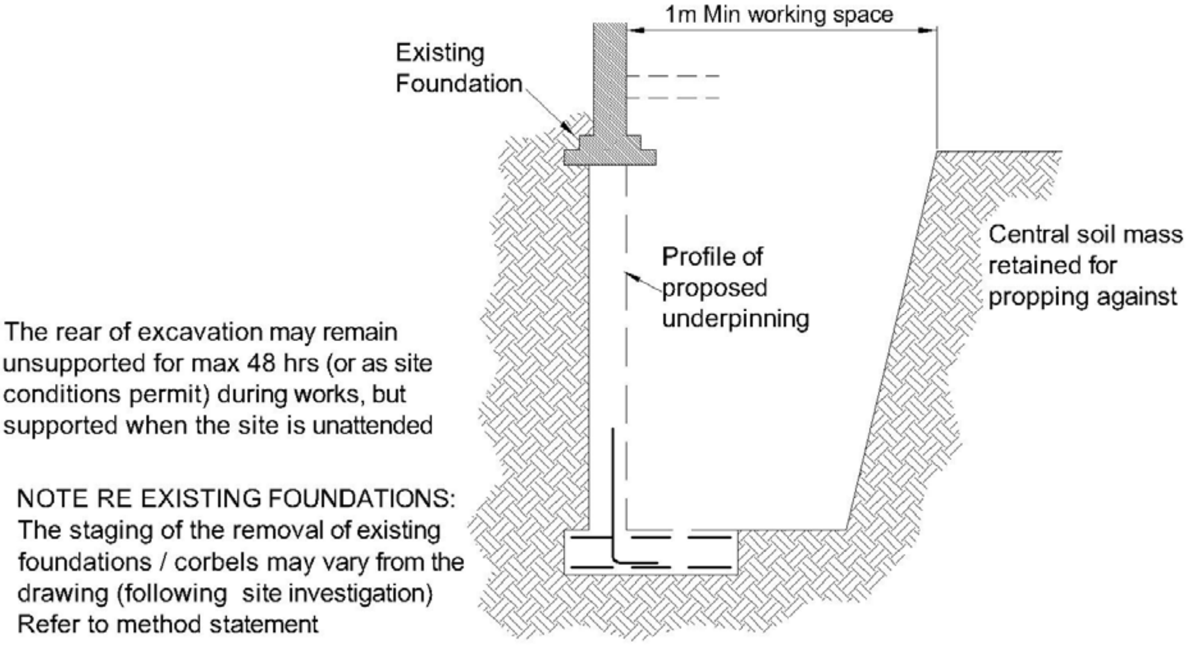
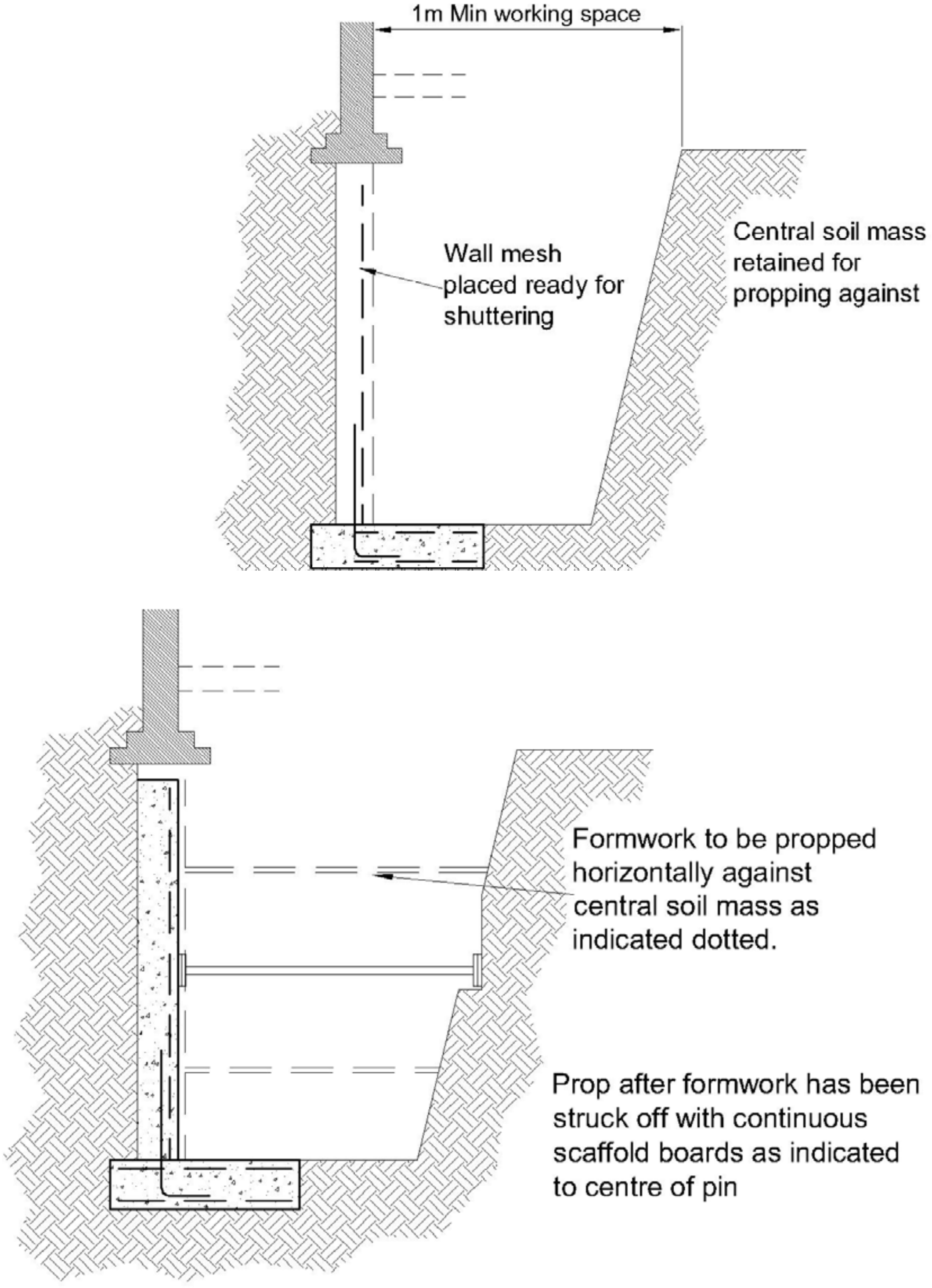


Figure 15 – Schematic Plan view of Soil Propping

Figure 16 Propping

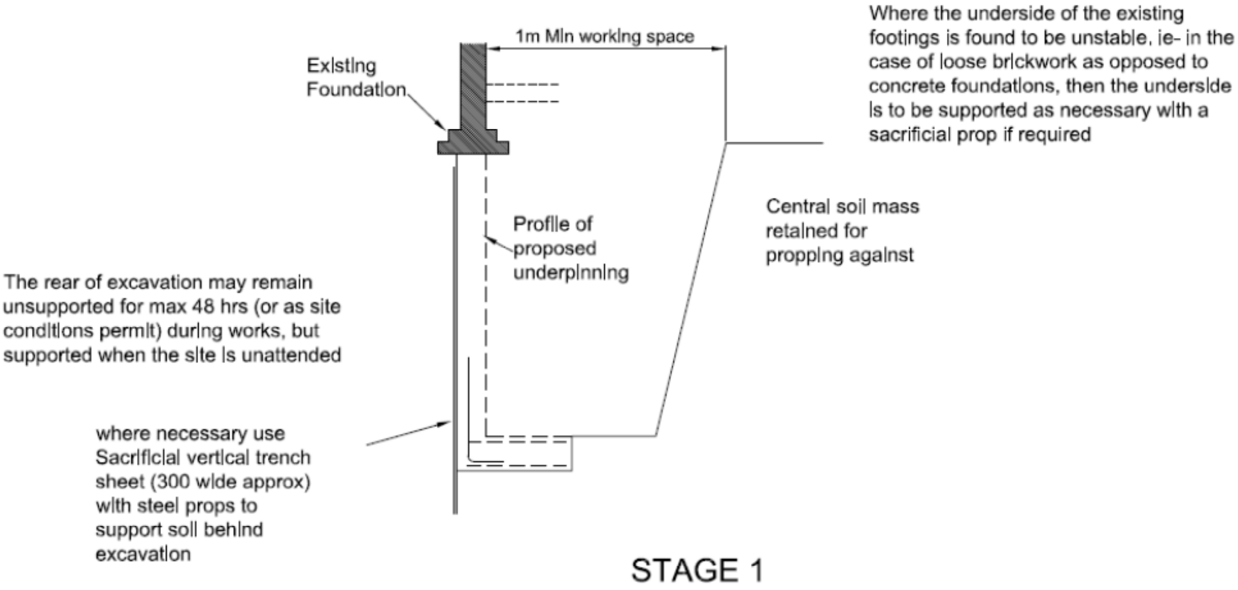


case of loose brickwork as opposed to concrete foundations, then the underside is to be supported as necessary with a sacrificial prop if required

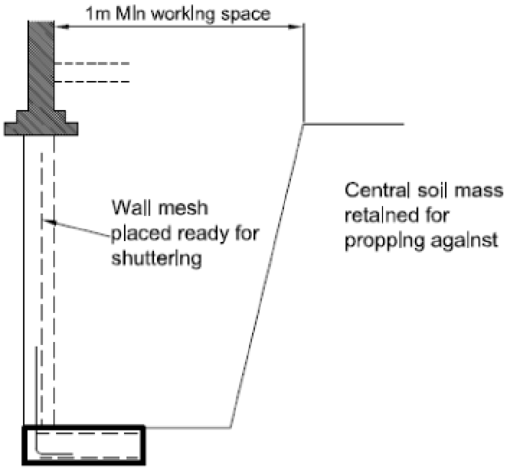


CLAY SOILS - STAGE 3

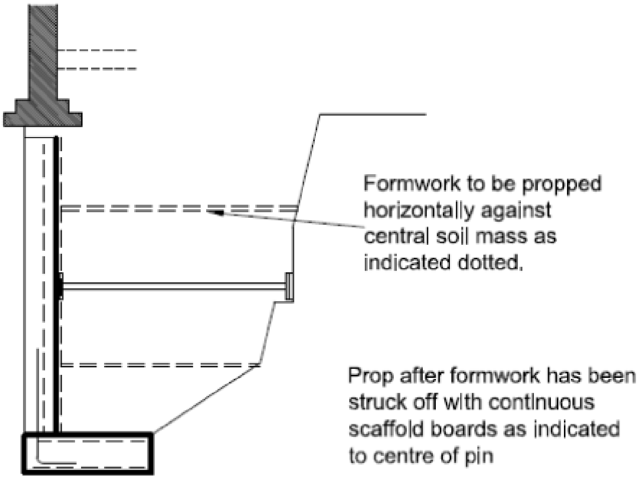
Granular soils:



STAGE 1

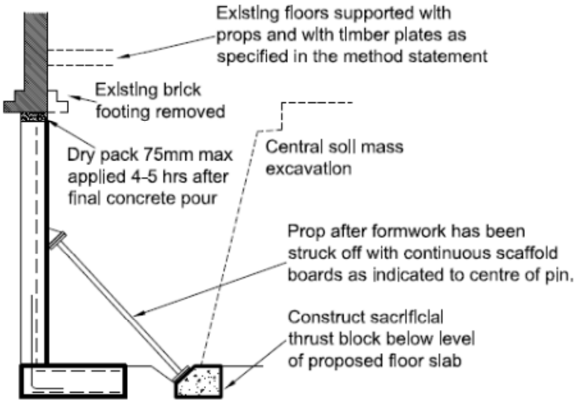


STAGE 2



STAGE 3

Image of Stage 3 on Site



STAGE 4

1.20.1. Where soft spots are encountered back prop with Precast lintels or trench sheeting. Where voids are present behind the lintels (or trench sheeting) grout behind. Prior to casting place layer of DPM between PC lintels (or trench sheeting) 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. . .

1.20.2. 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.

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

1.22. Provide propping to floor where necessary.

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

1.24. Clear underside of existing footing.

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

1.26. Place blinding.

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

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

1.29. 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.

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

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

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

1.33. Place shuttering & pour concrete for retaining wall. Stop a minimum of 75mm from the underside of existing footing. Take 2 cubes of concrete and store for testing

1.34. 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.)

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

1.36. Site supervisor to inspect and sign off for proceeding to the next stage.

Approval

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

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

1.39. 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.

