Job Number: 141208

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Basement Structural Method Statement

Property Details
106 Savernake Road
London
NW3 2JR

Client Information
Advantage Basement & Cellar Company Ltd
95 East Hill
Wands worth
London
SW18 2QD

Structural Design Reviewed by	Above Ground Drainage Reviewed by	Hydrogeology Reviewed by
Chris Tomlin MEng CEng MIStructE	Phil Henry BEng MEng MICE	Hannah Fraser BEng MEng CGeol

Revision	Date	Comment
-	14.01.2015	Draft Issue for Planning
1	26.01.2015	Issued for Planning
2	28-09-2015	Post-audit issue

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Contents

1. Camden Planning Guidance (CPG4)

The London Borough of Camden requires a Basement Impact Assessment (BIA) to be prepared for developments including basements and light wells within its area of responsibility. CGP4 – Basements and Light wells details the requirements for a BIA undertaken in support of proposed developments; in summary the Council will only allow basement construction to proceed if it does not:

1. Cause harm to the built environment and local amenity;
2. Result in flooding;
3. Lead to ground instability.

In order to comply with the above clauses a BIA must undertake 5 stages detailed in CPG 4:

Stage 1 – Screening
This stage should identify any areas for concern and therefore focus effort for further investigation.

Stage 2 – Scoping
Identifies the potential impacts of the areas of concern highlighted in the Screening phase.

Stage 3 – Site investigation and study
Allows greater understanding of the issues previously identified to be developed through focussed site investigation and data collection

Stage 4 – Impact assessment
Evaluation of impact, both direct and indirect, of the proposed scheme by comparison with the current situation

Stage 5 – Review and decision making
An audit of the information contained in the submitted BIA and a decision taken by the London Borough of Camden

This report is for planning purposes only and is not for construction: The information, drawings, calculations, method statement and other information in this report are for planning purposes. Croft provides no design warranty or insurances for the final design. Further information and design considerations must be undertaken before building regulations submission. The information provided in this document is not for construction.

2. Design Information - Structural

Structural Summary

Figure 1: 106 Savernake Road Front Elevation

Existing Building

106 Savernake Road is located in Camden, North west London. The current property is a 4 storeyed semi-detached building. The property concerned is ground floor flat with a small cellar. The floors of the building are built with timber. The external and internal walls are constructed with masonry. Some of the internal walls are load bearing walls. Structural steel work is also assumed to exist within the building. There is a front yard and a rear garden.

Neighbouring properties

The construction of neighbouring buildings is similar. During site visit no visual indication of basements were noted. A search on Camden Council Planning website confirmed this: No planning applications related to basement construction in the neighbouring properties are found since at least 1995.

Proposed works

The proposed works require the insertion of a new basement under the front portion of the property. In plan this will extend from the front of the building to approximately midway along the length of the ground floor footprint.

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. Begin by placing the MC pad foundations for the steel columns in cellar location as shown in the drawing SL-10.

2. Place conveyor and place steel columns.

3. Needle and prop the ground floor timber slab.

3. Slowly work by inserting 1000(max.) long cantilevered retaining walls sequentially as shown in SL-10 drawing.

4. Waterproof internal space with a drained cavity system.

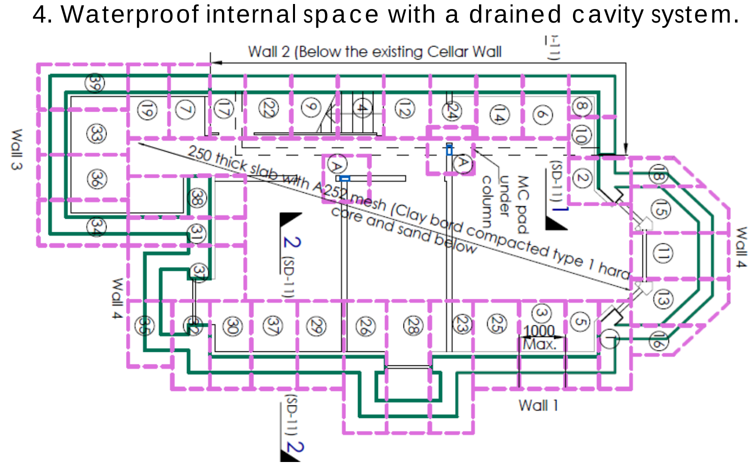


Figure 2 : Proposed basement plan

For further details of the proposed construction method, refer to the method statement in Appendix –C.

Structural Defects Noted

No defects were noted during the Chartered Engineers first visit.

Intended use of structure and user requirements	Family/domestic use

DP27	
A	**Maintain Structural Stability of the building & Neighbouring Properties**. The attached drawings show the reinforcement and construction required by maintaining the stability of the property, the neighbouring buildings, and the road. 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 relation 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.

3. Basement Impact: Stage 1 - Screening

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

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

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.

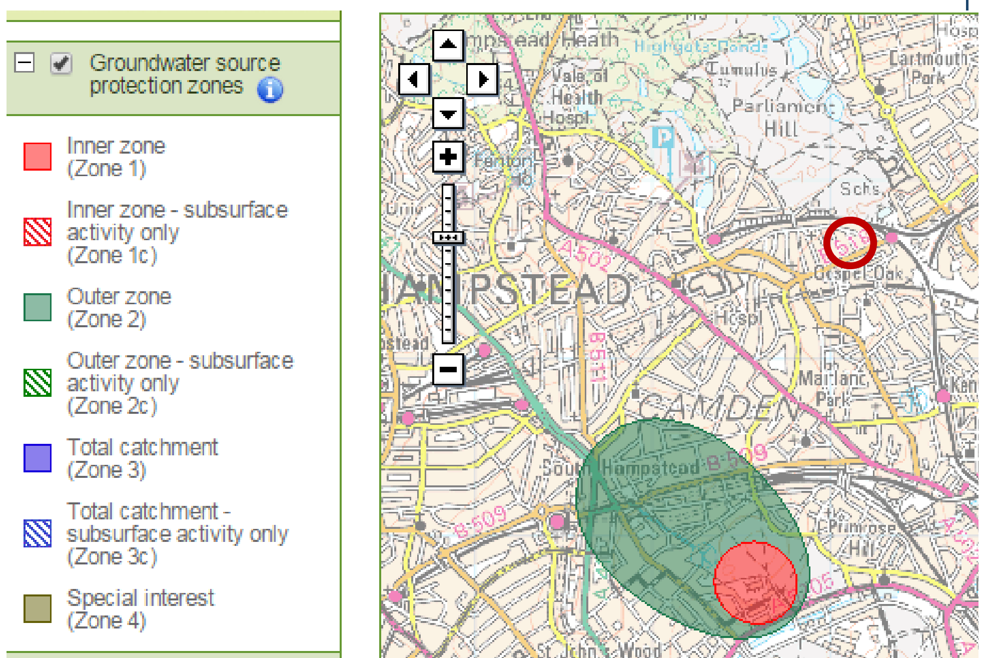


Figure 3: Environmental Map showing the ground water protection zones

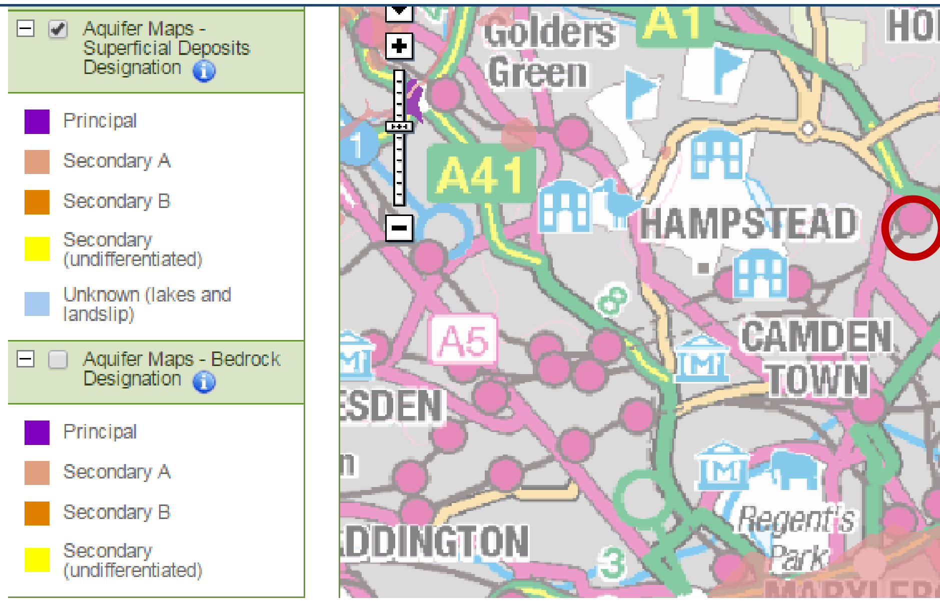


Figure 4: Environmental maps showing the aquifers

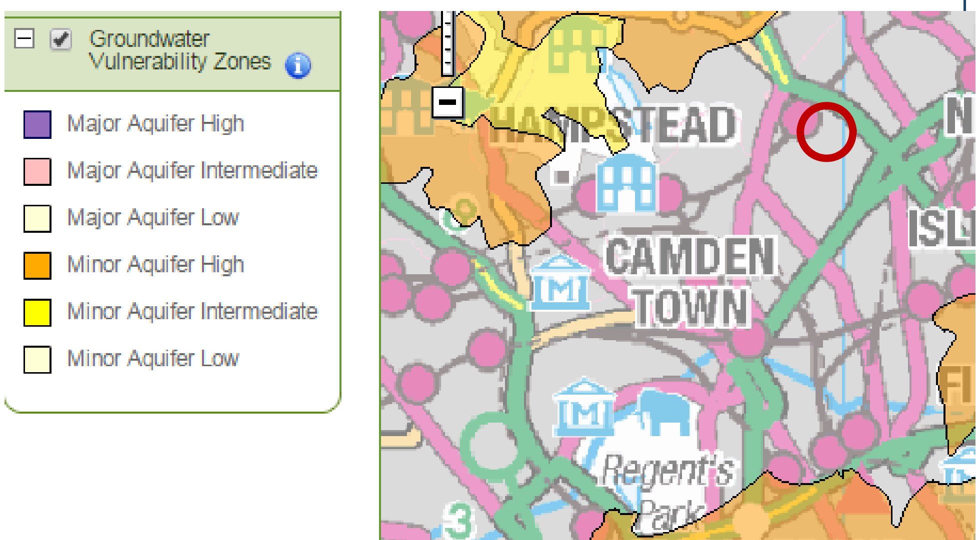


Figure 5: Environmental map showing the Ground water vulnerability zones

From figure 8 of Arup's Hydro Geological Report, it is observed that the site is in the zone of unproductive strata.

