Job Number:140407
22/04/14

CROFT
STRUCTURAL
ENGINEERS

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Basement Structural Method Statement

Property Details
25 Oakhill Avenue
Hampstead
NW3 7RD

Client Information
Mr Roy Hay

Structural Design Reviewed by	**Above Ground Drainage Reviewed by**
Chris Tomlin MEng CEng MIStructE	Phil Henry BEng MEng MICE

Revision	Date	Comment
-	23/06/14	First Issue for Comment
1	10/07/14	Updated to reflect Architect's comments

Table of Contents

1. Design Information - Structural

Structural Summary	This property is an existing semi-detached three-storey house with a loft. At the moment it consists of ground floor, first floor and second floor. The ground floor has an extended reception room facing rear garden. The general construction is masonry load bearing external walls with internal brick wall and stud timber walls. The floors are all built of timber and supported on load bearing walls. The timber roof is also supported by the load bearing external walls. **Proposed works** The proposed works require the insertion of a new lower ground floor under part of the property at the rear. This will be constructed in reinforced concrete retaining walls underpinning the existing walls. Croft Structural Engineers Ltd Structural Engineers has extensive knowledge of inserting 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 to allow for 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. **Structural Defects Noted** On the second floor, there are minor diagonal cracks on the internal wall. The level of cracking was consistent with a property of this age and construction and not considered a significant structural concern. The underpinning of the structure will place the footings on better ground. There are cracks on the ceiling on the second floor. This was through to be the brittle render resulting from the over stressed roof. No defects were noted at the front and the back external walls of the property, indicating there was no significant movement. The underpinning of the structure will place the footings on better ground.
Intended use of structure and user requirements	Family/domestic use

DP27	
A	**Maintain Structural Stability of the building & Neighbouring Properties.** The attached drawing shows the reinforcement and construction required by maintaining stability of the property, the neighbouring buildings (The adjacent Garden Wall has also been considered). Calculations results are shown in the Stage 4 – Impact Assessment
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 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 Stage 4 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 has been considered in Stage 4
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

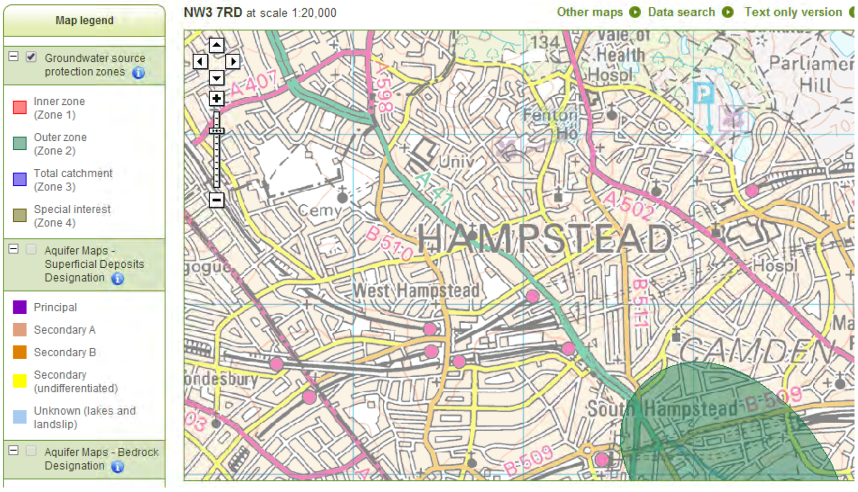
The questions below are taken from the Camden CPG 4 – Basements and Lightwells.

Groundwater flow

Figure 1 – Subterranean flow screening chart

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?

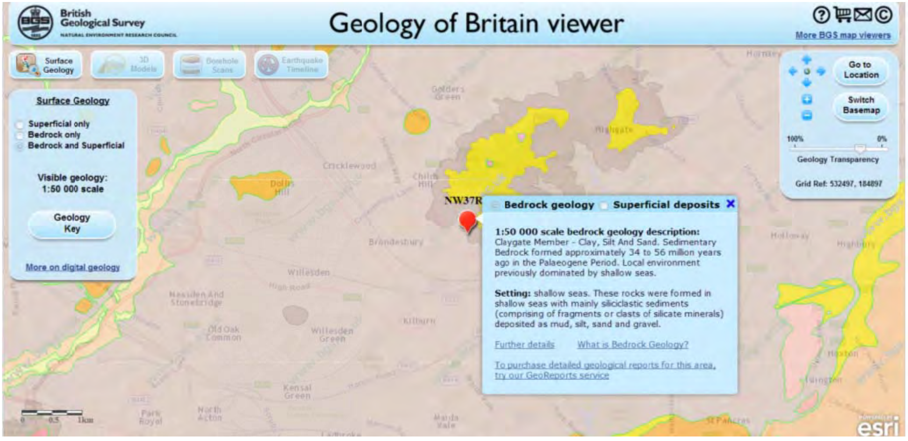
Unknown. Full geotechnical investigation has been conducted and this will show the findings in the scoping stage.

Carry forward to scoping stage.

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

OS maps and local walkover survey show no wells, watercourses.

The site is within 100m of the boundary of the Claygate and London clay interface that may act as a potential spring line.



Bedrock geology ☐ **Superficial deposits**

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

Carry forward to scoping stage.

3. Is the site within the catchment of the pond chains on Hampstead Heath?
No. The site lies outside the areas denoted by figure 13 of the Arup report.

Source – City of London, 2010, Welcome to Hampstead heath Leaflet

Camden Geological, Hydrogeological and Hydrological Study
Hampstead Heath Map

213923 FIGURE 13

Do not Carry forward to scoping stage.

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. The side extension is already covered.

Do not Carry forward to scoping stage.

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.

Do not Carry forward to scoping stage.

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?
No. From walkover and OS maps, there are no local ponds or springs of significance.

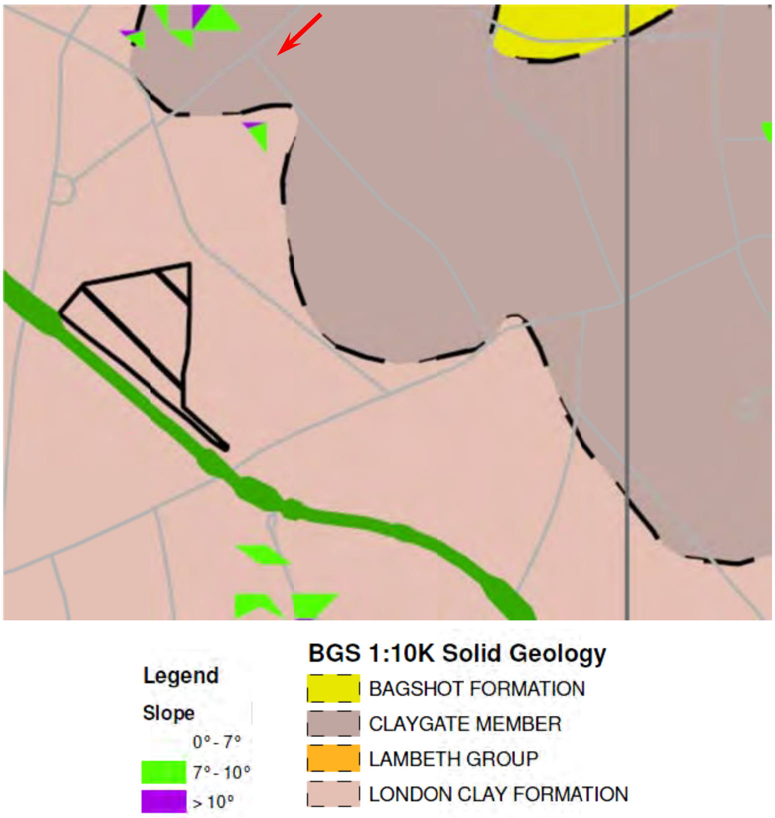
Do not Carry forward to scoping stage.

Slope Stability

Figure 2 – Slope Stability screening flowchart

1. Does the existing site include slopes, natural or man made greater than 7^{o} (approximately 1 in 8)?

No. Difference in height between the rear garden and front is less than 1 in 8 slope.



(Arup Report Figure 16)

<u>Do not Carry forward to scoping stage.</u>

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

No. Proposed landscaping does not affect the slope.

<u>Do not Carry forward to scoping stage.</u>

3. Does the development neighbour land including railway cuttings and the like with a slope greater than 7^{o} (approximately 1 in 8)?

No. Proposed landscaping does not affect the slope.

<u>Do not Carry forward to scoping stage.</u>

4. Is the site within a wider hillside setting in which the general slope is greater than 7^{o} (approximately 1 in 8)?

No. The slope of the wider hillside setting is as per the property, less than 7^{o}. From Figure 16 the slope angle is shown less than 7^{o}

<u>Do not Carry forward to scoping stage.</u>

5. Is the London Clay the shallowest strata on site?

Yes. The site sits on the Claygate beds part of the London Clay formation. The Claygate beds sit above the London Clay.

Carry forward to scoping stage.

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.

Do not 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. The site is founded on the London Clay and the area has a history of subsidence. But from the visual site inspection it showed that there were no signs of movement and only minor cracks possible from thermal effects.

Carry forward to scoping stage.

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

No. OS maps and local walkover survey show no wells, watercourses. The site is within 100m of the boundary of the Claygate and London clay interface.

Carry forward to scoping stage.

9. Is the site within an area of previously worked ground?

No. From the historical maps, the site has been residential for the past 100 years.

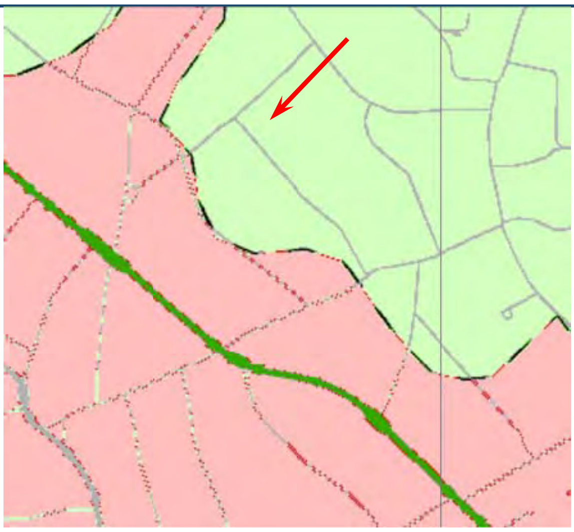
Carry forward to scoping stage: Soil investigation to be completed to confirm the ground conditions.

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.

However the Arups report shows the site to be with in the secondary A Aquifer. This is perched water on top of the permeable clays.

Carry forward to scoping stage.: Soil investigation to be completed.

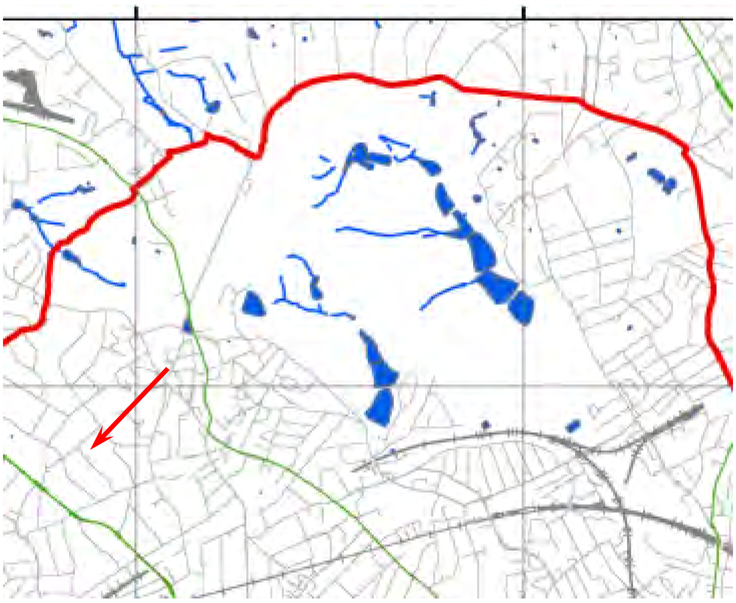


(Extract Arup Report Fig 8)

Carry forward to scoping stage.

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

(Arup Report Figure 12)



<u>Do not Carry forward to scoping stage.</u>

12. Is the site within 5m of a highway or pedestrian footway?
No. Site is not within 5m of the footpath/alleyway.

13. Will the proposed basement significantly increase the differential depth of foundations relative to the neighbouring properties?
Yes. Party wall will be underpinned. Existing footings are expected to be corbelled masonry approx. 600mm below ground level.

Carry forward to scoping stage.: Overall design to be considered.

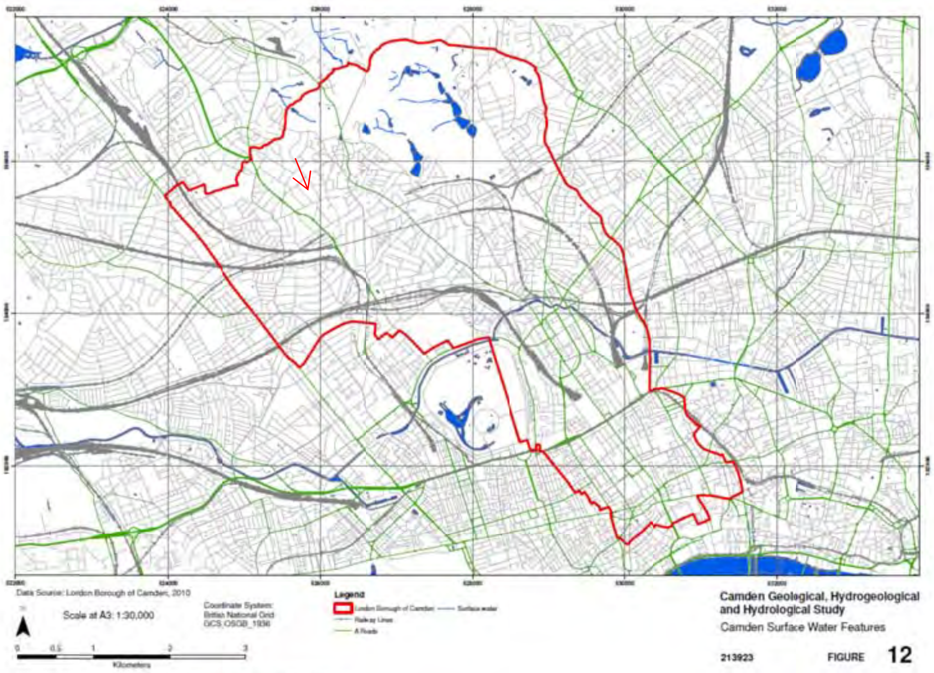
14. Is the site over (or within the exclusion zone) of any tunnels, e.g. railway lines?

No. Nearest is the LUL Line, >500m from site.

<u>Do not Carry forward to scoping stage.</u>

Surface flow and flooding

Figure 12 & 15 – Surface flow and flooding screening flowchart



Camden Geological, Hydrogeological and Hydrological Study
Camden Surface Water Features

213923 FIGURE 12

Figure 5 from Core Strategy, London Borough of Camden

Camden Geological, Hydrogeological and Hydrological Study
Flood Map

213923 FIGURE 15

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 14of the Camden Hydrological Study
<u>Do not Carry forward to scoping stage.</u>

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 surface water flows remains unchanged.
<u>Do not Carry forward to scoping stage.</u>

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

<u>Do not Carry forward to scoping stage.</u>

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.

<u>Do not Carry forward to scoping stage.</u>

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.

<u>Do not Carry forward to scoping stage.</u>

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?
25 Oakhill Avenue is not noted on the previously flooded streets in Camden.
The basement may be below the static water level and investigation is required.
In the appendix is the flood risk report which does not highlight elevated flood risks.
– This is not a ground water report as such this information will be clarified by the ground and water in their report.

3. Basement Impact: Scoping

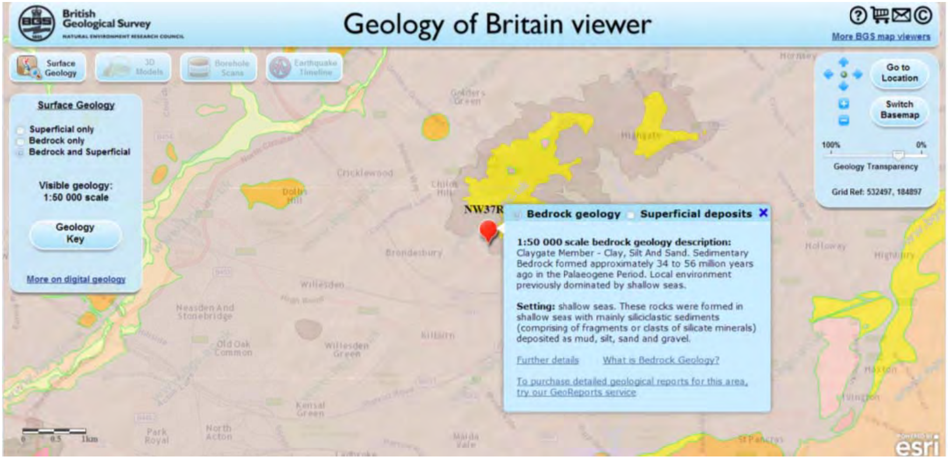
Groundwater flow	**Subterranean flow** Soil investigation has been completed with bore holes. The bores holes have a stand pipe inserted to confirm the water level. Water table noted at 6.4m BGL.
Slope Stability	The soil investigation found that Claygate beds to depth. The slope stability of the beds are in the region of 30˚. The design of the RC retaining walls will take this into account. The basement is not within 5m of the footpath; the retaining walls will be designed with a 10kN/m^2 surcharge. As party wall is to be underpinned and will leave the party wall with a deeper footing than the neighbours other walls, the design should look at the available bearing capacity. As part of the Party Wall agreement a pre-condition survey will be carried out. The design will consider the impact of the deeper footings.
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

A soil investigation has been carried out and shows that the soils are Claygate member of the London Clay Formation and no water has been encountered 7.8m below ground level.

The North London Geological Maps Indicates the site is underlain by London Claygate member. This is as expected in the area.



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?

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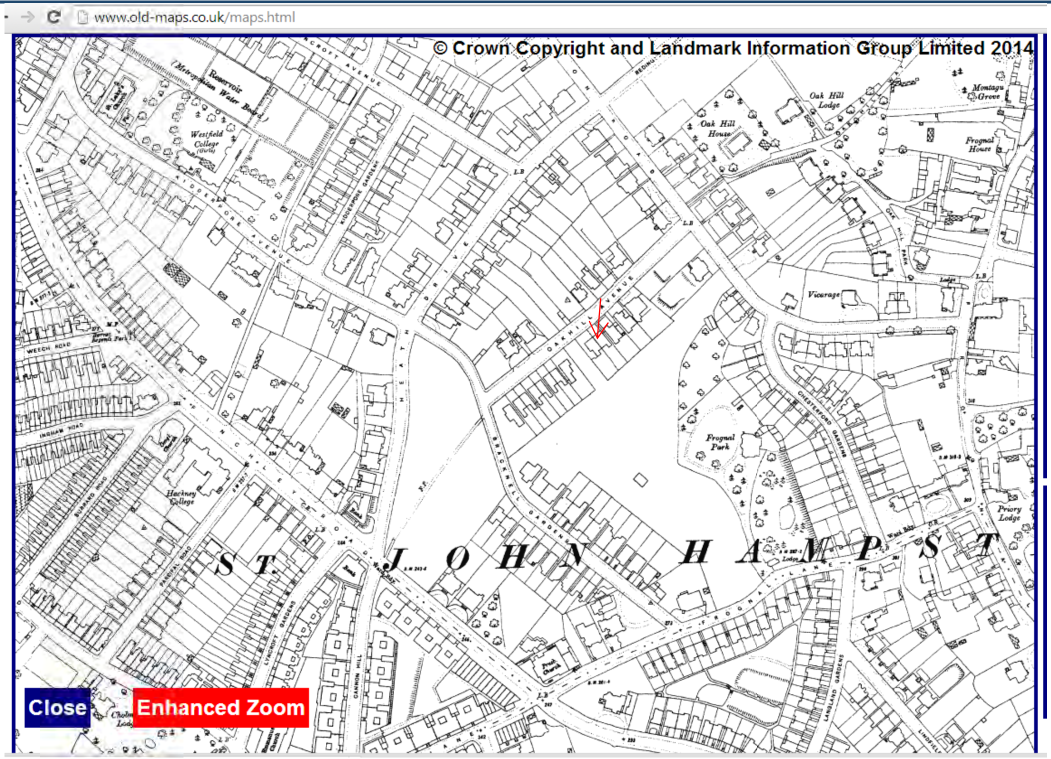
Figure 1 Extract From British Geological Survey Drift Sheet

Walk over Survey

	The existing building did not exhibit signs of minor cracking. The building is semi-detached and the effects of the development on the adjacent properties will need to be considered. Visual inspections were noted of the adjacent properties. No signs of movement were noted to the adjacent buildings. **At present, movement does not appear to be a concern to the local properties.** From Camden Council website, No. 23 and No. 27 Oakhill Avenue have been indicated that they both have their own basement to the rear, see Appendix F. Further investigation will be carried out by inspecting neighbour's property to find out their existing basement depth in the final design stage. After discussions with the neighbours we are aware of plans are being considered for a basement to number 27
Vicinity of Trees	No major trees noted.
Drainage effects on Structure	No build over agreements known
Underground	The site is >500m from the underground, but not within 65m of any LUL asset and therefore the effect of the basement is not considered significant.
Listed Building	Is the building listed? **Yes.** From Britishlistedbuildings.co.uk it is found out that this buiding is listed as Grade II. Are the adjacent buildings listed? **Yes.** From Britishlistedbuildings.co.uk it is found out that No 23 and 27 are both listed as Grade II.
Sources of Contaminates	From the Historic Maps it can be seen that the ground use was farming then residential, and has not been conducive to activities leading to poor ground or contamination. During the walk over survey no sources were noted that may lead to contamination.

	 1915 Pre-WWII Map
Water Course	No water course is shown on the historic maps attached. 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.

7. Site Investigation

Monitoring and Reporting

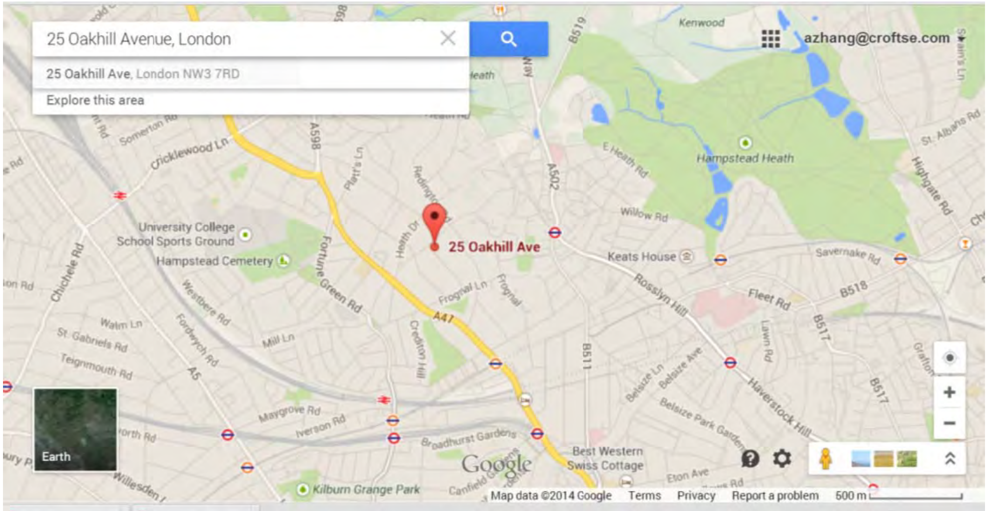
The Soil investigation was completed by Ground & Water.

From the Scoping stage we considered that their brief should cover:

- Two trial pits to the front side and rear to confirm the existing foundations. The purpose is to consider the effect of the works on the neighbouring properties and the find the ground conditions below the site.
- Two bore holes has been completed on this site. One is 7.8m below ground in front of the property and the other is 5m below ground in the back garden.
- Stand pipe to be inserted to monitor ground water; record initial strike and the water level.
- Site testing to determine insitu soil parameter. SPT testing to be undertaken.
- Laboratory testing to confirm soil make up and properties.
- The Historic maps and walk over survey did not highlight any significant contamination sources, therefore no site test of the ground has been requested.
- Factual Report on soil conditions.
- Calculation of Bearing pressures from SPT.
- Indication of Ø (angle of friction) from SPT.
- Indication of soil type
- The hydrogeological effects of the site.

See Appendix E for Soil report

8. OS Map extract showing location of Railway


25 Oakhill Avenue, London

25 Oakhill Ave, London NW3 7RD

Explore this area

9. Impact Assessment

Subterranean flow

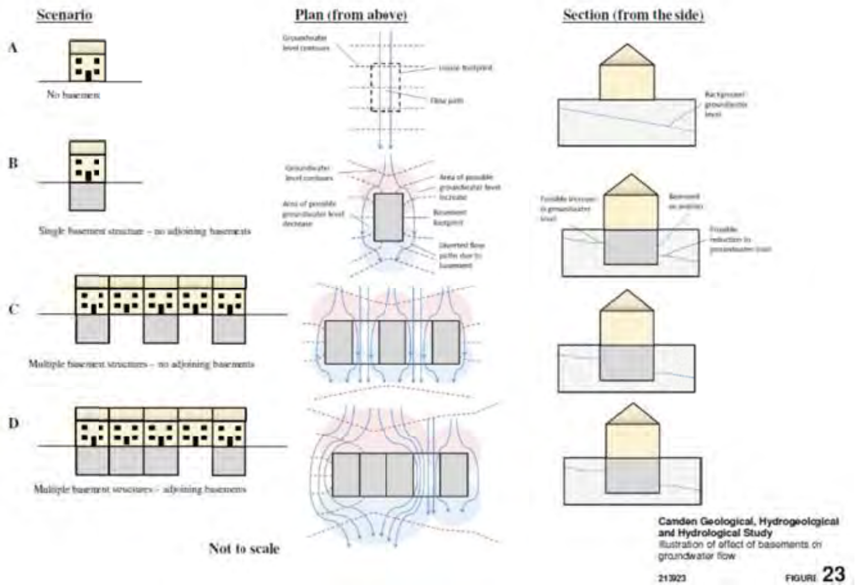
The site is not within the catchment of the Hampstead Heath Ponds. It is a considerable distance from the ponds and standing water courses in the area.

The development will not have an impact on the Hampstead heath ponds nor their catchement.

The proposed development depth is expected to be at 3.6m below external ground floor level.

The site investigation indicated that the water was encountered at 6.4m below ground. This is below the level of the basement, but was also taken in the summer. It is possible that the ground water may rise in the coming months and fluctuate throughout the year.

The local affect of the basement will be to divert any flowing ground water away from the foot print of the building. To the front side and rear of the property large areas over 10m wide are present. With a large dispersal area for the flow to be diverted around the affects on the surrounding area will be minimal.



Without field testing in the neighbouring properties or along the road there is a low residual risk that the ground wall flow may affect the external ground.

The basement design must allow for variants in ground water. The retaining walls must be designed to provide lateral resistance to water

	up to 1m from the top of the wall. The design must follow the recommendations as noted in BS8102. ***As no water was found in the bores a full hydrology report is not suitable.*** ***To allow for through flow of ground water the drawings SS03 shows a 150mm compacted Type (i) under the central slab. This will help though flow of any ground water that may build up around the edge of the building.***
Slope Stability	From the walk over survey, the OS map and the Arups report the slopes around the site are less than 7°. 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. Concrete will be exposed to sulphate attacks due to the surrounding soils. Concrete will be designed in accordance with the recommendations of T=the Building Research Establishment Special Digest 1, 2005, ‘Concrete in Aggressive Ground’. The designs for the retaining walls have been calculated using Finite element software designed by Masterseries. 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 retaining wall design and deflections check of walls 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 Investigations have highlight that water is a present. The walls are designed to cope with the hydrostatic pressure. The water table was low. The design of the walls however considers the long term items. It is possible that a water main may break causing local high water table. To account for this the wall is designed for water 1m from the top of the 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.

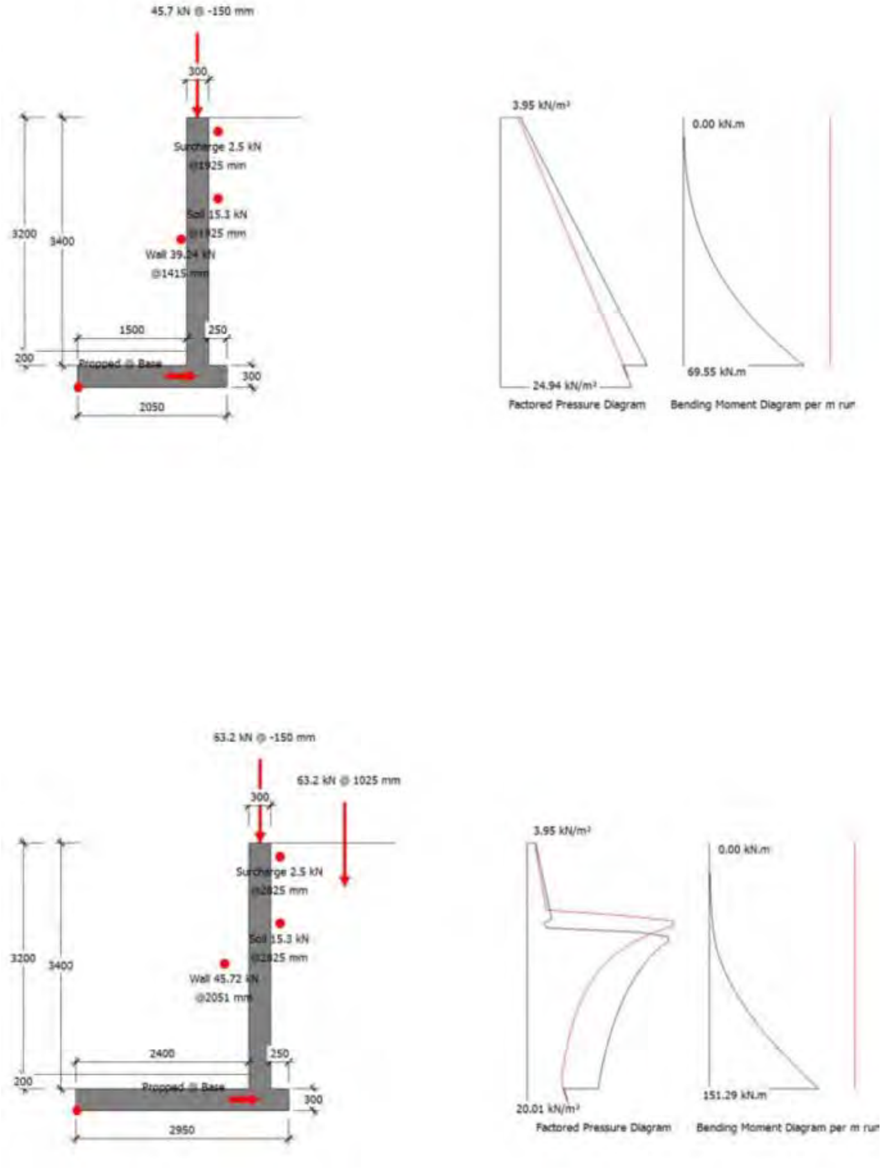


Figure 2 Loadings and Lateral stress patterns

<table>
<tr><td>Vicinity of Trees</td><td>No major trees nearby.</td></tr>
<tr><td>Special precautions due to trees</td><td>Basement depth will allow for footings to be placed outside the effects of the trees.</td></tr>
<tr><td>Drainage effects on Structure</td><td>No build over agreements known of.
Flooding. The site is not in an area of high risk flooding.</td></tr>
<tr><td>Roads</td><td>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.
5kN/m² to front light well
Garden Surcharge 2.5kN/m²
Surcharge for adjacent property 1.5kN/m² + 4kN/m² for concrete ground bearing slab</td></tr>
<tr><td>Intended use of structure and user requirements</td><td>Family/domestic use</td></tr>
<tr><td>Loading Requirements (EC1-1)</td><td>
<table>
<tr><th></th><th>UDL kN/m²</th><th>Concentrated Loads kN</th></tr>
<tr><td>**Domestic Single Dwellings**</td><td>1.5</td><td>2.0</td></tr>
<tr><td colspan="3">The basement does not line within a 45º angle of the highway. Therefore Highways HA loading is not required to be applied.</td></tr>
</table>
</td></tr>
<tr><td>Number of Storeys</td><td>3 storeys + loft becoming 4 storeys + loft
Is Live Load Reduction included in design No</td></tr>
</table>

<table>
<tr><td>Progressive Collapse</td><td>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 1</td><td>Single occupancy houses not exceeding 4 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>Existing Building Class</td><td>1</td></tr>
<tr><td>Proposed Building Class</td><td>1</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>Class1 – Design to satisfy EN 1990 to EN 1999 stability requirements</td></tr>
</table>

Lateral Stability Exposure and wind loading conditions	Basic wind speed Vb = 21 m/s to EC1-2 Site level +75.000 m above sea level. Topography not considered significant.
Stability Design	The cantilevered walls are suitable to carry the lateral loading applied from above
Lateral Actions	The soil loads apply a lateral load on the retaining walls. Hydrostatic pressure will be applied to the wall Imposed loading will surcharge the wall.