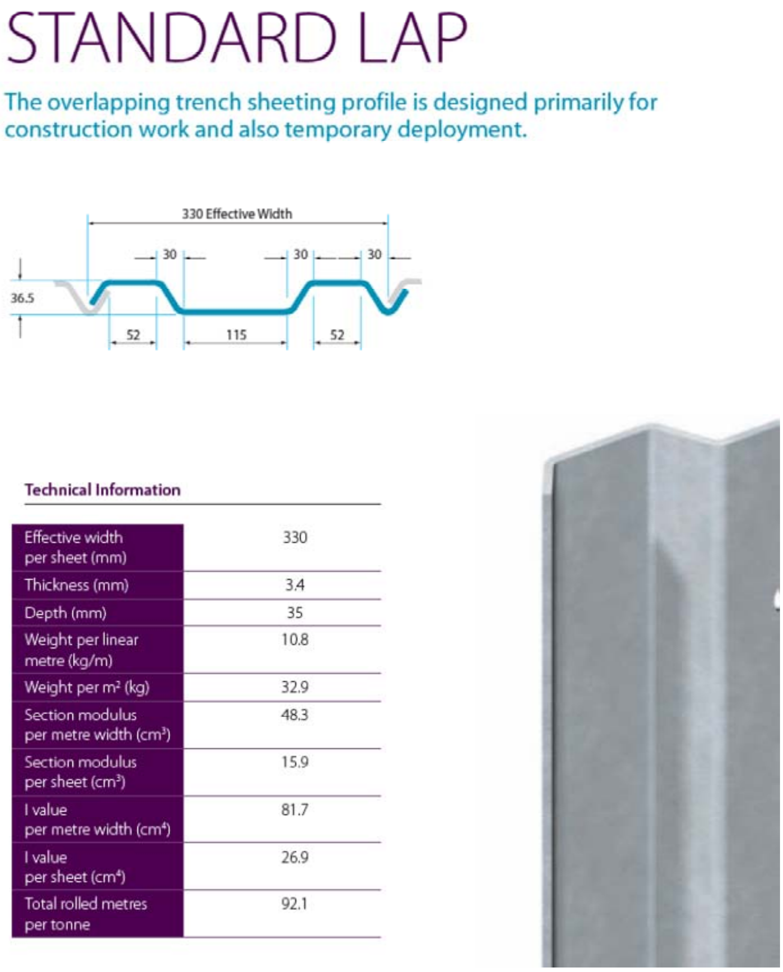
This calcualtion has been provided for the trench sheet and prop design of standard underpins in the temporary condition. There are gaps left between the sheeting and as such no water pressure will occur. Any water present will flow through the gaps betweenthe sheeting and will be required to pump out.

Trecnh 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 udnerpins to be placed at 1200c/c, in this condition the highest load on a trench sheet is when 2 nos 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 snad and gravels have been assummed 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



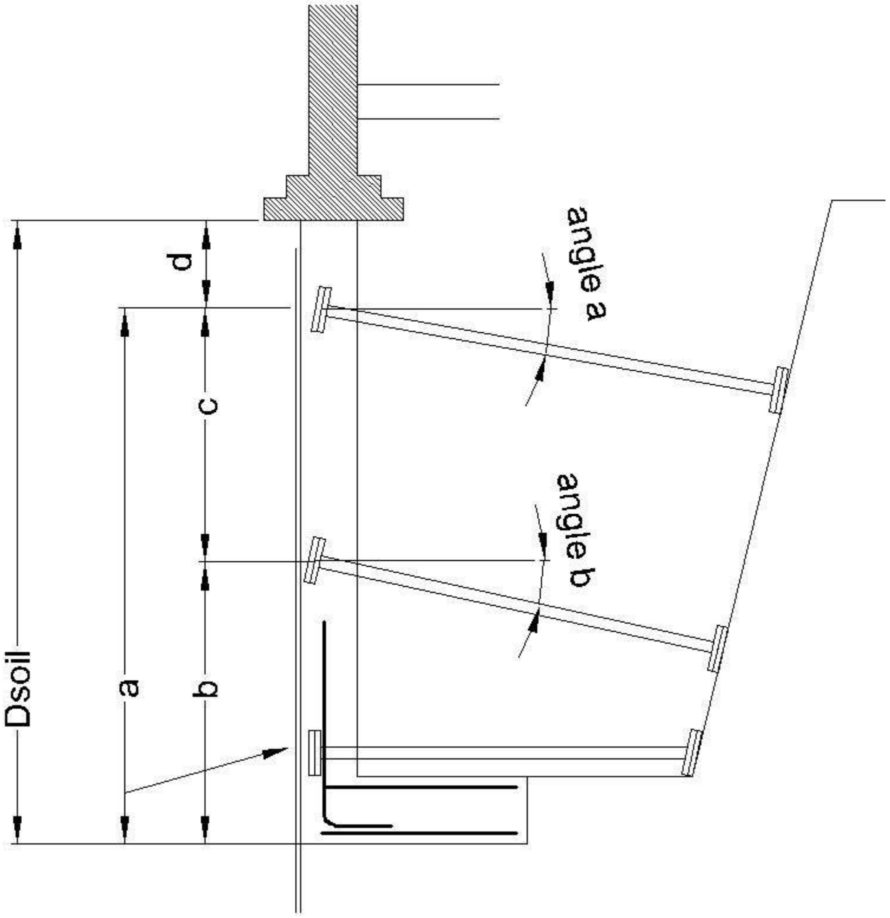
STANDARD LAP

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

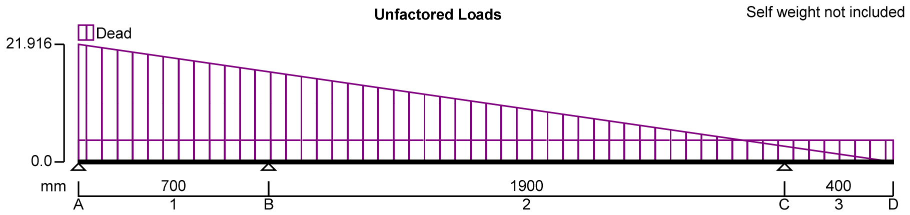


Length a a = **2.600** m

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

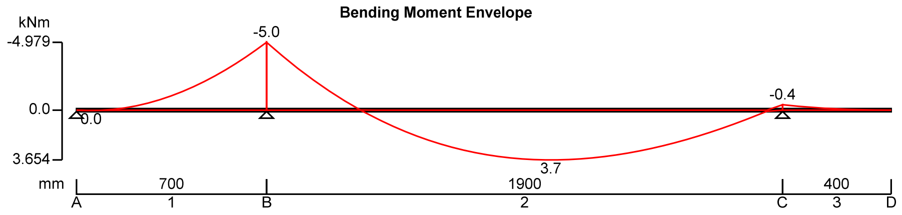
Support A	Max react = **-1.4** kN	Min react = **-1.4** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support B	Max react = **-32.8** kN	Min react = **-32.8** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support C	Max react = **-10.8** kN	Min react = **-10.8** kN	Max mom = **0.0** kNm	Min mom = **0.0**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

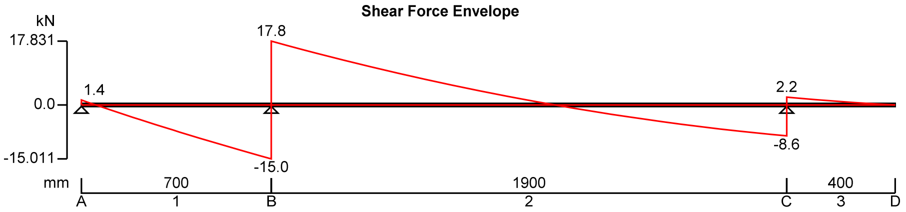
Beam Max/Min results - Combination Summary

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

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

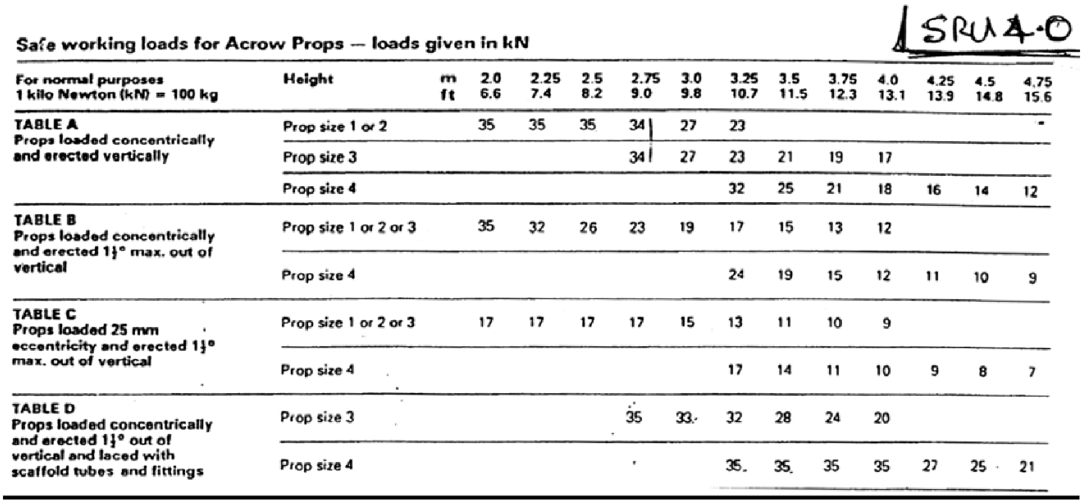
Maximum deflection = **21.0** mm   Minimum deflection = **-14.3** mm





Number of sheets Nos = 2

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


SRU4.0

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

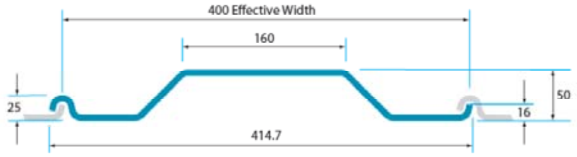
Shear V = (14.6kN + 13.4kN) /2 = 14.000kN

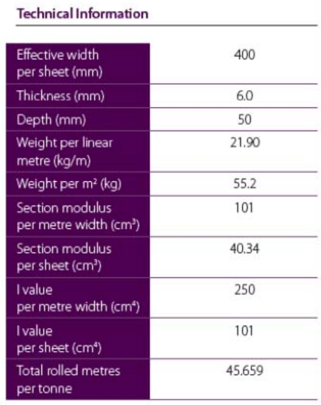
Any Acro Prop is accetpable

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

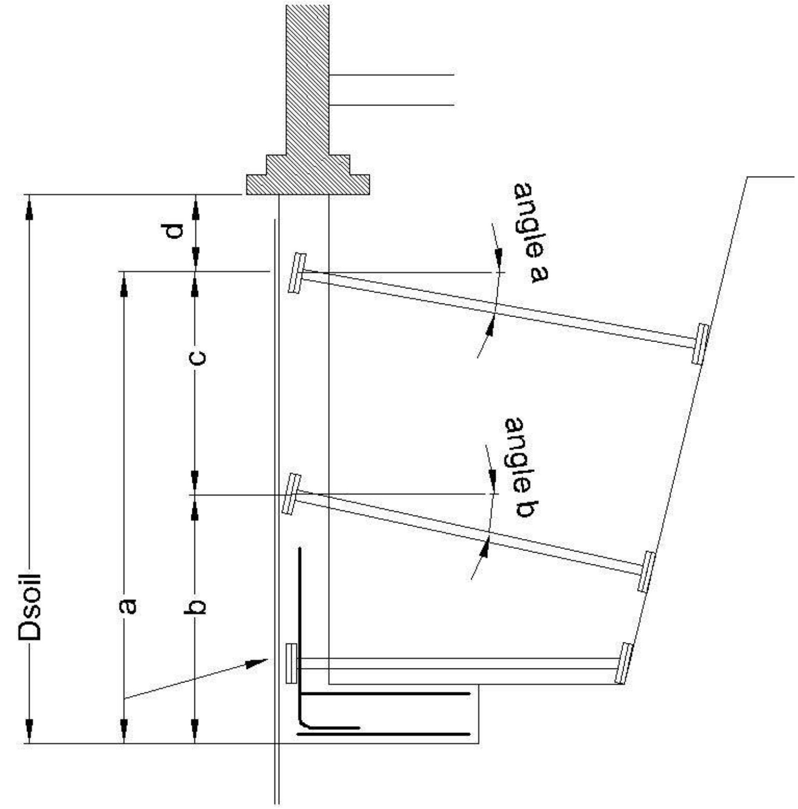
Effective width per sheet (mm)	400
Thickness (mm)	6.0
Depth (mm)	50
Weight per linear metre (kg/m)	21.90
Weight per m^2 (kg)	55.2
Section modulus per metre width (cm^3)	101
Section modulus per sheet (cm^3)	40.34
I value per metre width (cm^4)	250
I value per sheet (cm^4)	101
Total rolled metres per tonne	45.659

$Sxx = 48.3cm^3$

$py = 275N/mm^2$

$Ixx = 26.9cm^4$

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

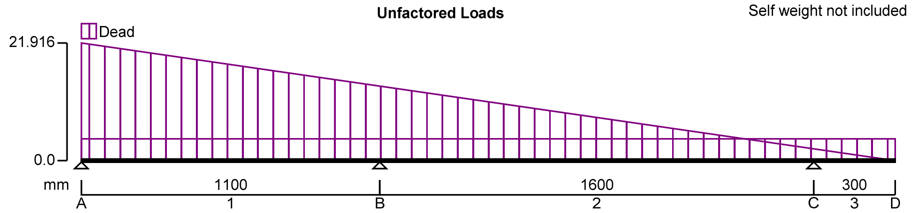


Length a a = **2.700** m

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.×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