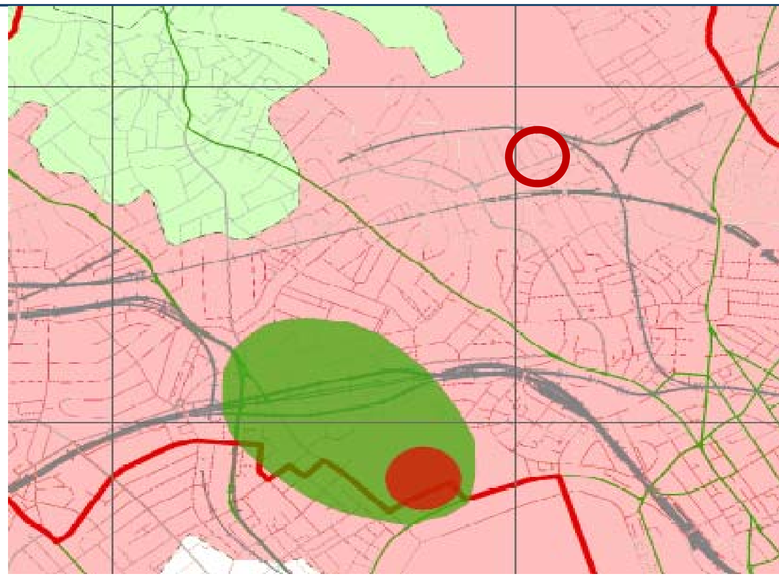


Figure 6: Extracted from Figure 8 of Arup's Hydro Geological Study

1b. Will the proposed basement extends beneath the water table surface?
Unknown. The proposed basement extends approximately 3.25 meters below ground level.
Requires scoping assessment and investigation

2. Is the site within 100m of a watercourse, well used/disused or potential spring line?
No. OS maps and local walkover survey show no wells, watercourses.

Figure 11 of Arup's report shows that the site is well away from (Approx. 200 metres) the former river Fleet.
Requires scoping assessment and investigation

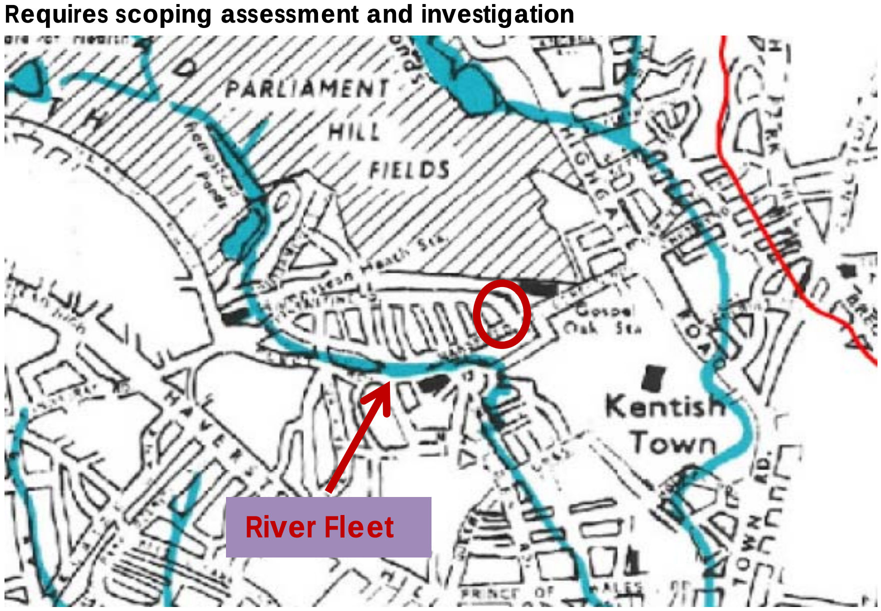


Figure 7: Extract from Figure 11 of Arup's Report showing the fleet river

Figure 8: Extract from BGS Geo Index Onshore Maps

3. Is the site within the catchment of the pond chains on Hampstead Heath?
No. The site lies outside the areas denoted by figure 14 of the Arup report.

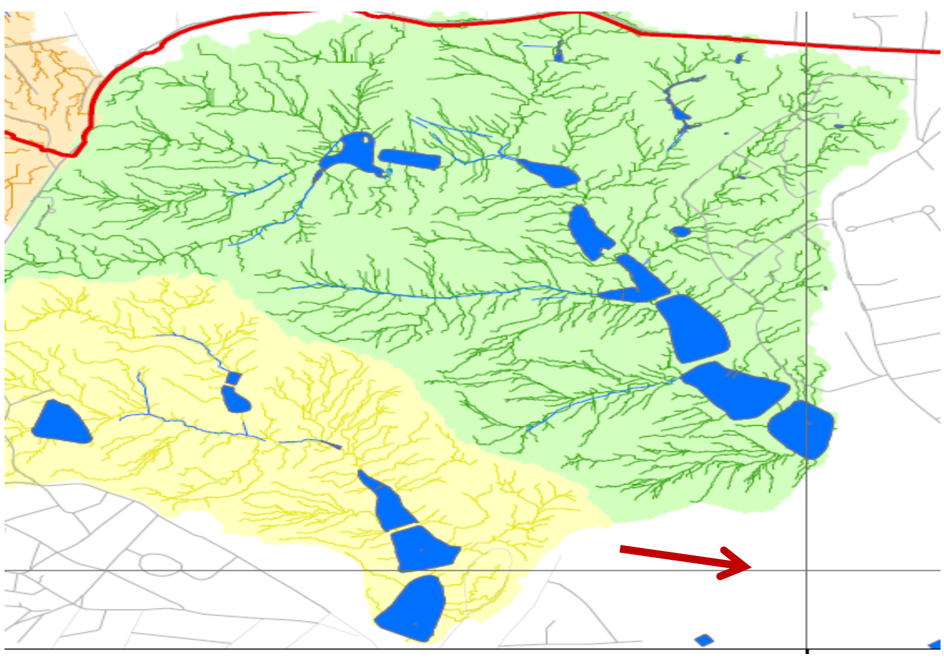
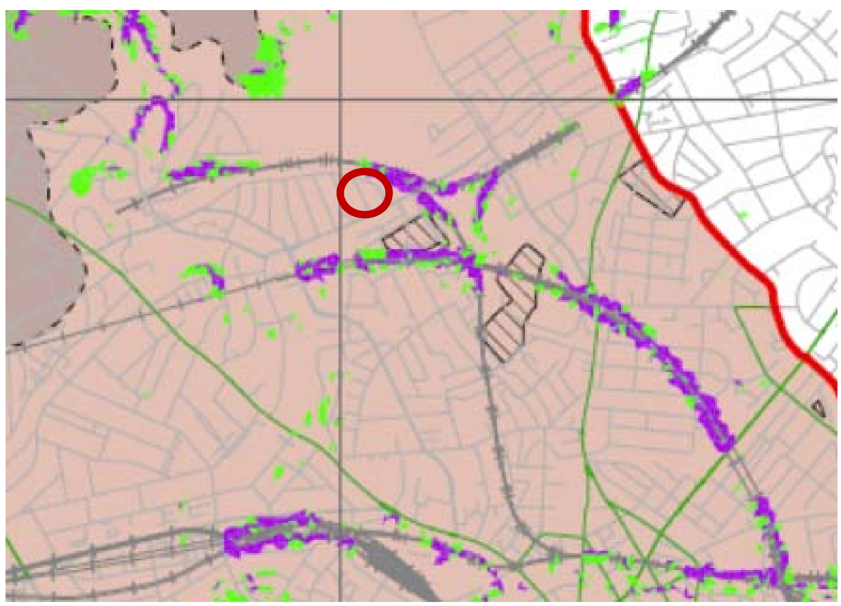


Figure 9: Extract from Figure 14 of Arup's Report

4. Will the proposed basement development result in a change in the proportion of hard surfaced/ paved areas?
Yes. There are three light wells one in the front, one on the side and one on

	the rear. These will increase slightly the hard surfaced area. **5. As part of the site drainage will more surface water (e.g. rainfall and run-off) than at present be discharged to the ground (e.g. via. Soakaways and or SUDS)?** **No**. Existing roof Drainage will run into the existing drainage system. Surface water will still discharge to ground. **6. Is the lowest point of the proposed excavation (allowing for any drainage and foundation space under the basement floor) close to or lower than, the mean water level in and local pond (not just the pond chains on Hampstead Heath) or spring line?** **No**. From walkover and OS maps, there are no local ponds or springs of significance.
Slope Stability	**Figure 2 – Slope Stability screening flowchart** **1. Does the existing site include slopes, natural or manmade greater than 7º (approximately 1 in 8)?** **No**. Difference in height between the rear garden and front is less than 1 in 8 slope (approx. flat)  *Figure 10 : Extract from Figure 16 of Arup's Report* **2. Will the proposed re profiling of landscaping at site change slopes at the property boundary to more than 7º (approximately 1in 8)?** **No**. Proposed landscaping does not affect the slope. **3. Does the development neighbour land including railway cuttings and the like with a slope greater than 7º (approximately 1 in 8)?**

No. The slope of the adjacent properties appears to match the site.

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

No. The slope of the wider hillside setting is as per the property, less than 7º

From Figure 16 of Arup's Report, the slope angle is shown less than 7º

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

Yes. The site sits on the London Clay formation. From the Soil Investigation Report it is clear that the London Clay is the shallowest strata.

6. Will any tree/s be felled as part of the proposed development and/or are any of the works proposed within any tree protection zones where trees are to be retained?

No. No local trees are to be felled. The impact of the basement on these trees should be considered.

7. Is there a history of seasonal shrink-swell subsidence in the local area, and/ or evidence of such effects at the site?

No. From the walk over survey Subsidence was not considered as an issue on this site.

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

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

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

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

No. From the historical maps, the site is within in an area of open field area and residential area.