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.

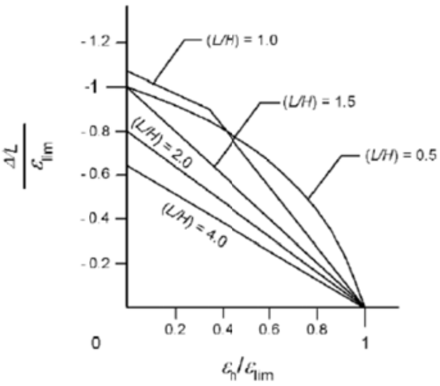
The design of the retaining walls is completed to K_O lateral design stress values. This increases the design stresses on the concrete retaining walls and limits the overall deflection of the retaining wall.

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.

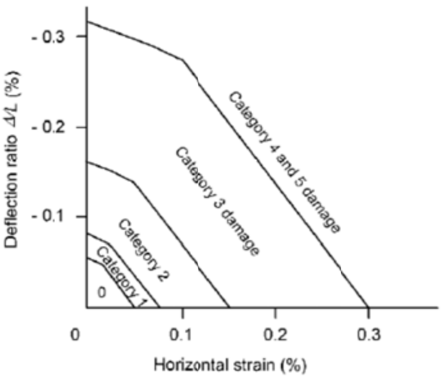
To reduce the risk the development:

- 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 be 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.



(b) Influence of horizontal strain on Δ/L / ε_{lim} (after Burland, 2001)



(c) Relationship between damage category and deflection ratio and horizontal tensile strain for hogging for (L/H) = 1.0 (after Burland, 2001)

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	Limiting Tensile strain	Definitions of cracks and repair types/considerations
0	**Up to 0.1**	0.0-0.05	**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	0.05-0.075	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	0.075-0.015	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	0.15-	SERIOUS – Internal cracks repaired as for

		0.3	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	>0.3	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, and second floor levels at rear as shown on the attached sketch.

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.

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

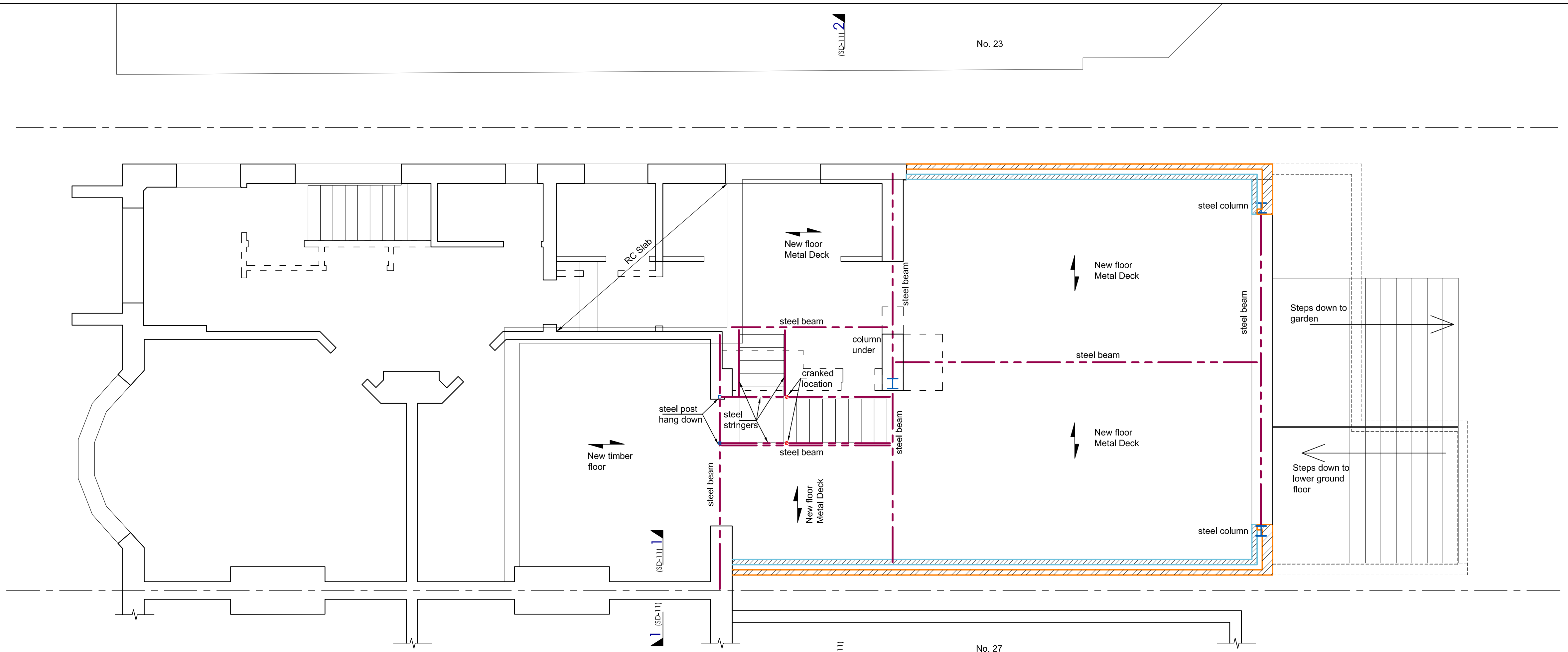
Noise and Nuisance	The contractor is to follow the good working practices and guidance laid down in the "Considerate Constructors Scheme". The hours of working will be limited to those allowed; 8am to 5pm Monday to Friday and Saturday Morning 8am to 1pm. None of the practices cause undue noise that one 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.

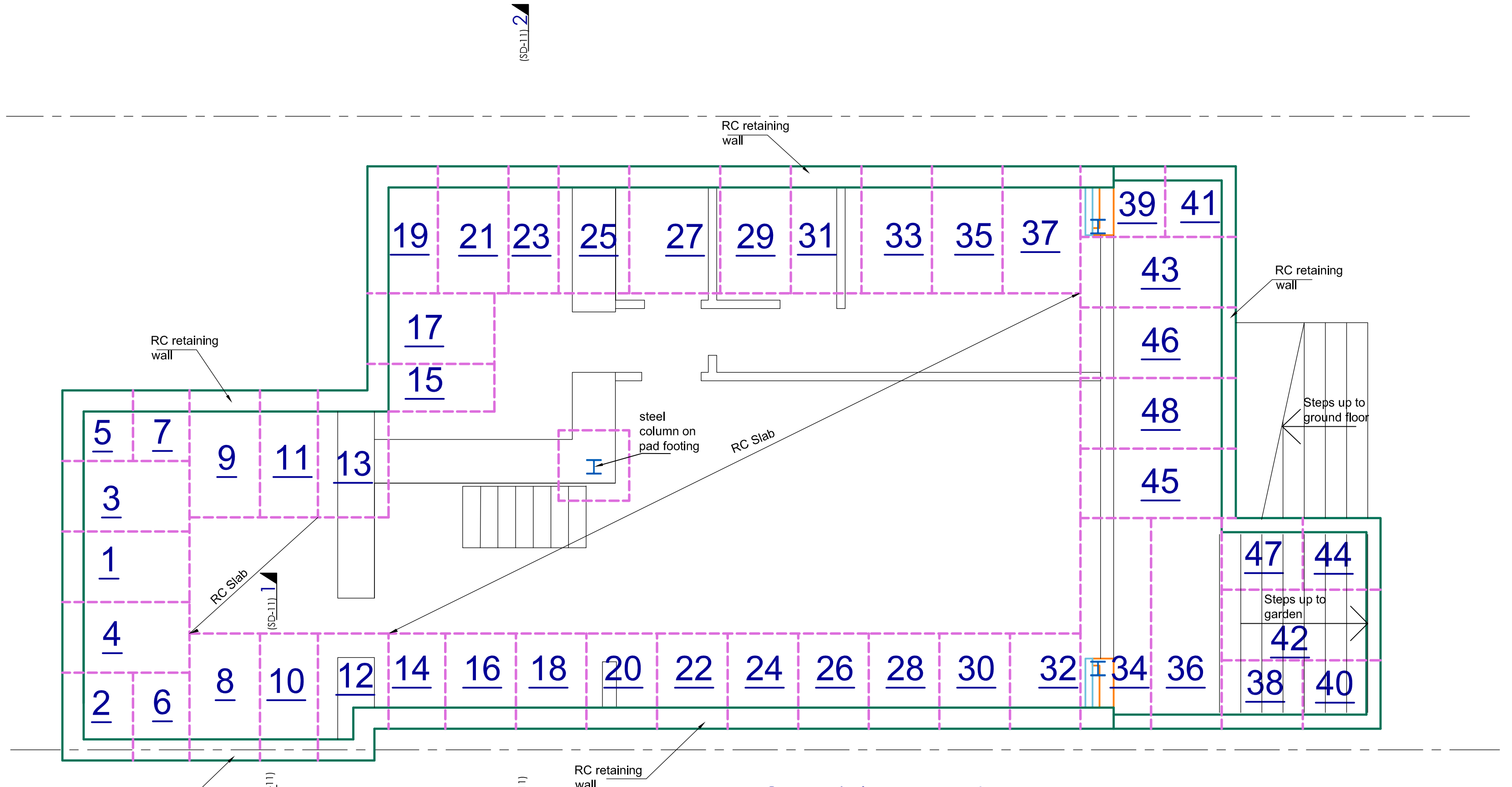
Appendix A

Structural Scheme Drawings

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



Ground Floor Layout
Scale 1:50



1 (SD-11)

2 (SD-11)

Lower Ground Floor Layout
Scale 1:50

PRELIMINARY SCHEME DESIGN
- NOT FOR CONSTRUCTION

Rev	Date	Amendments
-	23.6.14	First issue for BIA

Croft
Structural
Engineers

Clockshop Mews,
r/o 60 Saxon Rd,
London, SE25 5EH.

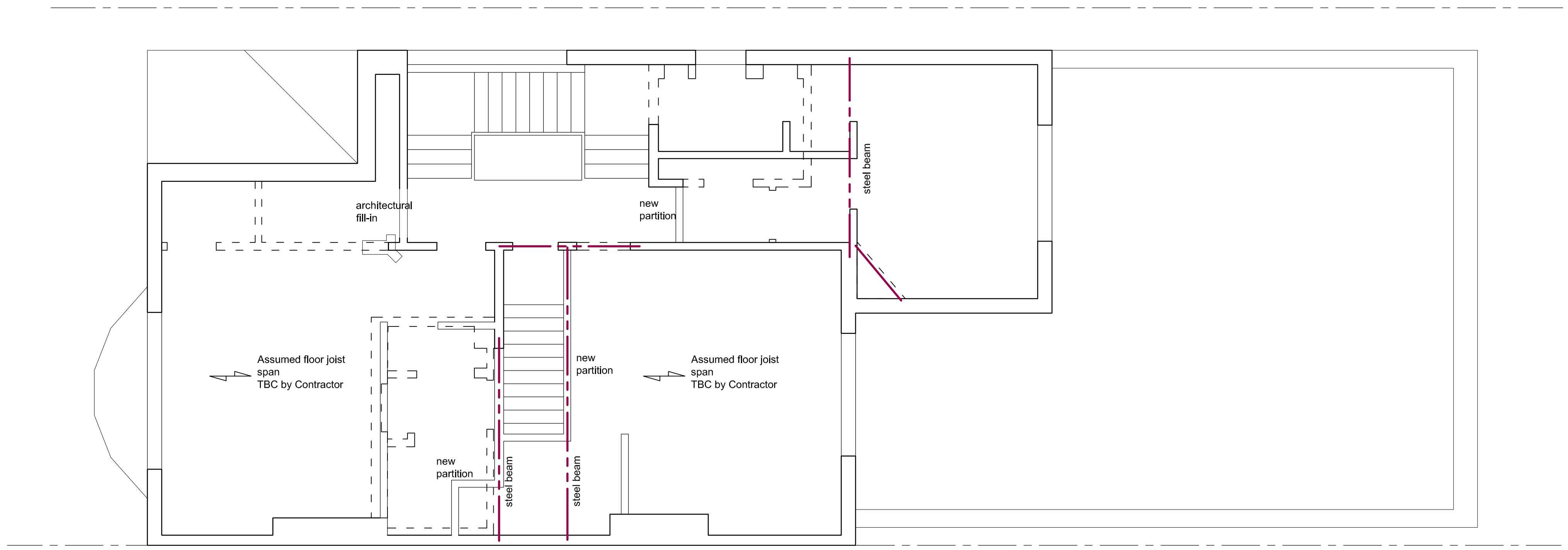
020 8684 4744
www.croftse.co.uk

Client: MR Roy Hay

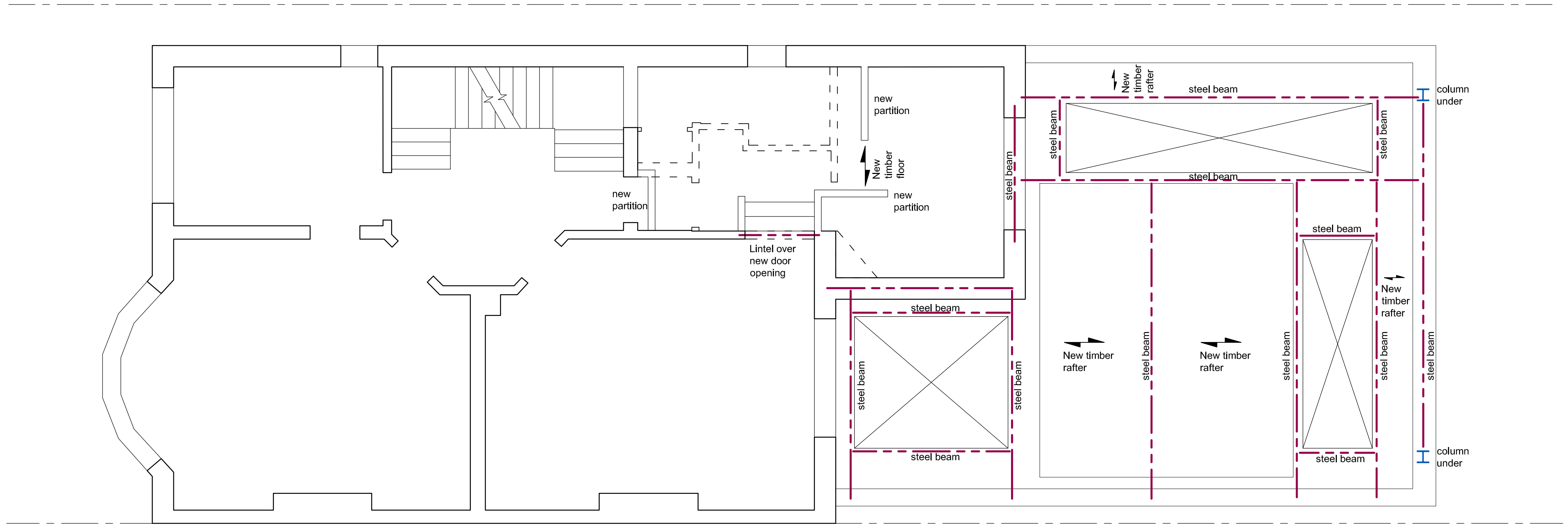
Project: 25 Oakhill Avenue

Title : Lower ground &
Ground Floors

Job nos 140407	drawn AZ Chk'd CT	Scale 1: 50 @ A1
Dwg Nos SL-10	Rev -	date June 14



Second Floor Layout
Scale 1:50



First Floor Layout
Scale 1:50

PRELIMINARY SCHEME DESIGN
- NOT FOR CONSTRUCTION

Rev	Date	Amendments
-	23.6.14	First issue for BIA

Croft
Structural
Engineers
Clockshop Mews,
r/o 60 Saxon Rd,
London, SE25 5EH.

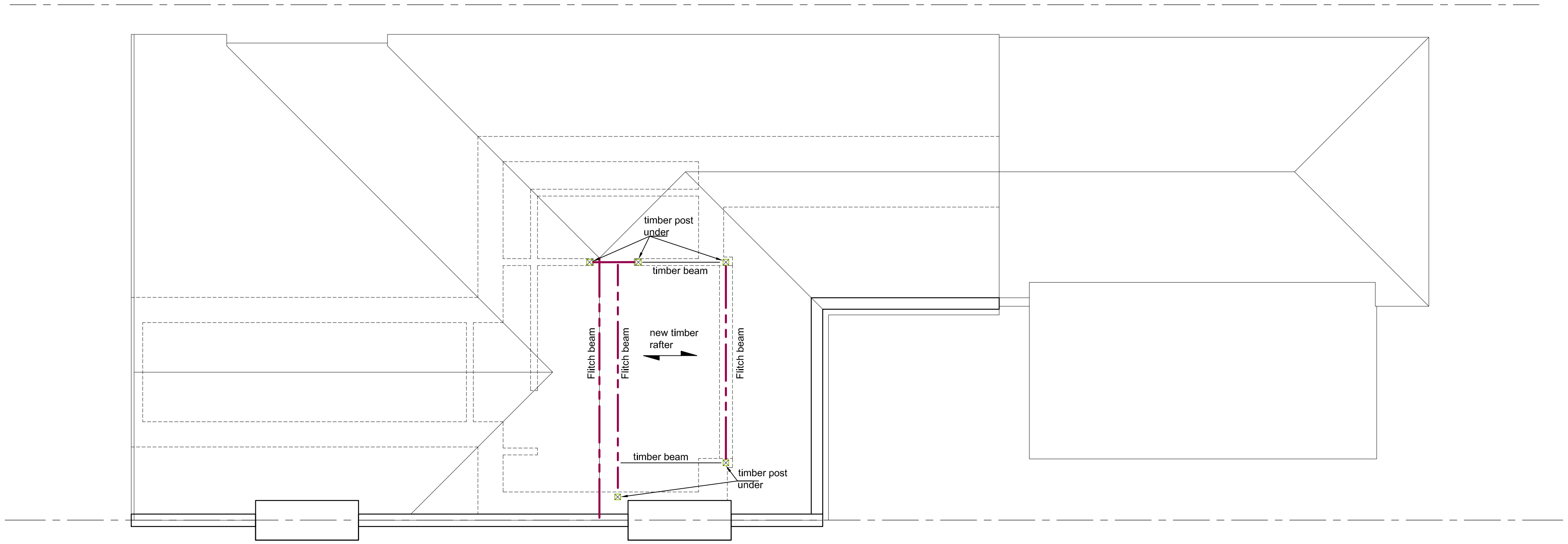
020 8684 4744
www.croftse.co.uk

Client: MR Roy Hay

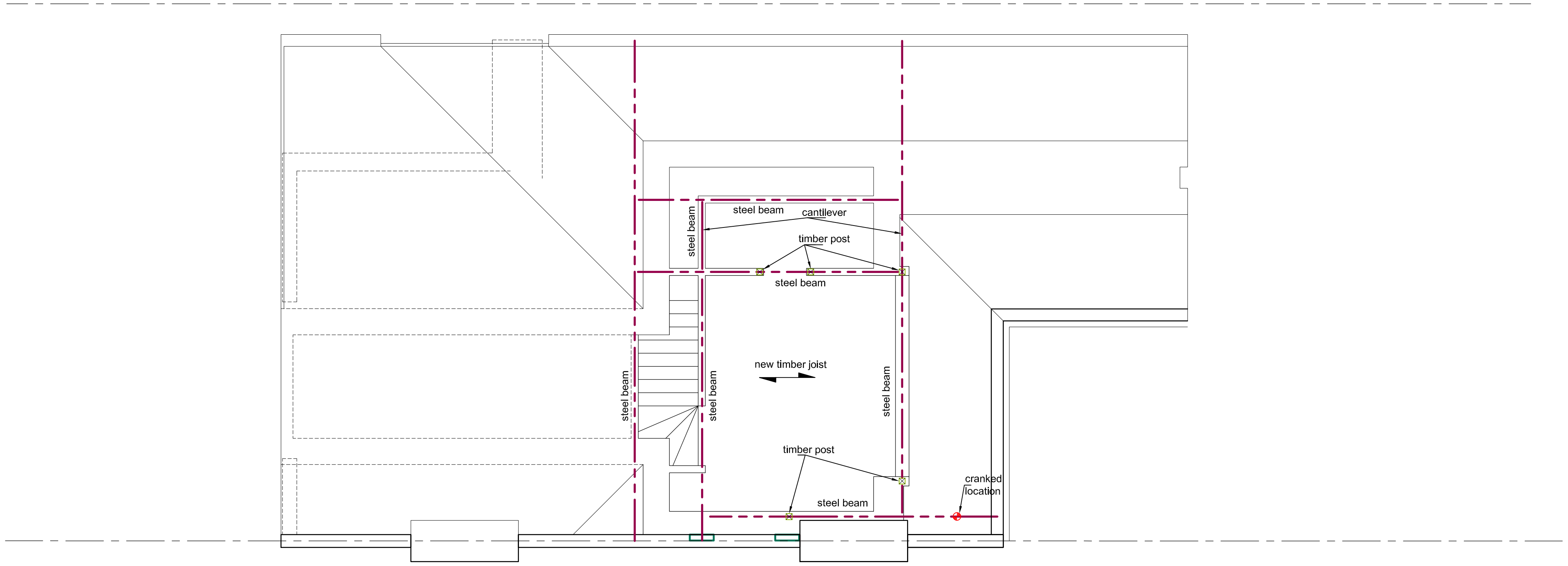
Project: 25 Oakhill Avenue

Title : First & Second Floors

Job nos 140407	drawn AZ Chk'd CT	Scale 1: 50 @ A1
Dwg Nos SL-11	Rev -	date June 14



Roof Layout
Scale 1:50



Loft Layout
Scale 1:50

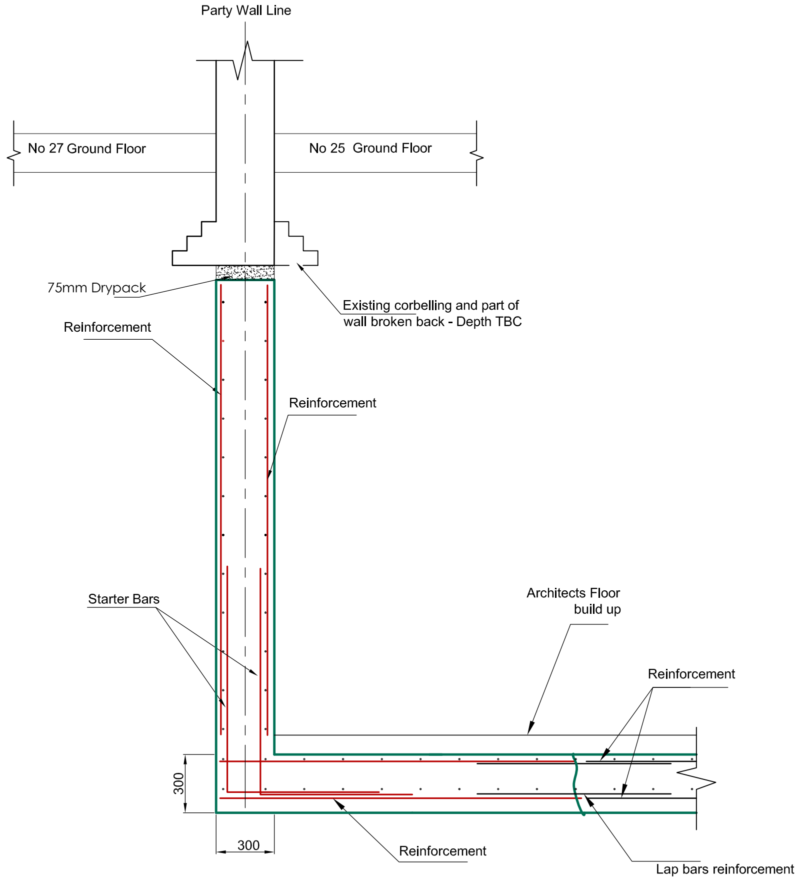
PRELIMINARY SCHEME DESIGN
- NOT FOR CONSTRUCTION

Rev	Date	Amendments
-	23.6.14	First issue for BIA

Croft
Structural
Engineers
Clockshop Mews,
r/o 60 Saxon Rd,
London, SE25 5EH.
020 8684 4744
www.croftse.co.uk

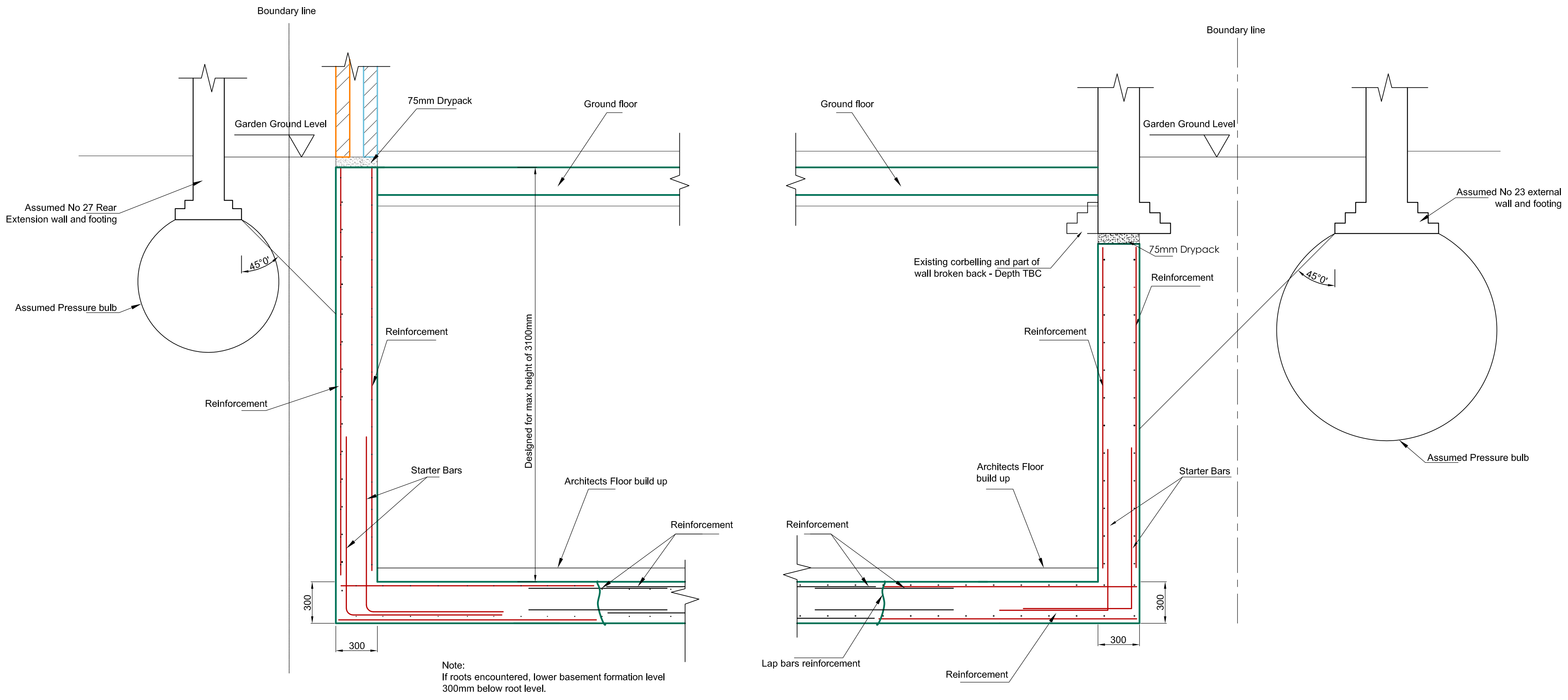
Client: MR Roy Hay
Project: 25 Oakhill Avenue
Title : Loft & Roof Floors

Job nos 140407	drawn AZ Chk'd CT	Scale 1: 50 @ A1
Dwg Nos SL-12	Rev -	date June 14



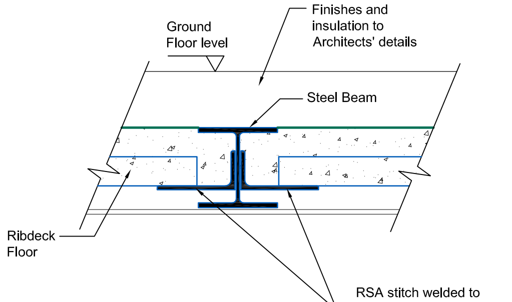
Note:
If roots encountered, lower basement formation level 300mm below root level.

Section 1-1
Scale 1:20



Note:
If roots encountered, lower basement formation level 300mm below root level.