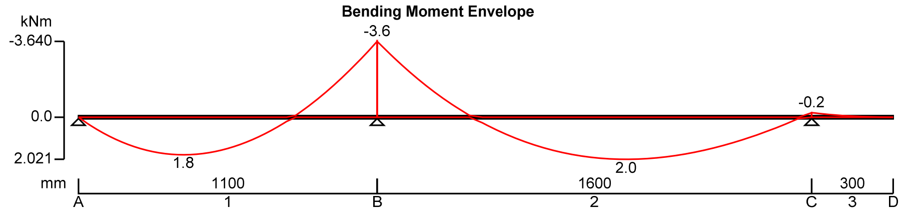
Support D Max react = **0.0** kN Min react = **0.0** kN Max mom = **0.0** kNm Min mom = **0.0** kNm

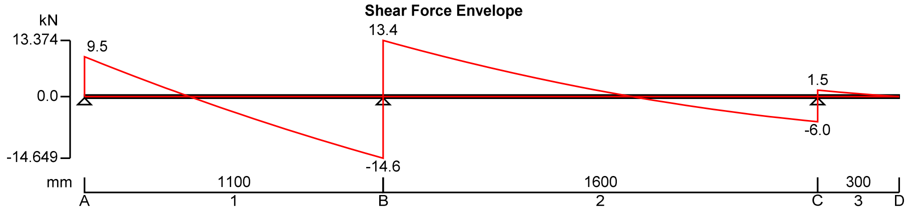
Beam Max/Min results - Combination Summary

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

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

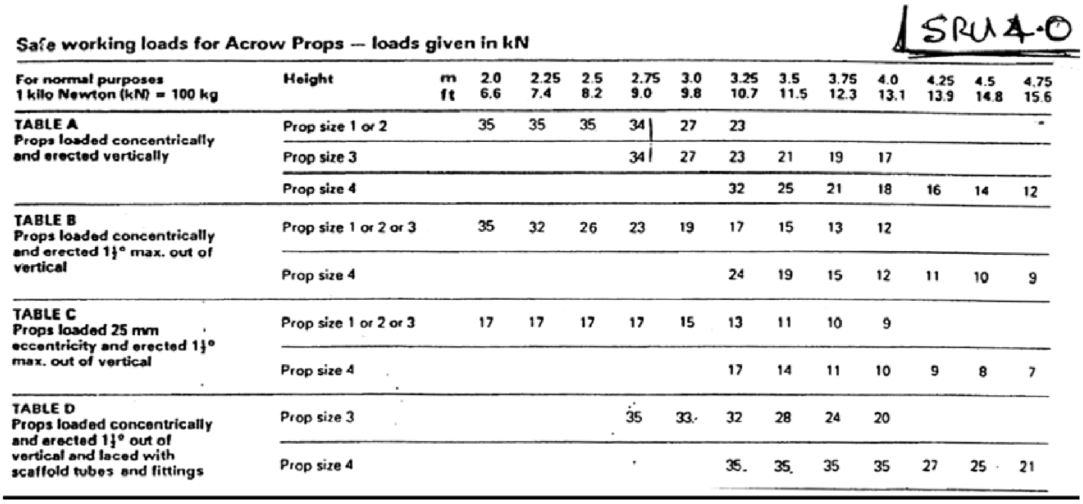
Maximum deflection = **7.7** mm Minimum deflection = **-4.9** mm





Number of sheets Nos = 2

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



SRU4.0

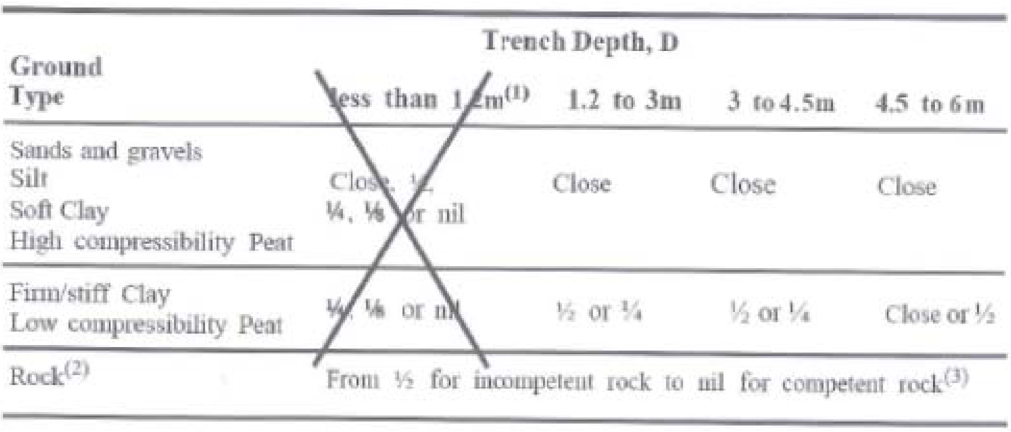
Safe working loads for Acrow Props — loads given in kN

For normal purposes 1 kilo Newton (kN) = 100 kg	Height	m	2.0	2.25	2.5	2.75	3.0	3.25	3.5	3.75	4.0	4.25	4.5	4.75
		ft	6.6	7.4	8.2	9.0	9.8	10.7	11.5	12.3	13.1	13.9	14.8	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

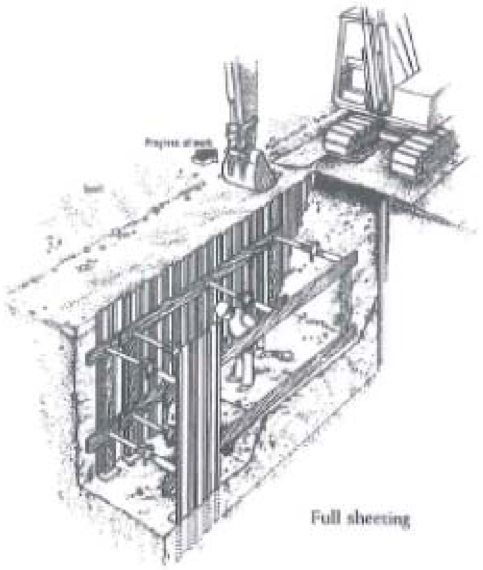
Any Acro Prop is accetpable

Sheeting requirements



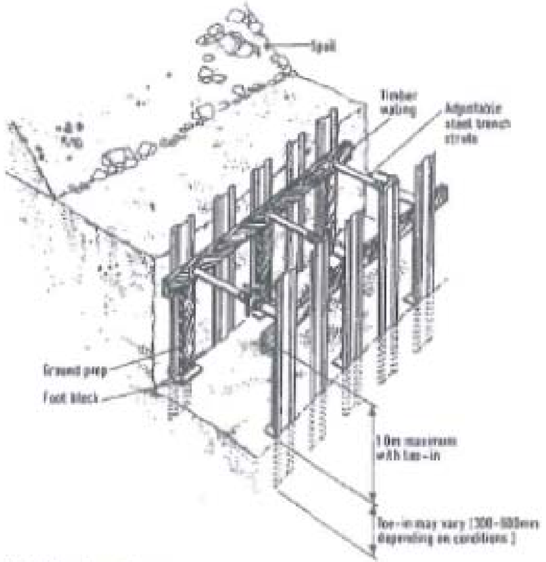
Ground Type	Trench Depth, D ~~less than 1.2m(1)~~	1.2 to 3m	3 to 4.5m	4.5 to 6m
Sands and gravels Silt Soft Clay High compressibility Peat	~~Close, ½, ¼, ⅛ or nil~~	Close	Close	Close
Firm/stiff Clay Low compressibility Peat	~~¼, ⅛ or nil~~	½ or ¼	½ or ¼	Close or ½
Rock(2)	From ½ for incompetent rock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

Sheeting requirements



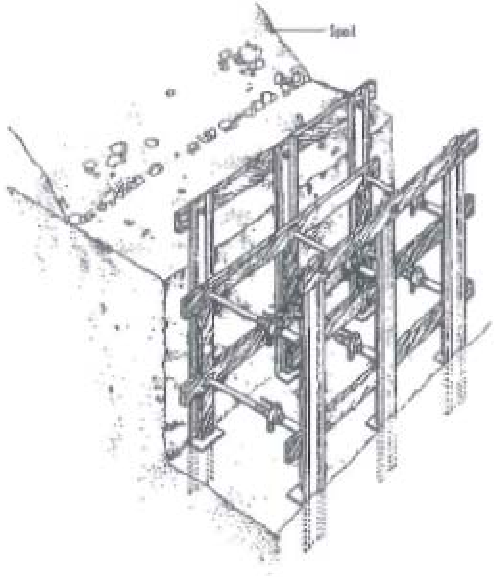
Full sheeting

Sheeting requirements



Half sheeting
shown for 1.5 m deep trench

Sheeting requirements



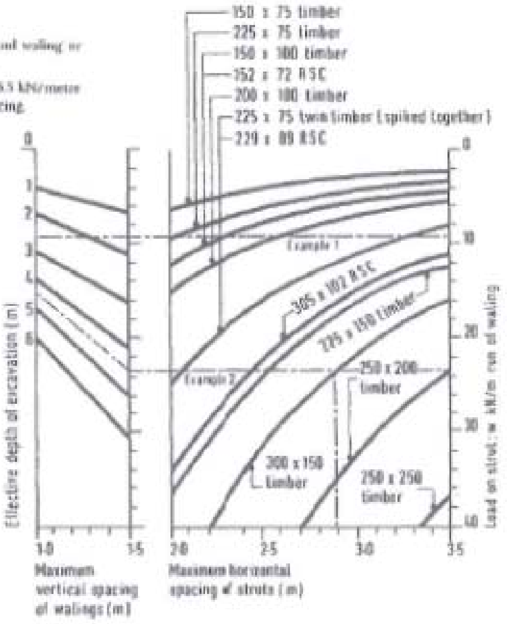
Quarter sheeting

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 proprietary system should be checked against manufacturer's latest information.



Use for:
Granular soils
Mixed soils
Short term trenches in clay
(see notes opposite)

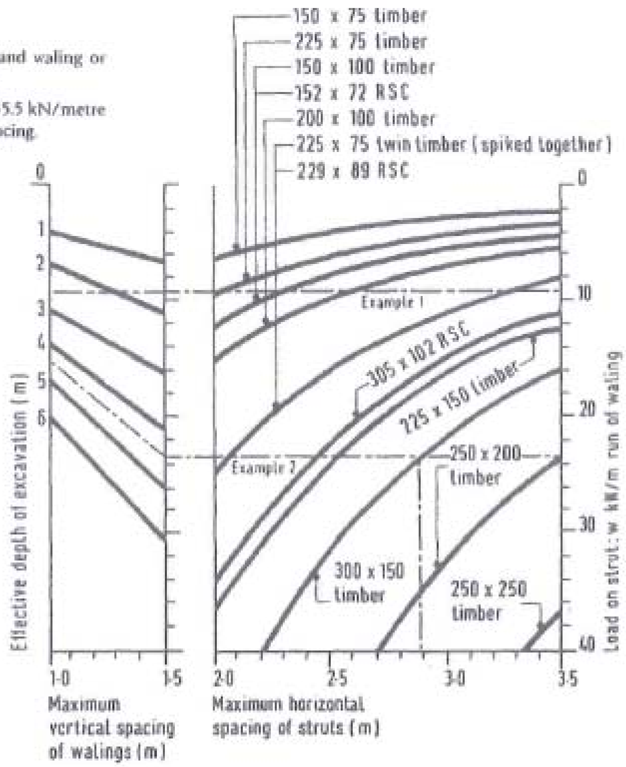
Note:

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.

BEWARE

Any proprietary system should be checked against manufacturer's latest information.



Use for:
- Granular soils
- Mixed soils
- Short term trenches in clay (see notes opposite)

Appendix D

Soil Investigation Report

GROUND INVESTIGATION REPORT

for the site at

51 FITZJOHN'S AVENUE, HAMPSTEAD, LONDON NW3 6PH

on behalf of

51 FITZJOHN'S DEVELOPMENT LIMITED
C/O CROFT STRUCTURAL ENGINEERS LIMITED

Report Reference: GWPR921/GIR/September 2014		Status: FINAL
Issue:	Prepared By:	Verified By:
V1.01 September 2014		T. Williams
	Mathias Gabrat BEng MSc. Geotechnical/Geo-environmental Engineer	Francis Williams M.Geol. (Hons) FGS CEnv AGS MSoBRA Director
File Reference: Ground and Water/Project Files/ GWPR921 51 Fitzjohn's Avenue, London NW3 6PH		

Ground and Water Limited 15 Bow Street, Alton, Hampshire GU34 1NY
Tel: 0333 600 1221 E-mail: enquiries@groundandwater.co.uk Website: www.groundandwater.co.uk

Site Investigations Environmental Consultants Geotechnical Engineers

CONTENTS

FIGURES

APPENDICES

1.0 INTRODUCTION

1.1 General

Ground and Water Limited were instructed by 51 Fitzjohn's Development Limited, c/o Croft Structural Engineers Limited, on the 29th April 2014 to undertake a Ground Investigation on a site at 51 Fitzjohn's Avenue, Hampstead, London NW3 6PH. The scope of the investigation was detailed within the Ground and Water Limited fee proposal ref: GWQ1968, dated 3rd 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 Geological, Hydrogeological and Hydrological Study, Guidance for Subterranean Development (November 2010) was reviewed with respect to this report.