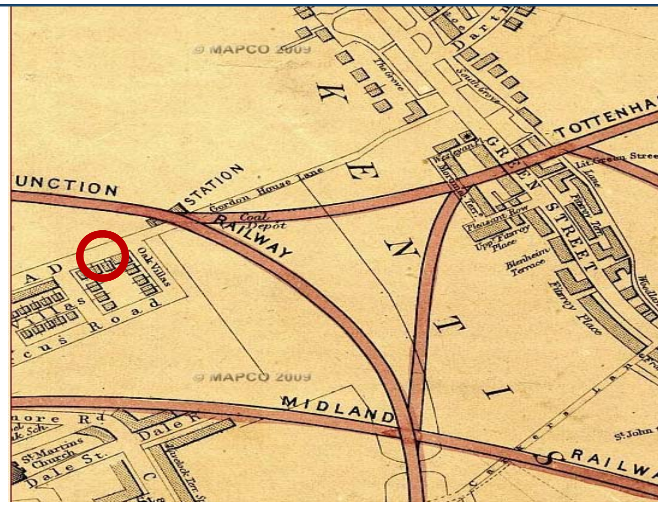


Figure 11: Extract from map of London 1868 by Edward Weller

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.

The site is on London Clay, which is relatively impermeable; as such it is not an aquifer.

In the bore hole taken at site, ground water seepage is noted at 2.3m below ground level and likely to represent perched groundwater migrating and collecting within standpipe installed within the impermeable soils of London Clay Formation. The ground water level was checked after a month's time and there was a standing water level at 1.65 m below ground level. Hence dewatering may be required during construction.

Carry forward to scoping stage.

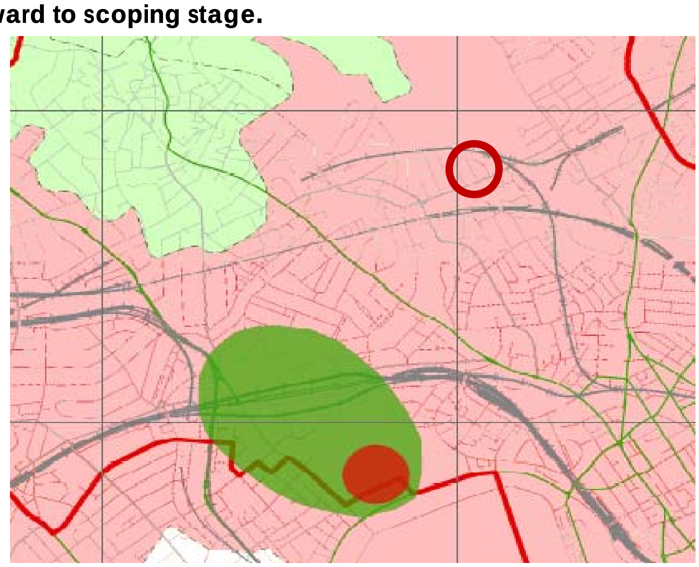


Figure 12: Extract from Figure 8 of Arup's Report

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

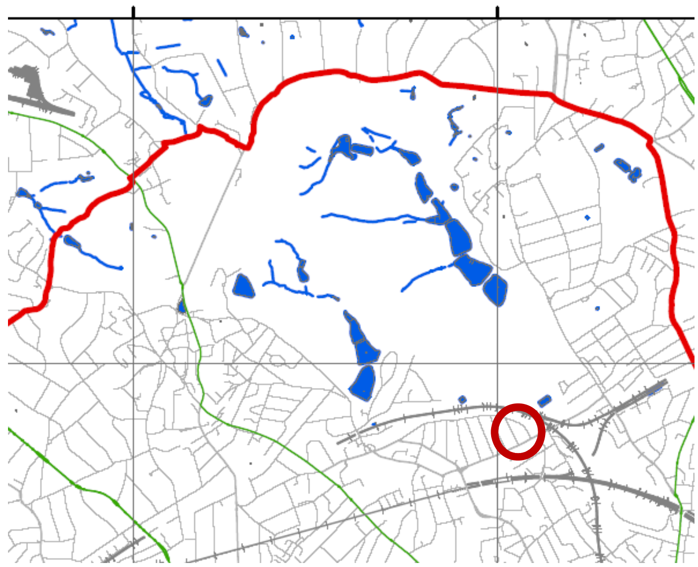


Figure 13: Extract from Figure 12 of Arup's Report

12. Is the site within 5m of a highway or pedestrian footway?
Yes. The site is within 5m of the highway.

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

13. Will the proposed basement significantly increase the differential depth of foundations relative to the neighbouring properties?
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. The nearest railway line is around 35m from the site.

Surface flow and flooding

Figure 3 – Surface flow and flooding screening flowchart

1. Is the site within a catchment of the pond chains on Hampstead Heath?
No. The site lies outside the catchment areas of the Hampstead heath ponds as shown on figure 14 of the Camden Hydrological Study (Arup's Report).

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 development will not result in a material change of the surface water flows into the existing sewers.

3. Will the proposed basement development result in a change to the hard surfaced /paved external areas?
Yes. There are three light wells one in the front, one on the side and one on the rear. These will increase slightly the hard surfaced area (approx. 5.3 m^2).

4. Will the proposed basement result in changes to the inflows (instantaneous and long term of surface water) being received by adjacent properties or downstream watercourses?
No. The proposed development will enter the current drainage system.

5. Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream watercourses?
No. The quality of water is unlikely to be altered.

6. Is the site in an area known to be at risk from surface water flooding, such as South Hampstead, West Hampstead, Gospel Oak and King's Cross or is it at risk from flooding, for example because the proposed basement is below the static water lever of a nearby surface water feature?

No. As per page 29 of the CPG4 (Camden planning guidance-Basements and Light wells), the site is not listed in the table which shows the streets at risk of water flooding.

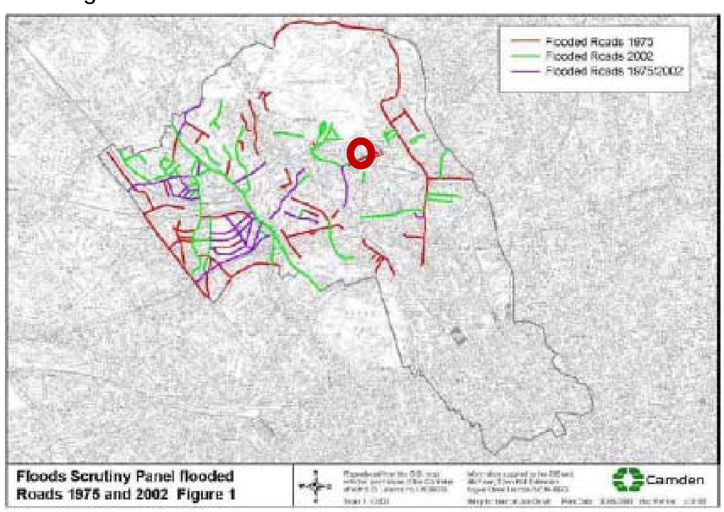


Figure 14: Extract from Camden Flood Risk Management Strategy (Cabinet Report Final)

The issues carried forward to the scoping stage are as follows:

1. The high volume change potential of the soils.
2. The site is within 5m of a pedestrian right of way and a highway

4. Basement Impact: Stage 2 – Scoping

Groundwater flow	**Subterranean flow** There is a need to find out groundwater table to see if basement will impact on the groundwater flow. This will be covered by a Soil Investigation. Soil investigation to be completed with bore holes. The bore holes are to have a stand pipe inserted to confirm the water level after a month's period.
Slope Stability	The site is close to the Railway line by about 35 m. Confirmation at design stage from TFL is required to confirm their assets are not affected. The top layer of the soil is London Clay formation. Soil Investigation confirms this. The slope stability of theses beds is in the region of 24˚. The design of the RC retaining walls will take this into account. It is possible made ground will be found on site. The soil investigation will confirm this to 1.5 m deep. The basement is within 5m of the footpath, and will therefore be designed conservatively 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 commissioned to determine the soil conditions and to confirm the water table

British Geological Survey maps shows that the site is located on London Clay formation. The same soil stratum is expected at site.

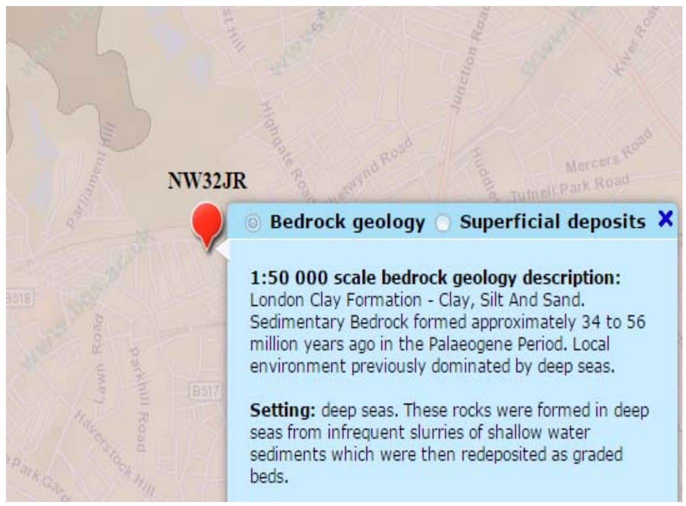


Figure 15: Extract from BGS maps

The Soil Investigation confirmed the ground conditions (Refer Appendix D for Soil Investigation Report). There is water seepage at 2.3 m below ground level.

Walk over Survey

Figure 16: Front and Rear of 106 Savernake Road

	 *Figure 17: Cellar and back side garden of 106 Savernake Road* There is a garden at the rear of the property but there are no mature trees observed. There are mature trees in the neighbouring property at around 30 metres from the proposed basement. Roots are noted during Soil Investigation at a depth of 2.0 metres below ground. The present basement will extend beyond the roots. Root barrier protection can be required to protect new basement walls. The base of the foundation excavations must extend at least 300 mm into non-root penetrated soils. The existing building did not exhibit any signs of subsidence not movement. The building is part of a semi-detached and the effects of the development on the adjacent properties will need to be considered.
Drainage effects on Structure	No build over agreements known of
Under ground	The proposed basement is 900m away from underground line. But the site is near to national rail line (35m).