Section 2-2
Scale 1:20



Metal Deck
(1:10)

PRELIMINARY SCHEME DESIGN
- NOT FOR CONSTRUCTION

Rev	Date	Amendments
-	23.6.14	First issue for BIA

Croft
Structural
Engineers
Clockshop Mews,
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020 8684 4744
www.croftse.co.uk

Client: MR Roy Hay
Project: 25 Oakhill Avenue
Title : Sections

Job nos	drawn	Scale
140407	AZ	1: 20 @ A1
	Chk'd CT	
Dwg Nos	Rev	date
SL-13	-	June 14

Appendix B

Structural Basement Calculations

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

Engineering Information Sheet/ Loadings

enquiries@croftse.co.uk

CROFT STRUCTURAL ENGINEERS

0208 684 4744

Project: 25 Oakhill Avenue

Section 01

Sheet 01

Date Apr-14

By AZ

Cheked CT

Rev

Date

Description

Job Number 140407

Status

Rev

Reference

General Loadings

Sloped Roof

Slate = 0.6 kN/m^2
Battens = 0.02
50x150@400c/c = 0.1125
Felt = 0.02
Insulation = 0.02
Plaster= 0.18
0.9525 **kN/m2**
Roof Angle = 30 deg
Plan Dead load = 1.1 kN/m2
Live Load = 0.6 kN/m2

Flat Roof

20mm Asphalt = 0.46
Felt underlay = 0.02
insulation = 0.04
Ply Sheeting = 0.1
Firring = 0.1
Roof joists 50x200@400 = 0.15
Plaster & Skim = 0.18
Plan Dead load = 1.05 kN/m2
Live Load = 0.75 kN/m2

Mansard Roof

Slate Tiles = 0.4
Battens = 0.02
Ply Sheeting = 0.125
Rafters = 0.125
100 Insulation = 0.06
plaster & Skim = 0.18
Felt = 0.02
0.93
Roof Angle = 75 deg
Plan Dead load = 3.596 kN/m2
Live Load = 0 kN/m2

Cavity Walls

100 Facing Brick = 2.2
100 Block (16kN/m3)= 1.6
Plaster & Skim = 0.18
Dead Load = **3.98 kN/m2**

Internal Walls

100 Block (20kN/m3)= 2
Plaster & Skim = 0.36
Dead Load = 2.36 kN/m2

Existing Internal Walls

100 Brick (20kN/m3)= 2.1
Plaster & Skim = 0.36
Dead Load = 2.46 kN/m2

Timber Floors

18mm Ply 0.15
Joists 50x225@400 = 0.16875
100 Insulation = 0.05
Plaster & Skim = 0.18
Dead Load = 0.54875 kN/m2
Live Load = 1.5 kN/m2

Terrace Floor

Promonade Tiles = 0.4
20mm Asphalt = 0.46
Felt underlay = 0.02
insulation = 0.04
Ply Sheeting = 0.1
Firring = 0.1
Roof joists 50x200@400 = 0.175
Plaster & Skim = 0.18
Dead Load = 1.475 kN/m2
Live Load = 1.5 kN/m2

Ceiling

50x100 Joists = 0.075
100 Insulation = 0.06
Plaster & Skim = 0.18
Dead Load = 0.315 kN/m2
Live Load = 0.25 kN/m2

Timber Partitions

50x100 Studs @ 400 = 0.15
Insulation = 0.04
Plaster & Skim = 0.36
Dead Load = 0.55

Existing Brick Walls

225 Facing Brick = 4.5
Plaster & Lathe = 0.15
Dead Load = 4.65

Metal Deck

Ribdeck 80 140 thick= 2.93
Screed = 1.20
Insulation = 0.07
Finishes = 0.05
Ceiling 0.25
Perm., g_k = 4.50
Var., q_k = 1.50

Ground RC slab

225mm thk slab 5.625
Insulation 0.07
Screed = 1.2
Ceiling 0.05
Dead Load = 6.945
Live Load = 2.5

Existing Brick Walls

300 Facing Brick = 6
Plaster & Lathe = 0.15
Dead Load = 6.15

Flat roof : Sedum

Sedum system= 1
Felt underlay = 0.02
insulation = 0.04
Ply Sheeting = 0.1
Firring = 0.1
Roof joists 50x200@400 = 0.15
Plaster & Skim = 0.18
Plan Dead load = 1.59
Live Load = 0.75

Table 3 Live Load Reduction

Area		Floors	
	0 0%		1 0%
	50 5%		2 10%
	100 10%		3 20%
	150 15%		4 30%
	200 20%		5 to 10 40%

Engineering Information Sheet/ Load Run Down

enquiries@croftse.co.uk

CROFT
STRUCTURAL
ENGINEERS

0208 684 4744

Project: 25 Oakhill Avenue
Section
Sheet

Date Apr-14
By AZ
Cheked CT

Rev Date Description

Job Number 140407
Status
Rev

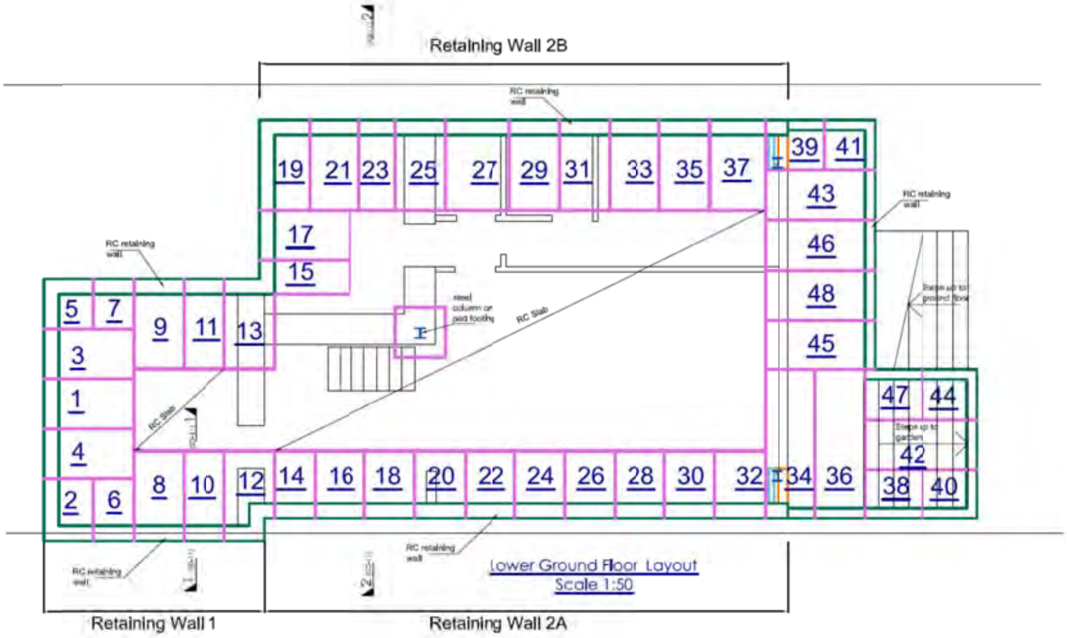
Reference
Load Run Downs

Location	Area			Type	L	Load kN/m2	Load kN			
	L	W	m2				Dead	%	Live	Total
Retaining wall 1										
roof	8.5	0.5	4.25	DL	1	1.10	9.4			
incl. adjacent buidin				LL		0.60		0.4	3.1	
Loft	5	0.5	2.5	DL	1	0.60	3.0			
incl. adjacent buidin				LL		1.50		0.4	4.5	
2nd floor	5	0.5	2.5	DL	1	0.60	3.0			
incl. adjacent buidin				LL		1.50		0.4	4.5	
1st floor	5	0.5	2.5	DL	1	0.60	3.0			
incl. adjacent buidin				LL		1.50		0.4	4.5	
ground floor	0	0.5	0	DL	1	4.50	0.0			
incl. adjacent buidin				LL		1.50		0.4	0.0	
masonry wall	3.55	1	3.55	DL	1	6.15	21.8			
masonry wall	8.95	1	8.95	DL	1	4.65	41.6			
							81.8	kN/m	16.6	kN/m
Retaining wall 2A										
sedum roof	5.5	0.5	2.75	DL	1	1.59	4.4			
				LL		0.75			2.1	
ground floor	4	0.5	2	DL	1	4.50	9.0			
				LL		1.50			3.0	
cavity wall	4.1	1	4.1	DL	1	3.98	16.3			
							29.7	kN/m	5.1	kN/m
surcharge from No 27's rear extension										
acting 950mm away from retaining wall										
flat roof	5.5	0.5	2.75	DL	1	1.05	2.9			
				LL		0.75			2.1	
ground floor	5.5	0.5	2.75	DL	1	4.50	12.4			
				LL		1.50			4.1	
cavity wall	4.1	1	4.1	DL	1	3.98	16.3			
							31.6	kN/m	6.2	kN/m
Equivalent surcharge load										
distance from retaining wall			0.95	m			19.9	kN/m		
Retaining wall 2B										
sedum roof	5.5	0.5	2.75	DL	1	1.59	4.4			
				LL		0.75			2.1	
ground floor	4	0.5	2	DL	1	4.50	9.0			
				LL		1.50			3.0	
cavity wall	4.1	1	4.1	DL	1	3.98	16.3			
							29.7	kN/m	5.1	kN/m
surcharge from No 23's rear extension										
flat roof	5.5	0.5	2.75	DL	1	1.05	2.9			

				LL		0.75			2.1	
1st floor	5.5	0.5	2.75	DL	1	0.55	1.5			
				LL		1.50			4.1	
ground floor	5.5	0.5	2.75	DL	1	4.50	12.4			
				LL		1.50			4.1	
cavity wall	6	1	6	DL	1	3.98	23.9			
							40.7	kN/m	10.3	kN/m
Equivalent surcharge load										
distance from retaining wall			2.1	m			12.1	kN/m		

Croft Structural Engineers Rear of 60 Saxon Rd Selhurst SE25 5EH	Project 25 Oakhill Avenue				Job Ref. 140407	
	Section Retaining wall				Sheet no./rev. 1	
	Calc. by AZ	Date 20/06/2014	Chk'd by CT	Date	App'd by	Date

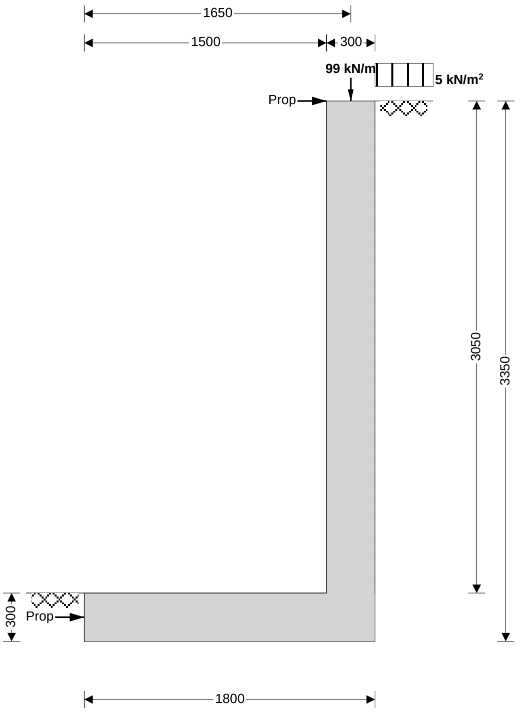
BASEMENT LAYOUT



RETAINING WALL 1 - TEMPORARY CASE

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type	**Cantilever**		
Height of wall stem	h_{stem} = **3050** mm	Wall stem thickness	t_{wall} = **300** mm
Length of toe	l_{toe} = **1500** mm	Length of heel	l_{heel} = **0** mm
Overall length of base	l_{base} = **1800** mm	Base thickness	t_{base} = **300** mm
Height of retaining wall	h_{wall} = **3350** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **300** mm
Position of downstand	l_{ds} = **900** mm		
Depth of cover in front of wall	d_{cover} = **0** mm	Unplanned excavation depth	d_{exc} = **0** mm
Height of ground water	h_{water} = **0** mm	Density of water	γ_{water} = 9.81 kN/m^3
Density of wall construction	γ_{wall} = **23.6** kN/m^3	Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of soil surface	β = **0.0** deg	Effective height at back of wall	h_{eff} = **3350** mm
Mobilisation factor	M = **1.5**		
Moist density	γ_m = **18.0** kN/m^3	Saturated density	γ_s = **21.0** kN/m^3
Design shear strength	ϕ' = **24.2** deg	Angle of wall friction	δ = **0.0** deg
Design shear strength	ϕ'_b = **24.2** deg	Design base friction	δ_b = **18.6** deg

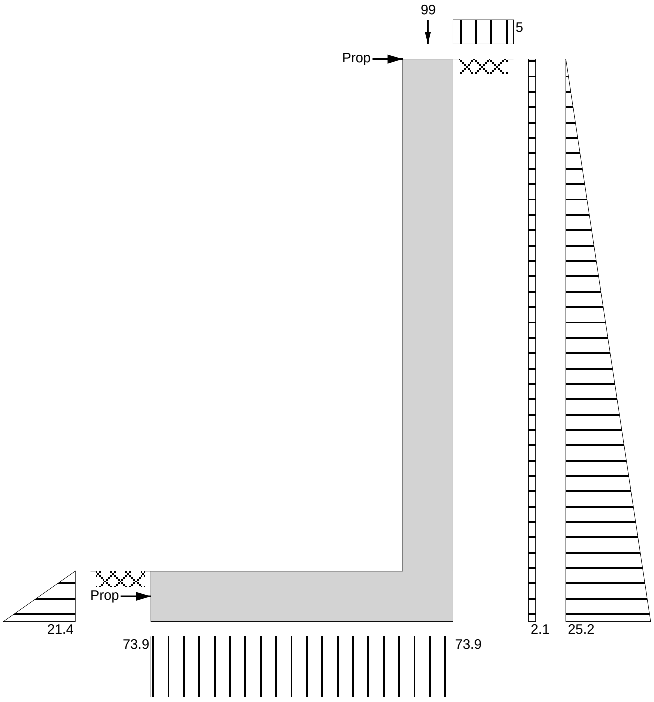
Moist density	γ_{mb} = **18.0** kN/m^3	Allowable bearing	$P_{bearing}$ = **100** kN/m^2

Using Coulomb theory

Active pressure	K_a =**0.419**	Passive pressure	K_p = **4.187**
At-rest pressure	K_0 = **0.590**		

Loading details

Surcharge load	Surcharge = **5.0** kN/m^2		
Vertical dead load	W_{dead} = **82.0** kN/m	Vertical live load	W_{live} = **16.6** kN/m
Horizontal dead load	F_{dead} = **0.0** kN/m	Horizontal live load	F_{live} = **0.0** kN/m
Position of vertical load	l_{load} = **1650** mm	Height of horizontal load	h_{load} = **0** mm



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

Calculate propping force

Propping force F_{prop} = **6.9** kN/m

Check bearing pressure

Total vertical reaction	R = **132.9** kN/m	Distance to reaction	x_{bar} = **900** mm
Eccentricity of reaction	e = **0** mm		

Reaction acts within middle third of base

Bearing pressure at toe	p_{toe} = **73.9** kN/m^2	Bearing pressure at heel	p_{heel} = **73.9** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

Calculate propping forces to top and base of wall

Propping force to top of wall	F_{prop_top} = **-1.515** kN/m	Propping force to base of wall	F_{prop_base} = **8.430** kN/m

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 pressure factor γ_{f_e} = **1.4**

Calculate propping force

Propping force F_{prop} = **6.9** kN/m

Calculate propping forces to top and base of wall

Propping force to top of wall $F_{prop\_top\_f}$ = **-4.695** kN/m Propping force to base of wall $F_{prop\_base\_f}$ = **44.635** kN/m

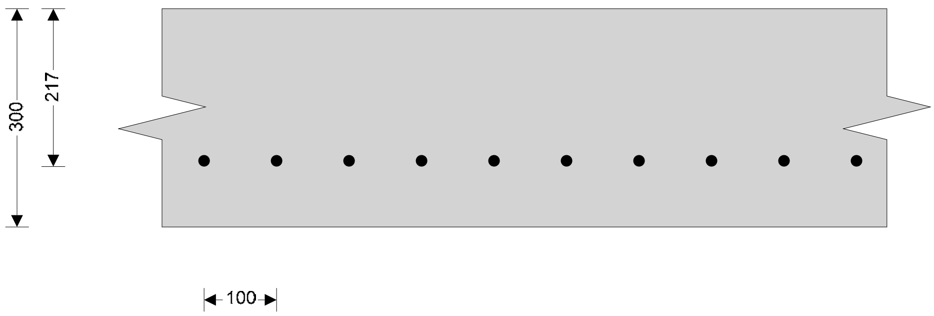
Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

Strength of concrete f_{cu} = **40** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Base details

Minimum reinforcement k = **0.13** % Cover in toe c_{toe} = **75** mm



Design of retaining wall toe

Shear at heel V_{toe} = **143.0** kN/m Moment at heel M_{toe} = **129.8** kNm/m

Compression reinforcement is not required

Check toe in bending

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

Area required $A_{s\_toe\_req}$ = **1500.0** mm^2/m Area provided $A_{s\_toe\_prov}$ = **2011** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress v_{toe} = **0.659** N/mm^2 Allowable shear stress v_{adm} = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_toe} = **0.840** N/mm^2

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

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

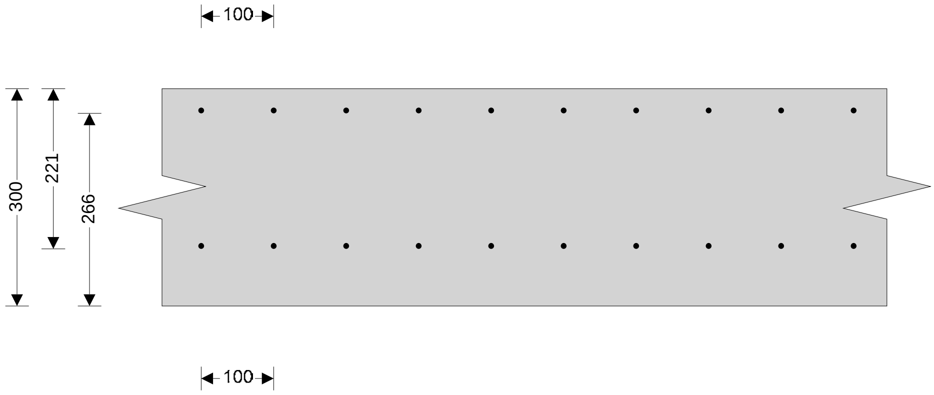
Material properties

Strength of concrete f_{cu} = **40** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Wall details

Minimum reinforcement k = **0.13** %

Cover in stem c_{stem} = **75** mm Cover in wall c_{wall} = **30** mm



Design of retaining wall stem

Shear at base of stem V_{stem} = **62.9** kN/m Moment at base of stem M_{stem} = **37.7** kNm/m

Compression reinforcement is not required

Check wall stem in bending

Reinforcement provided **B503 mesh**

Area required $A_{s\_stem\_req}$ = **413.3** mm^2/m Area provided $A_{s\_stem\_prov}$ = **503** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress v_{stem} = **0.285** N/mm^2 Allowable shear stress v_{adm} = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.523** N/mm^2

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

Design of retaining wall at mid height

Moment at mid height M_{wall} = **16.5** kNm/m

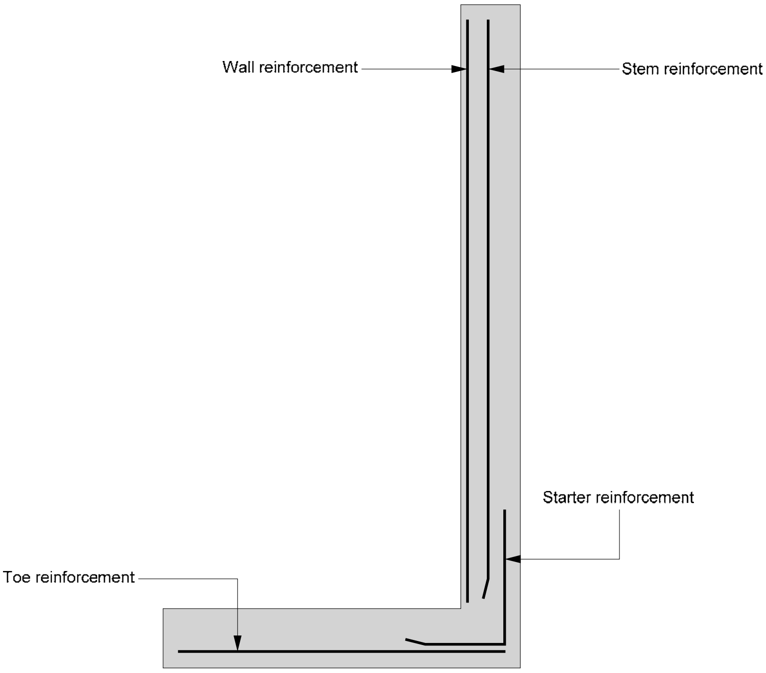
Compression reinforcement is not required

Reinforcement provided **B503 mesh**

Area required $A_{s\_wall\_req}$ = **390.0** mm^2/m Area provided $A_{s\_wall\_prov}$ = **503** mm^2/m

PASS - Reinforcement provided to the retaining wall at mid height is adequate

Indicative retaining wall reinforcement diagram



Toe bars - 16 mm dia.@ 100 mm centres - (2011 mm^2/m)

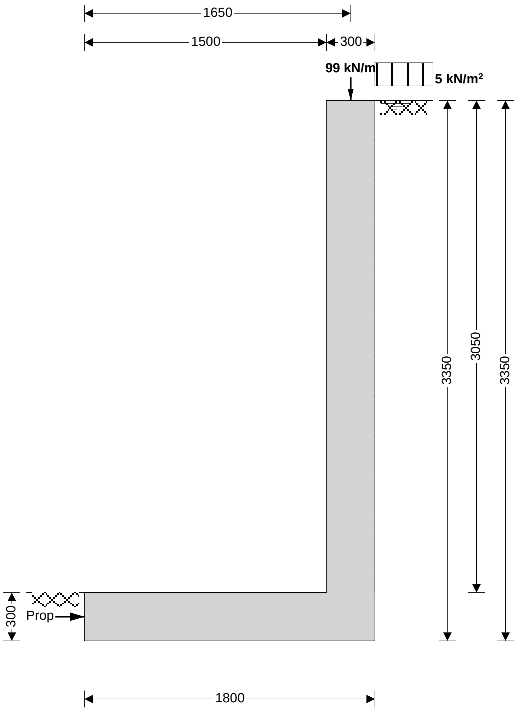
Wall mesh - B503 - (503 mm^2/m)

Stem mesh - B503 - (503 mm^2/m)

RETAINING WALL 1 - PERMANENT CASE

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type	**Cantilever**		
Height of wall stem	h_{stem} = **3050** mm	Wall stem thickness	t_{wall} = **300** mm
Length of toe	l_{toe} = **1500** mm	Length of heel	l_{heel} = **0** mm
Overall length of base	l_{base} = **1800** mm	Base thickness	t_{base} = **300** mm
Height of retaining wall	h_{wall} = **3350** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **300** mm
Position of downstand	l_{ds} = **900** mm		
Depth of cover in front of wall	d_{cover} = **0** mm	Unplanned excavation depth	d_{exc} = **0** mm
Height of ground water	h_{water} = **3350** mm	Density of water	γ_{water} = 9.81 kN/m^3
Density of wall construction	γ_{wall} = **23.6** kN/m^3	Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of soil surface	β = **0.0** deg	Effective height at back of wall	h_{eff} = **3350** mm
Mobilisation factor	M = **1.5**		
Moist density	γ_m = **18.0** kN/m^3	Saturated density	γ_s = **21.0** kN/m^3
Design shear strength	ϕ' = **24.2** deg	Angle of wall friction	δ = **0.0** deg
Design shear strength	ϕ'_b = **24.2** deg	Design base friction	δ_b = **18.6** deg
Moist density	γ_{mb} = **18.0** kN/m^3	Allowable bearing	$P_{bearing}$ = **100** kN/m^2

Using Coulomb theory

Active pressure	K_a =**0.419**	Passive pressure	K_p = **4.187**
At-rest pressure	K_0 = **0.590**		

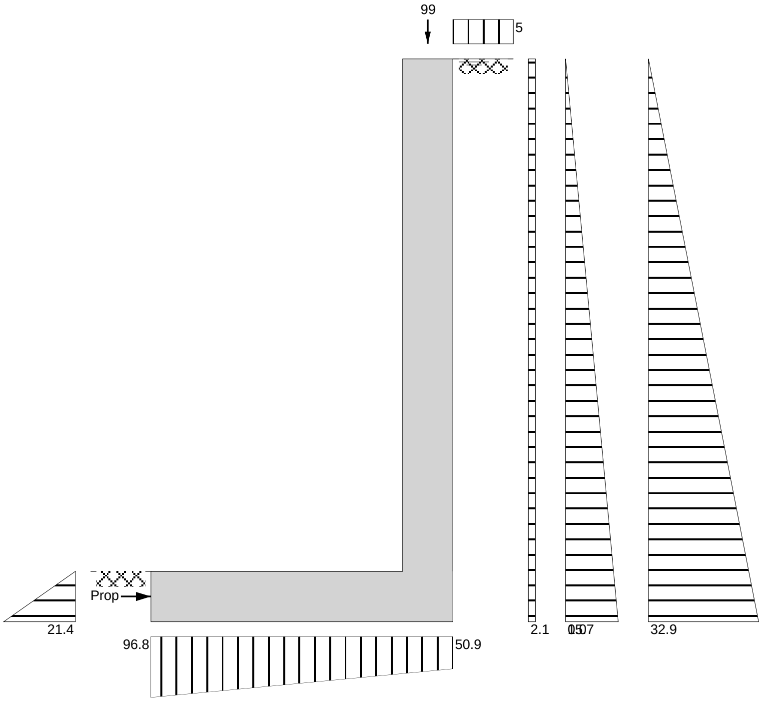
Loading details

Surcharge load — Surcharge = **5.0** kN/m^2

Vertical dead load — W_{dead} = **82.0** kN/m — Vertical live load — W_{live} = **16.6** kN/m

Horizontal dead load — F_{dead} = **0.0** kN/m — Horizontal live load — F_{live} = **0.0** kN/m

Position of vertical load — l_{load} = **1650** mm — Height of horizontal load — h_{load} = **0** mm



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

Calculate propping force

Propping force — F_{prop} = **46.0** kN/m

Check bearing pressure

Total vertical reaction — R = **132.9** kN/m — Distance to reaction — x_{bar} = **807** mm

Eccentricity of reaction — e = **93** mm

Reaction acts within middle third of base

Bearing pressure at toe — p_{toe} = **96.8** kN/m^2 — Bearing pressure at heel — p_{heel} = **50.9** 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 pressure factor γ_{f_e} = **1.4**

Calculate propping force

Propping force F_{prop} = **46.0** kN/m

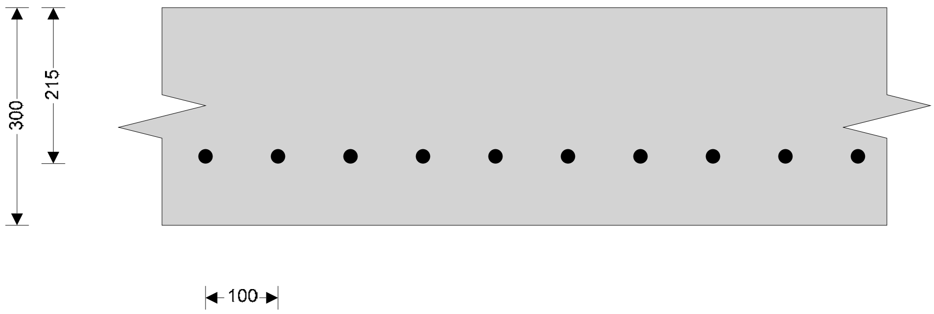
Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

Strength of concrete f_{cu} = **40** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Base details

Minimum reinforcement k = **0.13** % Cover in toe c_{toe} = **75** mm



Design of retaining wall toe

Shear at heel V_{toe} = **162.3** kN/m Moment at heel M_{toe} = **170.7** kNm/m

Compression reinforcement is not required

Check toe in bending

Reinforcement provided **20 mm dia.bars @ 100 mm centres**

Area required $A_{s\_toe\_req}$ = **2065.0** mm^2/m Area provided $A_{s\_toe\_prov}$ = **3142** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress v_{toe} = **0.755** N/mm^2 Allowable shear stress v_{adm} = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_toe} = **0.980** N/mm^2

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

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

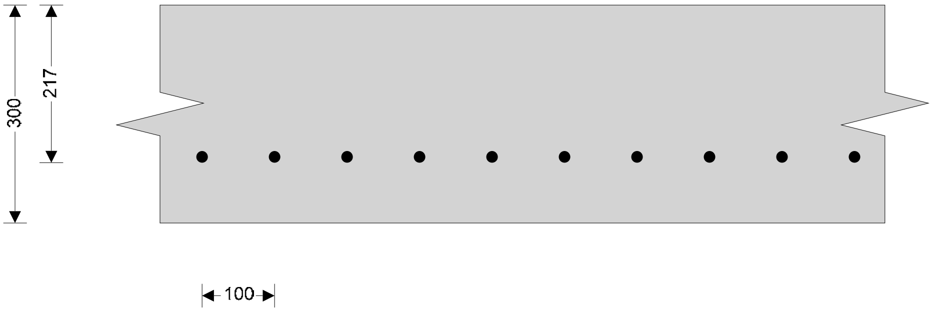
Material properties

Strength of concrete f_{cu} = **40** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Wall details

Minimum reinforcement k = **0.13** %

Cover in stem c_{stem} = **75** mm Cover in wall c_{wall} = **30** mm



Design of retaining wall stem

Shear at base of stem V_{stem} = **35.8** kN/m Moment at base of stem M_{stem} = **132.8** kNm/m

Compression reinforcement is not required

Check wall stem in bending

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

Area required $A_{s\_stem\_req}$ = **1538.4** mm^2/m Area provided $A_{s\_stem\_prov}$ = **2011** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

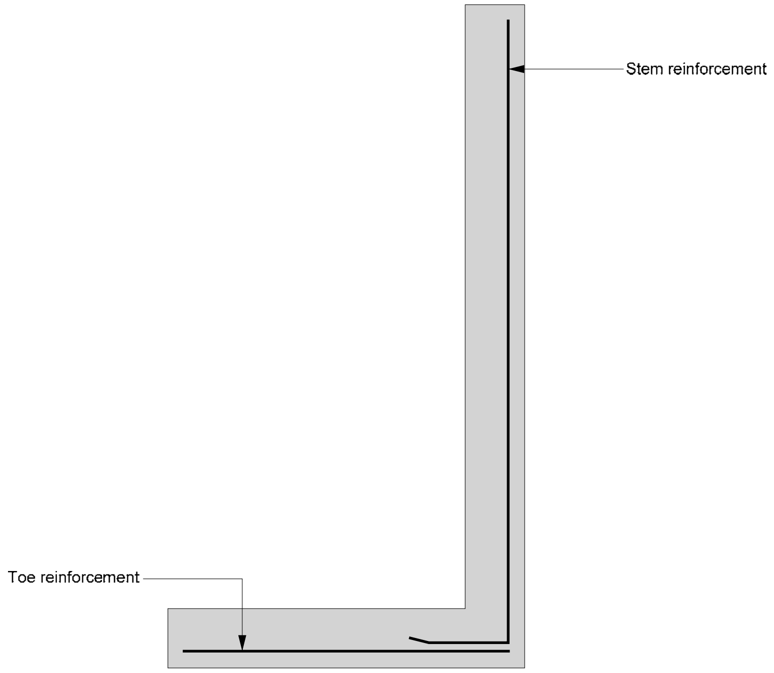
Design shear stress v_{stem} = **0.165** N/mm^2 Allowable shear stress v_{adm} = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.840** N/mm^2

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

Indicative retaining wall reinforcement diagram

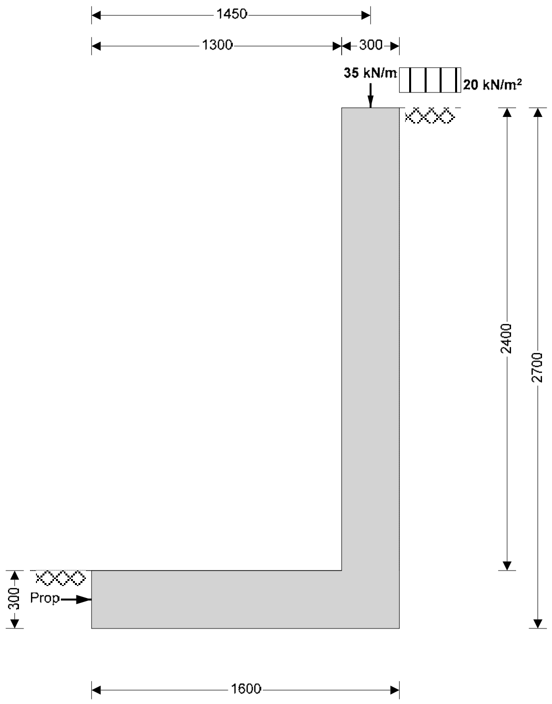


Toe bars - 20 mm dia.@ 100 mm centres - (3142 mm^2/m)
Stem bars - 16 mm dia.@ 100 mm centres - (2011 mm^2/m)