A Desk Study and 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 ~1440m^2 in area and orientated in an east to west direction, located on the western side of Fitzjohn's Avenue, ~50m south of its junction with Akenside Road and Lyndhurst Road. The site was located in Hampstead, north-west of London, within the London Borough of Camden.

The national grid reference for the centre of the site was approximately TQ 26524 85142. A site location plan is given within Figure 1. A plan showing the site area is given within Figure 2.

2.2 Site Description

The site was occupied by a detached four storey brick built block of residential flats with lower basement and roof accommodation. A paved driveway and parking area was accessed off Fitzjohn's Avenue and fronted the property. Access to the soft landscaped rear garden was through the bin store on the northern side of the property and via a narrow path.

The sites environs were noted to be sloping gently to moderately to the south.

2.3 Proposed Development

At the time of reporting, September 2014, the proposed development will comprise the enlargement of the existing basement so it spans almost the entire of the existing property. It is therefore understood that the basement (underside of the slab level) will be constructed at a depth of ~3.0 - 3.5m below existing ground level.

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 Hampstead area (Sheet No. 256 of the North London area) revealed the site to be situated on the Claygate Member of the London Clay Formation overlying 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.

Claygate Member of the London Clay Formation

The Claygate Member comprises the youngest part of the London Clay Formation and forms a transition between the deep water, dominantly argillaceous London Clay Formation and the succeeding shallow water arenaceous Bagshot Formation. The Claygate Member of the London Clay Formation comprises laminated orange sand and light grey to lilac clay, of a total thickness of 15m.

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.

Examination of the online BGS borehole records revealed a BGS borehole in similar geology to the north-west of the site revealed a thin capping of Made Ground to overlie light grey brown sandy clayey silts to silty clays and then dark grey silty clays.

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° 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. Areas prone to landslides were noted to the south of the site.

Figure 18 of the Camden Geological, Hydrogeological and Hydrological Study indicated that two major subterranean infrastructure (including existing and proposed tunnels) were noted ~120m north and ~100m 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 a **Secondary A Aquiver** relating to the bedrock of the Claygate Member of the London Clay Formation. No designation was given for any superficial deposits due to their likely absence.

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

Secondary Aquifers include a wide range of rock layers or drift deposits with an equally wide range of water permeability and storage. Secondary A Aquifers are permeable layers capable of supporting water supplies at a local rather than strategic scale, and in some cases forming an important source of base flow to rivers. These are generally aquifers formerly classified as minor aquifer.

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.

In accordance with Figure 11 of the Camden Geological, Hydrogeological and Hydrological Study a tributary of the lost River Tyburn was shown flowing down Fitzjohn's Avenue to the east and south of the site. A tributary of the lost River Westbourne was noted to the south-west of the site.

In accordance with Figure 12 of the Camden Geological, Hydrogeological and Hydrological Study there were no surface water features in a close proximity to the site.

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-westerly direction in accordance with the local topography.

Examination of the Environment Agency records showed that the site was not situated within a floodplain or flood warning area. Figure 5 the Camden Geological, Hydrogeological and Hydrological Study revealed that the site was not subject to surface water flooding.

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 8th May 2014 and comprised the drilling of one Premier Windowless Borehole (BH1) at the front of the property to a depth of 12.45m bgl, one window sampler borehole (WS2) at the rear of the property to a depth of 4.00m bgl and the hand excavation of two trial pit foundation exposures (TP/FE1 and TP/FE2).

A small diameter combined bio-gas and groundwater monitoring well was installed within BH1 to 5.00m bgl. 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)
BH1	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 Francis Williams 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 Topsoil (TP/FE1 only) and Made Ground was noted to overlie the soils of the Claygate Member of the London Clay Formation. Soils described as Head Deposits were noted between the Made Ground and the soils of the Claygate Member of the London Clay Formation in WS2 and TP/FE2 only

The ground conditions encountered during the investigation are described in this section. For more complete information about the Made Ground, the Head Deposits and the Claygate Member of 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 5.

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

Topsoil (TP/FE1 only)
Made Ground
Head Deposits (WS2 and TP/FE2 only)
Claygate Member of the London Clay Formation

Topsoil

Topsoil comprising a dark grey silty clay was encountered between GL-0.40m bgl in TP/FE1.

Made Ground

Made Ground was encountered underlying the Topsoil in TP/FE1 and from ground level in each of the remaining trial holes to a depth between 0.70-1.20m bgl.

In BH1, underlying a 0.30m thickness of paving slab resting on sharp sand, the Made Ground comprised a dark brown to mid brown, locally orange brown mottled, slightly gravelly sandy silty clay to 0.90m bgl. The sand was fine to coarse grained and the gravel was rare, fine to medium, sub-angular to sub-rounded flint and brick.

In WS2 a dark brown to black clayey gravelly sand was noted to a depth of 0.70m bgl. The sand was fine to coarse grained and the gravel was occasional, fine to coarse, sub-angular to sub-rounded brick and rare concrete and flint.

Soil described as dark reddy brown, dark brown to black clayey sandy gravel, locally a very sandy gravelly clay, were encountered to a depth of 0.86m in TP/FE2 and for the remaining depth of TP/FE1, a depth of 1.20m bgl. The sand was fine grained and the gravel was occasional, fine to medium, sub-angular to sub-rounded brick, concrete and flint.

Head Deposits
Soils described as Head Deposits were encountered underlying the Made Ground for the remaining depth of TP/FE2, a depth of 1.00m bgl, and to a proved depth of 1.25m bgl in WS2. The soils generally comprised an orange brown and light grey, to light grey brown, very slightly gravelly sandy silty clay. The sand was fine grained and the gravel was rare, fine, sub-angular to sub-rounded flint.

Claygate Member of the London Clay Formation
Soils described as Claygate Member of the London Clay Formation were encountered underlying the Made Ground in BH1 and the Head Deposits in WS2 and generally comprised an orange brown, dark orange brown and brown grey mottled silty sandy clay. The deposits were proved for the remaining depth of BH1, a depth of 12.45m bgl, and WS2, a depth of 4.00m 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, on its southern side. 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 1.00m bgl a brick wall was noted. The brick was noted to rest upon a concrete footing that stepped out by 0.19m and was >0.20m in thickness. It was not possible to hand excavate the trial pit below 1.20m bgl and therefore determine the thickness and founding stratum of the concrete footing. Topsoil and Made Ground was noted for the full depth of the trial pit. 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, on its northern side. 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 0.25m bgl a brick wall was noted where two brick steps (between 0.19-0.065m in width and 0.09-0.12m in thickness) out from the property were noted. A further brick wall was recorded, that stepped back into the property, for 0.24m. The brick wall rested upon a concrete footing that stepped out by 0.14m and was 0.16m in thickness. The foundation was noted to rest upon soils described as Head Deposits comprising an orange brown to light brown gravelly sandy clay. The sand was fine to coarse grained and the gravel was rare, fine to coarse, sub-rounded to angular flint. Made Ground was noted to a depth of 0.70m bgl in the trial pit. 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)**
BH1	Roots to 1.00m bgl. Occasional rootlets were noted within the geotechnical classification testing at 2.00m, 2.50m and 3.50m bgl.	Decaying, assumed relic, roots to 2.00m bgl
WS2	Roots to 1.50m 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

A groundwater strike was noted in BH1 at 5.50m bgl. Groundwater was not encountered in the remaining trial holes. A return site visit on the 20th June 2014 revealed that the groundwater was standing at a depth of 4.60m bgl within the standpipe installed in BH1.

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 May and June 2014, when groundwater levels are close to their annual minimum (lowest elevation).

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

4.5 Obstructions

No other 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

Standard Penetration Testing was undertaken within BH1 at 1.00m intervals. The results of the SPT's have not been amended to take into account hammer efficiency, rod length and overburden pressure in accordance with Eurocode 7. The test results are presented on the borehole logs within Appendix B.

Windowless Sampler Boreholes provide samples of the ground for assessment but they do not give any engineering data. The standard penetration test (SPT) is an in-situ dynamic penetration test designed to provide information on the geotechnical engineering properties of soil. The test uses a thick-walled sample tube, with an outside diameter of 50 mm and an inside diameter of 35 mm, and a length of around 650mm. This is driven into the ground at the bottom of a borehole by blows from a slide hammer with a weight of 63.5 kg falling through a distance of 760 mm. The sample tube is driven 150 mm into the ground and then the number of blows needed for the tube to penetrate each 150 mm up to a depth of 450 mm is recorded. The sum of the number of blows is termed the "standard penetration resistance" or the "N-value".

The cohesive soils of the Claygate Member of the London Clay Formation were classified based on the table below.

Undrained Shear Strength from Field Inspection/SPT 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	SPT “N” Blow Counts	Undrained Shear Strength kPa (based on Stroud, 1974)	Soil Type		Trial Hole
			Cohesive	Granular	
Claygate Member of the London Clay Formation	4 – 26	20 – 130	Very low/Low – High	-	BH1 (0.90 – 12.45m bgl)

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 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 Claygate Member of 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	3
Moisture Content	BS1377:1990:Part 2:Clause 3.2	8
Water Soluble Sulphate & pH	BS1377:1990:Part 3:Clause 5	1
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 Claygate Member 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
Claygate Member of the London Clay Formation	25 - 33	100	23 – 35	CI - CH	Soft - Stiff	Medium	Medium

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 Claygate Member 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**
Claygate Member of the London Clay Formation **BH1/3.00m bgl** (Brown and grey sandy silty CLAY)	33	21	23	0.52	Overconsolidated.
Claygate Member of the London Clay Formation **BH1/6.50m bgl** (Grey sandy silty CLAY)	28	24	29	0.14	Heavily Overconsolidated
Claygate Member of the London Clay Formation **BH2/2.50m bgl** (Brown CLAY with blue grey veins and orange brown sandy patches)	25	24	35	0.03	Heavily Overconsolidated.

Liquidity Index testing revealed no evidence for moisture deficit within the overconsolidated to heavily overconsolidated samples of the Claygate Member of 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**
Claygate Member of the London Clay Formation **BH1/3.00m bgl** (Brown and grey sandy silty CLAY)	33	44	17.6	MC > 0.4 x LL (No significant moisture deficit)
Claygate Member of the London Clay Formation **BH1/6.50m bgl** (Grey sandy silty CLAY)	28	53	21.2	MC > 0.4 x LL (No significant moisture deficit)
Claygate Member of the London Clay Formation **BH2/2.50m bgl** (Brown CLAY with blue grey veins and orange brown sandy patches)	25	59	23.6	MC > 0.4 x LL (No significant moisture deficit)

The results in the table above indicate that the samples of the the overconsolidated to heavily overconsolidated samples of the Claygate Member of the London Clay formation tested showed no evidence of a significant moisture deficit.

5.2.3 Moisture Content Profiling

The moisture content versus depth plot for BH1 can be seen within Figure 8.

The plot shows variations in moisture content that are most likely due to variations in lithology (the sand content) rather than the moisture demand from nearby trees.

5.2.4 Sulphate and pH Tests

Sulphate and pH tests were undertaken on one sample from the Claygate Member of the London Clay Formation (BH1/1.50m bgl). The sulphate concentration was 90mg/l with a pH of 7.6.

5.2.5 BRE Special Digest 1

In accordance with BRE Special Digest 1 'Concrete in Aggressive Ground' (BRE, 2005) one sample of the Claygate Member of the London Clay Formation (BH2/8.00m bgl) was scheduled for laboratory analysis to determine parameters for concrete specification.

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

Summary of Results of BRE Special Digest Testing

Determinand	Unit	Minimum	Maximum
pH	-	-	7.3
Ammonium as NH_4	mg/kg	-	4.2
Sulphur	mg/kg	-	3358
Chloride (water soluble)	mg/kg	-	22
Magnesium (water soluble)	g/l	-	0.0782
Nitrate (water soluble)	mg/kg	-	<3
Sulphate (water soluble)	g/l	-	0.82
Sulphate (total)	mg/kg	-	2602

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.

- A 0.70->1.20m capping of Topsoil/Made Ground was noted within the trial holes constructed.

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

- Soils described as Head Deposits were encountered underlying the Made Ground for the remaining depth of TP/FE2, a depth of 1.00m bgl, and to a proved depth of 1.25m bgl in WS2. The soils generally comprised an orange brown and light grey, to light grey brown very slightly gravelly sandy silty clay. The sand was fine grained and the gravel was rare, fine, sub-angular to sub-rounded flint.

  Given the limited thickness of the Head Deposits, these deposits are unlikely to be adopted as a bearing stratum for the proposed basement.

- Soils described as Claygate Member of the London Clay Formation were encountered underlying the Made Ground in BH1 and the Head Deposits in WS2 and generally comprised an orange brown, dark orange brown and brown grey mottled silty sandy clay for the remaining depth of BH1, a depth of 12.45m bgl, and WS2, a depth of 4.00m bgl. The sand was fine grained.