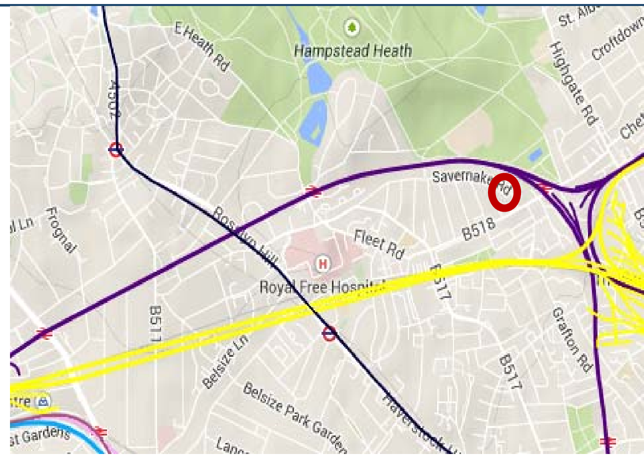


Figure 18: Extract from rail maps to show underground line

Sources of Contaminates

From the Historic Maps it can be seen that the ground use has not been conducive to activities leading to poor ground.

During the walk over survey no items were noted that may lead to contamination.

Water Course

No 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.

Figure 19: Extract from Figure 11 of Arup's report

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6. Historic Maps

From Historic maps, Savernake Road is seen from 1886 onwards.

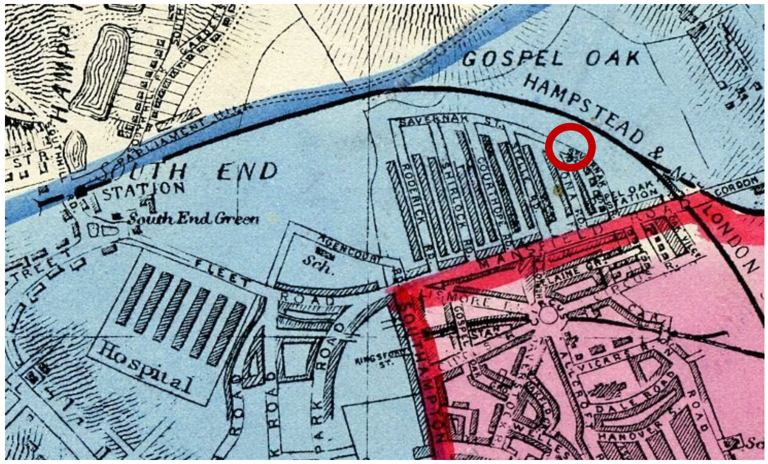

Figure 20: Map of London 1886

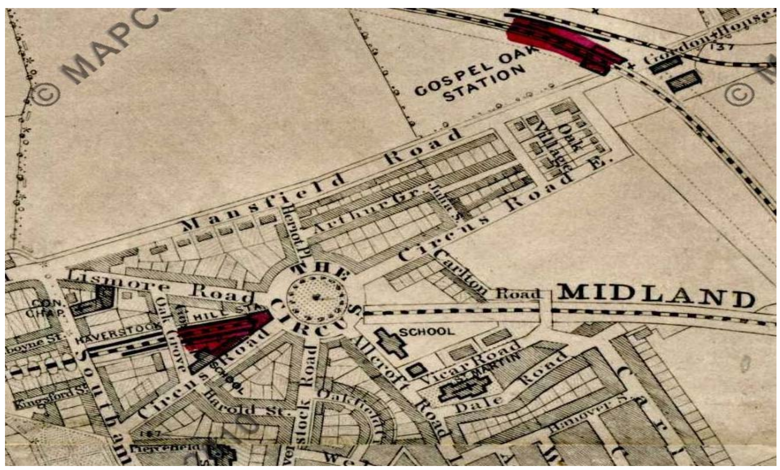

Figure 21: Map of London 1878

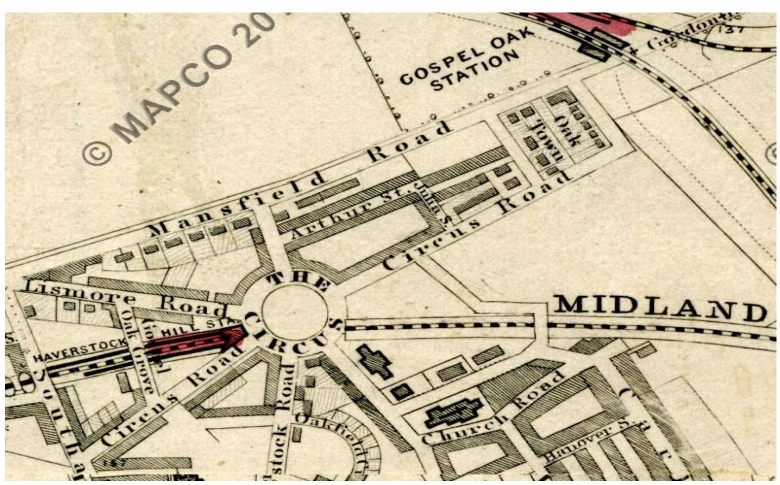

Figure 22: Map of London 1872

7. Flood Risk Assessment

As per page 29 of the CPG4 (Camden planning guidance-Basements and Light wells), the site is not listed in the table which shows the streets at risk of water flooding. Hence flood risk assessment is not required. However, as with all basements, suitable mitigation measures should be adopted to protect the basement from any possible flooding. The extent of the related damage can be reduced as follows:

- At ground level, an upstand can be constructed around the front lightwell. This should be considered at detailed design stage.
- A pumping mechanism will be installed for the proposed basement. There is a likelihood that this may fail and allow excess water to accumulate. If this were to occur, the build-up of water would be gradual and noticeable before it becomes a significant life-threatening hazard.
- Install a dual pumping system to maintain operation in the event of a failure. This should include a battery backup and a suitable alarm system for warning purposes.
- To reduce the impact of surface water flooding, sustainable drainage systems such as on site attenuation should be considered at detailed design stage.

8. Stage 3 - Site Investigation

Monitoring and Reporting

The Soil investigation was completed by Ground & Water

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

1. Two trial pits to the front side and rear of the proposed basement to confirm the existing foundations. The purpose is to consider the effect of the works on the neighbouring properties and to find the ground conditions below the site.
2. It would have been preferred to complete two bore holes on this site, but due to access it was only possible for the rig to access to the front of the property. With the size of site and our knowledge of the area it is not expected there to be a large variation across the small site, therefore one borehole 6m deep was completed.

	3. Stand pipe to be inserted to monitor ground water; record initial strike and the water level after 1 month. 1. Site testing to determine in-situ 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 See **Appendix D for Soil report**

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9. OS Map extract showing location of Railway

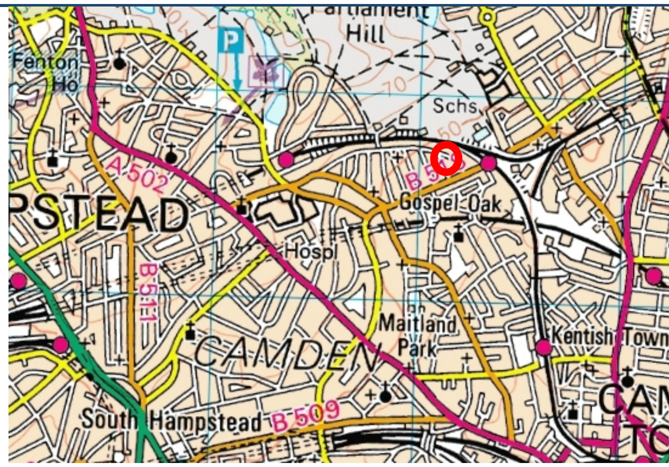


Figure 23: Extract from OS map showing proximity to nearest railways

10. Stage 4 - Impact Assessment

Subterranean flow

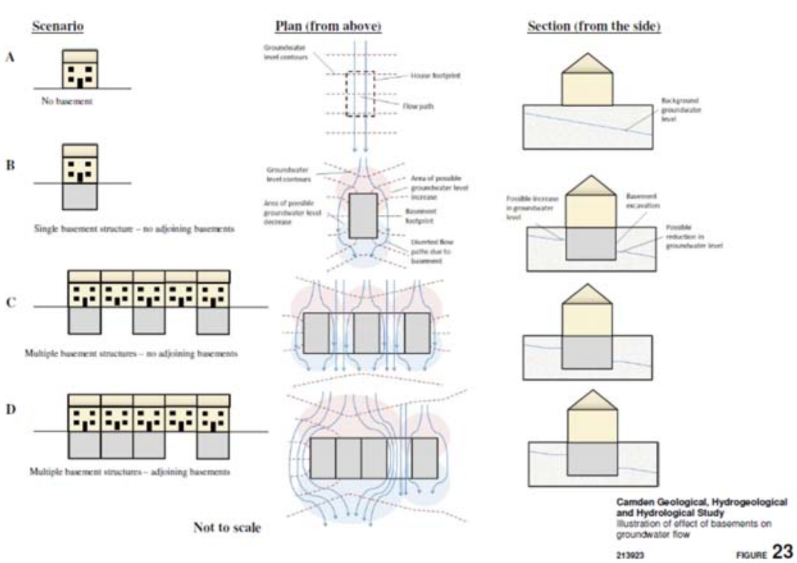
The site is not within the catchment of the Hampstead Heath Ponds. It is at a considerable distance from the ponds and standing water courses in the area.

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

The proposed development depth is expected to be at 3.4m below external ground floor level.

The soil investigation indicated that the ground water seepage was encountered at 2.3m below ground level (bgl) within BH1 and likely represents perched groundwater within London Clay Formation. The remaining trial holes were dry. Groundwater level was monitored after a month's period. It was revealed that a standing groundwater level of 1.65m bgl, with the well being recorded as 4.8m deep.It was considered likely that ponding within the well due to the cohesive, impermeable nature of deposits.

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 water 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.
	To allow for through flow of ground water the drawings SD-11 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 7º.
	Land slip is not a problem due to any circular failure patterns.
	The retaining walls must be designed to accommodate the lateral pressures from the soils.
Foundation type	Reinforced concrete cantilevered retaining walls
	The designs for the retaining walls have been calculated using TEDDS software. The software is specifically designed for retaining walls and ensures the design is kept to a limit to prevent damage to the adjacent property.
	Attached printout of Calculations and Deflections 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 highlighted that water is present. There is ground water seepage at 2.3m bgl in the first reading and raised to 1.65m bgl in the second reading taken after 1 month. The walls are designed to cope with the hydrostatic pressure.
	The Design also considers floatation as a risk. The design 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.