RETAINING WALL 2A-TEMPORARY CASE

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

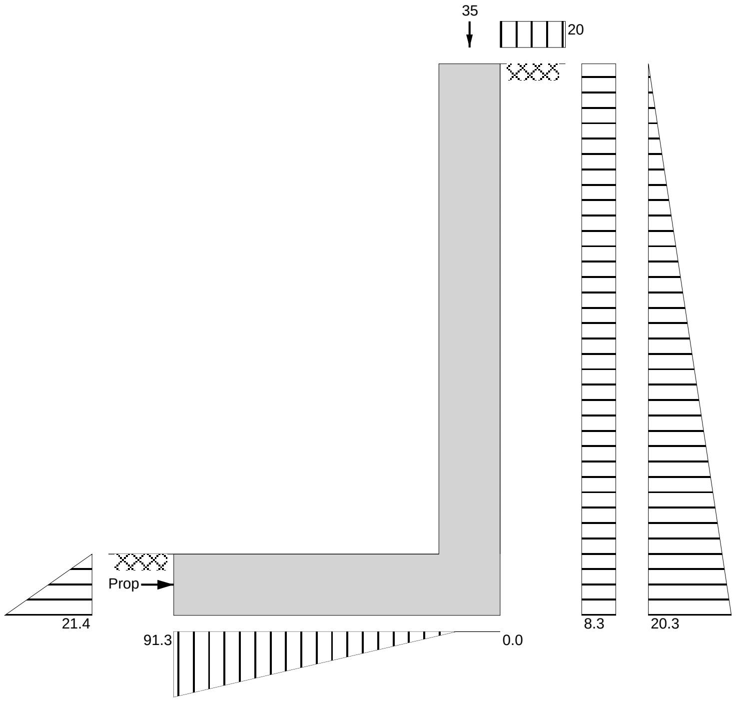
Retaining wall type	**Cantilever**		
Height of wall stem	h_{stem} = **2400** mm	Wall stem thickness	t_{wall} = **300** mm
Length of toe	l_{toe} = **1300** mm	Length of heel	l_{heel} = **0** mm
Overall length of base	l_{base} = **1600** mm	Base thickness	t_{base} = **300** mm
Height of retaining wall	h_{wall} = **2700** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **300** mm
Position of downstand	l_{ds} = **1000** mm		
Depth of cover in front of wall	d_{cover} = **0** mm	Unplanned excavation depth	d_{exc} = **0** mm
Height of ground water	h_{water} = **0** mm	Density of water	γ_{water} = 9.81 kN/m^3
Density of wall construction	γ_{wall} = **23.6** kN/m^3	Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of soil surface	β = **0.0** deg	Effective height at back of wall	h_{eff} = **2700** mm
Mobilisation factor	M = **1.5**		
Moist density	γ_m = **18.0** kN/m^3	Saturated density	γ_s = **21.0** kN/m^3
Design shear strength	ϕ' = **24.2** deg	Angle of wall friction	δ = **0.0** deg
Design shear strength	ϕ'_b = **24.2** deg	Design base friction	δ_b = **18.6** deg
Moist density	γ_{mb} = **18.0** kN/m^3	Allowable bearing	$P_{bearing}$ = **100** kN/m^2

Using Coulomb theory

Active pressure	K_a =**0.419**	Passive pressure	K_p = **4.187**
At-rest pressure	K_0 = **0.590**		

Loading details

Surcharge load	Surcharge = **19.9** kN/m^2		
Vertical dead load	W_{dead} = **29.7** kN/m	Vertical live load	W_{live} = **5.1** kN/m
Horizontal dead load	F_{dead} = **0.0** kN/m	Horizontal live load	F_{live} = **0.0** kN/m
Position of vertical load	l_{load} = **1450** mm	Height of horizontal load	h_{load} = **0** mm



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

Calculate propping force

Propping force	F_{prop} = **27.2** kN/m

Check bearing pressure

Total vertical reaction	R = **63.1** kN/m	Distance to reaction	x_{bar} = **461** mm
Eccentricity of reaction	e = **339** mm		

Reaction acts outside middle third of base

Bearing pressure at toe	p_{toe} = **91.3** kN/m^2	Bearing pressure at heel	p_{heel} = **0.0** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

Ultimate limit state load factors

Dead load factor γ_{f_d} = **1.4** Live load factor γ_{f_l} = **1.6**

Earth pressure factor γ_{f_e} = **1.4**

Calculate propping force

Propping force F_{prop} = **27.2** kN/m

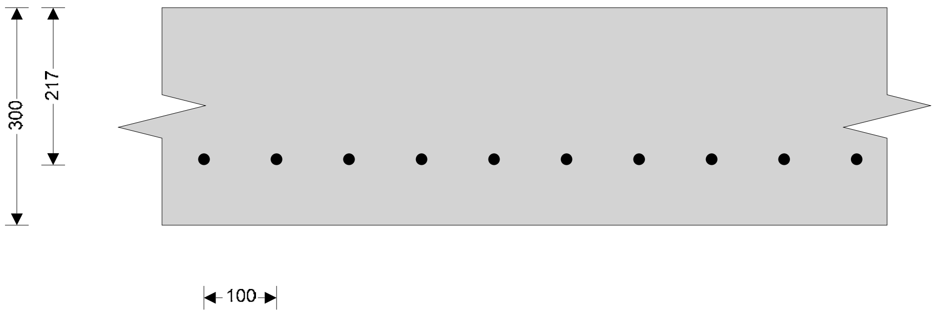
Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

Strength of concrete f_{cu} = **40** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Base details

Minimum reinforcement k = **0.13** % Cover in toe c_{toe} = **75** mm



Design of retaining wall toe

Shear at heel V_{toe} = **76.5** kN/m Moment at heel M_{toe} = **117.2** kNm/m

Compression reinforcement is not required

Check toe in bending

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

Area required $A_{s\_toe\_req}$ = **1326.9** mm^2/m Area provided $A_{s\_toe\_prov}$ = **2011** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress v_{toe} = **0.349** N/mm^2 Allowable shear stress v_{adm} = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_toe} = **0.689** N/mm^2

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

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

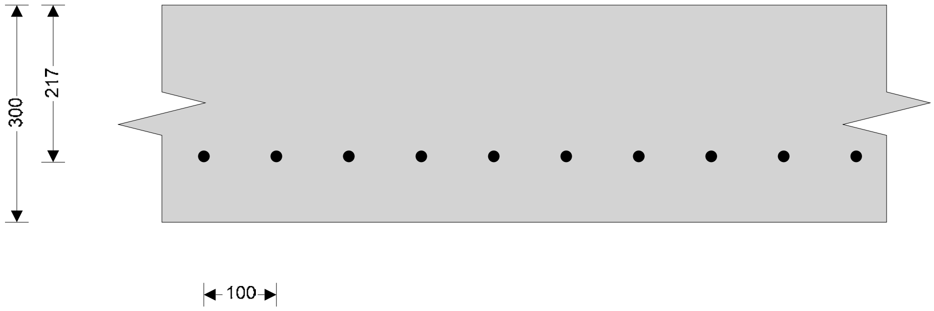
Material properties

Strength of concrete f_{cu} = **40** N/mm^2 Strength of reinforcement f_y = **500** N/mm^2

Wall details

Minimum reinforcement k = **0.13** %

Cover in stem c_{stem} = **75** mm Cover in wall c_{wall} = **30** mm



Design of retaining wall stem

Shear at base of stem V_{stem} = **14.8** kN/m

Moment at base of stem M_{stem} = **101.6** kNm/m

Compression reinforcement is not required

Check wall stem in bending

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

Area required $A_{s\_stem\_req}$ = **1137.4** mm^2/m

Area provided $A_{s\_stem\_prov}$ = **2011** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress v_{stem} = **0.068** N/mm^2

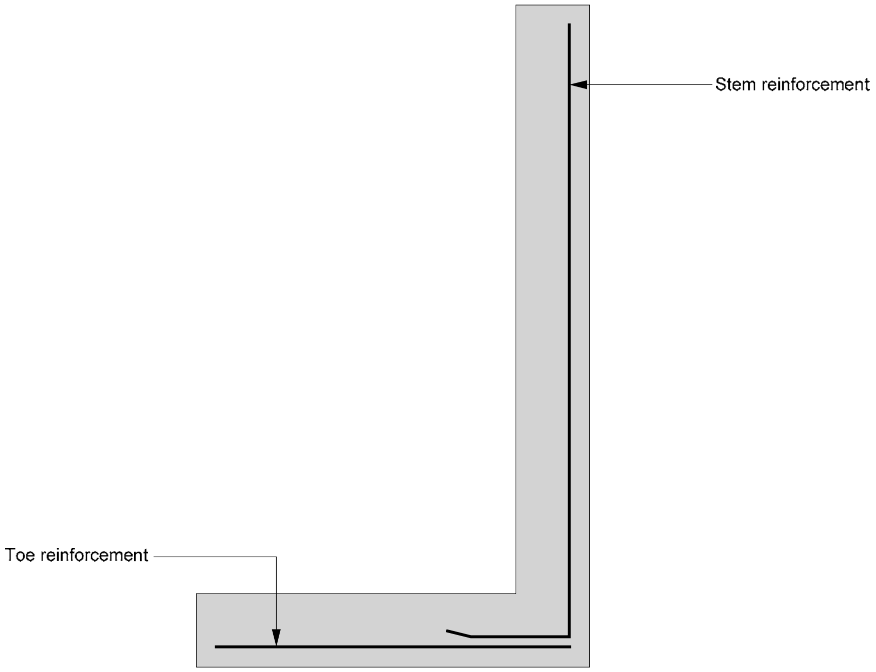
Allowable shear stress v_{adm} = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.689** N/mm^2

v_{stem} < v_{c_stem} - No shear reinforcement required

Indicative retaining wall reinforcement diagram



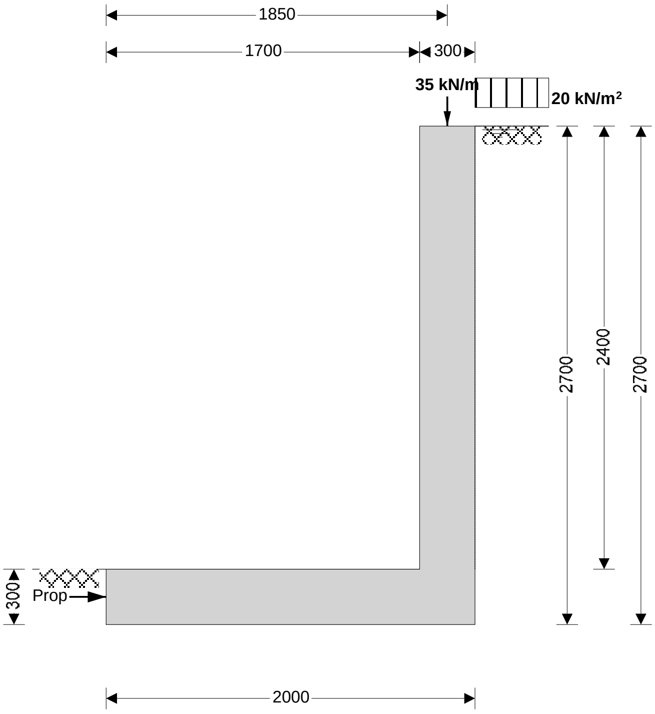
Toe bars - 16 mm dia.@ 100 mm centres - (2011 mm^2/m)

Stem bars - 16 mm dia.@ 100 mm centres - (2011 mm^2/m)

RETAINING WALL 2A - PERMANENT CASE

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type	**Cantilever**		
Height of wall stem	h_{stem} = **2400** mm	Wall stem thickness	t_{wall} = **300** mm
Length of toe	l_{toe} = **1700** mm	Length of heel	l_{heel} = **0** mm
Overall length of base	l_{base} = **2000** mm	Base thickness	t_{base} = **300** mm
Height of retaining wall	h_{wall} = **2700** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **300** mm
Position of downstand	l_{ds} = **900** mm		
Depth of cover in front of wall	d_{cover} = **0** mm	Unplanned excavation depth	d_{exc} = **0** mm
Height of ground water	h_{water} = **2700** mm	Density of water	γ_{water} = 9.81 kN/m^3
Density of wall construction	γ_{wall} = **23.6** kN/m^3	Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of soil surface	β = **0.0** deg	Effective height at back of wall	h_{eff} = **2700** mm
Mobilisation factor	M = **1.5**		
Moist density	γ_m = **18.0** kN/m^3	Saturated density	γ_s = **21.0** kN/m^3
Design shear strength	ϕ' = **24.2** deg	Angle of wall friction	δ = **0.0** deg
Design shear strength	ϕ'_b = **24.2** deg	Design base friction	δ_b = **18.6** deg
Moist density	γ_{mb} = **18.0** kN/m^3	Allowable bearing	$P_{bearing}$ = **100** kN/m^2

Using Coulomb theory

Active pressure	K_a =**0.419**	Passive pressure	K_p = **4.187**
At-rest pressure	K_0 = **0.590**		

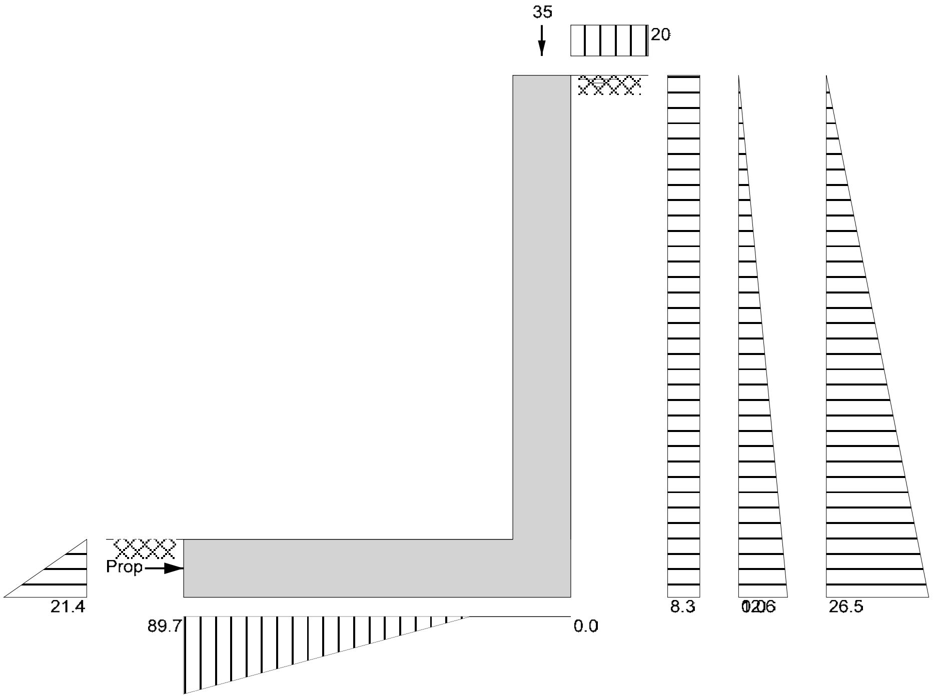
Loading details

Surcharge load Surcharge = **19.9** kN/m^2

Vertical dead load W_{dead} = **30.0** kN/m Vertical live load W_{live} = **5.1** kN/m

Horizontal dead load F_{dead} = **0.0** kN/m Horizontal live load F_{live} = **0.0** kN/m

Position of vertical load l_{load} = **1850** mm Height of horizontal load h_{load} = **0** mm



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

Calculate propping force

Propping force F_{prop} = **51.5** kN/m

Check bearing pressure

Total vertical reaction R = **66.3** kN/m Distance to reaction x_{bar} = **492** mm

Eccentricity of reaction e = **508** mm

Reaction acts outside middle third of base

Bearing pressure at toe p_{toe} = **89.7** kN/m^2 Bearing pressure at heel p_{heel} = **0.0** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

Ultimate limit state load factors

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

Calculate propping force

Propping force F_{prop} = **51.5** kN/m

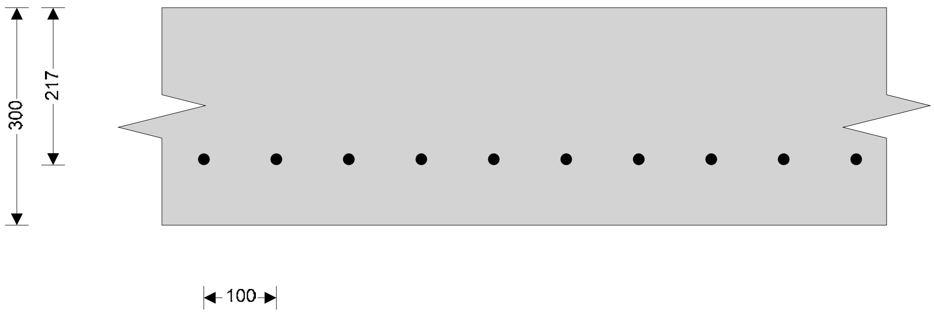
Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

Strength of concrete	f_{cu} = **40** N/mm^2	Strength of reinforcement	f_y = **500** N/mm^2

Base details

Minimum reinforcement	k = **0.13** %	Cover in toe	c_{toe} = **75** mm



Design of retaining wall toe

Shear at heel	V_{toe} = **76.9** kN/m	Moment at heel	M_{toe} = **143.8** kNm/m

Compression reinforcement is not required

Check toe in bending

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

Area required	$A_{s\_toe\_req}$ = **1680.0** mm^2/m	Area provided	$A_{s\_toe\_prov}$ = **2011** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress	v_{toe} = **0.354** N/mm^2	Allowable shear stress	v_{adm} = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_toe} = **0.840** N/mm^2

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

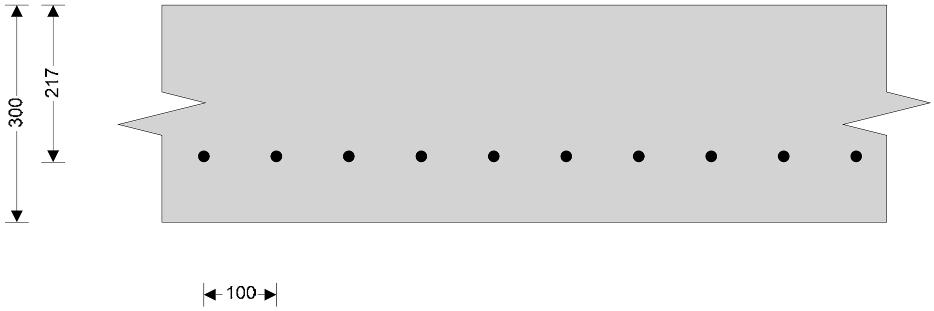
Design of reinforced concrete retaining wall stem (BS 8002:1994)

Material properties

Strength of concrete	f_{cu} = **40** N/mm^2	Strength of reinforcement	f_y = **500** N/mm^2

Wall details

Minimum reinforcement	k = **0.13** %		
Cover in stem	c_{stem} = **75** mm	Cover in wall	c_{wall} = **30** mm



Design of retaining wall stem

Shear at base of stem V_{stem} = **10.1** kN/m Moment at base of stem M_{stem} = **113.8** kNm/m

Compression reinforcement is not required

Check wall stem in bending

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

Area required $A_{s\_stem\_req}$ = **1286.0** mm^2/m Area provided $A_{s\_stem\_prov}$ = **2011** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

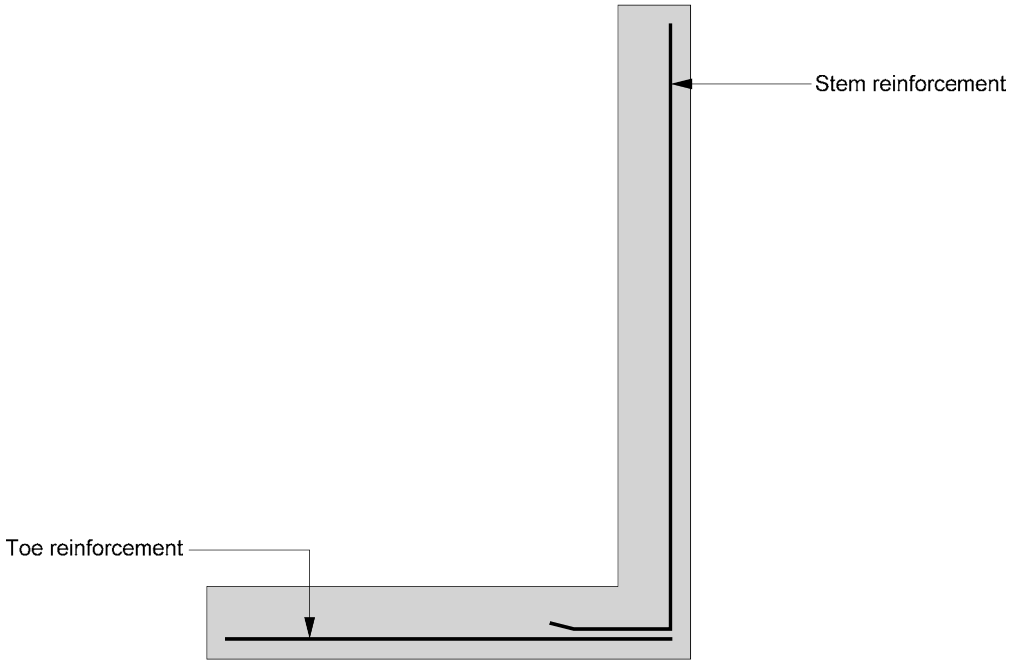
Design shear stress v_{stem} = **0.046** N/mm^2 Allowable shear stress v_{adm} = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.689** N/mm^2

v_{stem} < v_{c_stem} - No shear reinforcement required

Indicative retaining wall reinforcement diagram



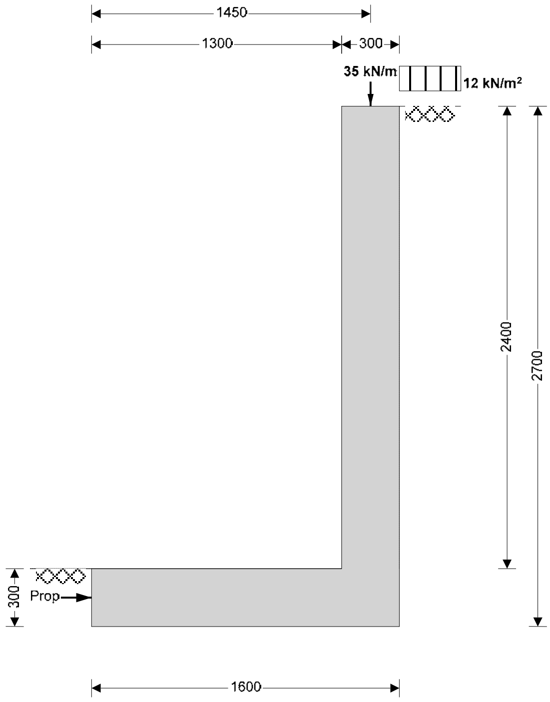
Toe bars - 16 mm dia.@ 100 mm centres - (2011 mm^2/m)

Stem bars - 16 mm dia.@ 100 mm centres - (2011 mm^2/m)

RETAINING WALL 2B - TEMPORARY CASE

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} = **2400** mm
Thickness of wall stem	t_{wall} = **300** mm
Length of toe	l_{toe} = **1300** mm
Length of heel	l_{heel} = **0** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1600** mm
Thickness of base	t_{base} = **300** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **900** mm
Thickness of downstand	t_{ds} = **300** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **2700** 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} = **0** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0\ mm)$ = **0** 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	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **2700** mm

Retained material details

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

Base material details

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

Using Coulomb theory

Active pressure coefficient for retained material

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

Passive pressure coefficient for base material

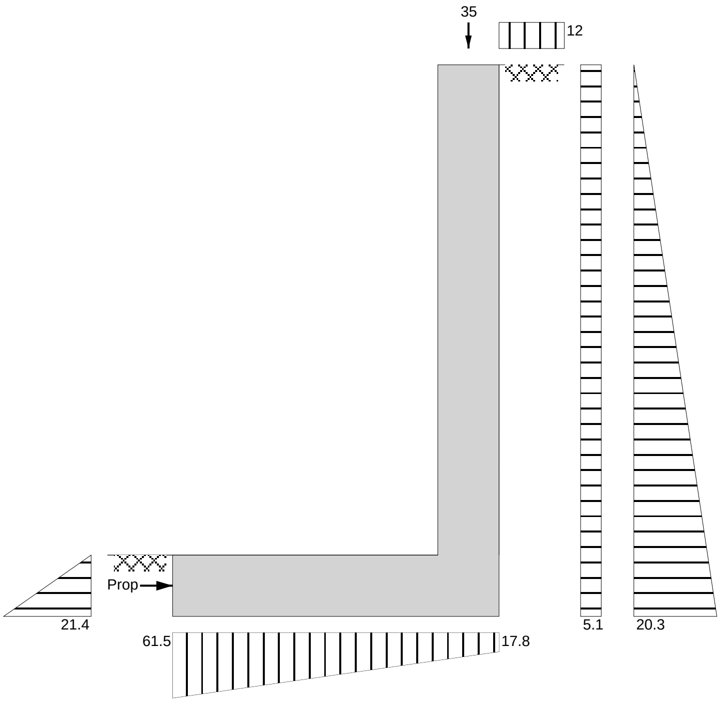
$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{4.187}$$

At-rest pressure

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

Loading details

Surcharge load on plan	Surcharge = **12.1** kN/m^2
Applied vertical dead load on wall	W_{dead} = **30.0** kN/m
Applied vertical live load on wall	W_{live} = **5.1** kN/m
Position of applied vertical load on wall	l_{load} = **1450** 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 $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **17** kN/m

Wall base $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **11.3** kN/m

Applied vertical load $W_v = W_{dead} + W_{live}$ = **35.1** kN/m

Total vertical load $W_{total} = w_{wall} + w_{base} + W_v$ = **63.4** kN/m

Horizontal forces on wall

Surcharge $F_{sur} = K_a \times \text{Surcharge} \times h_{eff}$ = **13.7** kN/m

Moist backfill above water table $F_{m\_a} = 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **27.5** kN/m

Total horizontal load $F_{total} = F_{sur} + F_{m\_a}$ = **41.1** kN/m

Calculate propping force

Passive resistance of soil in front of wall $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **3.2** kN/m

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

F_{prop} = **18.3** kN/m

Overturning moments

Surcharge $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **18.5** kNm/m

Moist backfill above water table $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **24.7** kNm/m

Total overturning moment $M_{ot} = M_{sur} + M_{m\_a}$ = **43.2** kNm/m

Restoring moments

Wall stem $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **24.6** kNm/m

Wall base $M_{base} = w_{base} \times l_{base} / 2$ = **9.1** kNm/m

Design vertical dead load $M_{dead} = W_{dead} \times l_{load}$ = **43.5** kNm/m

Total restoring moment $M_{rest} = M_{wall} + M_{base} + M_{dead}$ = **77.2** kNm/m

Check bearing pressure

Design vertical live load $M_{live} = W_{live} \times l_{load}$ = **7.4** kNm/m

Total moment for bearing $M_{total} = M_{rest} - M_{ot} + M_{live}$ = **41.4** kNm/m

Total vertical reaction $R = W_{total}$ = **63.4** kN/m

Distance to reaction $x_{bar} = M_{total} / R$ = **653** mm

Eccentricity of reaction $e = \text{abs}((l_{base} / 2) - x_{bar})$ = **147** mm

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe} = (R / l_{base}) + (6 \times R \times e / l_{base}^2)$ = **61.5** kN/m^2

Bearing pressure at heel $p_{heel} = (R / l_{base}) - (6 \times R \times e / l_{base}^2)$ = **17.8** 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 $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **23.8** kN/m

Wall base $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **15.9** kN/m

Applied vertical load $W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **50.2** kN/m

Total vertical load $W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f}$ = **89.8** kN/m

Factored horizontal at-rest forces on wall

Surcharge $F_{sur\_f} = \gamma_{f\_l} \times K_0 \times$ Surcharge $\times h_{eff}$ = **30.8** kN/m

Moist backfill above water table $F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times \gamma_m \times (h_{eff} - h_{water})^2$ = **54.2** kN/m

Total horizontal load $F_{total\_f} = F_{sur\_f} + F_{m\_a\_f}$ = **85** kN/m

Calculate propping force

Passive resistance of soil in front of wall $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **4.5** kN/m

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

F_{prop_f} = **53.1** kN/m

Factored overturning moments

Surcharge $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **41.6** kNm/m

Moist backfill above water table $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **48.8** kNm/m

Total overturning moment $M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f}$ = **90.4** kNm/m

Restoring moments

Wall stem $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **34.5** kNm/m

Wall base $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **12.7** kNm/m

Design vertical load $M_{v\_f} = W_{v\_f} \times l_{load}$ = **72.7** kNm/m

Total restoring moment $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f}$ = **119.9** kNm/m

Factored bearing pressure

Total moment for bearing $M_{total\_f} = M_{rest\_f} - M_{ot\_f}$ = **29.5** kNm/m

Total vertical reaction $R_f = W_{total\_f}$ = **89.8** kN/m

Distance to reaction $x_{bar\_f} = M_{total\_f} / R_f$ = **328** mm

Eccentricity of reaction $e_f = \text{abs}((l_{base} / 2) - x_{bar\_f})$ = **472** mm

Reaction acts outside middle third of base

Bearing pressure at toe $p_{toe\_f} = R_f / (1.5 \times x_{bar\_f})$ = **182.3** kN/m^2

Bearing pressure at heel p_{heel_f} = 0 kN/m^2 = **0** kN/m^2

Rate of change of base reaction rate $= p_{toe\_f} / (3 \times x_{bar\_f})$ = **185.06** kN/m^2/m

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

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

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

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

Material properties

Characteristic strength of concrete f_{cu} = **40** N/mm^2

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

Base details

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

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

Calculate shear for toe design

Shear from bearing pressure $V_{toe\_bear} = 3 \times p_{toe\_f} \times x_{bar\_f} / 2 = \mathbf{89.8}$ kN/m

Shear from weight of base $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} = \mathbf{12.9}$ kN/m

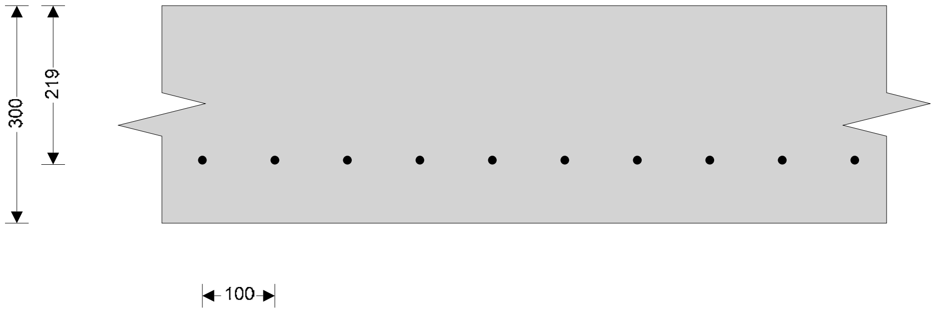
Total shear for toe design $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} = \mathbf{76.9}$ kN/m

Calculate moment for toe design

Moment from bearing pressure $M_{toe\_bear} = 3 \times p_{toe\_f} \times x_{bar\_f} \times (l_{toe} - x_{bar\_f} + t_{wall} / 2) / 2 = \mathbf{100.7}$ kNm/m

Moment from weight of base $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) = \mathbf{10.4}$ kNm/m

Total moment for toe design $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} = \mathbf{90.3}$ kNm/m



Check toe in bending

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

Depth of reinforcement $d_{toe} = t_{base} – c_{toe} – (\phi_{toe} / 2) = \mathbf{219.0}$ mm