  The cohesive soils of the Claygate Member of the London Clay Formation comprised very low/low to high undrained shear strength (20 -130kPa) soils between 0.90-12.45m bgl in BH1.

  The soils of the Claygate Member of the London Clay Formation 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 Claygate Member of the London Clay Formation to be soft to stiff. Liquidity Index testing revealed the soils to be overconsolidated to heavily overconsolidated.

  Geotechnical analysis revealed no potential significant moisture deficits were present within the samples of the Claygate Member of the London Clay Formation tested. Moisture content profiling indicated that the moisture profile with depth within the Claygate Member of the London Clay Formation was as expected with the variations noted associated with small changes in lithology (sand content).

  The soils of the Claygate Member of the London Clay Formation are overconsolidated to heavily overconsolidated cohesive soils and are therefore likely to be a suitable stratum for the proposed traditional strip or mat 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.

- A groundwater strike was noted in BH1 at 5.50m bgl. Groundwater was not encountered in the remaining trial holes. A return site visit on the 20th June 2014 revealed that the groundwater was standing at a depth of 4.60m bgl within the standpipe installed in BH1.

- Roots were noted to a depth of 1.00m bgl in BH1, 1.50m bgl in WS2. Decaying, assumed to be relic, roots were also noted to ~2.0m bgl in BH1.

6.2 Basement Foundations

At the time of reporting, September 2014, the proposed development will comprise the enlargement of the existing basement so it spans almost the entire of the existing property. It is therefore understood that the basement (underside of the slab level) will be constructed at a depth of ~3.0 - 3.5m below existing ground level.

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 **medium 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.

Fresh root penetration was noted by the supervising engineer within the samples collected to 1.00m bgl within BH1 and 1.50m bgl within WS2. Possible decaying roots were noted to 2.00m bgl within BH1. During the geotechnical classification testing possible rootlets were noted in the samples from BH1 at 2.00m bgl, 2.50m bgl and 3.50m bgl.

Geotechnical analysis revealed no potential significant moisture deficits were present within the samples of the Claygate Member of the London Clay Formation tested. Moisture content profiling indicated that the moisture profile with depth within the Claygate Member of the London Clay Formation was as expected with the variations noted associated with small changes in lithology (sand content).

Therefore it was considered likely the roots noted during the geotechnical classification testing are relic and pose not risk to the serviceability of the proposed structure. **However, the basement formation level must be carefully inspected for the presence of fresh/live roots. Should live roots be noted at basement formation level then the basement formation level should be extended at least 300mm into non-root penetrated soils. The void should be backfilled to the proposed slab level using a granular engineered fill.**

Should the above confirm that the roots in BH1 are live then any retaining wall strip footings may need to be extended to >3.80m bgl.

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 3.00m and 3.50m 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)
3.00m	5.00m by 0.75m Strip	224.15
	5.00m by 1.00m Strip	226.16
3.50m	5.00m by 0.75m Strip	227.40
	5.00m by 1.00m Strip	229.46

Serviceability State: Settlement Parameters Calculated

Depth (m BGL)	Foundation System	Limit Bearing Capacity (kN/m^2)	Settlement (mm)
3.00m	5.00m by 0.75m Strip	125	~25
	5.00m by 1.00m Strip	125	~25
3.50m	5.00m by 0.75m Strip	125	~25
	5.00m by 1.00m Strip	125	~25

It must be noted that a bearing capacity of less than 45KN/m^2 at 3.00 and 50kN/m^2 at 3.50m bgl may results in heave of the underlying soils.

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

Based on the results of the investigation a piled foundation is unlikely to be required for the proposed development.

6.4 Basement Excavations & Stability

Shallow excavations in the Topsoil, Made Ground, Head Deposits and Claygate Member of the 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 SPT 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	~15	0	12	0.66	1.52
Head Deposits	~17 - 18	0	20	0.49	2.04
Claygate Member of the 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 unlikely that groundwater would be encountered during basement construction.

Dewatering from sumps introduced into the floor of the excavation is likely to be required if perched groundwater is encountered within the Made Ground or sand horizons of the Head Deposits and the Claygate Member of the London Clay Formation, especially after a period of excessive rainfall.

6.5 Hydrogeological Effects

The proposed development is located on a **Secondary A Aquifer** relating to the bedrock of the Claygate Member of the London Clay Formation.

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

A groundwater strike was noted in BH1 at 5.50m bgl. Groundwater was not encountered in the remaining trial holes. A return site visit on the 20th June 2014 revealed that the groundwater was standing at a depth of 4.60m bgl within the standpipe installed in BH1.

Based on the above it is considered unlikely that the basement will be constructed below the groundwater level. Perched groundwater may be encountered during construction within the Made Ground or sand horizons of the Head Deposits and the Claygate Member of the London Clay Formation, especially after a period of excessive rainfall.

In relation to the basement, once constructed, the Made Ground, Head Deposits and the Claygate Member of the London Clay Formation will act as a barrier for groundwater migration and therefore additional drainage should be considered.

6.6 Sub-Surface Concrete

Sulphate concentrations measured in 2:1 water/soil extracts taken from the Claygate Member of the 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 Claygate Member of the London Clay Formation. For the classification given, the "mobile" and "natural" case was adopted given the cohesive nature of the deposits (permeability likely to exceed 10-7 m/sec given sand and silt lenses noted) and residential use of the site.

The sulphate concentration in the samples ranged from 90 - 860mg/l with a pH range of 7.3 -7.6. The total sulphate concentration recorded was 0.26%.

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 Claygate Member of the London Clay Formation and 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.

6.7 Surface Water Disposal

Infiltration tests were beyond the scope of the investigation.

Soakaway construction within the cohesive soils of the Head Deposits and the Claygate Member of the 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.

6.8 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.

6.9 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 classification was established 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).

6.10 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.

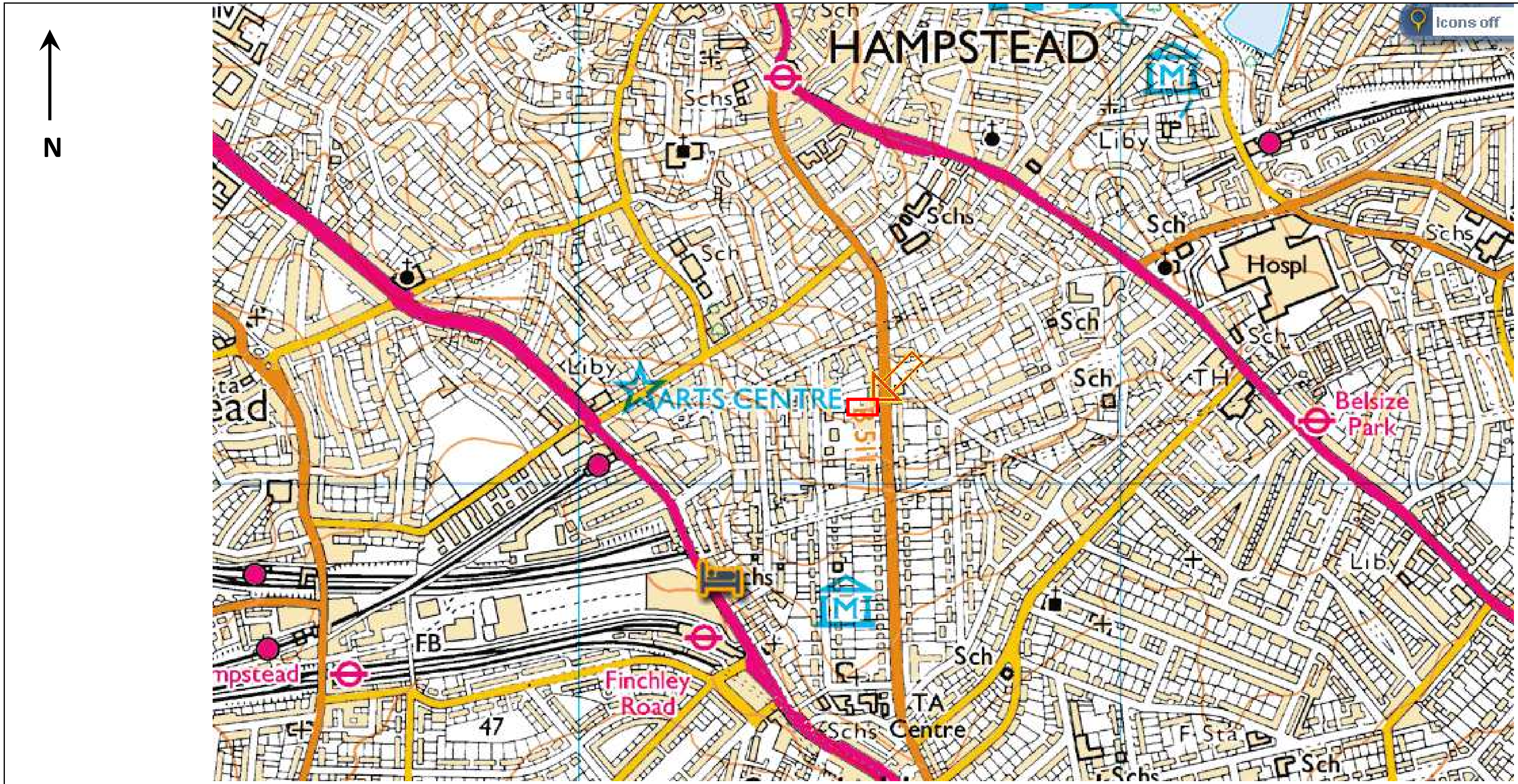
The Topsoil must be fit for purpose and must either be supplied with traceable chemical laboratory test certificates or be tested, either prior to placing (ideally) or after placing, to ensure that the human receptor cannot come into contact with compounds that could be detrimental to human health.

6.11 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.



Approximate Site Boundary

NOTE: NOT TO SCALE

Project:
51 Fitzjohn's Avenue, Hampstead, London NW3 6PH

Figure 1

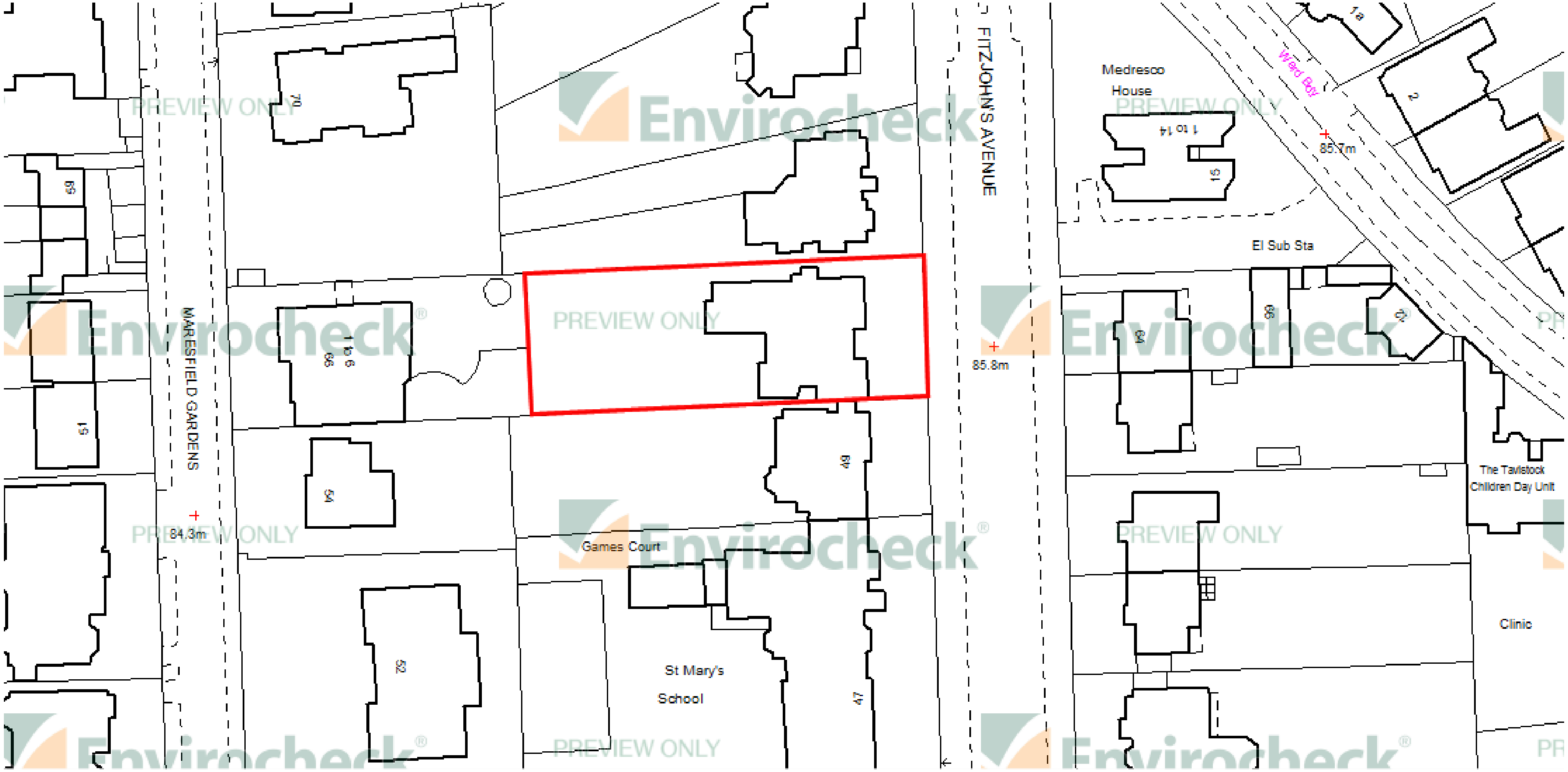
Client:
51 Fitzjohn's Development Limited