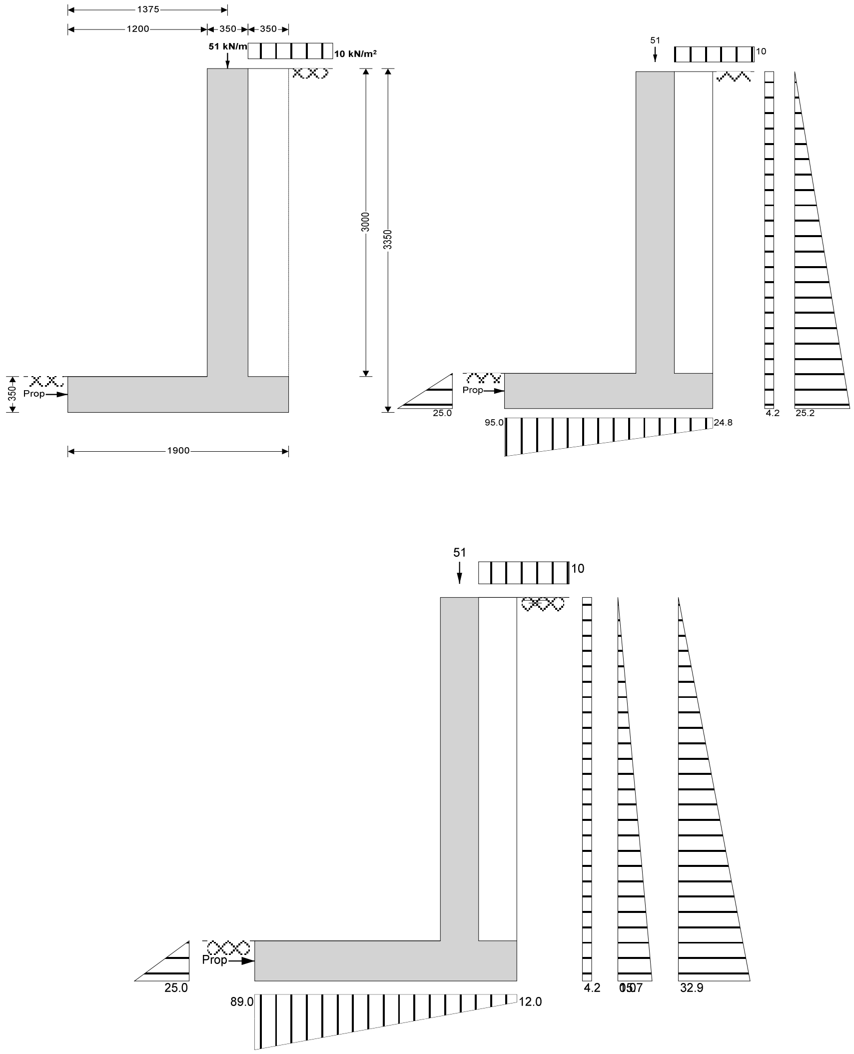


Figure 24: Loadings and Lateral stress patterns

Vicinity of Trees	There are few mature trees in the neighbour's garden which is at the back side of the property.
Special precautions due to trees	Design using NHBC guidance Basement depth will allow for footings to be placed outside the effects of the trees. Roots noted at 2 m below ground during soil investigation. Hence Root barrier protection can be required to protect new basement walls. The base of the foundation excavations must extend at least 300 mm into non-root penetrated soils.
Drainage effects on Structure	No build over agreements known of. Flooding. The site is not in an area of high risk flooding.
Roads	The building does not undermine the highway, but car parking is present to the front of the property. It is possible for heavier goods vehicles to reverse on to the property to allow for this risk loadings are to be taken from the Highways loading code. 10kN/m^2 if within 45^o of road 5 kN/m^2 if within 45^o of Pavement Garden Surcharge 2.5kN/m^2 Surcharge for adjacent property 1.5kN/m^2 + 4kN/m^2 for concrete ground bearing slab
Intended use of structure and user requirements	Family/domestic use
Loading Requirements (EC1-1)	(see table below)
Number of Storeys	4 Is Live Load Reduction included in design No / %

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

The basement does not line within a 45º angle of the highway. Therefore Highways HA loading is not required to be applied.

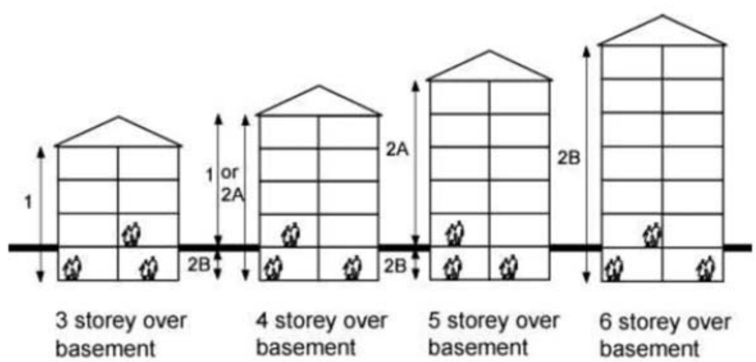
Progressive Collapse	Design for consequences of localized failure in building from an unspecified cause
Is the Building Multi Occupancy?	No.
Part A3 Progressive collapse	EN 1991-1-7:1996 Table A1

Class 2A	Hotels, Flats, Apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys

Progressive collapse Change of use	To NHBC guidance compliance is only required to other floors if a material change of use occurs to the property.

Proposed Building Class	2A
If class has changed material change has occurred	No

Additional Design Requirements to Comply with Progressive Collapse	Class 2B – Design provision of effective horizontal and vertical ties to all areas increased in class. The basement must be constructed to 2B standards and the above can be considered to remain unchanged as a 2A structure.



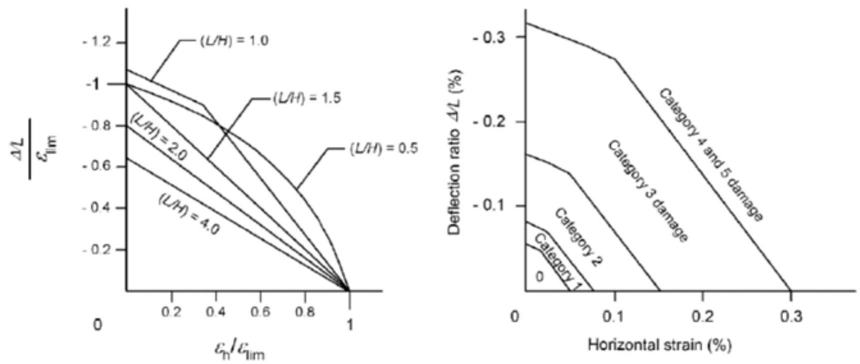
Lateral Stability	
Stability Design	The cantilevered walls are suitable to carry the lateral loading.
Lateral Actions	The soil loads apply a lateral load on the retaining walls. Hydrostatic pressure will be applied to the wall

	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 120 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 to the development: 1.0 Employ a reputable firm for extensive knowledge of basement works. 2.0 Employ suitably qualified consultants. Croft Structural engineer has completed over 120 basements in the last 4 years. 3.0 Design the underpins to the stable without the need for elaborate temporary propping or needing the floor slab to be present. 4.0 Provide method statements for the contractors to follow 5.0 Investigate the ground, now completed. 6.0 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.

7.0 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 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 $\Delta L / \varepsilon_{lim}$ (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)

Calculations have been done to predict the likely ground movement and the related damage. These calculations show the horizontal and vertical movements of the ground at the position of the wall and also movements that will occur further away; these movements decrease as the distance away from the excavation increases. These distances and their related ground movements include part (and for some all) of the neighbouring properties. From the position of negligible movement, right up to the location of the excavation, the predicted movements are found to be less than Damage Category 2. The calculations which show this are contained in Appendix F.

Given the presence of clay below the formation level, there is a potential for heave movement of the soil. This is accommodated by a void below the slab in the centre of the basement. Calculations related to the predicted heave are also included in Appendix F.

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	Approximate	Limiting	Definitions of cracks and repair

of Damage	crack width	Tensile strain	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	0.15-0.3	SERIOUS – Internal cracks repaired as for MODERATE, plus perhaps reconstruction if seriously cracked. Re-bonding 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 party wall at ground floor at 2m intervals along the length of the property as shown on the attached sketch M-10 in Appendix E.

	The surveying points on the adjacent structures are to be set up using an EDM prior to commencement of the works and to be read daily and reported against the following control values. Limits on ground and adjacent structures movement during underpinning and throughout the construction works. Movement of survey points must not exceed: Settlement: Action values: 5mm (stop work) Trigger values: 65% of action values (submit proposals for ensuring action values are not exceeded) Lateral displacement: Action values: 6mm (stop work) Trigger values: 65% of action values (submit proposals for ensuring action values are not exceeded) Movement approaching critical values: Trigger: Submit proposals for ensuring action values are not exceeded Action: Stop work The reporting format will be in the form of a table as attached. **Predicted Category of Damage** The predicted category of damage is likely to be within BRE Category Slight, with possible localised crack widths 2mm to 5mm Classification Aesthetic.