Constant $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) = \mathbf{0.047}$

Compression reinforcement is not required

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

$z_{toe} = \mathbf{207}$ mm

Area of tension reinforcement required $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) = \mathbf{1004}$ mm^2/m

Minimum area of tension reinforcement $A_{s\_toe\_min} = k \times b \times t_{base} = \mathbf{390}$ mm^2/m

Area of tension reinforcement required $A_{s\_toe\_req} = \mathrm{Max}(A_{s\_toe\_des}, A_{s\_toe\_min}) = \mathbf{1004}$ mm^2/m

Reinforcement provided **B1131 mesh**

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

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress $v_{toe} = V_{toe} / (b \times d_{toe}) = \mathbf{0.351}$ N/mm^2

Allowable shear stress $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1 \text{ N/mm}^2)}, 5) \times 1 \text{ N/mm}^2 = \mathbf{5.000}$ 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 $v_{c\_toe} = \mathbf{0.689}$ N/mm^2

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

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

Material properties

Characteristic strength of concrete $f_{cu} = \mathbf{40}$ 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 $F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds})$ = **27.4** kN/m

Moist backfill above water table $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2$ = **42.8** kN/m

Calculate shear for stem design

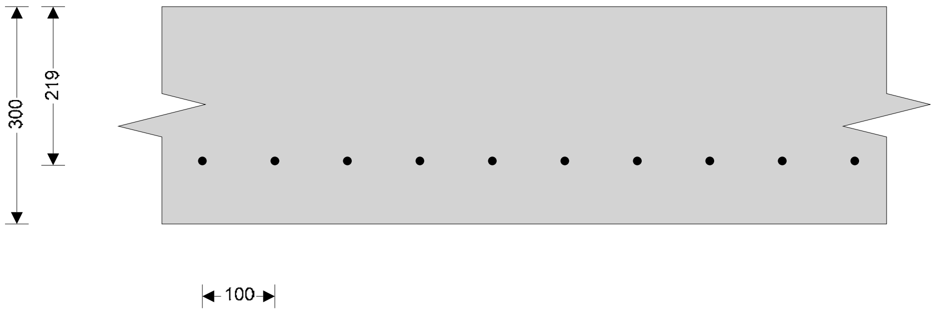
Shear at base of stem $V_{stem} = F_{s\_sur\_f} + F_{s\_m\_a\_f} - F_{prop\_f}$ = **17.2** kN/m

Calculate moment for stem design

Surcharge $M_{s\_sur} = F_{s\_sur\_f} \times (h_{stem} + t_{base}) / 2$ = **37** kNm/m

Moist backfill above water table $M_{s\_m\_a} = F_{s\_m\_a\_f} \times (2 \times h_{sat} + h_{eff} - d_{ds} + t_{base} / 2) / 3$ = **40.7** kNm/m

Total moment for stem design $M_{stem} = M_{s\_sur} + M_{s\_m\_a}$ = **77.7** kNm/m



Check wall stem in bending

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

Depth of reinforcement $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2)$ = **219.0** mm

Constant $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu})$ = **0.041**

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} = **208** mm

Area of tension reinforcement required $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem})$ = **859** mm^2/m

Minimum area of tension reinforcement $A_{s\_stem\_min} = k \times b \times t_{wall}$ = **390** mm^2/m

Area of tension reinforcement required $A_{s\_stem\_req} = \text{Max}(A_{s\_stem\_des}, A_{s\_stem\_min})$ = **859** mm^2/m

Reinforcement provided **B1131 mesh**

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

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress $v_{stem} = V_{stem} / (b \times d_{stem})$ = **0.078** N/mm^2

Allowable shear stress $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1\ \text{N/mm}^2)}, 5) \times 1\ \text{N/mm}^2$ = **5.000** N/mm^2

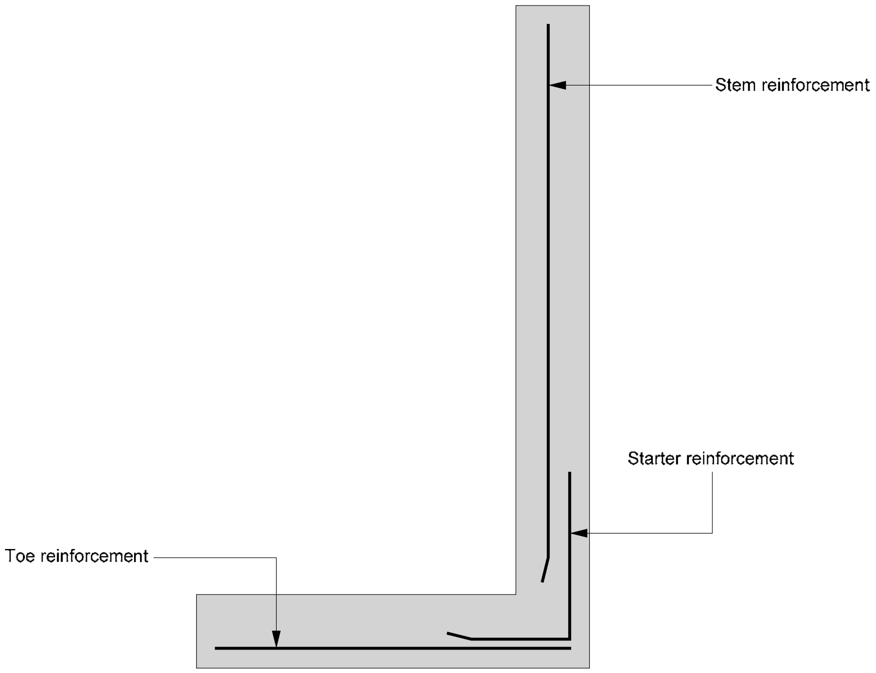
PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress v_{c_stem} = **0.689** N/mm^2

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

Indicative retaining wall reinforcement diagram



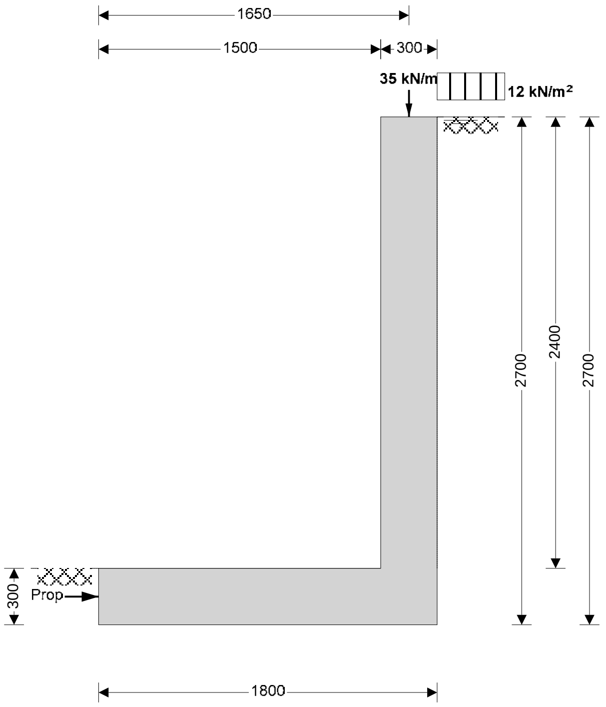
Toe mesh - B1131 - (1131 mm^2/m)

Stem mesh - B1131 - (1131 mm^2/m)

RETAINING WALL 2B - PERMANENT CASE

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} = **2400** mm
Thickness of wall stem	t_{wall} = **300** mm
Length of toe	l_{toe} = **1500** mm
Length of heel	l_{heel} = **0** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1800** mm
Thickness of base	t_{base} = **300** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **1500** mm
Thickness of downstand	t_{ds} = **300** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **2700** 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} = **2700** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0 \text{ mm})$ = **2400** 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 $h_{eff} = h_{wall} + l_{heel} \times \tan(\beta) = \mathbf{2700}$ 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 $P_{bearing} = \mathbf{100}$ kN/m^2

Using Coulomb theory

Active pressure coefficient for retained material

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

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 = **12.1** kN/m^2

Applied vertical dead load on wall $W_{dead} = \mathbf{29.7}$ kN/m

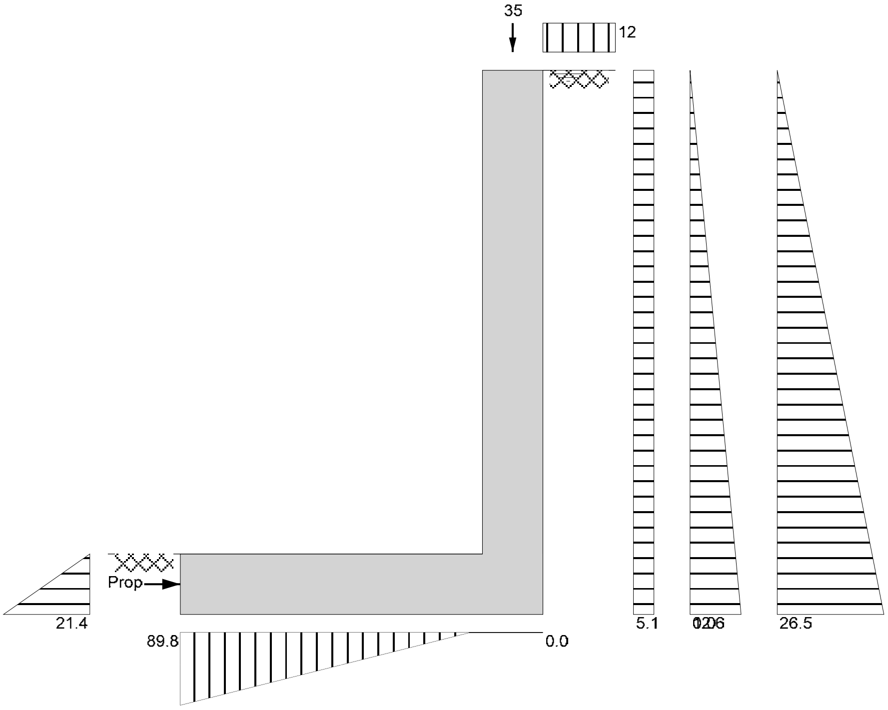
Applied vertical live load on wall $W_{live} = \mathbf{5.1}$ kN/m

Position of applied vertical load on wall $l_{load} = \mathbf{1650}$ 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 $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **17** kN/m

Wall base $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **12.7** kN/m

Applied vertical load $W_v = W_{dead} + W_{live}$ = **34.8** kN/m

Total vertical load $W_{total} = w_{wall} + w_{base} + W_v$ = **64.5** kN/m

Horizontal forces on wall

Surcharge $F_{sur} = K_a \times \text{Surcharge} \times h_{eff}$ = **13.7** kN/m

Saturated backfill $F_s = 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **17.1** kN/m

Water $F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **35.8** kN/m

Total horizontal load $F_{total} = F_{sur} + F_s + F_{water}$ = **66.5** kN/m

Calculate propping force

Passive resistance of soil in front of wall $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **3.2** kN/m

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

F_{prop} = **43.3** kN/m

Overturning moments

Surcharge $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **18.5** kNm/m

Saturated backfill $M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **15.4** kNm/m

Water $M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **32.2** kNm/m

Total overturning moment $M_{ot} = M_{sur} + M_s + M_{water}$ = **66** kNm/m

Restoring moments

Wall stem $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **28** kNm/m

Wall base $M_{base} = w_{base} \times l_{base} / 2$ = **11.5** kNm/m

Design vertical dead load $M_{dead} = W_{dead} \times l_{load}$ = **49** kNm/m

Total restoring moment $M_{rest} = M_{wall} + M_{base} + M_{dead}$ = **88.5** kNm/m

Check bearing pressure

Design vertical live load $M_{live} = W_{live} \times l_{load} = \mathbf{8.4}$ kNm/m

Total moment for bearing $M_{total} = M_{rest} - M_{ot} + M_{live} = \mathbf{30.9}$ kNm/m

Total vertical reaction $R = W_{total} = \mathbf{64.5}$ kN/m

Distance to reaction $x_{bar} = M_{total} / R = \mathbf{479}$ mm

Eccentricity of reaction $e = abs((l_{base} / 2) - x_{bar}) = \mathbf{421}$ mm

Reaction acts outside middle third of base

Bearing pressure at toe $p_{toe} = R / (1.5 \times x_{bar}) = \mathbf{89.8}$ kN/m^2

Bearing pressure at heel $p_{heel} = 0$ kN/m^2 = $\mathbf{0}$ 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 $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall} = \mathbf{23.8}$ kN/m

Wall base $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base} = \mathbf{17.8}$ kN/m

Applied vertical load $W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live} = \mathbf{49.7}$ kN/m

Total vertical load $W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f} = \mathbf{91.4}$ kN/m

Factored horizontal at-rest forces on wall

Surcharge $F_{sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times h_{eff} = \mathbf{30.8}$ kN/m

Saturated backfill $F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{water}^2 = \mathbf{33.7}$ kN/m

Water $F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water} = \mathbf{50.1}$ kN/m

Total horizontal load $F_{total\_f} = F_{sur\_f} + F_{s\_f} + F_{water\_f} = \mathbf{114.6}$ kN/m

Calculate propping force

Passive resistance of soil in front of wall $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb} = \mathbf{4.5}$ kN/m

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

$F_{prop\_f} = \mathbf{82.1}$ kN/m

Factored overturning moments

Surcharge $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2 = \mathbf{41.6}$ kNm/m

Saturated backfill $M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3 = \mathbf{30.3}$ kNm/m

Water $M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3 = \mathbf{45.1}$ kNm/m

Total overturning moment $M_{ot\_f} = M_{sur\_f} + M_{s\_f} + M_{water\_f} = \mathbf{117}$ kNm/m

Restoring moments

Wall stem $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2) = \mathbf{39.3}$ kNm/m

Wall base $M_{base\_f} = w_{base\_f} \times l_{base} / 2 = \mathbf{16.1}$ kNm/m

Design vertical load $M_{v\_f} = W_{v\_f} \times l_{load} = \mathbf{82.1}$ kNm/m

Total restoring moment $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f} = \mathbf{137.4}$ kNm/m

Factored bearing pressure

Total moment for bearing $M_{total\_f} = M_{rest\_f} - M_{ot\_f} = \mathbf{20.4}$ kNm/m

Total vertical reaction $R_f = W_{total\_f} = \mathbf{91.4}$ kN/m

Distance to reaction $x_{bar\_f} = M_{total\_f} / R_f = \mathbf{223}$ mm

Eccentricity of reaction $e_f = \text{abs}((l_{base} / 2) - x_{bar\_f}) = \mathbf{677}$ mm

Reaction acts outside middle third of base

Bearing pressure at toe $p_{toe\_f} = R_f / (1.5 \times x_{bar\_f}) = \mathbf{273.4}$ kN/m^2

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

Rate of change of base reaction $\text{rate} = p_{toe\_f} / (3 \times x_{bar\_f}) = \mathbf{408.93}$ kN/m^2/m

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

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

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

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

Material properties

Characteristic strength of concrete f_{cu} = **40** 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 $V_{toe\_bear} = 3 \times p_{toe\_f} \times x_{bar\_f} / 2 =$ **91.4** kN/m

Shear from weight of base $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} =$ **14.9** kN/m

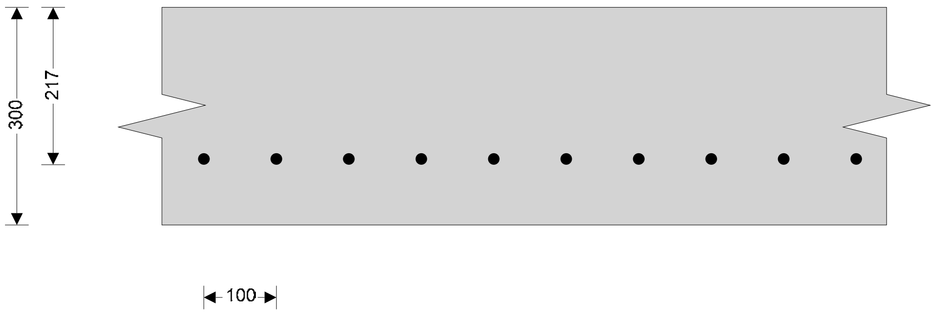
Total shear for toe design $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} =$ **76.5** kN/m

Calculate moment for toe design

Moment from bearing pressure $M_{toe\_bear} = 3 \times p_{toe\_f} \times x_{bar\_f} \times (l_{toe} - x_{bar\_f} + t_{wall} / 2) / 2 =$ **130.4** kNm/m

Moment from weight of base $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) =$ **13.5** kNm/m

Total moment for toe design $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} =$ **116.9** kNm/m



Check toe in bending

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

Depth of reinforcement $d_{toe} = t_{base} – c_{toe} – (\phi_{toe} / 2) =$ **217.0** mm

Constant $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) =$ **0.062**

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} = **201** mm

Area of tension reinforcement required $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) =$ **1338** mm^2/m

Minimum area of tension reinforcement $A_{s\_toe\_min} = k \times b \times t_{base} =$ **390** mm^2/m

Area of tension reinforcement required $A_{s\_toe\_req} = \mathrm{Max}(A_{s\_toe\_des}, A_{s\_toe\_min}) =$ **1338** mm^2/m

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

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

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress $v_{toe} = V_{toe} / (b \times d_{toe}) =$ **0.353** N/mm^2

Allowable shear stress $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1\ \mathrm{N/mm^2})}, 5) \times 1\ \mathrm{N/mm^2} =$ **5.000** 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} = **0.840** N/mm^2

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

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

Material properties

Characteristic strength of concrete $f_{cu} = \mathbf{40}$ 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 $c_{stem} = \mathbf{75}$ mm

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

Factored horizontal at-rest forces on stem

Surcharge $F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds}) = \mathbf{27.4}$ kN/m

Saturated backfill $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{sat}^2 = \mathbf{26.6}$ kN/m

Water $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2 = \mathbf{39.6}$ kN/m

Calculate shear for stem design

Shear at base of stem $V_{stem} = F_{s\_sur\_f} + F_{s\_s\_f} + F_{s\_water\_f} - F_{prop\_f} = \mathbf{11.5}$ kN/m

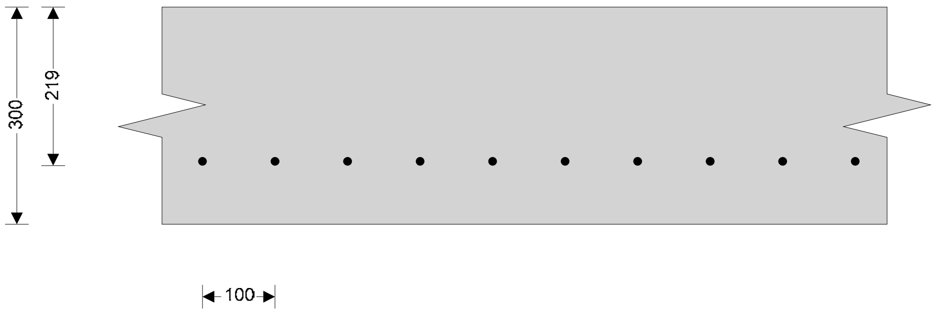
Calculate moment for stem design

Surcharge $M_{s\_sur} = F_{s\_sur\_f} \times (h_{stem} + t_{base}) / 2 = \mathbf{37}$ kNm/m

Saturated backfill $M_{s\_s} = F_{s\_s\_f} \times h_{sat} / 3 = \mathbf{21.3}$ kNm/m

Water $M_{s\_water} = F_{s\_water\_f} \times h_{sat} / 3 = \mathbf{31.6}$ kNm/m

Total moment for stem design $M_{stem} = M_{s\_sur} + M_{s\_s} + M_{s\_water} = \mathbf{90}$ kNm/m



Check wall stem in bending

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

Depth of reinforcement $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2) = \mathbf{219.0}$ mm

Constant $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu}) = \mathbf{0.047}$

Compression reinforcement is not required

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

$z_{stem} = \mathbf{207}$ mm

Area of tension reinforcement required $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem}) = \mathbf{999}$ mm^2/m

Minimum area of tension reinforcement $A_{s\_stem\_min} = k \times b \times t_{wall} = \mathbf{390}$ mm^2/m

Area of tension reinforcement required $A_{s\_stem\_req} = \text{Max}(A_{s\_stem\_des}, A_{s\_stem\_min}) = \mathbf{999}$ mm^2/m

Reinforcement provided **B1131 mesh**

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

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress $v_{stem} = V_{stem} / (b \times d_{stem}) = \mathbf{0.052}$ N/mm^2

Allowable shear stress $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1\text{ N/mm}^2)}, 5) \times 1\text{ N/mm}^2 = \mathbf{5.000}$ N/mm^2

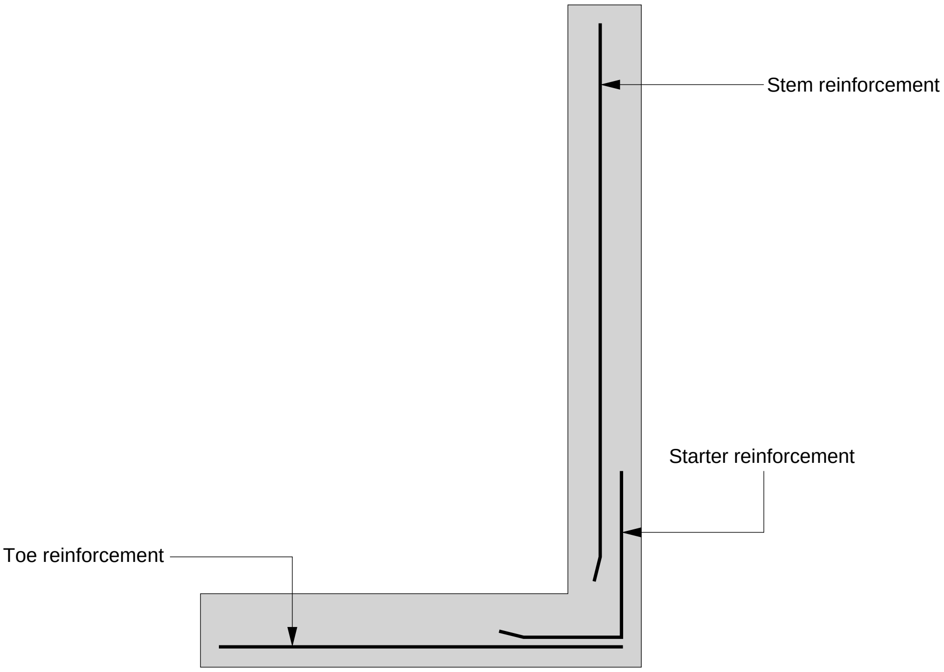
PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress $v_{c\_stem} = \mathbf{0.689}$ N/mm^2

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

Indicative retaining wall reinforcement diagram



Toe bars - 16 mm dia.@ 100 mm centres - (2011 mm^2/m)
Stem mesh - B1131 - (1131 mm^2/m)

Appendix C

Method Statement

Job Number: 140407
Date: 30/04/14

CROFT
STRUCTURAL
ENGINEERS

Croft Structural Engineers
Clock Shop Mews
Rear of 60 Saxon Road
London SE25 5EH

T: 020 8684 4744
E: enquiries@croftse.co.uk
W: www.croftse.co.uk

Basement Method Statement

Property Details:
25 Oakhill Avenue
Hampstead
NW3 7RD

Client Information:
Mr R Hay

Revision	Date	Comment
-	23/06/14	First Issue for BIA

LABC Regional winner 2013 awards
constructionline
TRADA
ice Institution of Civil Engineers
The Institution of Structural Engineers

Contents

25 Oakhill Avenue

1. Basement Formation Suggested Method Statement.

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

1.2. This method statement 25 Oakhill Avenue has been written by a Chartered Engineer. The sequencing has been developed considering guidance from ASUC.

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

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

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

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

1.7. A soil investigation has been undertaken. The soil conditions are claygate member of the London Clay Formation.

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

1.9. The water table is not encountered below ground level 7.8m.

2. Enabling Works

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

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

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

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

3. Basement Sequencing

3.1. Remove the existing ground floor structure above the proposed basement.

3.2. Erect conveyor to the front of the property.

3.3. Needle & prop the ground floor/ walls over.

3.4. Begin by placing cantilevered walls 1, 2, 3 etc. noted on plans. (Cantilevered walls to be placed in accordance with section 4.)

3.5. Insert steel over and sit on cantilevered walls.

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

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

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

3.6.1. 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)

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

3.7. Cast base to internal columns. Construct columns to provide support to floor and steels as works progress.

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

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

4. Underpinning and Cantilevered Walls

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

4.2. Provide propping to floor where necessary.

4.3. Excavate first section of retaining wall (no more than 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.

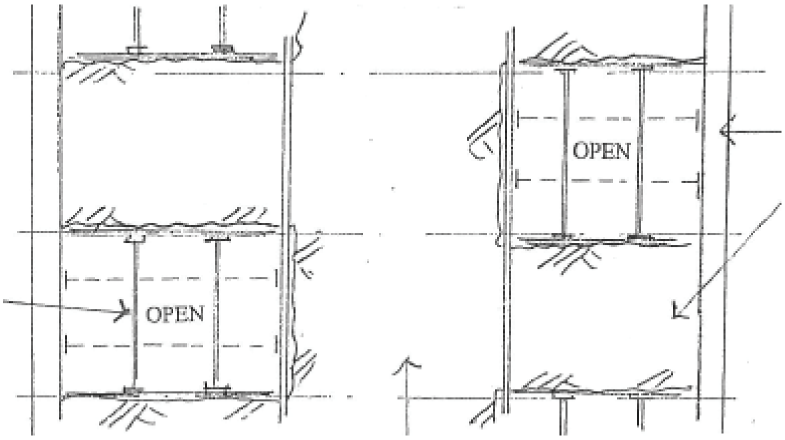


Figure 1 – Schematic Plan view of Soil Propping

Figure 2 Propping

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

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

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

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

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

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

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

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

4.8. Clear underside of existing footing.

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

4.10. Place blinding.

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

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

4.13. 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.

Ensure that Concrete is of sufficient strength, check engineers specifications

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

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

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

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

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

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

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

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

5. Floor Support

Timber Floor

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

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

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

Concrete Ground bearing slabs

5.4. The support of the existing concrete floor will be undertaken in conjunction with the underpinning process. Two opposite pins are constructed and allowed to cure as described elsewhere.

5.5. Locally prop concrete floors with Acros at 2m centres with timbers between. If the underside is found be in poor condition then temporary boarding and props are to be introduced.

5.6. Insert Steelwork and dry pack to underside of floor

5.7. Between steelwork place 215wide x 65dp PC lintels at a maximum spacing of 600mm

5.8. If necessary Brick up to the 50mm below underside of floor

5.9. Dry pack between lintel/brickwork to underside of slab.

5.10. Remove props

5.11. This process is to continue one pin width at a time.

6. Supporting existing walls above basement excavation

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

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

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

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

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

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

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

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

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

7. Approval

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

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

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

8. Trench sheet desgin and temporary prop Calculations

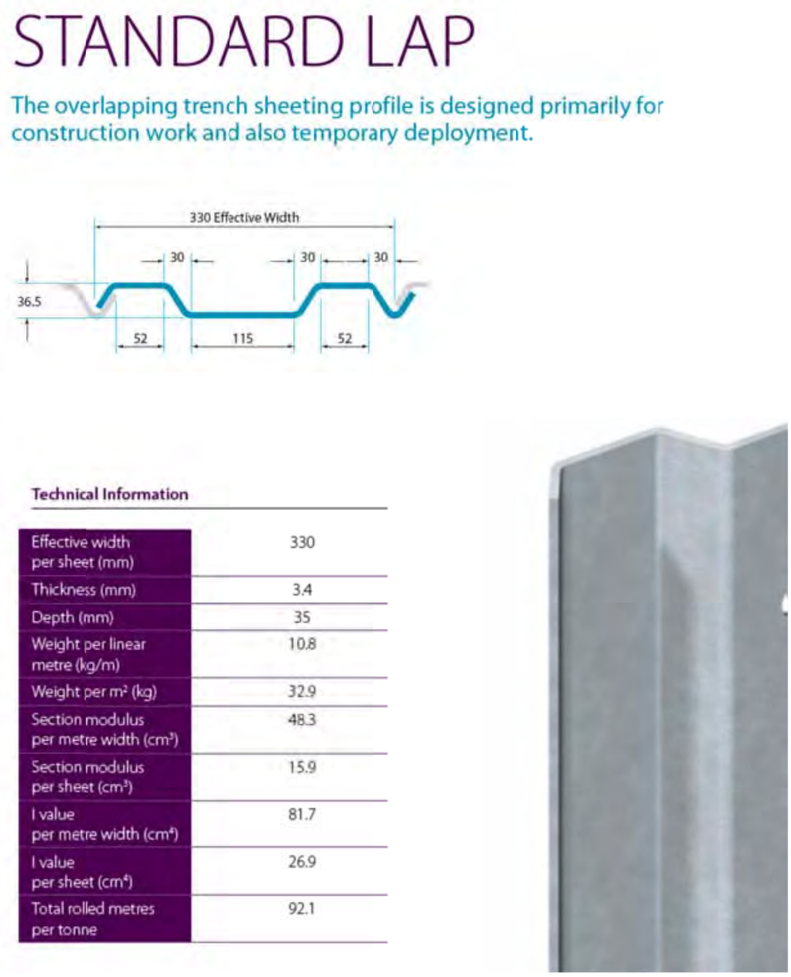
This calcualtion has been provided for the trench sheet and prop design of standard underpins in the temporary condition. There are gaps left between the sheeting and as such no water pressure will occur. Any water present will flow through the gaps betweenthe sheeting and will be required to pump out.