Date:
September 2014

ground&water

Site Location Plan

Ref:
GWPR921



Approximate Site Boundary

NOTE: NOT TO SCALE

Project:
51 Fitzjohn's Avenue, Hampstead, London NW3 6PH

Figure 2

Client:
51 Fitzjohn's Development Limited

Date:
September 2014

Site Development Area

Ref:
GWPR921

ground&water

Approximate Site Boundary

NOTE: NOT TO SCALE

Project:
51 Fitzjohn's Avenue, Hampstead, London NW3 6PH

Client:
51 Fitzjohn's Development Limited

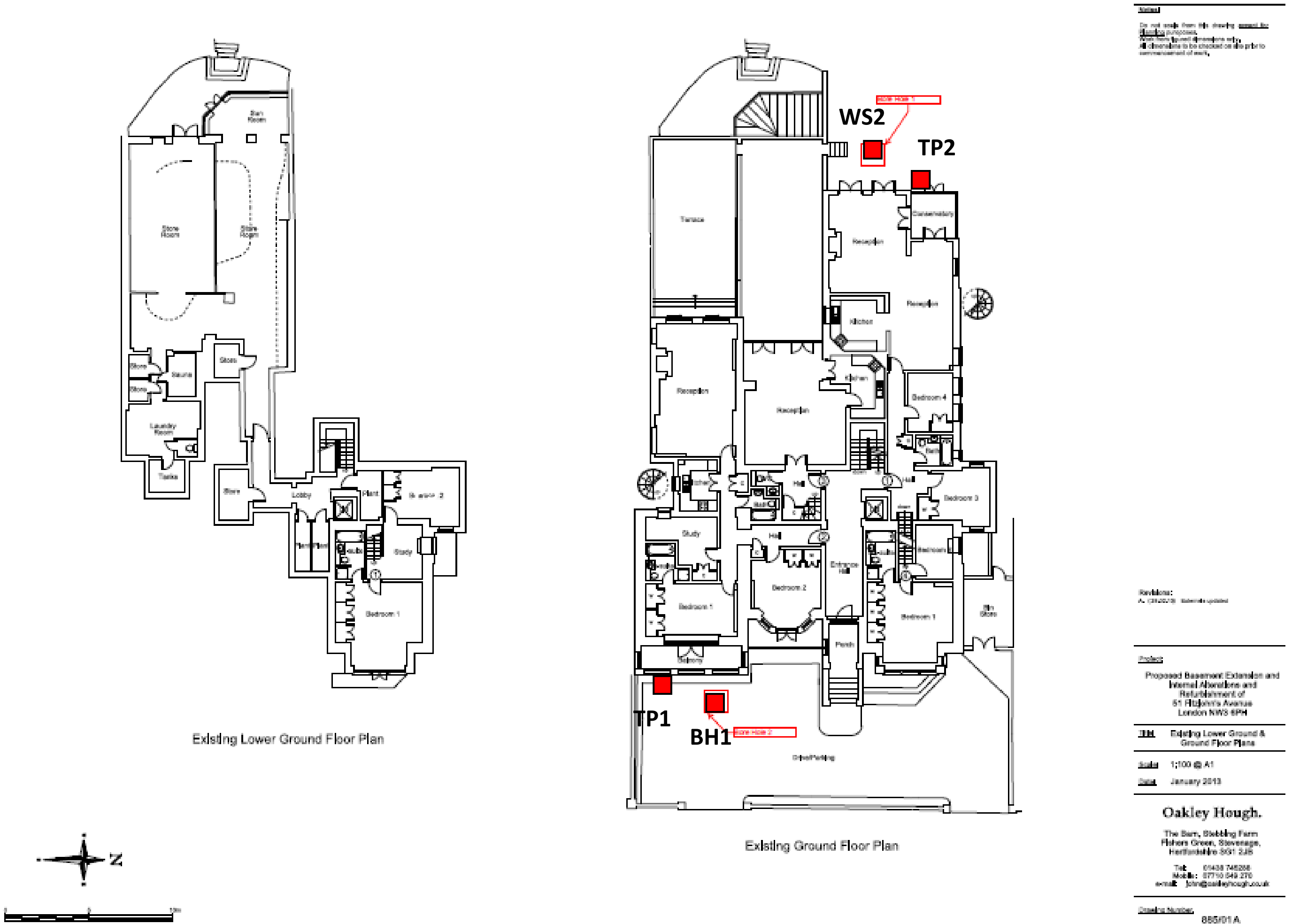
Date:
September 2014

Aerial View of the Site

Ref:
GWPR921

Figure 3

ground&water



Existing Lower Ground Floor Plan

Existing Ground Floor Plan

Notes:
Do not scale from this drawing except for planning purposes.
Work from figured dimensions only.
All dimensions to be checked on site prior to commencement of work.

Revisions:
A. (31/2013) Externals updated

Project:
Proposed Basement Extension and
Internal Alterations and
Refurbishment of
51 Fitzjohn's Avenue
London NW3 6PH

Title: Existing Lower Ground &
Ground Floor Plans

Scale: 1:100 @ A1

Date: January 2013

Oakley Hough.

The Barn, Stebbing Farm
Fishers Green, Stevenage,
Hertfordshire SG1 2JB

Tel: 01438 745288
Mobile: 07710 849 276
e-mail: john@oakleyhough.co.uk

Drawing Number:
885/01 A

Project:
51 Fitzjohn's Avenue, Hampstead, London NW3 6PH

Client:
51 Fitzjohn's Development Limited

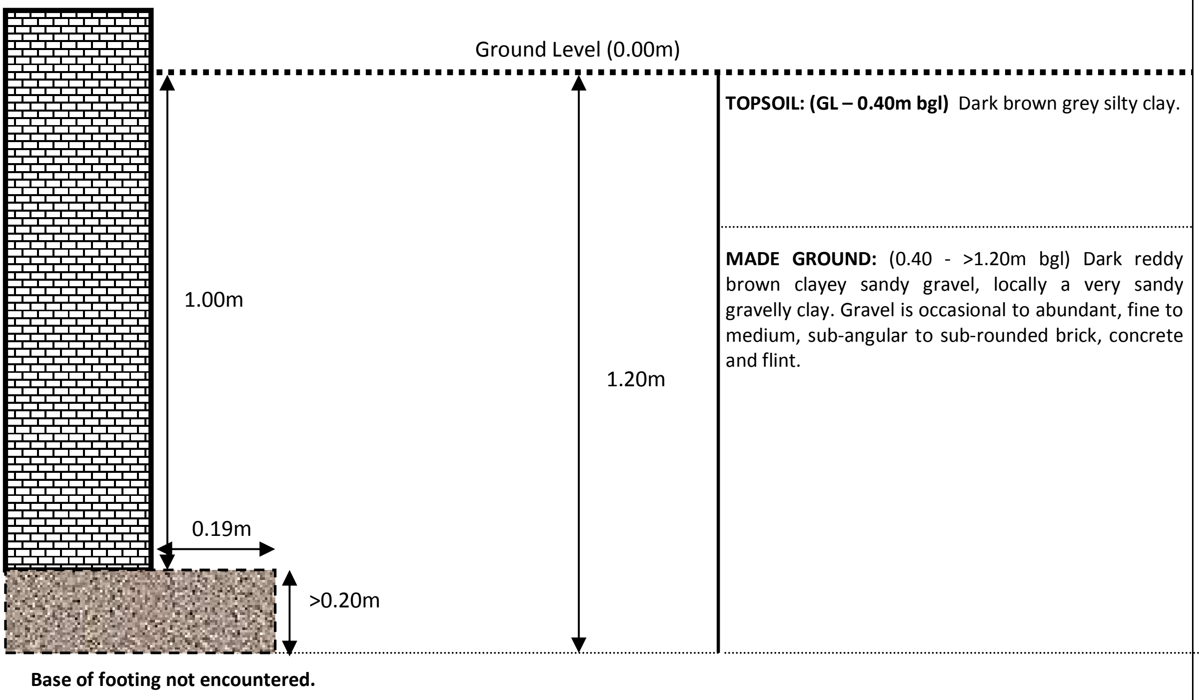
Date:
September 2014

Trial Hole Location Plan

Ref:
GWPR921

Figure 4

ground&water



TOPSOIL: (GL – 0.40m bgl) Dark brown grey silty clay.

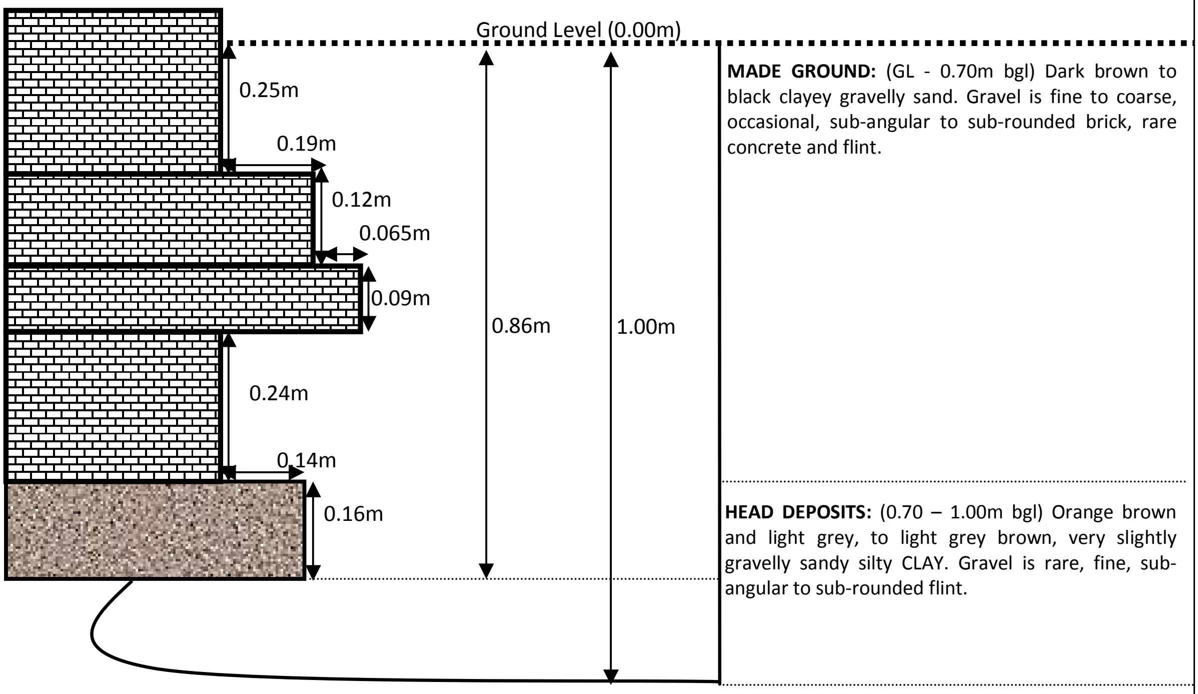
MADE GROUND: (0.40 - >1.20m bgl) Dark reddy brown clayey sandy gravel, locally a very sandy gravelly clay. Gravel is occasional to abundant, fine to medium, sub-angular to sub-rounded brick, concrete and flint.

▦ Brick

▩ Concrete

NOTE: NOT TO SCALE

Project: **51 Fitzjohn's Avenue, London NW3 6PH**		**Figure 5** ground&water
Client: **51 Fitzjohn's Development Limited**	**Date:** **September 2014**	
Section Drawing: Foundation Exposure TP/FE1	**Ref:** **GWPR921**	



MADE GROUND: (GL - 0.70m bgl) Dark brown to black clayey gravelly sand. Gravel is fine to coarse, occasional, sub-angular to sub-rounded brick, rare concrete and flint.

HEAD DEPOSITS: (0.70 – 1.00m bgl) Orange brown and light grey, to light grey brown, very slightly gravelly sandy silty CLAY. Gravel is rare, fine, sub-angular to sub-rounded flint.