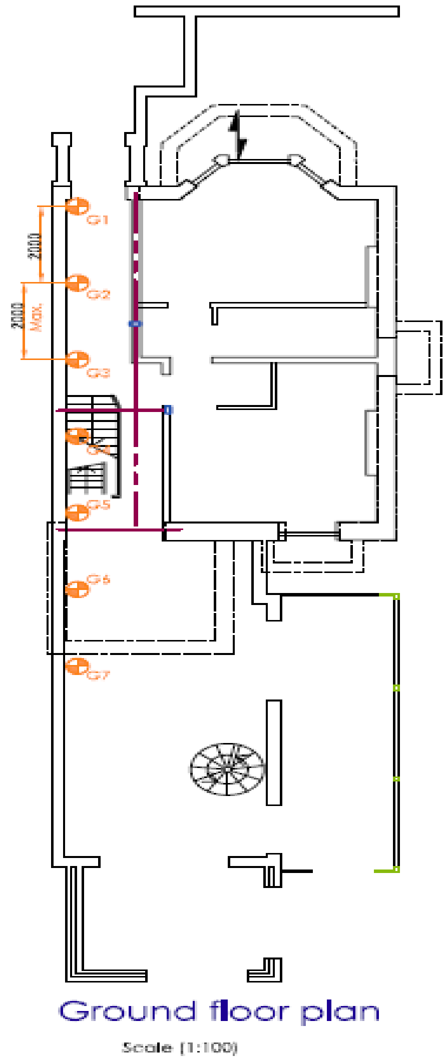


Figure 25: Figure showing the monitoring points

Movement	CATEGORY	-
0mm-7mm	Green	No action required
7mm-12mm	AMBER	Crack Monitoring: Carry out a local structural review; Preparation for the implementation of remedial measures should be required.
>12mm	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

	Basement Monitoring Statement included in Appendix E

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

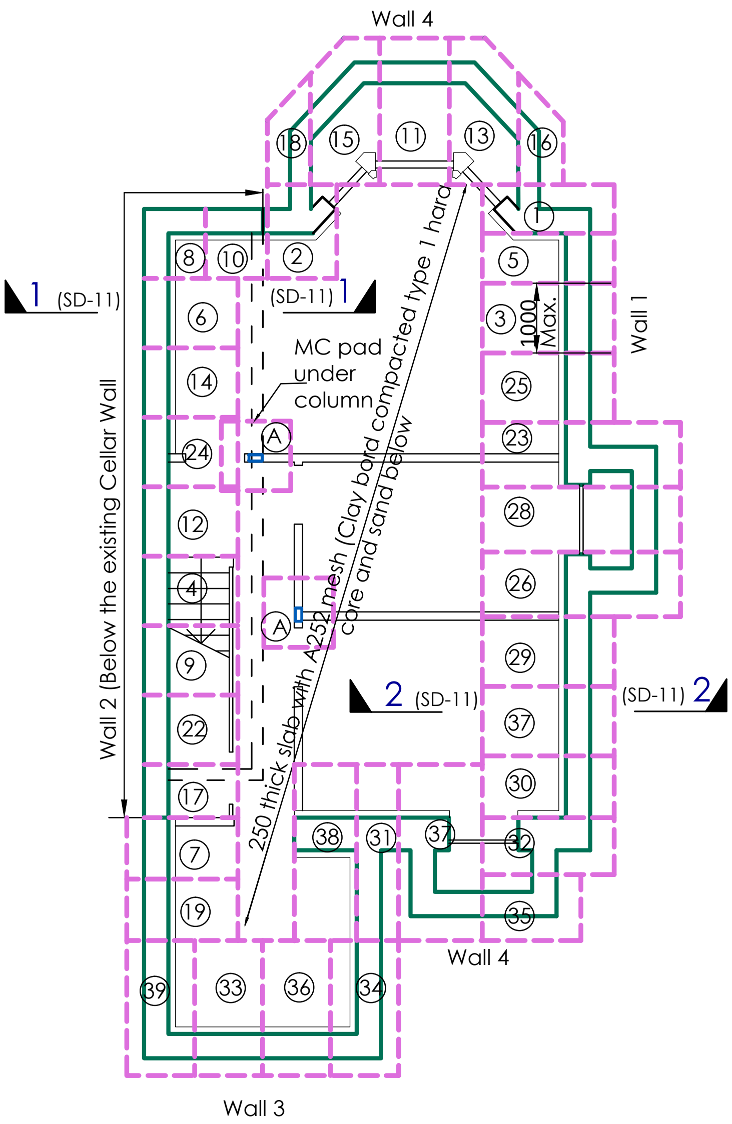
Noise and Nuisance	The contractor is to follow the good working practices and guidance laid down in the "Considerate Constructors Scheme". The hours of working will be limited to those allowed; 8am to 5pm Monday to Friday and Saturday Morning 8am to 1pm. None of the practices cause undue noise that one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

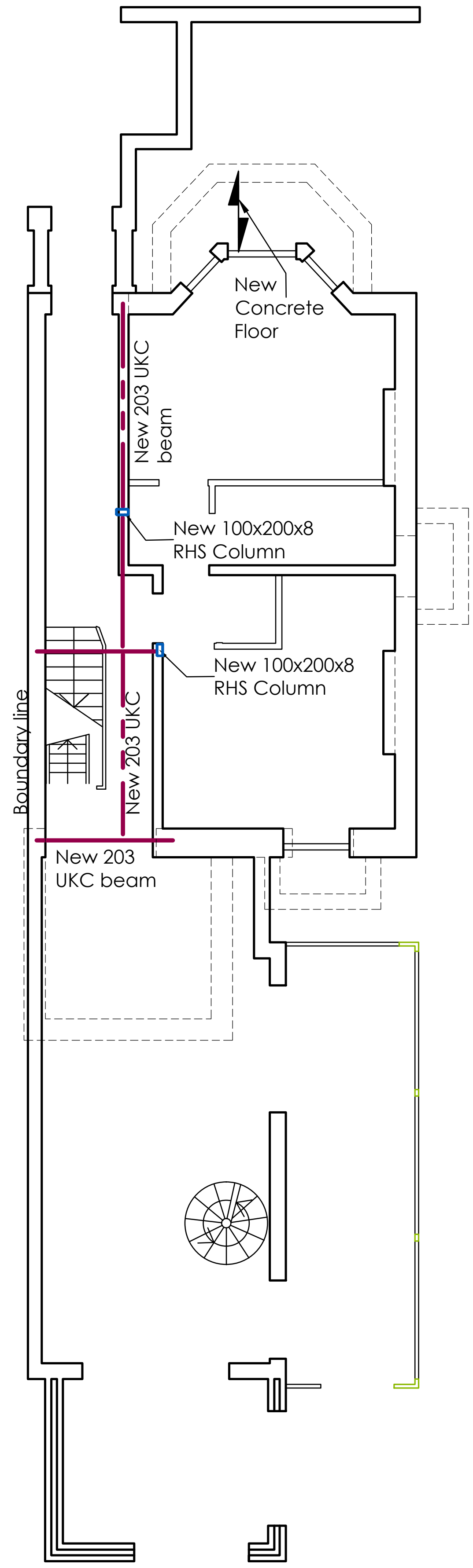
Structural Scheme Drawings

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



Basement floor plan

Scale (1:100)



Ground floor plan

Scale (1:100)

Rev	Date	Amendments
1	28/09/15	Post Audit Issue
-	14/01/15	First issue for comment

Client: Advantage Basement & Cellar Company Ltd.