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

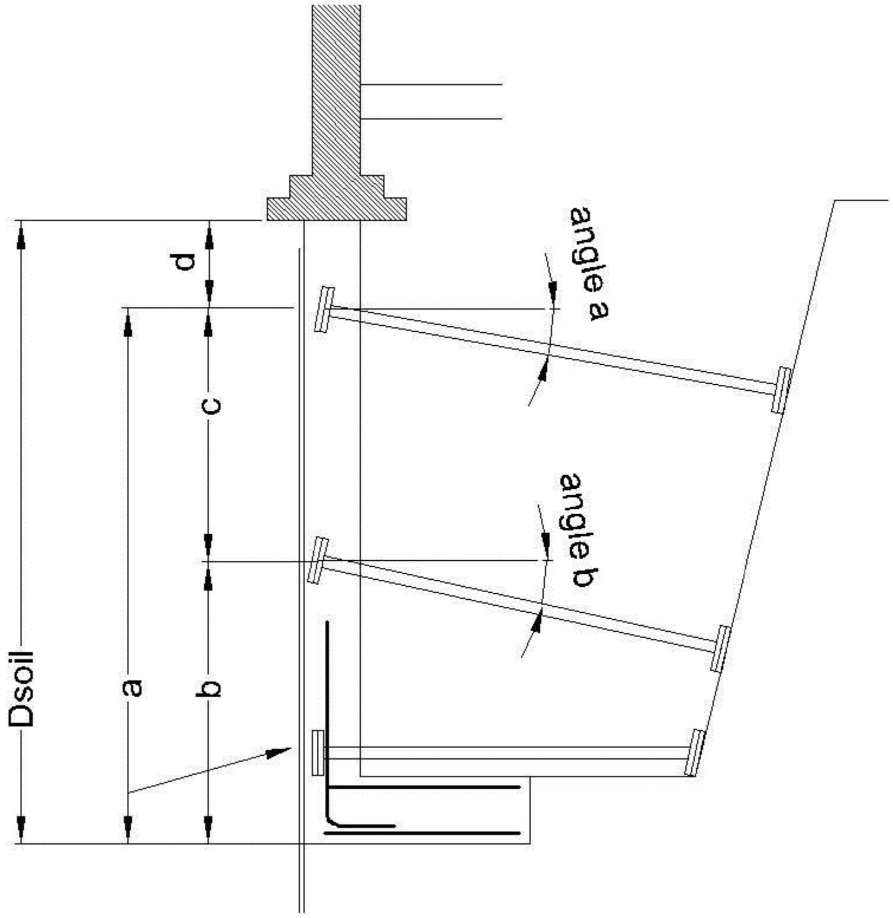
Effective width per sheet (mm)	330
Thickness (mm)	3.4
Depth (mm)	35
Weight per linear metre (kg/m)	10.8
Weight per m² (kg)	32.9
Section modulus per metre width (cm³)	48.3
Section modulus per sheet (cm³)	15.9
I value per metre width (cm⁴)	81.7
I value per sheet (cm⁴)	26.9
Total rolled metres per tonne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

$Sxx = 15.9\ cm^3$

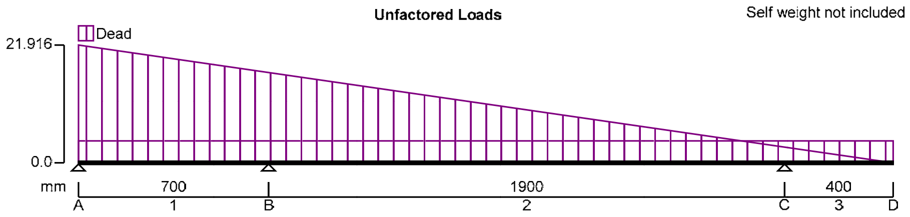
$py = 275N/mm^2$

$Ixx = 26.9cm^4$

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



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

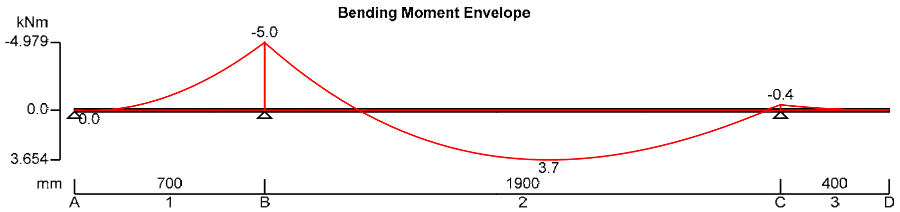
Support D Max react = **0.0** kN Min react = **0.0** kN Max mom = **0.0** kNm Min mom = **0.0** kNm

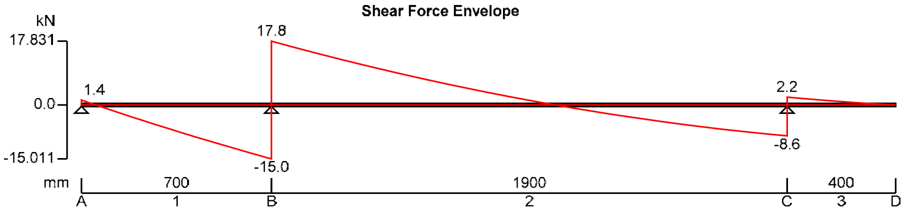
Beam Max/Min results - Combination Summary

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

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

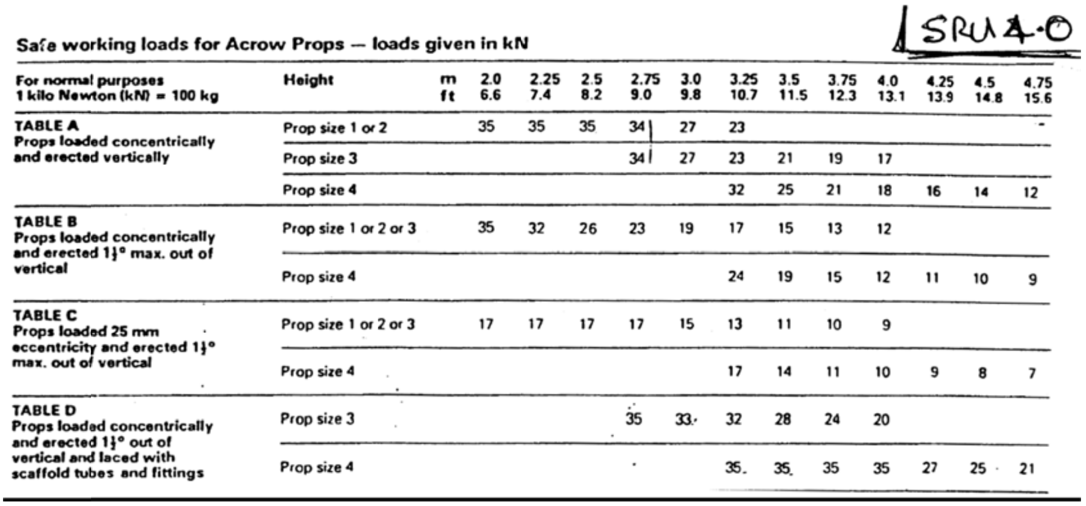
Maximum deflection = **21.0** mm Minimum deflection = **-14.3** mm





Number of sheets Nos = 2

Mallowable = Sxx * py * Nos = **8.745k**Nm



SRU4·0

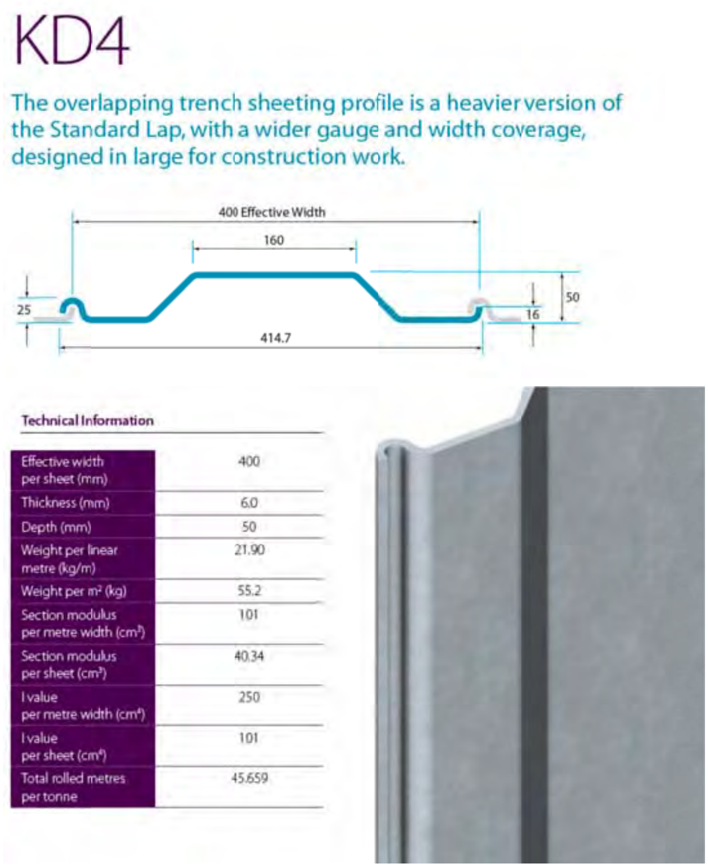
Safe working loads for Acrow Props — loads given in kN

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

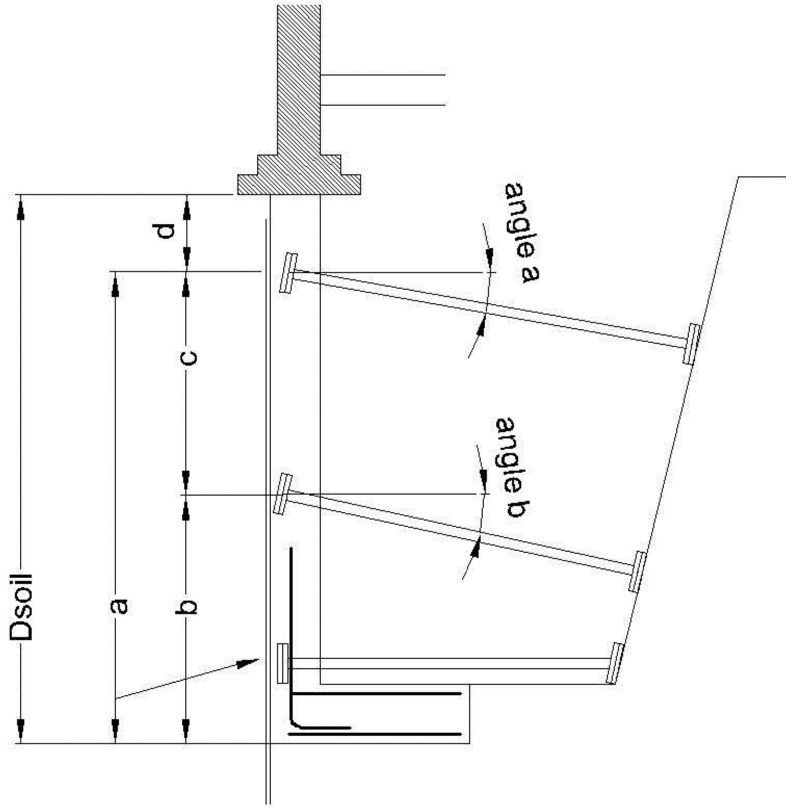
Effective width per sheet (mm)	400
Thickness (mm)	6.0
Depth (mm)	50
Weight per linear metre (kg/m)	21.90
Weight per m² (kg)	55.2
Section modulus per metre width (cm³)	101
Section modulus per sheet (cm³)	40.34
I value per metre width (cm⁴)	250
I value per sheet (cm⁴)	101
Total rolled metres per tonne	45.659

$Sxx = 48.3cm^3$

$py = 275N/mm^2$

$Ixx = 26.9cm^4$

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

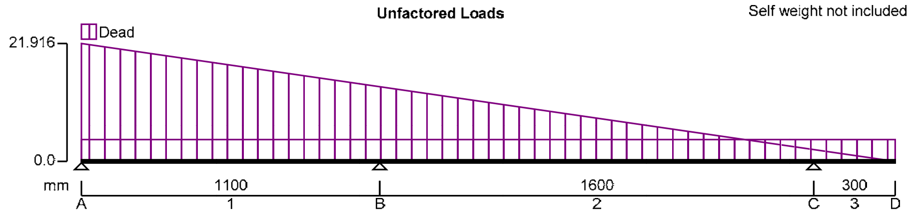


Length a a = **2.700** m

Length b bottom b = **1.100** m

Length c Middle c = a – b = **1.600**m

Length d top d = Dsoil – a = **0.300**m



CONTINUOUS BEAM ANALYSIS - INPUT

BEAM DETAILS

Number of spans = **3**

Material Properties:

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

Support Conditions:

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

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

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

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

Span Definitions:

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

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

Span 3 Length = **300** mm Cross-sectional area = **17806** mm^2 Moment of inertia = **269.×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

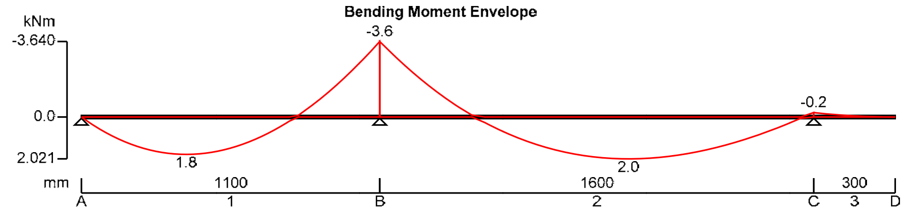
Support D Max react = **0.0** kN Min react = **0.0** kN Max mom = **0.0** kNm Min mom = **0.0** kNm

Beam Max/Min results - Combination Summary

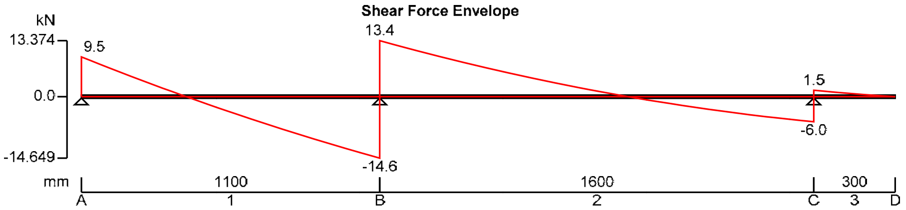
Maximum shear = **13.4** kN Minimum shearF_{min} = **-14.6** kN

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

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



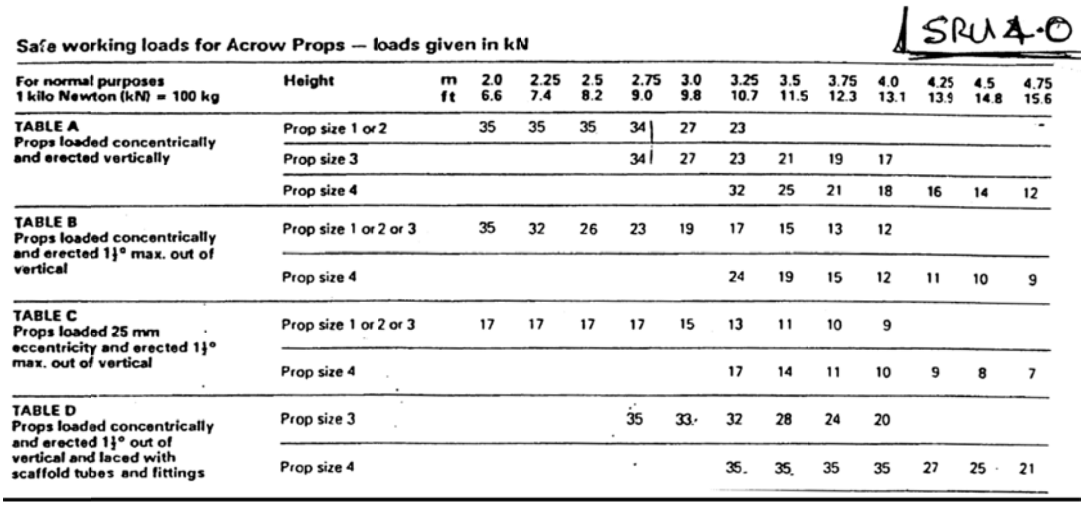
Bending Moment Envelope



Shear Force Envelope

Number of sheets Nos = 2

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



SRU 4.0

Safe working loads for Acrow Props — loads given in kN

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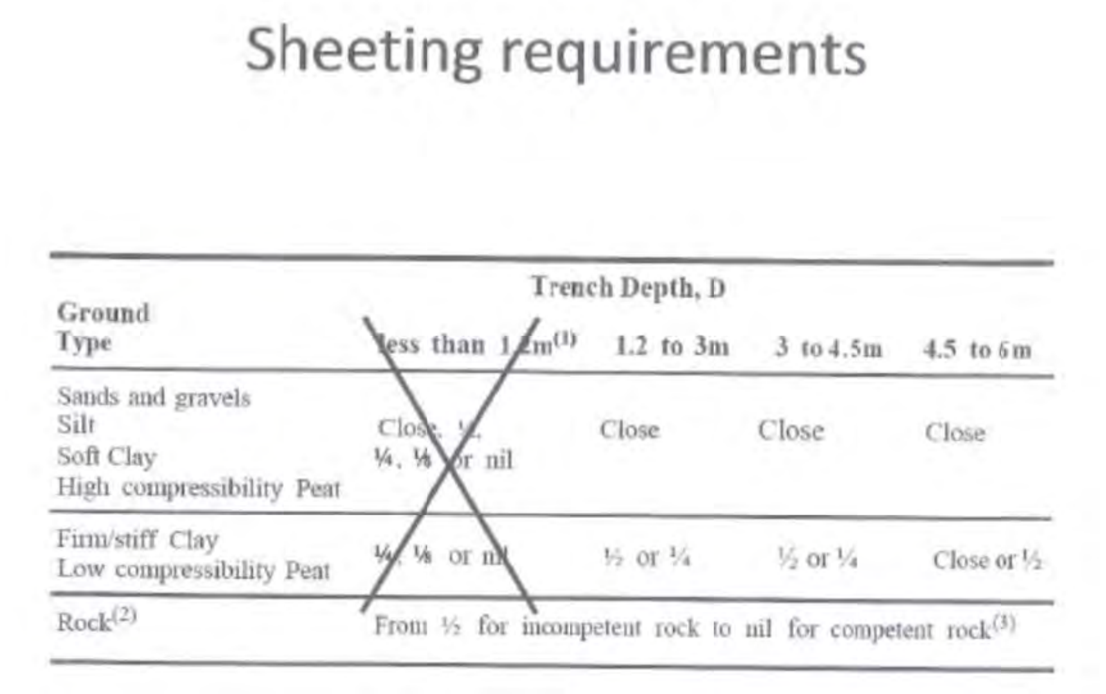
TABLE A Props loaded concentrically and erected vertically	Prop size 1 or 2		35	35	35	34	27	23						-
	Prop size 3					34	27	23	21	19	17			
	Prop size 4							32	25	21	18	16	14	12
TABLE B Props loaded concentrically and erected 1½° max. out of vertical	Prop size 1 or 2 or 3		35	32	26	23	19	17	15	13	12			
	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

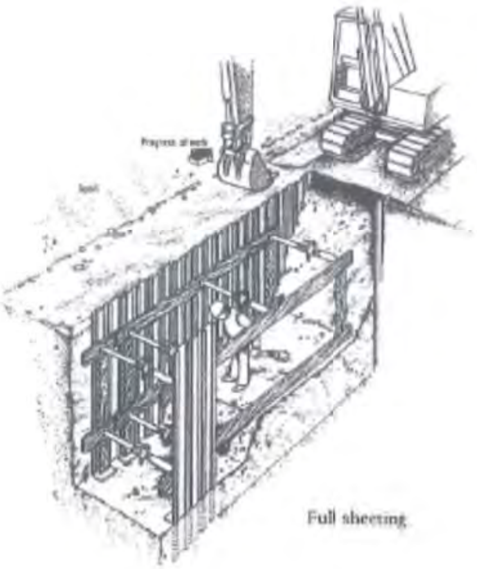
Any Acro Prop is accetpable



Sheeting requirements

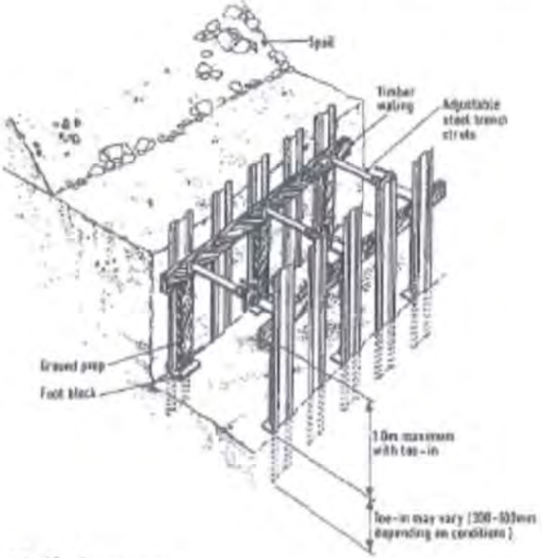
Ground Type	Trench Depth, D ~~less than 1.2m(1)~~	 1.2 to 3m	 3 to 4.5m	 4.5 to 6m
Sands and gravels Silt Soft Clay High compressibility Peat	~~Close, ½, ¼, ⅛ or nil~~	Close	Close	Close
Firm/stiff Clay Low compressibility Peat	~~¼, ⅛ or nil~~	½ or ¼	½ or ¼	Close or ½
Rock(2)	From ½ for incompetent rock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

Sheeting requirements

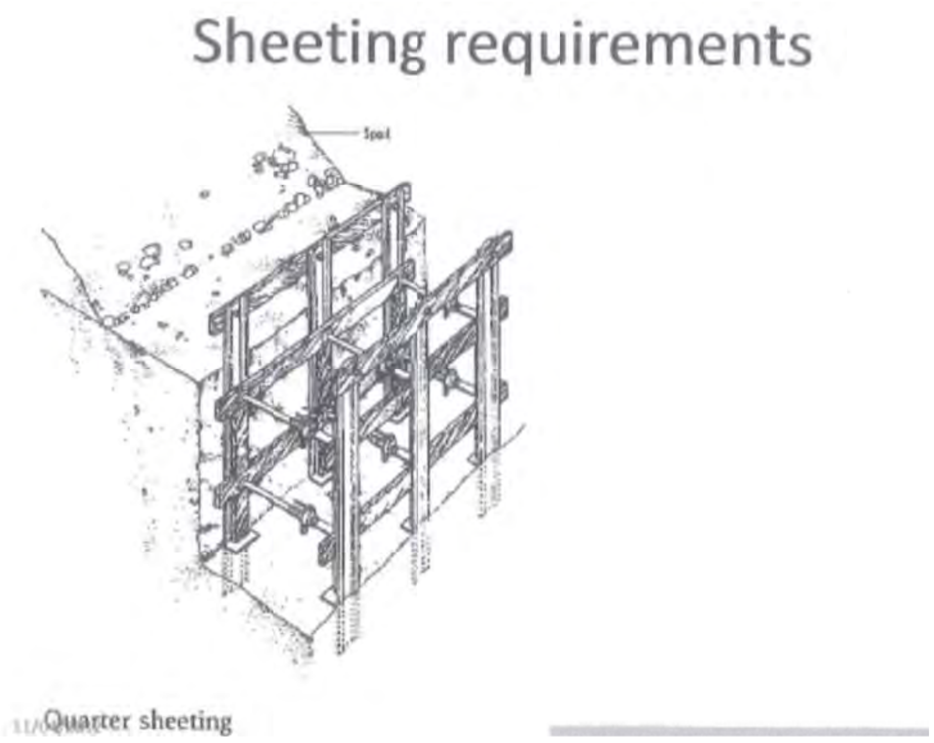


Full sheeting

Sheeting requirements

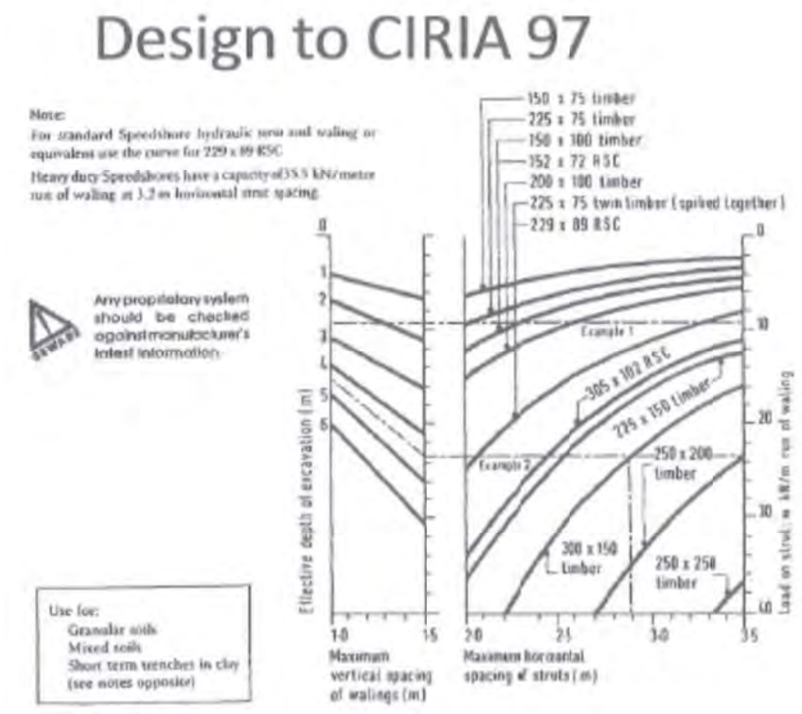


Half sheeting
shown for 1.5 m deep trench



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

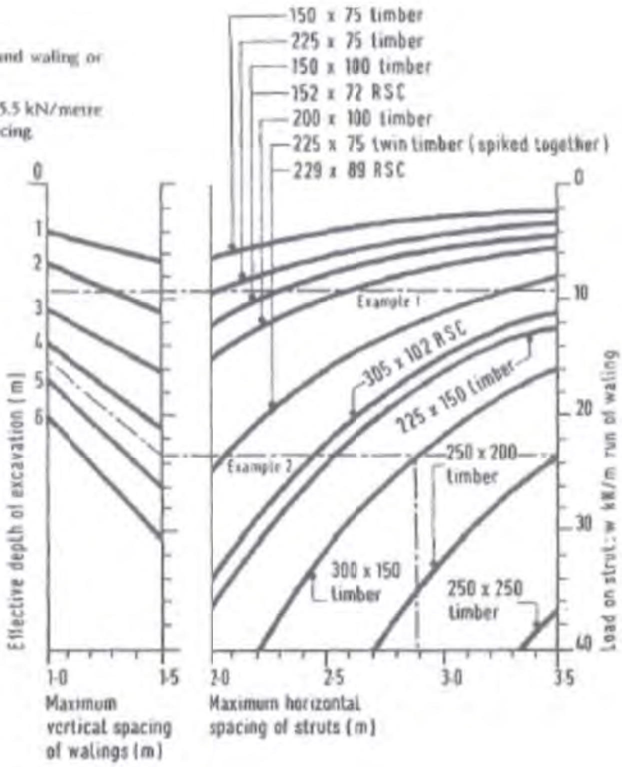
Use for:
Granular soils
Mixed soils
Short term trenches in clay
(see notes opposite)

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.

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&water

GROUND INVESTIGATION REPORT

for the site at

25 OAKHILL AVENUE, HAMPSTEAD, LONDON NW3 7RD

on behalf of

ROY HAY c/o CROFT STRUCTURAL ENGINEERS LIMITED

Report Reference: GWPR914/GIR/June 2014		Status: FINAL
Issue:	Prepared By:	Verified By:
V1.01 June 2014		T. Williams
	Roger Foord BA (Hons) MSc DIC FGS MSoBRA Senior Geotechnical Engineer	Francis Williams M.Geol. (Hons) FGS CEnv AGS MSoBRA Director
File Reference: Ground and Water/Project Files/ GWPR914 25 Oakhill Avenue, Hampstead		

CONTENTS

FIGURES

APPENDICES

1.0 INTRODUCTION

1.1 General

Ground and Water Limited were instructed by Roy Hay, c/o Croft Structural Engineers Limited, on the 22nd April 2014 to undertake a Ground Investigation on a site at 25 Oakhill Avenue, Hampstead, London NW3 7RD. The scope of the investigation was detailed within the Ground and Water Limited fee proposal ref: GWQ2104, dated 15th April 2014.

1.2 Aims of the Investigation

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

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

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

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

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

1.3 Conditions and Limitations

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

2.0 SITE SETTING

2.1 Site Location

The site comprised an approximately rectangular shaped plot of land, totalling ~475m^2 in area and orientated in a north by north-west to south by south-east direction, located on the south-eastern side of Oakhill Avenue. The site was located ~30m north-east of its junction with Bracknell Gardens and ~60m south-west of its junction with Greenway Gardens. The site was located in Hampstead in the London Borough of Camden.

The national grid reference for the centre of the site was approximately TQ 25652 85618. A site location plan is given within Figure 1 and a plan. A plan showing the site area is given within Figure 2.

2.2 Site Description

The site was occupied by a semi-detached two storey brick built residential house with roof accommodation. A soft landscaped front garden with paved pathway was accessed via a ~0.80m wide gate. The rear garden of the property was accessible through the existing building or via a gated access down the north-east side of the property. Bracknell Gardens, ~30m south-west of the site, was noted to be at ~90m AOD.

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

2.3 Proposed Development

At the time of reporting, June 2014, the proposed development will comprise the construction of a basement beneath the front half of the house. The basement is anticipated to be founded at ~3.0 – 3.5m below existing ground level (bgl).

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 North London) 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 borehole, in similar geology, located ~450m north-west of the site. The borehole encountered ~0.60m of Made Ground to overlie grey and brown clayey silts, becoming sandy with depth, to ~5.70m bgl. Dark grey silty clays were then proved to the base of the borehole at 15.45m bgl.

2.5 Slope Stability and Subterranean Developments

The site was not situated within an area where a natural or man-made slope of greater than 7^o was present (Figure 16 Camden Geological, Hydrogeological and Hydrological Study). The site was located close, to the south-east, to areas where a natural or man-made slope of greater than 7^o was noted.

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

Figure 18 of the Camden Geological, Hydrogeological and Hydrological Study indicated that no major subterranean infrastructure (including existing and proposed tunnels) was noted within close proximity to the site.

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) Aquifer** relating to the bedrock of the Claygate Member of the London Clay Formation underlain by **Unproductive Strata** comprising the bedrock of the London Clay Formation. No designation was given for any superficial deposits due to their likely absence.

Secondary aquifers include a wide range of drift deposits with an equally wide range of water permeability and storage capacities. Secondary (A) Aquifers consist of deposits with 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 Aquifers.

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

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

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

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 11 revealed the site was located close to where a southerly flowing tributary of the “Lost” Westbourne River was present.

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

From analysis of hydrogeological and topographical maps groundwater was anticipated to be encountered at moderate to deep depth (4-6m below existing ground level (bgl)) and it was considered that the groundwater was flowing in a south-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 15 the Camden Geological, Hydrogeological and Hydrological Study revealed that whilst Oakhill Avenue was not subject to surface water flooding, Bracknell Gardens, ~30m to the south-west, suffered surface water flooding in 2002.

2.7 Radon

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

3.0 FIELDWORK

3.1 Scope of Works

Fieldwork was undertaken on the 2nd May 2014 and comprised the drilling of one Premier Windowless Borehole (BH1) at the front of the property to a depth of 7.80m bgl, one window sampler borehole (WS1) at the rear of the property to a depth of 5.00m bgl and the hand excavation of two trial pit foundation exposures (TP/FE1 and TP/FE2).

A groundwater monitoring standpipe was installed in BH1 to a depth of 5.00m bgl to enable the measurement of standing groundwater levels.

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

Combined Bio-gas and Groundwater Monitoring Well Construction

Trial Hole	Depth of Installation (m bgl)	Thickness of slotted piping with gravel filter pack (m)	Depth of plain piping with bentonite seal (m bgl)	Piping external diameter (mm)
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 David McMillan of Ground and Water Limited generally in accordance with BS EN 14688 'Geotechnical Investigation and Testing – Identification and Classification of Soil'.

The ground conditions encountered within the trial holes constructed on the site generally conformed to that anticipated from examination of the geology map. A capping of Made Ground was noted to overlie the soils of the Claygate Member of the London Clay Formation. The deposits of the London Clay Formation were noted at depth.

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

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

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

Made Ground
Claygate Member of the London Clay Formation
London Clay Formation (BH1 and WS1)

Made Ground

Made Ground was encountered from ground surface in BH1 and TP/FE1 and beneath decking in WS1 and TP/FE2 to a depth of between 0.30m bgl in WS2 and 0.90m bgl in BH1 and TP/FE2.

The Made Ground comprised a dark brown to black gravelly sandy clay to a depth of 0.30m bgl in WS2 and 0.40m bgl in BH1 and TP/FE2. The sand was fine to medium grained and the gravel was rare, fine, angular flint, brick and carbonaceous material (clinker).

A brown to orange brown, locally grey, gravelly sandy, locally silty, clay was encountered between 0.40-0.90m bgl in BH1 and 0.30-0.84m bgl in TP/FE1. The sand was fine grained and the gravel was rare, fine, angular flint, brick and carbonaceous material (clinker).

Claygate Member of the London Clay Formation

Soils described as Claygate Member of the London Clay Formation and generally comprising a light brown, brown, orange brown and grey, locally sandy and gravelly, silty clay was encountered for the remaining depth of TP/FE1, a depth of 1.00m bgl, and TP/FE2, a depth of 1.20m bgl, and to a depth of 4.00m bgl in BH1 and 4.50m bgl in WS1. The sand, where encountered, was fine grained and the gravel was rare, fine to coarse, sub-rounded to angular flint.