▦ Brick

▨ Concrete

NOTE: NOT TO SCALE

Project:
51 Fitzjohn’s Avenue, London NW3 6PH

Client:
51 Fitzjohn’s Development Limited

Date:
September 2014

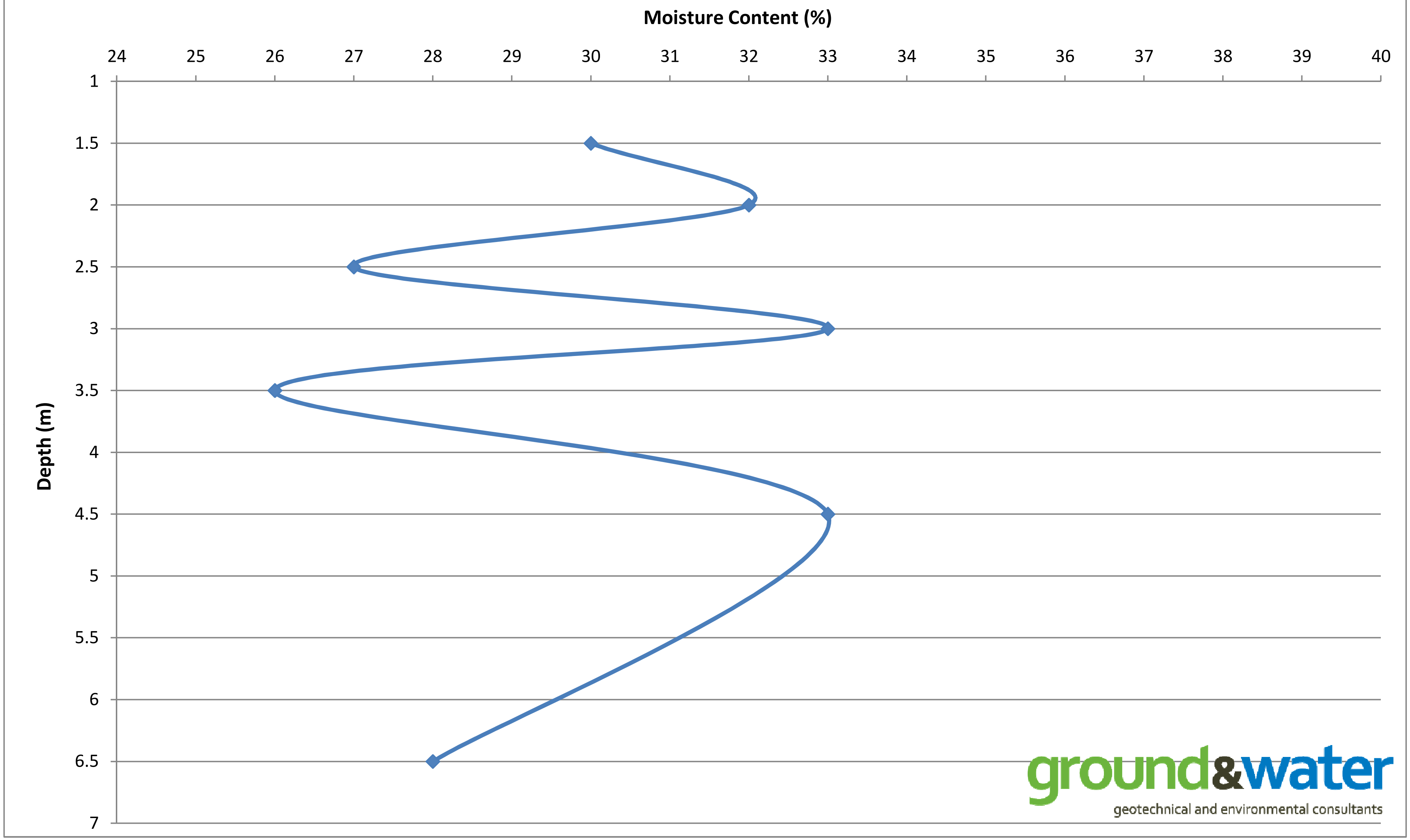
Section Drawing: Foundation Exposure TP/FE2

Ref:
GWPR921

Figure 6

ground&water

Figure 7: Change in Moisture Content With Depth Within BH1



ground&water

geotechnical and environmental consultants

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 51 Fitzjohn’s Avenue, Hampstead, London NW3 6PH.

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.

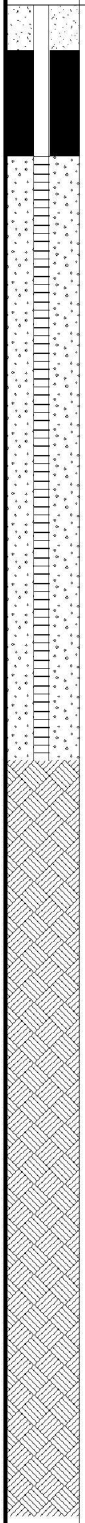
The depth to roots and/or of desiccation may vary from that found during the investigation. The client is responsible for establishing the depth to roots and/or of desiccation on a plot-by-plot basis prior to the construction of foundations. Where trees are mentioned in the text this means existing trees, 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	**BH1**
Sheet	1 of 2
Project Name	51 Fitzjohn's Avenue
Project No.	GWPR921
Co-ords:	-
Hole Type	WLS
Location:	London NW3 6PH
Level:	-
Scale	1:50
Client:	51 Fitzjohn's Developments Limited
Dates:	09/05/2014
Logged By	FW



Water Strikes	Depth (m)	Type	Results
	0.30	D	
	0.50	D	
	0.80	D	
	1.00	SPT	N=4 (1,1/ 1,1,1,1)
	1.00	D	
	1.50	D	
	2.00	SPT	N=5 (1,2/ 1,1,2,1)
	2.00	D	
	2.50	D	
	3.00	SPT	N=13 (2,2/ 3,3,3,4)
	3.00	D	
	3.50	D	
	4.00	SPT	N=13 (2,2/ 3,3,3,4)
	4.00	D	
	4.50	D	
	5.00	SPT	N=9 (3,2/ 3,2,2,2)
	5.00	D	
▽ 5.50	5.50	D	
	6.00	SPT	N=9 (2,2/ 3,2,2,2)
	6.00	D	
	6.50	D	
	7.00	SPT	N=16 (3,3/ 4,4,4,4)
	7.00	D	
	7.50	D	
	8.00	SPT	N=13 (3,4/ 3,3,3,4)
	8.00	D	
	8.50	D	
	9.00	SPT	N=17 (4,4/ 3,5,4,5)
	9.00	D	
	9.50	D	

Depth (m)	Level (m AOD)	Stratum Description
0.30		MADE GROUND: Paving slab over sharp sand.
0.90		MADE GROUND: Dark brown to mid brown, locally orange brown mottled, very slightly gravelly sandy silty clay. Gravel is fine rare, sub-angular to sub-rounded flint and occasional, fine to medium, sub-angular to sub-rounded brick. Sand is fine to coarse grained.
4.30		CLAYGATE MEMBER OF THE LONDON CLAY FORMATION: Mid to dark brown, orange brown and brown grey silty sandy CLAY with dark orange brown fine sand and light to mid grey silt lenses. Sand is fine grained.
		CLAYGATE MEMBER OF THE LONDON CLAY FORMATION: Dark grey silty sandy CLAY with light grey silt and fine sand lenses. Strata becoming less sandy with depth. Sand is fine grained.

Continued next sheet

Remarks: Fresh roots noted to ~1.0m bgl. Possible decaying roots noted to ~2.0m bgl.
Groundwater strike at 5.50m bgl.

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

Borehole No
BH1
Sheet 2 of 2

Project Name 51 Fitzjohn's Avenue	Project No. GWPR921	Co-ords: -	Hole Type WLS
Location: London NW3 6PH		Level: -	Scale 1:50
Client: 51 Fitzjohn's Developments Limited		Dates: 09/05/2014	Logged By FW

Well	Water Strikes	Samples & In Situ Testing Depth (m)	Type	Results	Depth (m)	Level (m AOD)	Legend	Stratum Description
		10.00	SPT	N=23				CLAYGATE MEMBER OF THE LONDON CLAY FORMATION: Dark grey silty sandy CLAY with light grey silt and fine sand lenses. Strata becoming less sandy with depth. Sand is fine grained.
		10.00	D	(4,5/ 6,5,6,6)				
		10.50	D					
		11.00	SPT	N=22				
		11.00	D	(5,5/ 6,5,5,6)				
		11.50	D					
		12.00	SPT	N=26				
		12.00	D	(5,6/ 7,6,7,6)				
					12.45			End of Borehole at 12.45 m

Remarks: Fresh roots noted to ~1.0m bgl. Possible decaying roots noted to ~2.0m bgl.
Groundwater strike at 5.50m bgl.

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

Borehole No
WS2
Sheet 1 of 1

Project Name 51 Fitzjohn's Avenue	Project No. GWPR921	Co-ords: -	Hole Type WS
Location: London NW3 6PH		Level: -	Scale 1:50
Client: 51 Fitzjohn's Developments Limited		Dates: 09/05/2014	Logged By FW

Well	Water Strikes	Samples & In Situ Testing Depth (m)	Type	Results	Depth (m)	Level (m AOD)	Legend	Stratum Description
▨		0.30	D				▨	MADE GROUND: Dark brown to black clayey gravelly sand. Gravel is fine to coarse, occasional, sub-angular to sub-rounded brick and rare concrete and flint. Sand is fine to coarse grained.
		0.50	D					
					0.70			
		0.80	D				▨	HEAD DEPOSITS: Orange brown, with light grey to grey brown mottling, silty sandy gravelly CLAY. Gravel is rare, fine, sub-angular to sub-rounded flint. Sand is fine grained.
		1.00	D					
					1.25			
		1.50	D				▨	CLAYGATE MEMBER OF THE LONDON CLAY FORMATION: Orange brown and brown grey mottled silty sandy CLAY. Sand is fine grained.
					1.80			
		2.00	D				▨	CLAYGATE MEMBER OF THE LONDON CLAY FORMATION: Mid brown, dark orange brown and grey mottled silty sandy CLAY with fine sand and silt lenses and orange brown ferruginous nodules with depth.
		2.50	D					
		3.00	D					
		3.50	D					
		4.00	D		4.00			End of Borehole at 4.00 m

Type	Results

Remarks: Fresh roots noted to ~1.50m bgl.
No groundwater encountered.

APPENDIX C
Geotechnical Laboratory Test Results

Project Name:	51 Fitzjohns Avenue, London NW3 6PH	**Samples Received:**	01/07/2014
		Project Started:	04/08/2014
Client:	Ground and Water Ltd	**Testing Started:**	18/08/2014
Project No:	GWPR921 **Our job/report no:** 17172	**Date Reported:**	20/08/2014

K4 SOILS

Borehole No:	Sample No:	Depth (m)	Description	Moisture content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing 0.425 mm (%)	Remarks
BH1	-	1.50	Brown silty sandy CLAY with occasional rootlets	30					
BH1	-	2.00	Brown mottled orange brown and blue grey sandy CLAY with occasional rootlets	32					
BH1	-	2.50	Brown mottled orange brown and blue grey silty sandy CLAY with occasional rootlets	27					
BH1	-	3.00	Brown and grey sandy silty CLAY	33	44	21	23	100	
BH1	-	3.50	Brown mottled orange brown and blue grey silty sandy CLAY with occasional rootlets	26					
BH1	-	4.50	Dark grey silty sandy CLAY	33					
BH1	-	6.50	Grey sandy silty CLAY	28	53	24	29	100	
BH2	-	2.50	Brown CLAY with blue grey veins and orange brown sandy patches	25	59	24	35	100	

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: 19/08/2014

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)

All samples connected with this report ,incl any on 'hold' will be stored and disposed off according to Company policy.Acopy of this policy is available on request.

K4 SOILS

Project Name:	51 Fitzjohns Avenue, London NW3 6PH		
Client:	Ground and Water Ltd	Project no:	GWPR921
		Our job no:	17172

Borehole No:	Sample No:	Depth m	Description	pH	Sulphate content (g/l)
BH1	-	1.50	Brown silty sandy CLAY with occasional rootlets	7.6	0.09

Date
19/08/2014

Summary of Test Results

BS 1377 : Part 3 :Clause 5 : 1990

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

Checked and Approved
Initials : kp

UKAS TESTING
4480

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: 14-23486

Site Reference:	49/51 Fitzjohns Avenue, London NW3 6PH
Project / Job Ref:	GWPR921
Order No:	None Supplied
Sample Receipt Date:	31/07/2014
Sample Scheduled Date:	31/07/2014
Report Issue Number:	1
Reporting Date:	06/08/2014

Authorised by:

Russell Jarvis
Director
On behalf of QTS Environmental Ltd

Authorised by:

Kevin Old
Director
On behalf of QTS Environmental Ltd

Soil Analysis Certificate						
QTS Environmental Report No: 14-23486	**Date Sampled**	08/05/14	08/05/14	09/05/14		
Ground & Water Ltd	**Time Sampled**	None Supplied	None Supplied	None Supplied		
Site Reference: 49/51 Fitzjohns Avenue, London NW3 6PH	**TP / BH No**	49 BH1	49 BH1	51 BH2		
Project / Job Ref: GWPR921	**Additional Refs**	None Supplied	None Supplied	None Supplied		
Order No: None Supplied	**Depth (m)**	3.00	6.00	8.00		
Reporting Date: 06/08/2014	**QTSE Sample No**	112818	112819	112820		