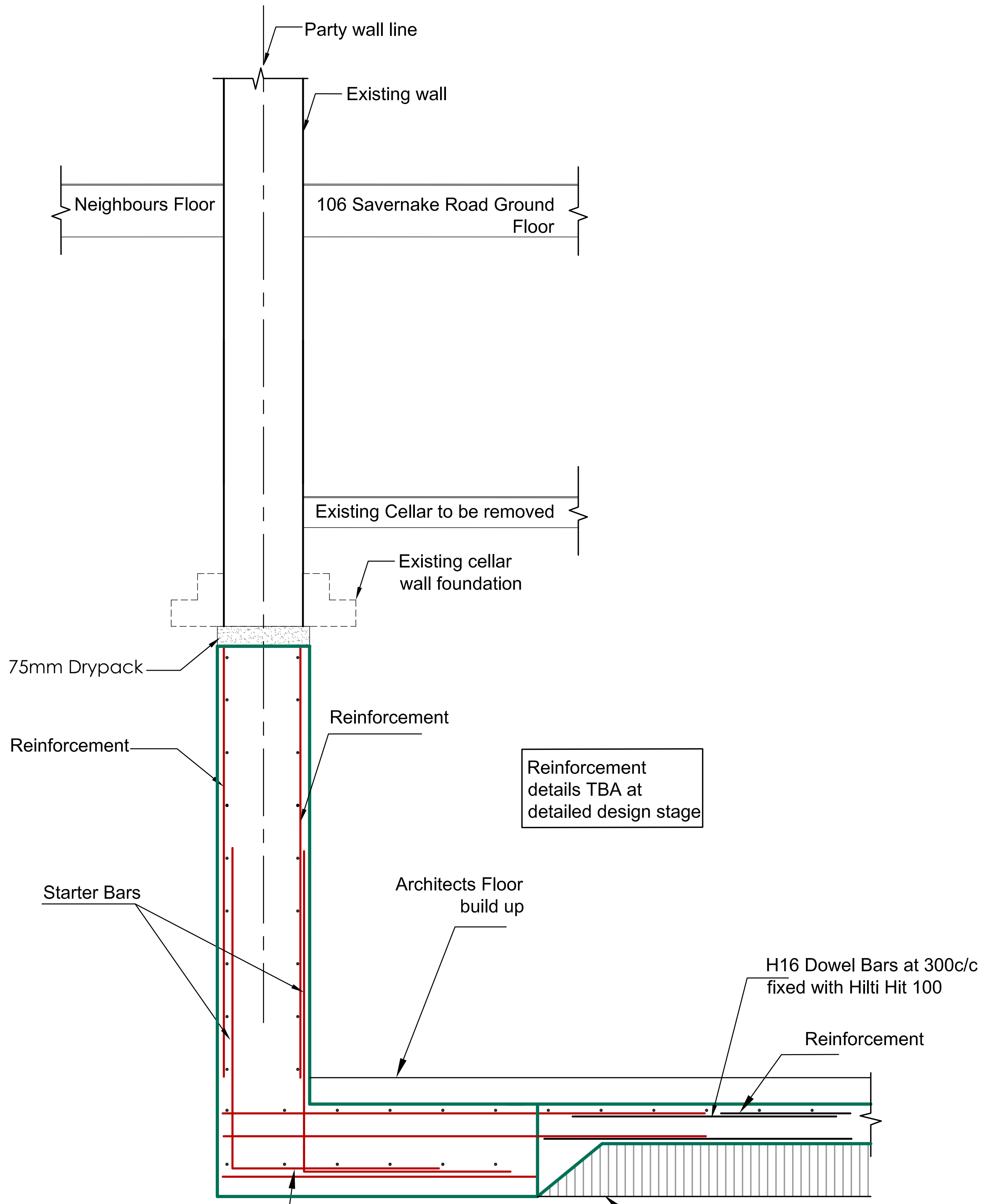
Project: 106 Savernake Road

Title : Ground Floor and Basement Plans

Job No.s	Drawn	Date
141208	VLD	Jan 15

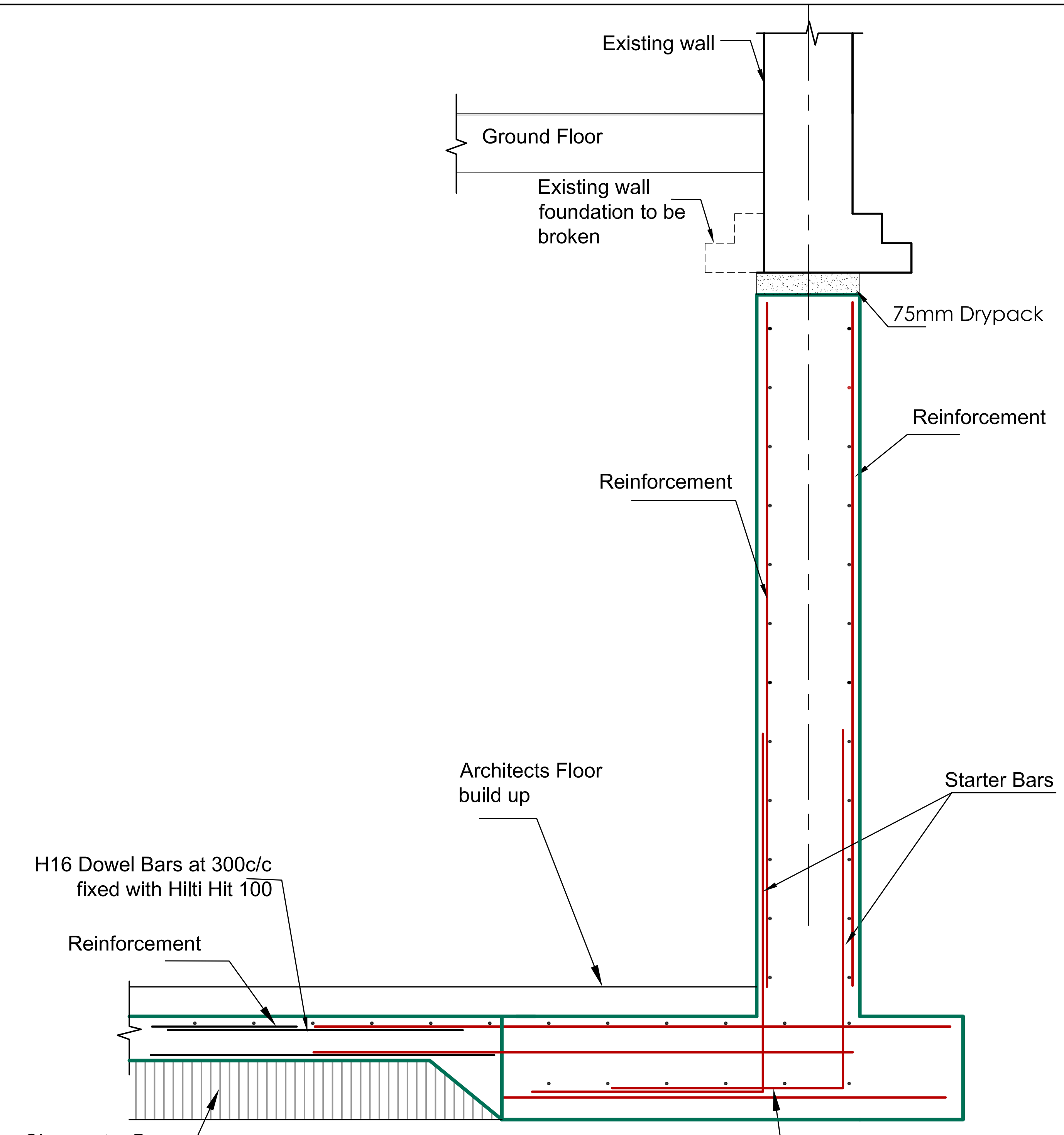
Dwg Nos	Rev	Scale
SL-10	1	As Shown @ A3

Croft Structural Engineers
Clockshop Mews, Rear 60 Saxon Rd, London, SE25 5EH
0208 684 4744
www.croftse.co.uk



Section 1-1 (For wall 2)
Scale 1:20

Reinforcement details TBA at detailed design stage



Section 2-2 (For walls 1,3&4)
Scale 1:20

Rev	Date	Amendments
-	14/01/15	First issue for comment

Job No.s: 141208	Client: Advantage Basement & Cellar Company Ltd.
Dwg Nos: SD-11	Project: 106 Savernake Road
Date: Jan 15; Drawn: VLD; Chk'd: CT; Scale: As shown @ A3; Rev: -	Title: Basement Retaining wall Sections

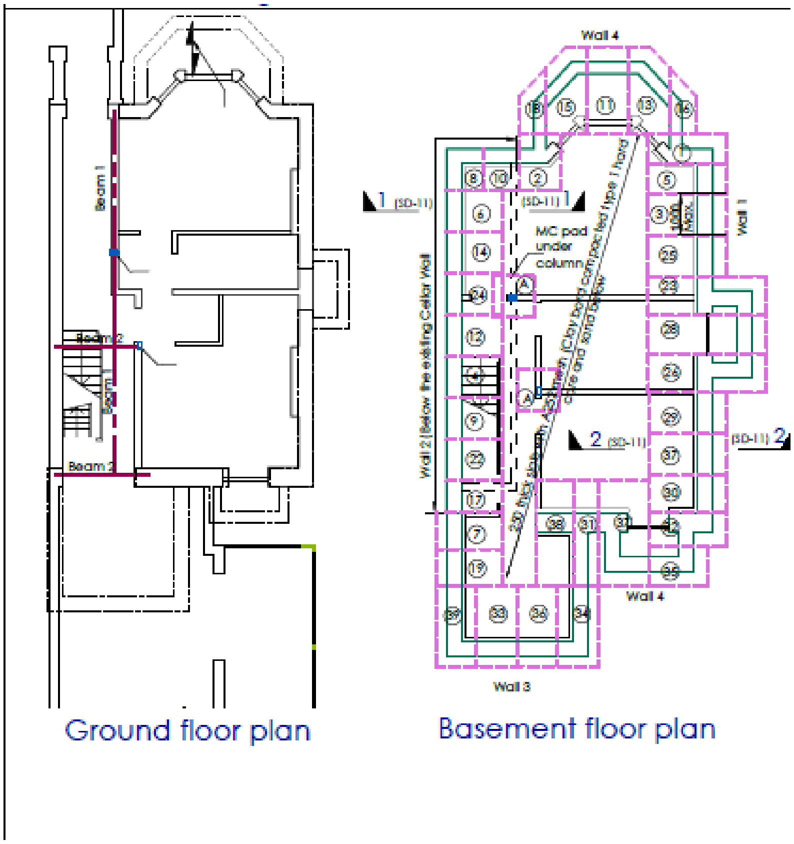
Croft Structural Engineers
Clockshop Mews,
r/o 60 Saxon Rd,
London, SE25 5EH.
020 8684 4744
www.croftse.co.uk

Appendix B

Structural Basement Calculations

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

CROFT STRUCTURAL ENGINEERS

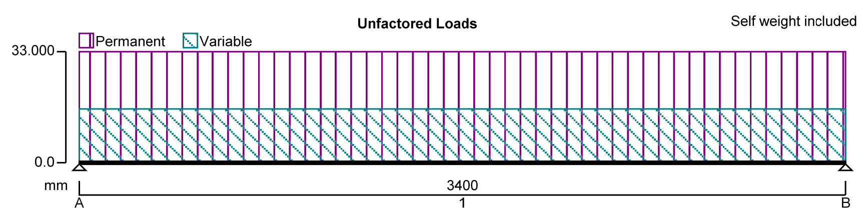


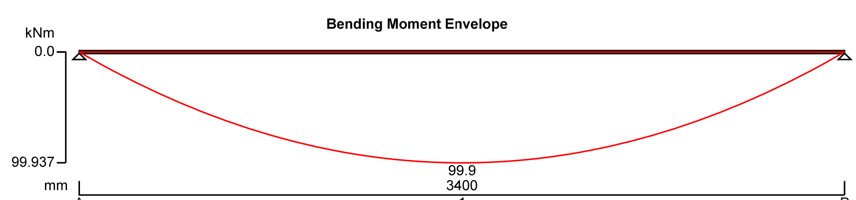
BEAM 1 (GROUND FLOOR)

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

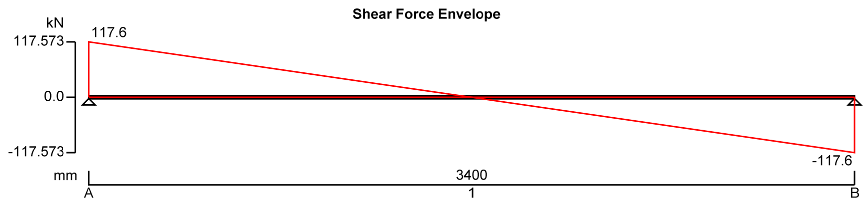
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.06





CROFT STRUCTURAL ENGINEERS



Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	Permanent full UDL 33 kN/m
	Variable full UDL 16 kN/m

Load combinations

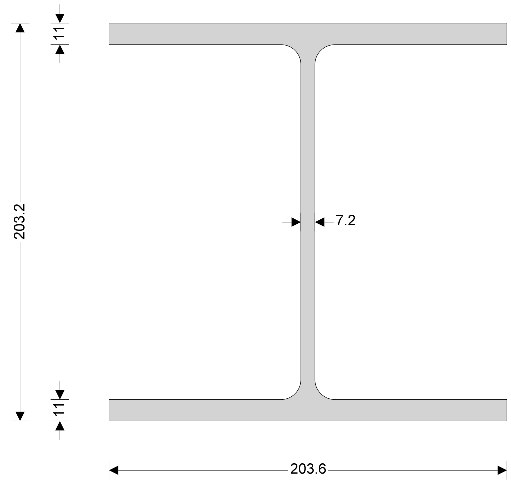
Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
	Span 1	Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

Maximum moment	M_{max} = **99.9** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **117.6** kN	V_{min} = **-117.6** kN
Deflection	δ_{max} = **9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **117.6** kN	R_{A_min} = **117.6** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **56.9** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **27.2** kN	
Maximum reaction at support B	R_{B_max} = **117.6** kN	R_{B_min} = **117.6** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **56.9** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **27.2** kN	