London Clay Formation

Soils of the London Clay Formation, generally comprising a dark brown to dark grey silty clay, were encountered underlying the Claygate Member of the London Clay Formation for the remaining depth of each of the boreholes, a depth of 7.80m bgl in BH1 and 5.00m bgl in WS1. In BH1 rare shell fragments were noted between 5.60-7.80m bgl.

4.2 Foundation Exposures

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

TP/FE1

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

The foundation exposure was measured from ground level.

The foundation layout encountered consisted of a brick wall to ground level. From ground level to a depth of 0.30m bgl a brick wall was noted. Two brick steps out (both 0.06m in width and 0.07m in thickness) from the property were noted to rest upon a concrete footing that stepped out by 0.15m and was 0.40m in thickness. The foundation was noted to rest upon soils described as Claygate Member of the London Clay Formation comprising an orange brown to light brown sandy silty clay at 0.84m bgl. Made Ground was noted to 0.84m bgl in 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. 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 0.70m bgl a brick wall was noted. Three brick steps out (each 0.07m in width and 0.06m in thickness) from the property were noted to rest upon a concrete footing that stepped out by 0.50m and was 0.20m in thickness. The foundation was noted to rest upon soils described as Claygate Member of the London Clay Formation comprising a grey to orange brown slightly gravelly sandy clay at 1.08m bgl. Made Ground was noted to 0.90m 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 1.50m bgl	Decaying, assumed relic, roots to 3.50m bgl
WS2	Roots to 1.00m bgl	Decaying, assumed relic, roots to 3.00m bgl
TP/FE1	None	None
TP/FE2	Roots to 0.80m bgl	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 seepage was noted in BH1 at 6.40m bgl. Groundwater was not encountered in the remaining trial holes. The standpipe installed in BH1 had been tampered with and it was not possible to take a reading during a return site visit on the 30th May 2014.

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 2014, when groundwater levels are falling from their annual maximum (highest elevation).

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

4.5 Obstructions

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

5.0 INSITU AND LABORATORY GEOTECHNICAL TESTING

5.1 In-Situ Geotechnical Testing

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 and 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	5 – 13	25 – 65	Low – Medium	-	BH1 (0.90 – 4.00m bgl)
London Clay Formation	12 – >37	60 - >185	Medium – Very High		BH1 (4.00 – 7.80m 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 and the London Clay Formation. The results of the tests are presented in Appendix C.

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

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

Standard Methodology for Laboratory Geotechnical Testing

Test	Standard	Number of Tests
Atterberg Limit Tests	BS1377:1990:Part 2:Clauses 3.2, 4.3 & 5	6
Moisture Content	BS1377:1990:Part 2:Clause 3.2	15
Water Soluble Sulphate & pH	BS1377:1990:Part 3:Clause 5	2
BRE Special Digest 1 (incl. Ph, Electrical Conductivity, Total Sulphate, W/S Sulphate, Total Chlorine, W/S Chlorine, Total Sulphur, Ammonium as NH4, W/S Nitrate, W/S Magnesium)	BRE Special Digest 1 “Concrete in Aggressive Ground (BRE, 2005).	1

5.2.1 Atterberg Limit Tests

A précis of Atterberg Limit Tests undertaken on five samples of the Claygate Member of the London Clay Formation and one sample 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	26 - 28	100	28 – 33	CI	Stiff	Medium	Medium
London Clay Formation	30	100	48	CH	Stiff	High	High

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

5.2.2 Comparison of Soil's Moisture Content with Index Properties

5.2.2.1 Liquidity Index Analyses

The results of the Atterberg Limit tests undertaken on five samples of the Claygate Member of the London Clay Formation and one sample 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 **BH1/1.50m bgl** (Brown, orange brown and pale grey slightly fine sandy silty CLAY)	28	22	29	0.000	Heavily Overconsolidated.
Claygate Member of the London Clay Formation **BH1/2.50m bgl** (Brown and pale grey sandy silty CLAY)	27	21	29	0.207	Overconsolidated
Claygate Member of the London Clay Formation **BH1/3.50m bgl** (Brown and pale grey sandy silty CLAY)	27	21	29	0.207	Overconsolidated.
London Clay Formation **BH1/7.00m bgl** (Grey silty CLAY)	30	28	48	0.042	Heavily Overconsolidated.
Claygate Member of the London Clay Formation **WS1/1.00m bgl** (Brown and grey silty CLAY with occasional sand patches)	27	21	33	0.182	Heavily Overconsolidated.
Claygate Member of the London Clay Formation **WS1/2.00m bgl** (Brown and pale grey sandy silty CLAY)	26	21	28	0.179	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 or the heavily overconsolidated sample 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 overpage.

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 **BH1/1.50m bgl** (Brown, orange brown and pale grey slightly fine sandy silty CLAY)	28	51	20.4	MC > 0.4 x LL (No significant moisture deficit)
Claygate Member of the London Clay Formation **BH1/2.50m bgl** (Brown and pale grey sandy silty CLAY)	27	50	20.0	MC > 0.4 x LL (No significant moisture deficit)
Claygate Member of the London Clay Formation **BH1/3.50m bgl** (Brown and pale grey sandy silty CLAY)	27	50	20.0	MC > 0.4 x LL (No significant moisture deficit)
London Clay Formation **BH1/7.00m bgl** (Grey silty CLAY)	30	76	30.4	**MC < 0.4 x LL** **(Potentially significant moisture deficit)**
Claygate Member of the London Clay Formation **WS1/1.00m bgl** (Brown and grey silty CLAY with occasional sand patches)	27	54	21.6	MC > 0.4 x LL (No significant moisture deficit)
Claygate Member of the London Clay Formation **WS1/2.00m bgl** (Brown and pale grey sandy silty CLAY)	26	49	19.6	MC > 0.4 x LL (No significant moisture deficit)

The results in the table above indicate that a potential significant moisture deficit was present within one sample of the London Clay Formation tested (BH1/7.00m). The moisture content value was marginally below 40% of the liquid limit.

The sample was described as a grey silty clay. Roots were noted to a depth of 1.50m with decaying/relic roots to 3.50m bgl in BH1. Geotechnical testing on a shallower samples in the borehole showed no potential moisture deficit. The apparent moisture deficit is most likely to be related to the lithology of the soil (heavily overconsolidated soils) rather than the water demand from the roots.

The results in the table above indicate that the remaining 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

Moisture content versus depth plots for BH1 and WS1 can be seen within Figures 7 and 8 and show minor variations in moisture content (1-2%) 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 two samples from the Claygate Member of the London Clay Formation (BH1/3.00m and WS1/2.00m bgl). The sulphate concentration ranged from 20-40mg/l with a pH range of 7.6-7.7.

5.2.5 BRE Special Digest 1

In accordance with BRE Special Digest 1 ‘Concrete in Aggressive Ground’ (BRE, 2005) one sample of the Claygate Member of the London Clay Formation (WS1/3.00m) were 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	-	6.6	-
Ammonium as NH_4	mg/kg	8.1	-
Sulphur	mg/kg	<200	-
Chloride (water soluble)	mg/kg	9	-
Magnesium (water soluble)	g/l	0.048	-
Nitrate (water soluble)	mg/kg	5	-
Sulphate (water soluble)	g/l	0.01	-
Sulphate (total)	mg/kg	<200	-

6.0 ENGINEERING CONSIDERATIONS

6.1 Soil Characteristics and Geotechnical Parameters

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

- Made Ground was encountered from ground surface in BH1 and TP/FE1 and beneath a decking in WS1 and TP/FE2 to a depth of between 0.30m bgl in WS2 and 0.90m bgl in BH1 and TP/FE2.

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

- Soils described as Claygate Member of the London Clay Formation and generally comprising a light brown, brown, orange brown and grey, locally sandy and gravelly, silty clay was encountered for the remaining depth of TP/FE1, a depth of 1.00m bgl, and TP/FE2, a depth of 1.20m bgl, and to a depth of 4.00m bgl in BH1 and 4.50m bgl in WS1. The sand, where encountered, was fine grained and the gravel was rare, fine to coarse, sub-rounded to angular flint.

  The cohesive soils of the Claygate Member of the London Clay Formation comprised low to medium undrained shear strength (25-65kPa) soils between 0.90-4.00m 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 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 minor variation 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.

- Soils of the London Clay Formation, generally comprising a dark brown to dark grey silty clay, were encountered underlying the Claygate Member of the London Clay Formation for the remaining depth of each of the boreholes, a depth of 7.80m bgl in BH1 and 5.00m bgl in WS1. In BH1 rare shell fragments were noted between 5.60-7.80m bgl.

The cohesive soils of the London Clay Formation comprised medium to very high undrained shear strength (60->185kPa) soils from 4.00-7.80m bgl with the strength generally increasing with depth.

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

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

Geotechnical analysis revealed a potential significant moisture deficit was present within the sample of the London Clay Formation tested because the moisture content value was marginally below 40% of the liquid limit. The sample was described as a grey silty clay. Roots were noted to a depth of 1.50m with decaying/relic roots to 3.50m bgl in BH1. Geotechnical testing on shallower samples in the borehole showed no potential moisture deficit. The apparent moisture deficit is most likely to be related to the lithology of the soil (heavily overconsolidated soils) rather than the water demand from the roots.

Moisture content profiling indicated that the moisture profile with depth within the London Clay Formation was as expected with minor variation noted associated with small changes in lithology.

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

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

- A groundwater seepage was noted in BH1 at 6.40m bgl. Groundwater was not encountered in the remaining trial holes. The standpipe installed in BH1 had been tampered with and it was not possible to take a reading during the return site visit on the 30th May 2014.

- Roots were noted to a depth of 1.50m bgl in BH1, 1.00m bgl in WS1 and 0.80m bgl in TP/FE1. Decaying, assumed to be relic, roots were also noted to 3.50m bgl in BH1 and 3.00m bgl in WS2.

6.2 Basement Foundations

At the time of reporting, June 2014, the proposed development will comprise the construction of a basement beneath the front half of the house. The basement is anticipated to be founded at ~3.0 – 3.5m below existing ground level (bgl).

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

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

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

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

Roots were noted to a depth of 1.50m bgl in BH1, 1.00m bgl in WS1 and 0.80m bgl in TP/FE1. Decaying, assumed to be relic roots were also noted to 3.50m bgl in BH1 and 3.00m bgl in WS2. The moisture content profile showed minor variations in soil moisture content which could be attributed to minor changes in lithology. None of the samples taken from the Claygate Member of the London Clay Formation showed evidence for a potential moisture deficit. Therefore it was assessed that the roots noted between 3.00 – 3.50m bgl are relic and unlikely to pose a risk to the serviceability of the proposed development.

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.

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	124.81
	5.00m by 1.00m Strip	125.84
3.50m	5.00m by 0.75m Strip	128.44
	5.00m by 1.00m Strip	129.14

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	120	~19
	5.00m by 1.00m Strip	125	~24
3.50m	5.00m by 0.75m Strip	125	~16
	5.00m by 1.00m Strip	125	~20

It must be noted that a bearing capacity of less than 50kN/m^2 at 3.00-3.50m 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 Made Ground, Claygate Member of the London Clay Formation and London Clay Formation are likely to be marginally stable at best. Long, deep excavations, through both of these strata are likely to become unstable.

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

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

Based on the ground conditions encountered within the boreholes the following parameters could be used in the design of retaining walls. These have been designed based on the 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
Claygate Member of the London Clay Formation	~20-22	0	24	0.42	2.37
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 perched 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 Claygate Member of the London Clay Formation, especially after a period of excessive rainfall. Consideration should be given to creating a coffer dam using contiguous piled or sheet piled walls to aid basement construction below the perched water table.

6.5 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 Made Ground over cohesive soils of the Claygate Member of the London Clay Formation and the London Clay Formation. Based on a visual appraisal of the soils encountered the permeability of the both the Claygate Beds and London Clay Formation was likely to be very low to negligible permeability.

A groundwater seepage was noted in BH1 at 6.40m bgl. Groundwater was not encountered in the remaining trial holes. The standpipe installed in BH1 had been tampered with and it was not possible to take a reading during the return site visit on the 30th May 2014.

The Environment Agency records show that the highest recorded tide for the nearest river station on the River Thames at Westminster is 4.50m AOD with high tides generally at ~3.00m AOD. The elevation of the site is ~90.00m AOD. Based on a 3.00-3.50m bgl deep basement slab a formation level of 37.00-39.50m AOD is assumed. This means that the basement will be constructed above general high tide levels of the River Thames.

Based on the above it is considered 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 Claygate Member of the London Clay Formation, especially after a period of excessive rainfall.

In relation to the basement, once constructed, the Made Ground will act as a slightly porous medium for water to migrate however additional drainage should be considered as the Claygate

Member of the London Clay Formation and the London Clay Formation will act as a barrier for groundwater migration.

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 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-1s for foundations within the Claygate Member of the London Clay Formation. For the classification given, the "static" and "natural" case was adopted given the cohesive nature of the deposits (permeability unlikely to exceed 10-7 m/se) and residential use of the site.

The sulphate concentration in the samples ranged from 10-40mg/l with a pH range of 6.6-7.7. The total sulphate concentration recorded was <0.02%.

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 Claygate Member of the London Clay Formation and the underlying 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 classified the waste can be removed to the appropriately licensed facilities, with some waste requiring pre-treatments prior to disposal.

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

6.10 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.

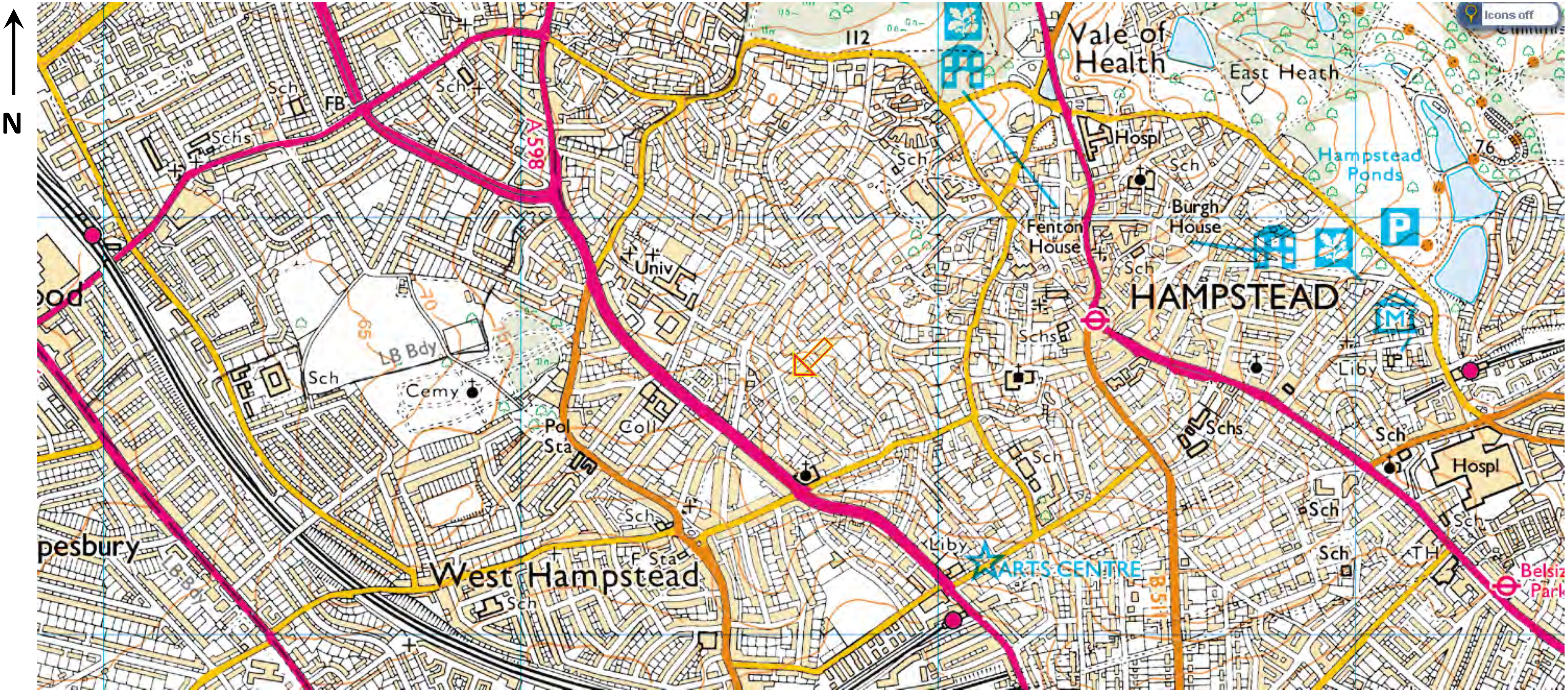
The Topsoil must be fit for purpose and must either be supplied with traceable chemical laboratory test certificates or be tested, either prior to placing (ideally) or after placing, to ensure that the human receptor cannot come into contact with compounds that could be detrimental to human health.

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

NOT TO SCALE

Project: 25 Oakhill Avenue, Hampstead, London NW3 7RD	**Figure 1**
Client: Roy Hay c/o Croft Structural Engineers Limited	**Date:** June 2014
Site Location Plan	**Ref:** GWPR914

ground&water

— Approximate Site Boundary

NOT TO SCALE

Project:
25 Oakhill Avenue, Hampstead, London NW3 7RD

Figure 2

Client:
Roy Hay c/o Croft Structural Engineers Limited

Date:
June 2014

Site Development Area

Ref:
GWPR914

ground&water

— Approximate Site Boundary

NOT TO SCALE

Project:

25 Oakhill Avenue, Hampstead, London NW3 7RD

Client:

Roy Hay c/o Croft Structural Engineers Limited

Date:

June 2014

Aerial View of the Site

Ref:

GWPR914

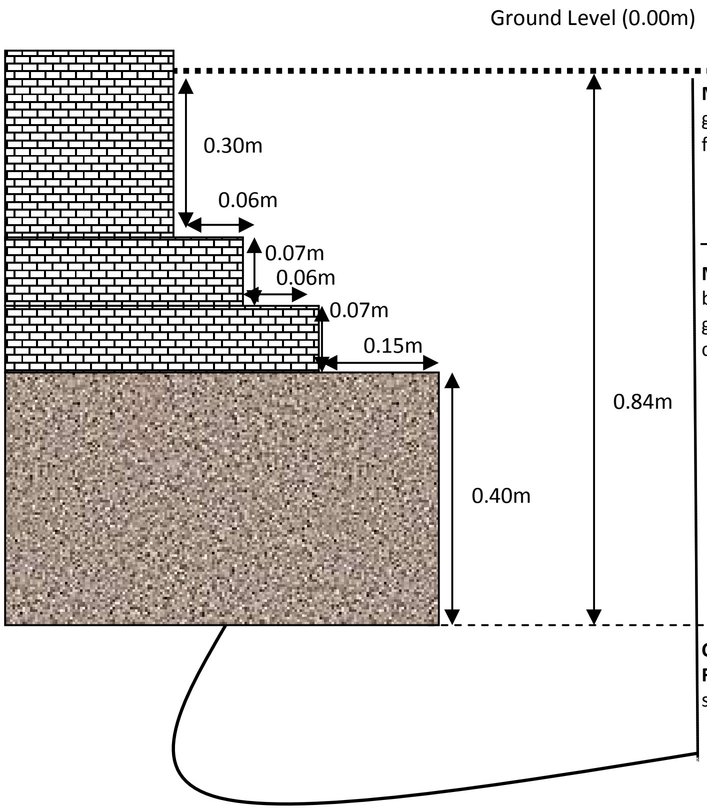
Figure 3

ground&water

🟥 Approximate Site Boundary

NOT TO SCALE

Project: **25 Oakhill Avenue, Hampstead, London NW3 7RD**		**Figure 4** ground&water
Client: **Roy Hay c/o Croft Structural Engineers Limited**	**Date:** **June 2014**	
Trial Hole Location Plan	**Ref:** **GWPR914**	



MADE GROUND: (GL - 0.30m bgl) Grey and orange gravelly sandy clay. Sand is fine grained. Gravel is rare, fine, angular flint and clinker**.**

MADE GROUND: (0.30 – 0.84m bgl) Brown to orange brown slightly gravelly sandy silty clay. Sand is fine grained. Gravel is rare, fine, angular flint, brick and clinker.

CLAYGATE MEMBER OF THE LONDON CLAY FORMATION: (0.84– 1.00m bgl) Orange to light brown sandy silty CLAY. Sand is fine grained.

▦ Brick

▨ Concrete

NOTE: NOT TO SCALE

Project:
25 Oakhill Avenue, Hampstead, London NW3 7RD

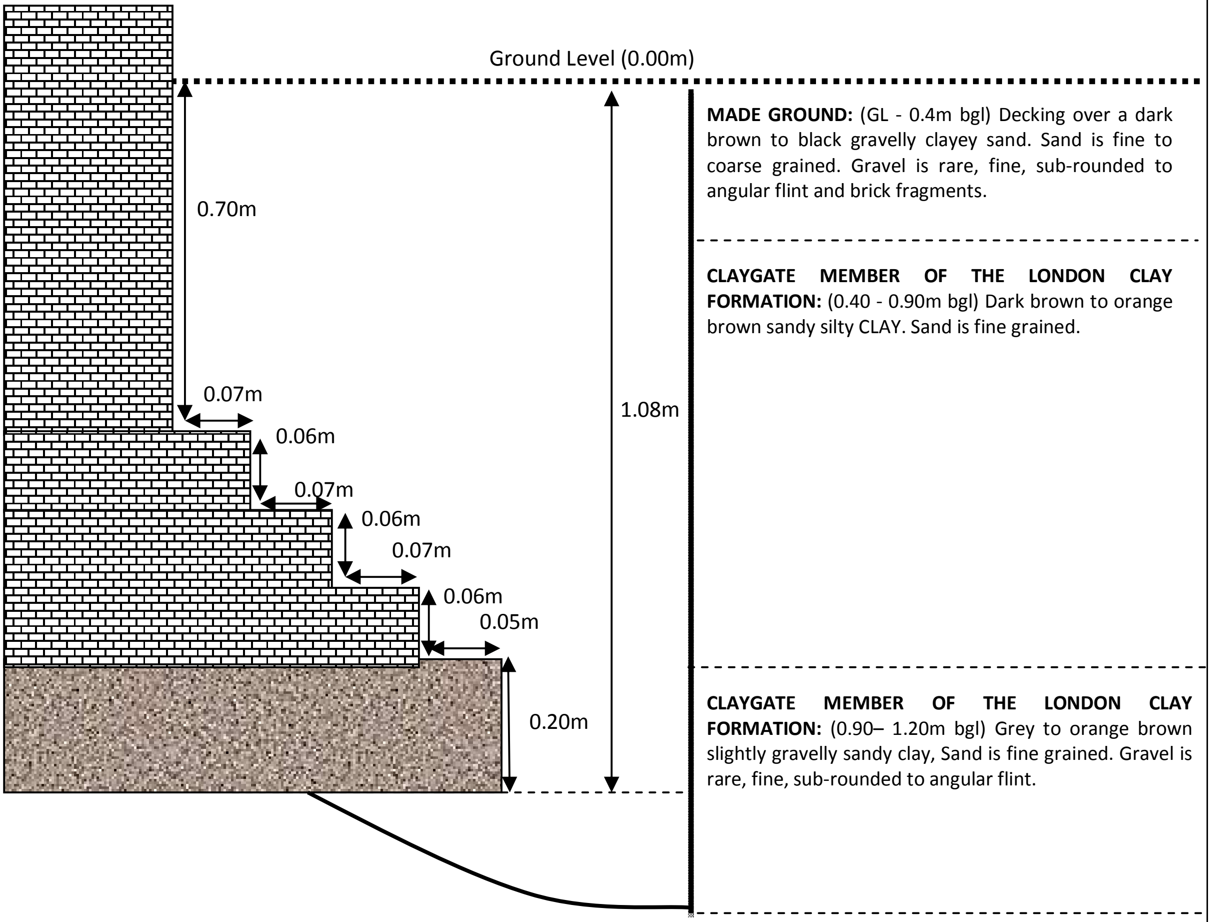
Client:
Roy Hay c/o Croft Structural Engineers Limited

Date:
June 2014

Section Drawing: Foundation Exposure TP/FE1

Ref:
GWPR914

Figure 5



MADE GROUND: (GL - 0.4m bgl) Decking over a dark brown to black gravelly clayey sand. Sand is fine to coarse grained. Gravel is rare, fine, sub-rounded to angular flint and brick fragments.

CLAYGATE MEMBER OF THE LONDON CLAY FORMATION: (0.40 - 0.90m bgl) Dark brown to orange brown sandy silty CLAY. Sand is fine grained.

CLAYGATE MEMBER OF THE LONDON CLAY FORMATION: (0.90– 1.20m bgl) Grey to orange brown slightly gravelly sandy clay, Sand is fine grained. Gravel is rare, fine, sub-rounded to angular flint.

▦ Brick

▒ Concrete

NOTE: NOT TO SCALE

Project:
25 Oak Hill Avenue, Hampstead, London NW3 7RD

Client:
Roy Hay c/o Croft Structural Engineers Limited

Date:
June 2014

Section Drawing: Foundation Exposure TP/FE2

Ref:
GWPR914

Figure 6

ground&water

Figure 7: Change in Moisture Content With Depth Within BH1

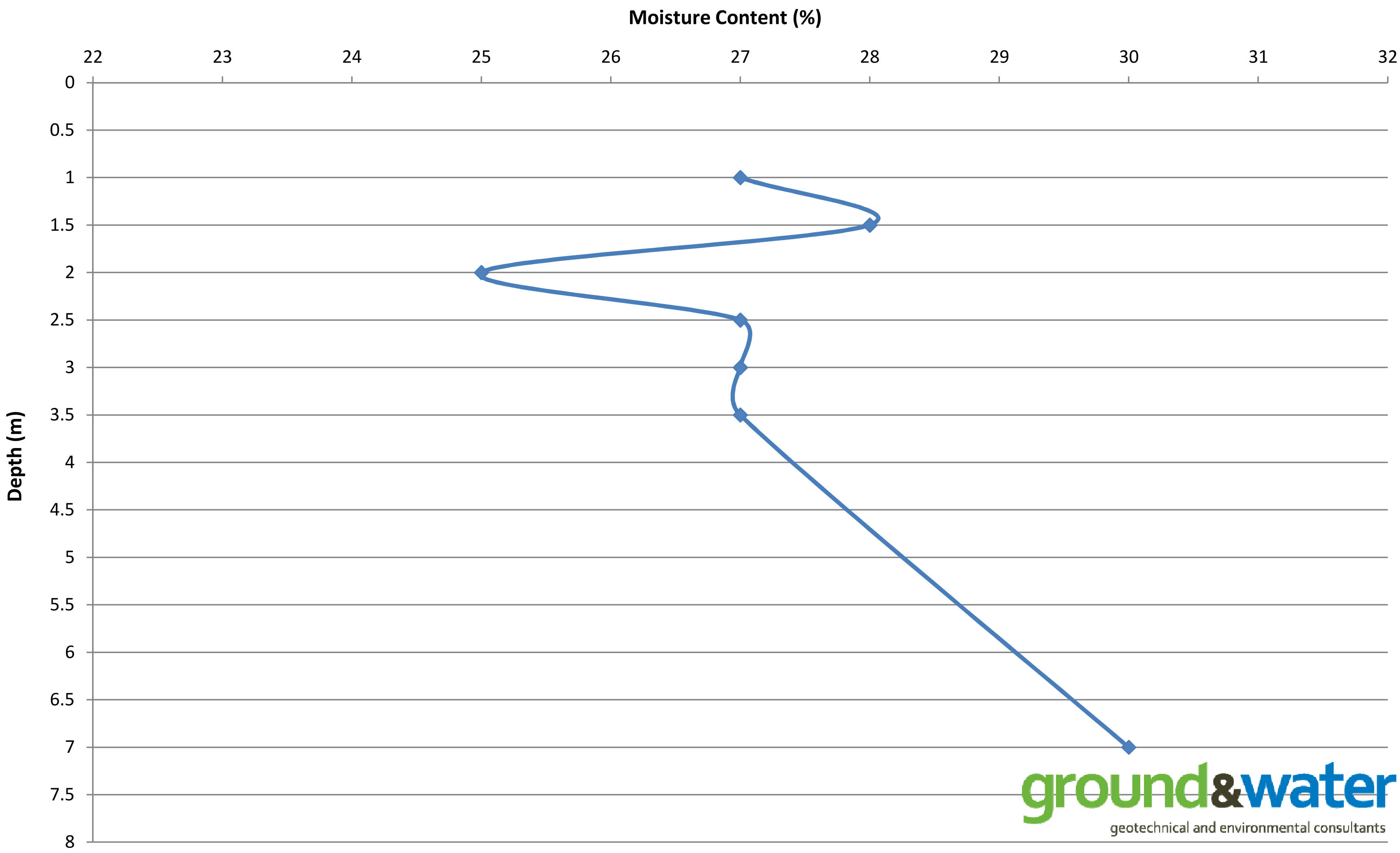
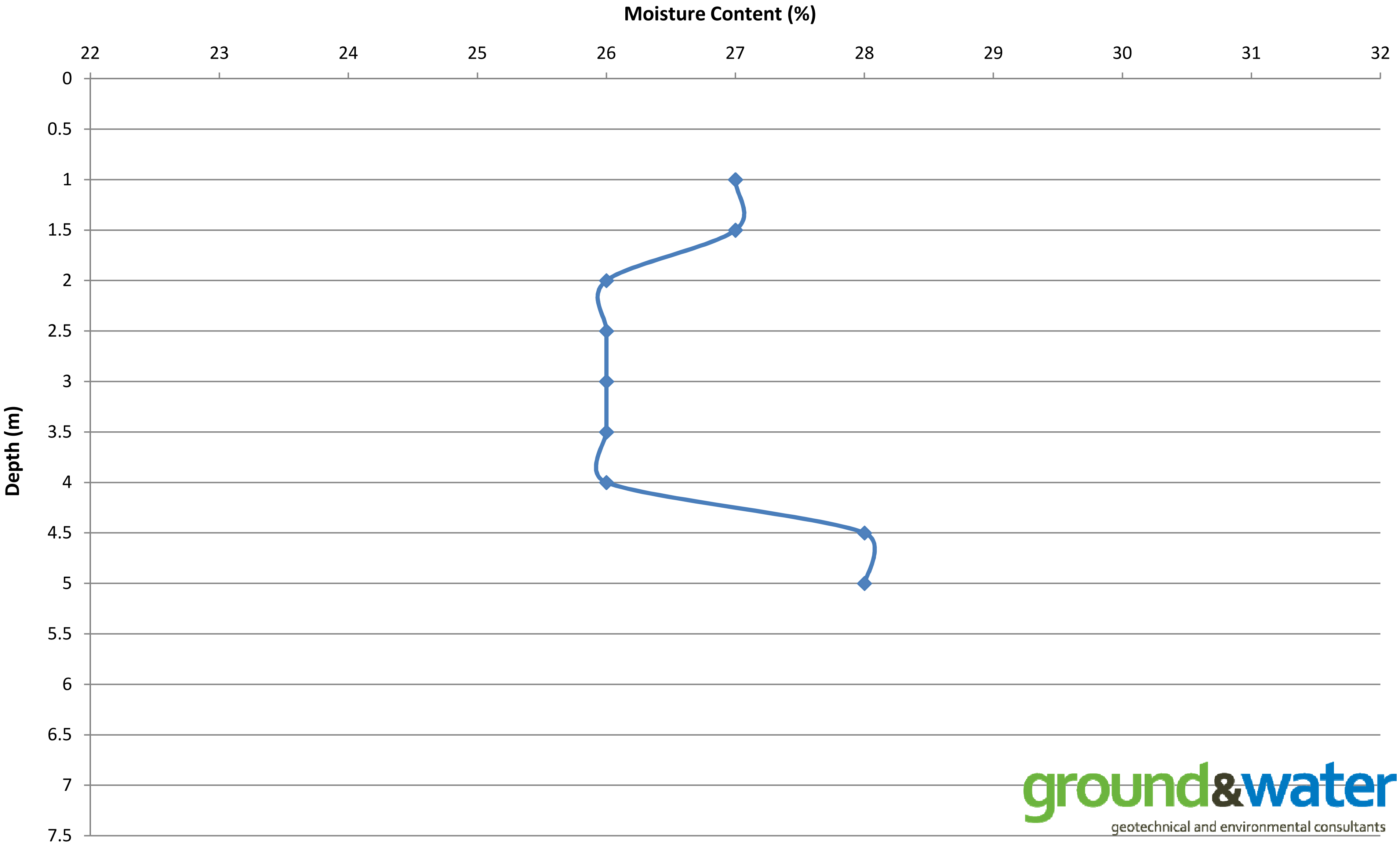


Figure 8: Change in Moisture Content With Depth Within WS1



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 25 Oakhill Avenue, Hampstead, London NW3 7RD.