Determinand	**Unit**	**RL**	**Accreditation**					
pH	pH Units	N/a	**MCERTS**	7.7	7.1	7.3		
Total Sulphate as SO_4	mg/kg	< 200	NONE	3297	3483	2602		
W/S Sulphate as SO4 (2:1)	g/l	< 0.01	**MCERTS**	0.24	0.96	0.82		
Total Sulphur	mg/kg	< 200	NONE	1092	4868	3358		
Ammonium as NH_4	mg/kg	< 0.5	NONE	4.5	3.8	4.2		
W/S Chloride (2:1)	mg/kg	< 1	**MCERTS**	44	23	22		
Water Soluble Nitrate (2:1) as NO_3	mg/kg	< 3	**MCERTS**	< 3	< 3	< 3		
W/S Magnesium	g/l	< 0.0001	NONE	0.0176	0.1130	0.0782		

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: 14-23486
Ground & Water Ltd
Site Reference: 49/51 Fitzjohns Avenue, London NW3 6PH
Project / Job Ref: GWPR921
Order No: None Supplied
Reporting Date: 06/08/2014

QTSE Sample No	TP / BH No	Additional Refs	Depth (m)	Moisture Content (%)	Sample Matrix Description
$ 112818	49 BH1	None Supplied	3.00	17.4	Light brown clayey sand
$ 112819	49 BH1	None Supplied	6.00	16.4	Brown clayey sand
$ 112820	51 BH2	None Supplied	8.00	17.8	Brown clayey sand

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

Soil Analysis Certificate - Methodology & Miscellaneous Information

QTS Environmental Report No: 14-23486
Ground & Water Ltd
Site Reference: 49/51 Fitzjohns Avenue, London NW3 6PH
Project / Job Ref: GWPR921
Order No: None Supplied
Reporting Date: 06/08/2014

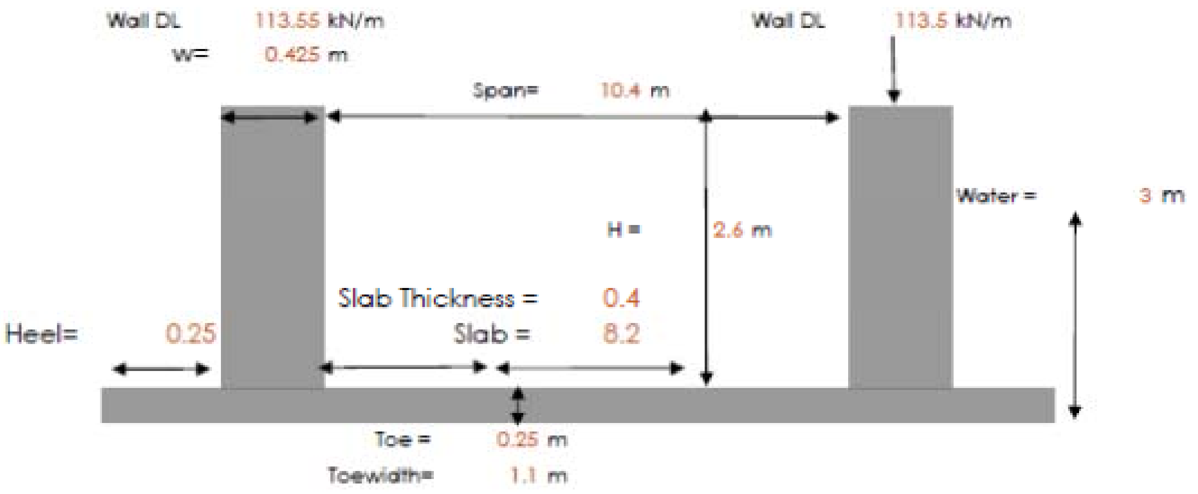
Matrix	Analysed On	Determinand	Brief Method Description	Method No
Soil	D	Boron - Water Soluble	Determination of water soluble boron in soil by 2:1 hot water extract followed by ICP-OES	E012
Soil	AR	BTEX	Determination of BTEX by headspace GC-MS	E001
Soil	D	Cations	Determination of cations in soil by aqua-regia 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

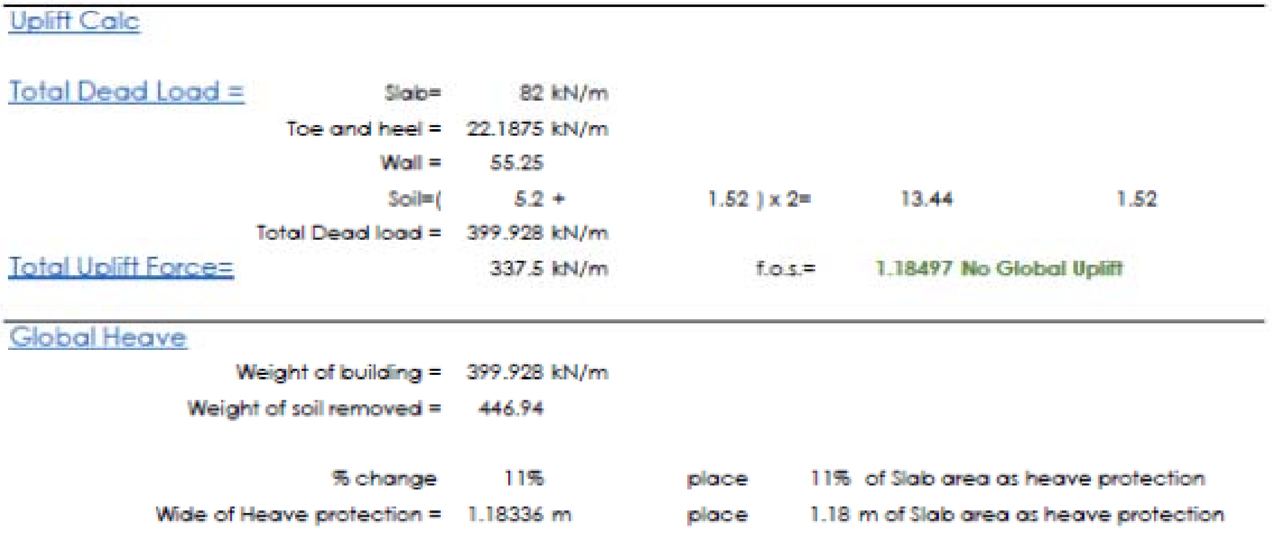
D **Dried**
AR **As Received**

Appendix E

Uplift, Heave & Initial settlement check

Uplift & Heave calculation





Uplift Calc

Total Dead Load = Slab= 82 kN/m
Toe and heel = 22.1875 kN/m
Wall = 55.25
Soil=(5.2 + 1.52) x 2= 13.44 1.52
Total Dead load = 399.928 kN/m

Total Uplift Force= 337.5 kN/m f.o.s.= 1.18497 No Global Uplift

Global Heave

Weight of building = 399.928 kN/m
Weight of soil removed = 446.94

% change 11% place 11% of Slab area as heave protection
Wide of Heave protection = 1.18336 m place 1.18 m of Slab area as heave protection

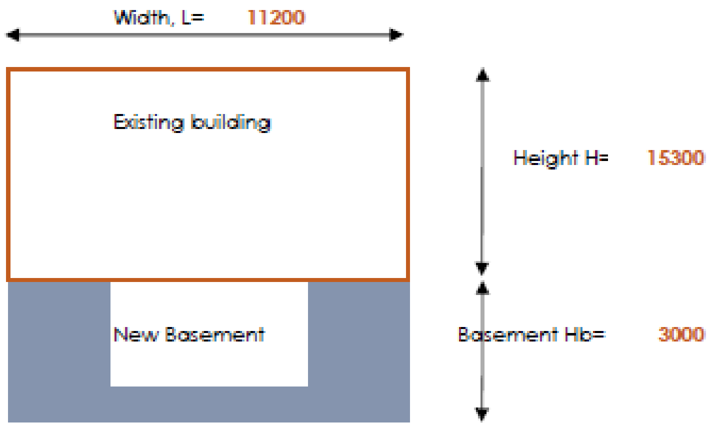
As per soil investigation, the bearing capacity of less than 45kN/m² may result in heave. The load underneath the RC retaining walls is greater; therefore there is no need for a heave protection under. An RC ground bearing slab span will be broken by introducing the thickening to the slab. The thickening must be loaded with a load greater than the 45kN/m.

Load from RC ground bearing slab	W = (24kN/m^3 × 0.25m + 1.5kN/m^2) × 7.5m / 2 = **28.125**kN/m
Required width of the foundation	D = 0.4m
Bearing pressure under the foundation	σf = W / D = **70.313**kN/m^2
Soil bearing pressure;	σ = 125kN/m^2

The proposed thickening will be 400mm wide. Clayboard heave protection to be introduced underneath the rest of the ground bearing slab.

Initial Settlement check

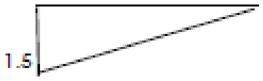
L/H = 0.73203



Horizontal movement Assessment CIRIA C580: Embedded Retaining walls - Guide to Ecomonic Design

Potential Movement Due to wall installation

Horizontal surface movement = 0.05%
DeltaH = 0.05% x 3000 = 1.5 mm



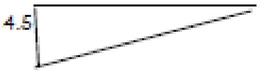
Vertical Surface Movement = 0.05%
Delta V = 0.05% x 3000 = 1.5 mm

= 0.33333 mm/m

Distance behind wall wall to neglibible movement
lh = 3000 x 1.5 = 4500 mm

Potential Movement Due to wall Excavation

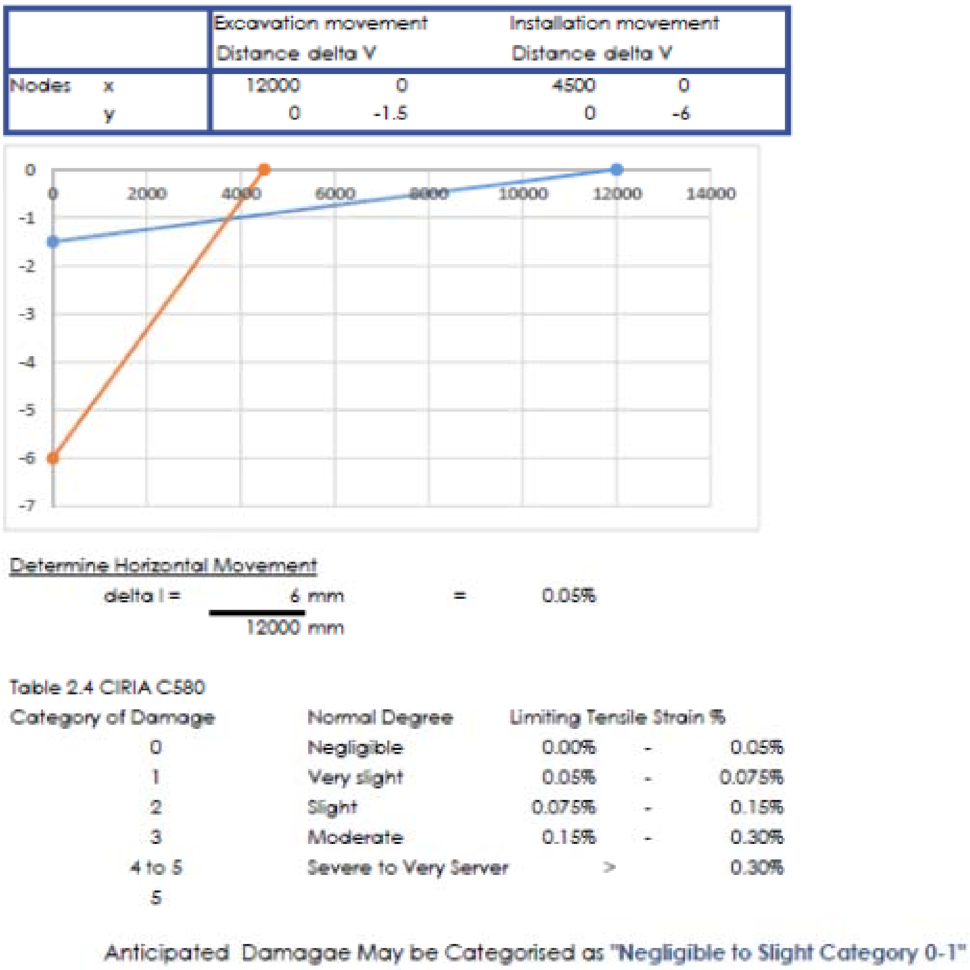
Horizontal surface movement = 0.15%
DeltaH = 0.15% x 3000 = 4.5 mm



= 0.375 mm/m

Vertical Surface Movement = 0.10%
Delta V = 0.10% x 3000 = 3 mm

Distance behind wall wall to neglibible movement
lh = 3000 x 4 = 12000 mm



		Excavation movement		Installation movement	
		Distance	delta V	Distance	delta V
Nodes	x	12000	0	4500	0
	y	0	-1.5	0	-6

Determine Horizontal Movement

delta l = 6 mm / 12000 mm = 0.05%

Table 2.4 CIRIA C580

Category of Damage	Normal Degree	Limiting Tensile Strain %		
0	Negligible	0.00%	-	0.05%
1	Very slight	0.05%	-	0.075%
2	Slight	0.075%	-	0.15%
3	Moderate	0.15%	-	0.30%
4 to 5	Severe to Very Server		>	0.30%
5				

Anticipated Damagae May be Categorised as "Negligible to Slight Category 0-1"