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 1

Project Name 25 Oakhill Avenue	Project No. GWPR914	Co-ords: -	Hole Type WLS
Location: Hampstead, London NW3 7RD		Level: -	Scale 1:50
Client: Roy Hay		Dates: 02/05/2014	Logged By DM

Water Strikes	Depth (m)	Type	Results
	0.30	D	
	0.50	D	
	0.80	D	
	1.00	SPT	N=5 (1,1/ 1,2,1,1)
	1.00	D	
	1.50	D	
	2.00	SPT	N=12 (1,2/ 3,3,3,3)
	2.00	D	
	2.50	D	
	3.00	SPT	N=13 (2,2/ 3,3,4,3)
	3.00	D	
	3.50	D	
	4.00	SPT	N=12 (2,2/ 3,3,3,3)
	4.00	D	
	4.50	D	
	5.00	SPT	N=16 (2,3/ 3,4,4,5)
	5.00	D	
	5.50	D	
	6.00	SPT	N=22 (3,4/ 5,5,6,6)
	6.00	D	
▽			
	6.50	D	
	7.00	SPT	N=26 (4,4/ 6,6,7,7)
	7.00	D	
	7.50	D	
	7.80	SPT	37 (10,27/ 37)
	7.80	D	

Depth (m)	Level (m AOD)	Stratum Description
0.40		MADE GROUND. Dark brown to black gravelly sandy clay. Sand is fine to medium grained. Gravel is rare, fine, angular flint and clinker.
0.90		MADE GROUND. Brown to orange brown, with grey mottling, gravelly sandy silty clay. Sand is fine grained. Gravel is rare, fine, angular flint and clinker.
4.00		CLAYGATE MEMBER OF THE LONDON CLAY FORMATION. Orange to light brown sandy silty CLAY. Sand is fine grained.
5.60		LONDON CLAY FORMATION. Dark brown silty CLAY.
7.80		LONDON CLAY FORMATION. Dark grey silty CLAY. Rare shell fragments noted.

End of Borehole at 7.80 m

Remarks: Groundwater strike at 6.40m bgl
Live roots noted to 1.5m with decaying roots to 3.5m.

AGS

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

Borehole No
WS1
Sheet 1 of 1

Project Name 25 Oakhill Avenue	Project No. GWPR914	Co-ords: -	Hole Type WS
Location: Hampstead, London NW3 7RD		Level: -	Scale 1:50
Client: Roy Hay		Dates: 02/05/2014	Logged By DM

Well	Water Strikes	Samples & In Situ Testing Depth (m)	Type	Results	Depth (m)	Level (m AOD)	Legend	Stratum Description	
		0.30	D		0.30			MADE GROUND. Decking over a dark brown to black gravelly sandy clay. Sand is fine to medium grained. Gravel is rare, fine, angular flint and clinker.	
		0.50	D					CLAYGATE MEMBER OF THE LONDON CLAY FORMATION. Light brown to orange, with grey mottling, sandy silty CLAY. Sand is fine grained.	
		0.80	D						
		1.00	D		1.00			LONDON CLAY FORMATION. Dark grey brown silty sandy CLAY.	1
		1.50	D						
		2.00	D						2
		2.50	D						
		3.00	D						3
		3.50	D						
		4.00	D						4
		4.50	D		4.50			LONDON CLAY FORMATION: Dark grey brown silty CLAY.	
		5.00	D		5.00			End of Borehole at 5.00 m	5
									6
									7
									8
									9
			Type	Results					

Remarks: Groundwater not encountered.
Live roots to ~1.0m and decaying roots to ~3.0m.

APPENDIX C
Geotechnical Laboratory Test Results

Project Name: 25 Oakhill Avenue		**Samples Received:**	15/05/2014
		Project Started:	16/05/2014
Client: Ground and Water Ltd		**Testing Started:**	28/05/2014
Project No: GWPR914	**Our job/report no:** 16700	**Date Reported:**	29/05/2014

K4 SOILS

Borehole No:	Sample No:	Depth (m)	Description	Moisture content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing 0.425 mm (%)	Remarks
BH1	-	1.00	Brown and occasional grey slightly fine sandy silty CLAY	27					
BH1	-	1.50	Brown, orange brown and pale grey slightly fine sandy silty CLAY	28	51	22	29	100	
BH1	-	2.00	Brown and occasional grey slightly fine sandy silty CLAY	25					
BH1	-	2.50	Brown and pale grey sandy silty CLAY	27	50	21	29	100	
BH1	-	3.00	Brown mottled blue grey slightly fine sandy silty CLAY	27					
BH1	-	3.50	Brown and pale grey sandy silty CLAY	27	50	21	29	100	
BH1	-	7.00	Grey silty CLAY	30	76	28	48	100	
WS1	-	1.00	Brown and grey silty CLAY with occasional sandy patches	27	54	21	33	100	
WS1	-	1.50	Brown mottled blue grey slightly fine sandy silty CLAY	27					
WS1	-	2.00	Brown and pale grey sandy silty CLAY	26	49	21	28	100	
WS1	-	2.50	Brown mottled blue grey fine sandy silty CLAY	26					
WS1	-	3.50	Brown mottled blue grey fine sandy silty CLAY	26					
WS1	-	4.00	Brown mottled blue grey slightly fine sandy silty CLAY	26					
WS1	-	4.50	Dark grey brown silty CLAY	28					
WS1	-	5.00	Dark grey brown silty CLAY	28					

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: 29/05/2014

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

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

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.

Project Name: 25 Oakhill Avenue
Client: Ground and Water Ltd
Project no: GWPR914
Our job no: 16700

K4 SOILS

Borehole No:	Sample No:	Depth m	Description	pH	Sulphate content (g/l)
BH1	-	3.00	Brown mottled blue grey slightly fine sandy silty CLAY	7.7	0.04
WS1	-	2.00	Brown and pale grey sandy silty CLAY	7.6	0.02

Summary of Test Results

BS 1377 : Part 3 :Clause 5 : 1990

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

Date
29/05/2014

Checked and Approved
Initials : kp

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

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

QTS Environmental Report No: 14-21681

Site Reference:	25 Oakhill Avenue
Project / Job Ref:	GWPR914
Order No:	None Supplied
Sample Receipt Date:	19/05/2014
Sample Scheduled Date:	19/05/2014
Report Issue Number:	1
Reporting Date:	23/05/2014

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: 14-21681	**Date Sampled**	02/05/14				
Ground & Water Ltd	**Time Sampled**	None Supplied				
Site Reference: 25 Oakhill Avenue	**TP / BH No**	WS1				
Project / Job Ref: GWPR914	**Additional Refs**	None Supplied				
Order No: None Supplied	**Depth (m)**	3.00				
Reporting Date: 23/05/2014	**QTSE Sample No**	104344				

Determinand	**Unit**	**RL**	**Accreditation**					
pH	pH Units	N/a	**MCERTS**	6.6				
Total Sulphate as SO_4	mg/kg	< 200	NONE	< 200				
W/S Sulphate as SO4 (2:1)	g/l	< 0.01	**MCERTS**	0.01				
Total Sulphur	mg/kg	< 200	NONE	< 200				
Ammonium as NH_4	mg/kg	< 0.5	NONE	8.1				
W/S Chloride (2:1)	mg/kg	< 1	**MCERTS**	9				
Water Soluble Nitrate (2:1) as NO_3	mg/kg	< 3	**MCERTS**	5				
W/S Magnesium	g/l	< 0.0001	NONE	0.0048				

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: 14-21681
Ground & Water Ltd
Site Reference: 25 Oakhill Avenue
Project / Job Ref: GWPR914
Order No: None Supplied
Reporting Date: 23/05/2014

QTSE Sample No	TP / BH No	Additional Refs	Depth (m)	Moisture Content (%)	Sample Matrix Description
$ 104344	WS1	None Supplied	3.00	15.6	Light brown clay

Moisture content is part of procedure E003 & is not an accredited test

Insufficient Sample $^{I/S}$

Unsuitable Sample $^{U/S}$

$ samples exceeded recommended holding times

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

4480

Soil Analysis Certificate - Methodology & Miscellaneous Information
QTS Environmental Report No: 14-21681
Ground & Water Ltd
Site Reference: 25 Oakhill Avenue
Project / Job Ref: GWPR914
Order No: None Supplied
Reporting Date: 23/05/2014

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

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

Appendix E

Monitoring Statement and Plan

Job Number: 140407
20/06/14

CROFT
STRUCTURAL
ENGINEERS

Croft Structural Engineers
Clock Shop Mews
Rear of 60 Saxon Road
London SE25 5EH

T: 020 8684 4744
E: enquiries@croftse.co.uk
W: www.croftse.co.uk

Structural Monitoring Statement

Property Details:
25 Oakhill Avenue
Hampstead
NW3 7RD

Client Information:
Mr Roy Hay

Revision	Date	Comment
-	23/06/14	First Issue for Comment

Contents

25 Oakhill Avenue

1. Introduction

Basement works are intended to the above address. To undertake these works structural works will be undertaken that require party wall awards.

2. Risk assessment

The purpose of this risk assessment is to consider the impact of the proposed works and how they impact the party wall. There are varying levels of inspection that can be undertaken and not all works, soil conditions and properties required the same level of protection.

Monitoring 1

Visual inspection and production of condition survey by Party wall surveyors at the beginning of the works and also at the end of the works.

Monitoring 2

Visual inspection and production of condition survey by Party wall surveyors at the beginning of the works and also at the end of the works.
Visual inspection of existing party wall during the works.
Inspection of the footing to ensure that the footings are stable and adequate.

Monitoring 3

Visual inspection and production of condition survey by Party wall surveyors at the beginning of the works and also at the end of the works.
Visual inspection of existing party wall during the works.
Inspection of the footing to ensure that the footings are stable and adequate.
Vertical monitoring movement by standard optical equipment

Monitoring 4

Visual inspection and production of condition survey by Party wall surveyors at the beginning of the works and also at the end of the works.
Visual inspection of existing party wall during the works.
Inspection of the footing to ensure that the footings are stable and adequate.
Vertical monitoring movement by standard optical equipment
Lateral movement between walls by laser measurements

Monitoring 5

Visual inspection and production of condition survey by Party wall surveyors at the beginning of the works and also at the end of the works.
Visual inspection of existing party wall during the works.
Inspection of the footing to ensure that the footings are stable and adequate.
Vertical & Lateral monitoring movement by theodolite at specific times during the projects.

Monitoring 6

Visual inspection and production of condition survey by Party wall surveyors at the beginning of the works and also at the end of the works.
Visual inspection of existing party wall during the works.
Inspection of the footing to ensure that the footings are stable and adequate.
Vertical & Lateral monitoring movement by electronic means with live data gathering. Weekly interpretation

Monitoring 7

Visual inspection and production of condition survey by Party wall surveyors at the beginning of the works and also at the end of the works.
Visual inspection of existing party wall during the works.
Inspection of the footing to ensure that the footings are stable and adequate.
Vertical & Lateral monitoring movement by electronic means with live data gathering with data transfer.

3. Scheme Details

This document has been prepared by .Croft Structural Engineers Ltd. It covers the proposed construction of a new basement underneath the existing structure at 25 Oakhill Avenue.

Scope of Works

The works comprise:

- Visual Monitoring of the party wall
- Attachment of Tell tales or Demec Studs to accurately record movement of significant cracks.
- Attachment of levelling targets to monitor settlement.
- The monitoring of the above instrumentation is in accordance with Appendix A. The number and precise locations of instrumentation may change during the works; this shall be subject to agreement with the PC.
- All instruments are to be adequately protected against any damage from construction plant or private vehicles using clearly visible markings and suitable head protection e.g. manhole rings or similar. Any damaged instruments are to be immediately replaced or repaired at the contractors own cost.
- Reporting of all data in a manner easily understood by all interested parties.

- Co-ordination of these monitoring works with other site operations to ensure that all instruments can be read and can be reviewed against specified trigger values both during and post construction.
- Regular site meetings by the Principal Contractor (PC) and the Monitoring Surveyor (MS) to review the data and their implications.
- Review of data by Croft Structural Engineers

In addition, the PC will have responsibility for the following:

- Review of methods of working/operations to limit movements, and
- Implementation of any emergency remedial measures if deemed necessary by the results of the monitoring.

The Monitoring Surveyor shall allow for settlement and crack monitoring measures to be installed and monitored on various parts of the structure described in Table 1 as directed by the PC and Party Wall Surveyor (PWS) for the Client.

Item	Instrumentation Type
Party Wall Brickwork	
Settlement monitoring	Levelling equipment & targets
Crack monitoring	Visual inspection of Cracking, demec studs where necessary

Table 1: Instrumentation

General

The site excavations and substructure works up to finished ground slab stage have the potential to cause vibration and ground movements in the vicinity of the site due to the following:

a) Removal of any existing redundant foundations / obstructions;
b) Installation of reinforced concrete retaining walls under the existing footings;
c) Excavations within the site

The purpose of the Monitoring is a check to confirm wall movements are not excessive.

This Specification is aimed at providing a strategy for monitoring of potential ground and building movements at the site. The monitoring is a check to confirm building movements are not excessive.

This Specification is intended to define a background level of monitoring. The PC may choose to carry out additional monitoring during critical operations. Monitoring that is to be carried out is as follows:

a) Visual inspection of the party wall and any pre-existing cracking
b) Settlement of Party Wall

All instruments are to be protected from interference and damage as part of these works.

Access to all instrumentation or monitoring points for reading shall be the responsibility of the Monitoring Surveyor (MS). The MS shall be in sole charge for ensuring that all instruments or monitoring points can be read at each visit and for reporting of the data in a form to be agreed with the PWS. He shall inform the PC if access is not available to certain instruments and the PC will, wherever possible, arrange for access. He shall immediately report to the PC any damage. The Monitoring Surveyor and the Principal Contractor will be responsible for ensuring that all the instruments that fall under their respective remits as specified are fully operational at all times and any defective or damaged instruments are immediately identified and replaced.

The PC shall be fully responsible for reviewing the monitoring data with the MS, before passing onto the Croft Structural Engineers, determining its accuracy and assessing whether immediate action is to be taken by him and/or other contractors on site to prevent damage to instrumentation or to ensure safety of the site and personnel. All work shall comply with the relevant legislation, regulations and manufacturer's instructions for installation and monitoring of instrumentation.

Applicable Standards and References

The following British Standards and civil engineering industry references are applicable to the monitoring of ground movements related to activities on construction works sites:

1. BS 5228: Part 1: 1997 - Noise and Vibration Control on Construction and Open Sites -Part 1.Code of practice for basic information and procedures for noise and vibration control, Second Edition, BSI 1999.
2. BS 5228: Part 2: 1997 - Noise and Vibration Control on Construction and Open Sites -Part 2.Guide to noise and vibration control legislation for construction and demolition including road construction and maintenance, Second Edition, BSI 1997.
3. BS 7385-1: 1990 (ISO 4866:1990) - Evaluation and measurement for vibration in buildings - Part 1: Guide for measurement of vibrations and evaluation of their effects on buildings, First Edition, BSI 1990.
4. BS 7385-2: 1993 - Evaluation and measurement for vibration in buildings - Part 2: Guide to damage levels from ground-borne vibration, First Edition, BSI 1999.
5. CIRIA SP 201 - Response of buildings to excavation-induced ground movements, CIRIA 2001.

SPECIFICATION FOR INSTRUMENTATION

General

The Monitoring Contractor is required to monitor, protect and reinstall instruments as described. The readings are to be reported as specified in Section 4 of this Specification.
The following instruments are defined:

a) Automatic level and targets: A device which allows the measurement of settlement in the vertical axis. To be installed by the MS.

b) Tell-tales and 3 stud sets: A device which allows measurement of movement to be made in two axes perpendicular to each other. To be installed by the MS.

Monitoring of existing cracks

The locations of tell-tales or Demec studs to monitor existing cracks shall be agreed with Croft Structural Engineers.

Instrument Installation Records and Reports

Where instrumentation is to be installed or reinstalled, the Monitoring Surveyor, or the Principal Contractor, as may be applicable, shall make a complete record of the work, including the position and level of each instrument. The records shall include base readings and measurements taken during each monitoring visit. Both tables and graphical outputs of these measurements shall be presented in a format to be agreed with the CM. The report shall include photographs of each type of instrumentation installed and clear scaled sections and plans of each instrument installed. This report shall also include the supplier's technical fact sheet on the type of instrument used and instructions on monitoring.

Two signed copies of the report shall be supplied to the PWS within one week of completion of site measurements for approval.

Installation

All instruments shall be installed to the satisfaction of the PC. No loosening or disturbance of the instrument with use or time shall be acceptable. All instruments are to be clearly marked to avoid damage.

All setting out shall be undertaken by the Monitoring Surveyor or the Principal Contractor as may be applicable. The precise locations will be agreed by the PC prior to installation of the instrument.

The installations are to be managed and supervised by the Instrumentation Engineer or the Measurement Surveyor as may be applicable.

Monitoring

The frequencies of monitoring for each Section of the Works are given in Appendix A.

The following accuracies shall be achieved:

Party Wall settlement	±0.5mm
Crack monitoring	±0.5mm

REPORT OF RESULTS AND TRIGGER LEVELS

General

Within 24 hours of taking the readings, the Monitoring Surveyor will submit a single page summary of the recorded movements. All readings shall be immediately reviewed by Croft Structural Engineers prior to reporting to the PWS.

Within one working day of taking the readings the Monitoring Contractor shall produce a full report (see below).

The following system of control shall be employed by the PC and appropriate contractors for each section of the works. The Trigger value, at which the appropriate action shall be taken, for each section, is given in Appendix C.

The method of construction by use of sequential underpins limits the deflections in the party wall. The maximum movement across the length of the party wall must not exceed 5 mm.

Between the trigger points, which are no greater than 2 m apart, there should be no more than 3 mm movement.

During works measurements are taken, these are compared with the limits set out below:

Movement	CATEGORY	ACTION
0mm-5mm	Green	No action required
5mm-9mm	AMBER	Crack Monitoring: Carry out a local structural review; Preparation for the implementation of remedial measures should be required.
>9mm	RED	Crack Monitoring: Implement structural support as required; Cease works with the exception of necessary works for the safety and stability of the structure and personnel; Review monitoring data and implement revised method of works

Table 2 – Movement limits between adjacent sets of Tell-tales or stud sets

Any movements which exceed the individual amber trigger levels for a monitoring measure given in Appendix B shall be immediately reported to the PWS, and a review of all of the current monitoring data for all monitoring measures must be implemented to determine the possible causes of the trigger level being exceeded. Monitoring of the affected location must be increased and the actions described above implemented. Assessment of exceeded trigger levels must not be carried out in isolation from an assessment of the entire monitoring regime as the monitoring measures are inter-related. Where required, measures may be implemented or prepared as determined by the specific situation and combination of observed monitoring measurement data.

Standard Reporting

1 No. electronic copy of the report in PDF format shall be submitted to the PWS.

The Monitoring Surveyor shall report whether the movements are within (or otherwise) the Trigger Levels indicated in Appendix B. A summary of the extent of completion of any of the elements of works and any other significant events shall be given. These works shall be shown in the form of annotated plans (and sections) for each survey visit both local to the instrumentation and over a wider area. The associated changes to readings at each survey or monitoring point shall be then regulated to the construction activity so that the cause of any change, if it occurs, can be determined.

The Monitoring Surveyor shall also give details of any events on site which in his opinion could affect the validity of the results of any of the surveys.

The report shall contain as a minimum, for each survey visit the following information:

a) The date and time of each reading:
b) The weather on the day:
c) The name of the person recording the data on site and the person analysing the readings together with their company affiliations;
d) Any damage to the instrumentation or difficulties in reading;
e) Tables comparing the latest reading with the last reading and the base reading and the changes between these recorded data;
f) Graphs showing variations in crack width with time for the crack measuring gauges; and
g) Construction activity as described. It is very important that each set of readings is associated with the extent of excavation and construction at that time. Readings shall be accompanied by information describing the extent of works at the time of readings. This shall be agreed with the PC.

Spread-sheet columns of numbers should be clearly labelled together with units. Numbers should not be reported to a greater accuracy than is appropriate. Graph axis should be linear and clearly labelled together with units. The axis scales are to be agreed with the PC before the start of monitoring and are to remain constant for the duration of the job unless agreed otherwise. The specified trigger values are also to be plotted on all graphs.

The reports are to include progress photographs of the works both general to the area of each instrument and globally to the main Works. In particular, these are to supplement annotated plans/sections described above. Wherever possible the global photographs are to be taken from approximately the same spot on each occasion. The locations of these points on site are to be Croft Structural Engineers drawing SD-22.

Erroneous Data

All data shall be checked for errors by the Monitoring Surveyor prior to submission. If a reading that appears to be erroneous (i.e. it shows a trend which is not supported by the surrounding instrumentation), he shall notify the PC immediately, resurvey the point in question and the neighbouring points and if the error is repeated, he shall attempt to identify the cause of the error. Both sets of readings shall be processed and submitted, together with the reasons for the errors and details of remedial works. If the error persists at subsequent survey visits, the Monitoring Surveyor shall agree with the PC how the data should be corrected. Correction could be achieved by correcting the readings subsequent to the error first being identified to a new base reading.

The Monitoring surveyor shall rectify any faults found in or damage caused to the instrumentation system for the duration of the specified monitoring period, irrespective of cause, at his own cost.

Trigger Values

Trigger values for maximum movements as listed in Appendix B. If the movement exceeds these values then action may be required to limit further movement. The PC should be immediately advised of the movements in order to implement the necessary works.

It is important that all neighbouring points (not necessarily a single survey point) should be used in assessing the impact of any movements which exceed the trigger values, and that rechecks are carried out to ensure the data is not erroneous. A detailed record of all activities in the area of the survey point will also be required as specified elsewhere.

Responsibility for Instrumentation

The Monitoring Surveyor shall be responsible for: managing the installation of the instruments or measuring points, reporting of the results in a format which is user friendly to all parties; and immediately reporting to all parties any damage. The Monitoring Surveyor shall be responsible for informing the PC of any movements which exceed the specified trigger values listed in Appendix B so that the PC can implement appropriate procedures. He shall immediately inform the PWS of any decisions taken.

APPENDIX A

MONITORING FREQUENCY

INSTRUMENT	FREQUENCY OF READING
Settlement monitoring and Monitoring existing cracks	**Pre-construction** Monitored once. **During construction** Monitored after every pin is cast for first 4 no. pins to gauge effect of underpinning. If all is well, monitor after every other pin. **Post construction works** Monitored once.

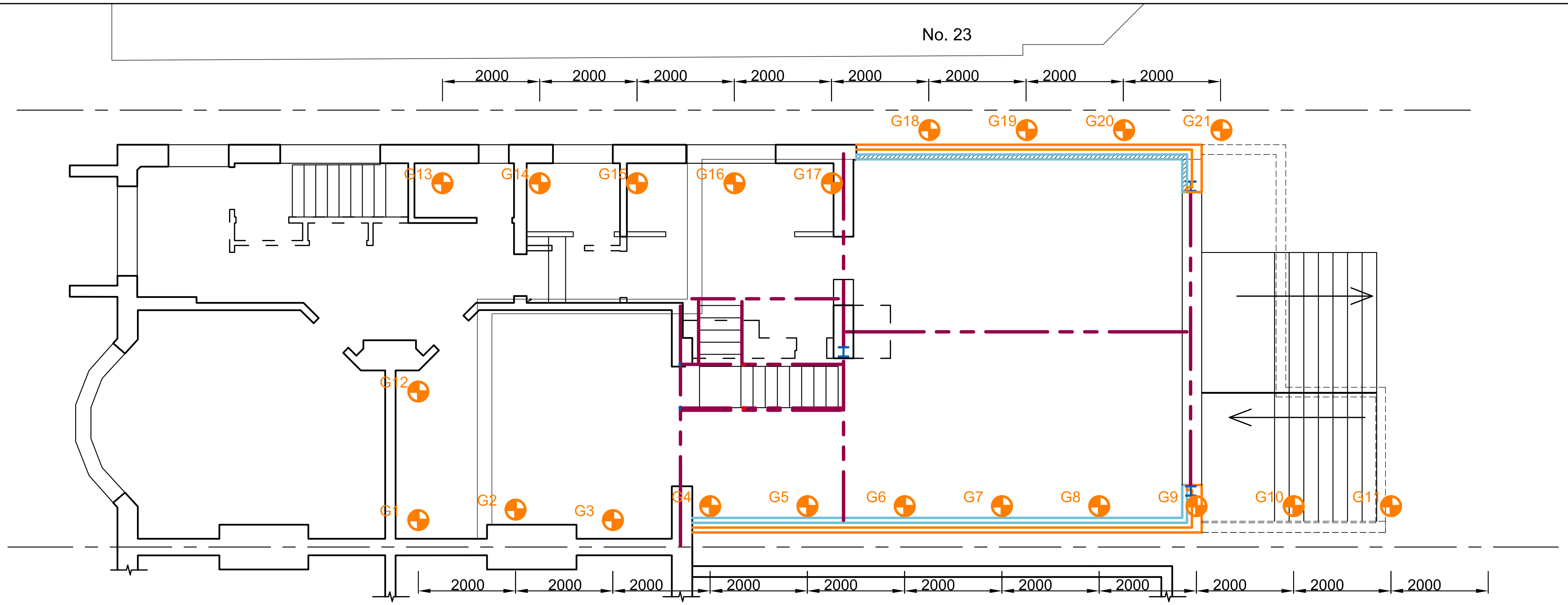
APPENDIX B

TRIGGER LEVELS

(ii) Settlement monitoring – all stages

Trigger values for Settlement Monitoring

	AMBER	RED
Party wall	5mm	10mm



Ground Floor Layout
Scale 1:100

Key

● Denotes position of Leveling Targets, fixed to party wall 500mm & 2000mm above Ground Floor Level.
Additional monitoring may be required for any cracking noted in the Party Wall Surveyor's survey.
Levels to be checked both vertically and horizontally

Settlement limit along length of wall	Category	Action
0-5mm	Green	Carry on Works
5-9mm	Amber	Carry out a local structural review; Preparation for the implementation of remedial measures should they be required.
>10mm	Red	Implement structural support as required; Cease works within a 5m zone of the affected section; Review monitoring data and implement revised method of works. Inform Croft Structural Engineers.

Rev	Date	Amendments
-	23/6/14	First issue for BIA

Job nos
140407

Dwg Nos
SL-14

date June 14

drawn AZ

Chk'd CT

Scale
1:100 @ A3

Rev
-

Client: Mr Roy Hay

Project: 25 Oakhill Avenue

Title : Monitoring Layout Plan

Croft
Structural
Engineers
Clockshop Mews,
r/o 60 Saxon Rd,
London, SE25 5EH.

020 8684 4744
www.croftse.co.uk

Appendix F

Information of neighbour's property

London Borough of Camden

Planning and Transport Department

Camden Town Hall
Argyle Street Entrance Euston Road
London WC1H 8EQ Tel: 278 4444

David Pike MSc CEng MICE MRTPI
Director of Planning and Transport

Our Reference: PL/8803789/
Case File No: E5/13/22
Tel.Inqu:
Ian Pestel ext. 2520
(Please ring after 2.00pm unless enquiring about Tree applications.)

Mr A J Moore
38 Ludlow Road
London
W5 1NY

Ref:AJ/8704

Date: 16 MAR 1989

Dear Sir(s)/Madam,

Town and Country Planning Act 1971 (as amended)

Permission for Development

The Council, in pursuance of its powers under the above-mentioned Act and Orders made thereunder, hereby permits the development referred to in the undermentioned Schedule subject to the conditions set out therein and in accordance with the plans submitted, save insofar as may otherwise be required by the said conditions.

Your attention is drawn to the General Information attached hereto.

Your attention is also drawn to the Statement of Applicants Rights.

SCHEDULE

Date of Original Application : 29th March 1988

Address : 23 Oakhill Avenue NW3

Proposal : The erection of conservatory and basement extension to the rear, as shown on drawing nos. AJ/8704/1 and 2.

Standard Condition:
1. The development hereby permitted must be begun not later than the expiration of five years from the date of this permission.

Reason for Standard Condition:
1. In order to comply with the provisions of Section 41 of the Town and Country Planning Act 1971.

Additional Condition(s):
01 All new external work shall be carried out in materials which resemble, as closely as possible in colour and texture those of the existing building or, in the case of the proposed conservatory, those of the adjacent conservatory.

Reason(s) for Additional Condition(s):
01 To ensure that the external appearance of the building will be satisfactory.

Yours faithfully

London Borough of Camden

Planning and Transport Department

Camden Town Hall
Argyle Street Entrance Euston Road
London WC1H 8EQ Tel: 278 4444

David Pike MSc CEng MICE MRTPI
Director of Planning and Transport

(Cont.)

David Pike JAT

(Our Reference: PL/8803789/)
(Case File No: E5/13/22)

Director of Planning and Transport
(Duly authorised by the Council to sign this document)

Camden

ENVIRONMENT DEPARTMENT
Planning, Transport and Health Service
Head of Planning,Transport and Health Service . Richard Rawes BA Hons . MICE C.Eng Dip TE

London Borough of Camden
Camden Town Hall
Argyle Street Entrance
Euston Road
London WC1H 8EQ

Tel 071 – 278 4444
Fax 071 – 860 5556

Our Reference: PL/9301084/
Case File No: E5/13/21
Tel.Inqu:
Miss Jay Turner ext. 5623

Campbell Charles Associates
47A Lewes Road
Brighton
BN2 3HW

Date: 21 FEB 1994

Dear Sir(s)/Madam,

[illegible]own and Country Planning Act 1990
Town and Country Planning General Development Order 1988 (as amended)
Town and Country Planning (Applications) Regulations 1988

Permission for Development

The Council, in pursuance of its powers under the above-mentioned Act and Orders and Regulations made thereunder, hereby permits the development referred to in the undermentioned Schedule subject to the conditions set out therein and in accordance with the plans submitted, save insofar as may otherwise be required by the said conditions.

Your attention is drawn to the Appeal Rights and other information at the end of this letter.

SCHEDULE

Date of Original Application : 23rd August 1993

Address : 27 Oakhill Avenue, NW3

Proposal : The enclosure of part of the rear basement area with a glazed roof to form a conservatory,
as shown on drawing no(s) 360/1,2,3,and 4

Standard Condition:
1. The development hereby permitted must be begun not later than the expiration of five years from the date of this permission.

Reason for Standard Condition:
1. In order to comply with the provisions of Section 91 of the Town and Country Planning Act 1990.

Yours faithfully,

Head of Planning, Transport & Health Services
(Duly authorised by the Council to sign this document)