Section details

Section type **UKC 203x203x46 (Corus Advance)** Steel grade **S275**



Section classification	**Class 1**		

Check shear - Section 6.2.6

Design shear force V_{Ed} = **118** kN   Design shear resistance $V_{c,Rd}$ = **269.5** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = **99.9** kNm   Des.bending resist.moment $M_{c,Rd}$ = **136.8** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT}$ = **0.532**   Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = **0.400**

$\bar{\lambda}_{LT}$ > $\bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = **133** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection δ_{lim} = **9.4** mm   Maximum deflection δ = **8.97** mm

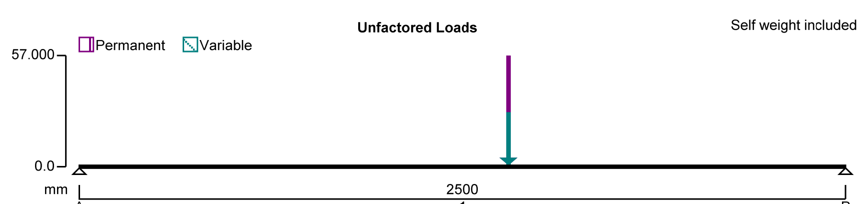
PASS - Maximum deflection does not exceed deflection limit

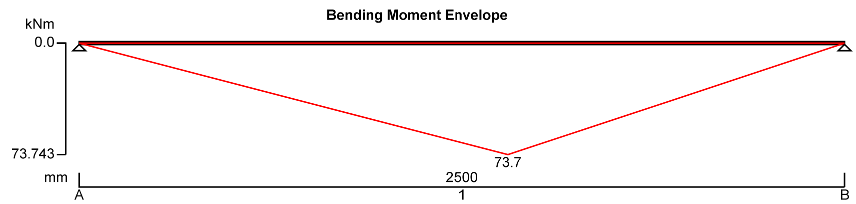
BEAM 2 (GROUND FLOOR)

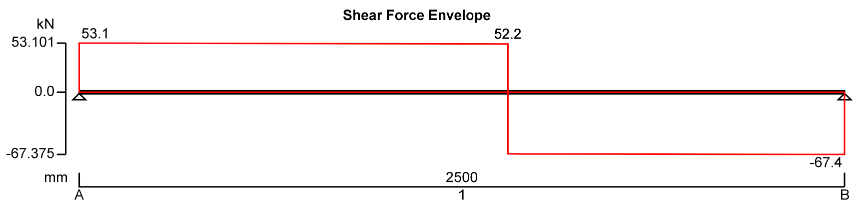
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.06







Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1	
	Permanent point load 57 kN at 1400 mm	
	Variable point load 28 kN at 1400 mm	

Load combinations

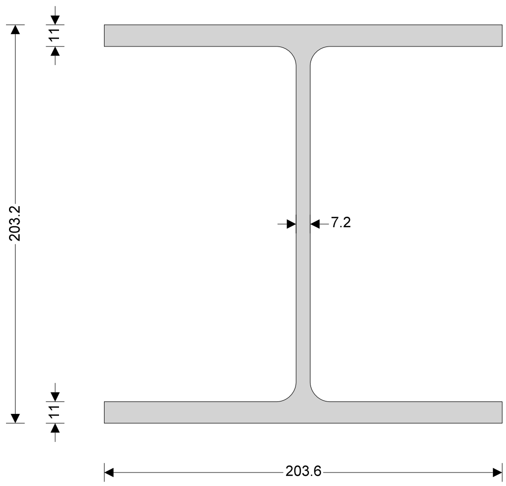
Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
	Span 1	Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

Maximum moment	M_{max} = **73.7** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **53.1** kN	V_{min} = **-67.4** kN
Deflection	δ_{max} = **2.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **53.1** kN	R_{A_min} = **53.1** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **25.6** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **12.3** kN	
Maximum reaction at support B	R_{B_max} = **67.4** kN	R_{B_min} = **67.4** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **32.5** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **15.7** kN	

Section details

Section type	**UKC 203x203x46**	Steel grade	**S275**



Section classification **Class 1**

Check shear - Section 6.2.6

Design shear force V_{Ed} = **67** kN Design shear resistance $V_{c,Rd}$ = **269.5** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = **73.7** kNm Des.bending resist.moment $M_{c,Rd}$ = **136.8** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT}$ = **0.377** Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = **0.400**

$\bar{\lambda}_{LT}$ < $\bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection δ_{lim} = **6.9** mm Maximum deflection δ = **2.854** mm

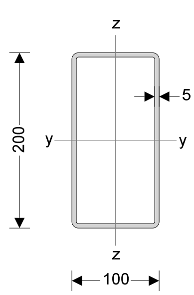
PASS - Maximum deflection does not exceed deflection limit

COLUMN

STEEL COLUMN DESIGN (EN 1993-1-1)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 1.0.09



Column and loading details

Column details

Column section	**RHS 200x100x5.0**		
System length y axis buckling	L_y = **3500** mm	System length z axis buckling	L_z = **3500** mm

Sway

The column is not part of a sway frame in the direction of the z axis

The column is not part of a sway frame in the direction of the y axis

Column loading

Axial load	N_{Ed} = **120** kN (Compression)		
Moment about y axis at end 1	$M_{y,Ed1}$ = **12.0** kNm	Moment about y axis at end 2	$M_{y,Ed2}$ = **0.0** kNm
		Single curvature bending about y axis	
Moment about z axis at end 1	$M_{z,Ed1}$ = **0.0** kNm	Moment about z axis at end 2	$M_{z,Ed2}$ = **0.0** kNm
Shear force parallel to z axis	$V_{z,Ed}$ = **0** kN	Shear force parallel to y axis	$V_{y,Ed}$ = **0** kN

Material details

Steel grade	**S275**		
Yield strength	f_y = **275** N/mm^2	Ultimate strength	f_u = **410** N/mm^2
Modulus of elasticity	E = **210** kN/mm^2	Shear modulus	G = **80.8** kN/mm^2

Buckling length for flexural buckling about y axis

End restraint factor	K_y = **1.000**	Buckling length	L_{cr_y} = **3500** mm

Buckling length for flexural buckling about z axis

End restraint factor	K_z = **1.000**	Buckling length	L_{cr_z} = **3500** mm

Section classification (Table 5.2)

Web classification	**1**	Flange classification	**1**

The section is class 1

Resistance of cross section (cl. 6.2)

Shear parallel to z axis (cl. 6.2.6)

Design shear force	$V_{z,Ed}$ = **0.0** kN	Plastic shear resistance	$V_{pl,z,Rd}$ = **304.1** kN

PASS - Shear resistance parallel to z axis exceeds the design shear force

$V_{z,Ed} <= 0.5 \times V_{pl,z,Rd}$ - No reduction in f_y required for bending/axial force

Shear parallel to y axis (cl. 6.2.6)

Design shear force	$V_{y,Ed}$ = **0.0** kN	Plastic shear resistance	$V_{pl,y,Rd}$ = **152.1** kN

PASS - Shear resistance parallel to y axis exceeds the design shear force

$V_{y,Ed} <= 0.5 \times V_{pl,y,Rd}$ - No reduction in f_y required for bending/axial force

Compression (cl. 6.2.4)

Design force	N_{Ed} = **120** kN	Design resistance	$N_{c,Rd}$ = **790** kN

PASS - The compression design resistance exceeds the design force

Bending about y axis (cl. 6.2.5)

Design bending moment	$M_{y,Ed}$ = **12.0** kNm	Design resistance	$M_{c,y,Rd}$ = **50.9** kNm

PASS - The bending design resistance about the y axis exceeds the design moment

Combined bending and axial force (cl. 6.2.9)

Bending about y axis (cl. 6.2.9.1)

Design bending moment	$M_{y,Ed}$ = **12.0** kNm	Modified design resistance	$M_{N,y,Rd}$ = **50.9** kNm

PASS - Bending resistance about y axis in presence of axial load exceeds design moment

Buckling resistance (cl. 6.3)

Axial buckling resistance

Flexural buck resist about y $N_{b,y,Rd}$ = **715.0** kN Flexural buck resist about z $N_{b,z,Rd}$ = **547.1** kN

Min. buckling resistance $N_{b,Rd}$ = **547.1** kN

PASS - The axial load buckling resistance exceeds the design axial load

Buckling resistance moment (cl.6.3.2.1)

Design bending moment $M_{y,Ed}$ = **12.0** kNm

Lat. torsional buck length fact K_{LT} = **1.00** Design buckling resistance mt $M_{b,Rd}$ = **50.9** kNm

PASS - The design buckling resistance moment exceeds the maximum design moment

Combined bending and axial compression (cl. 6.3.3)

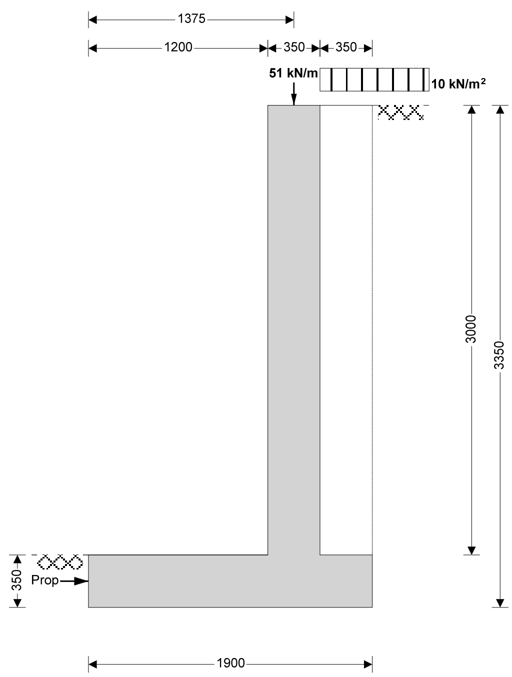
Section utilisation UR_{B_1} = **0.318** Section utilisation UR_{B_2} = **0.441**

PASS - The buckling resistance is adequate

WALL 1 (TEMPORARY CONDITION)

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} = **3000** mm	Wall stem thickness	t_{wall} = **350** mm
Length of toe	l_{toe} = **1200** mm	Length of heel	l_{heel} = **350** mm
Overall length of base	l_{base} = **1900** mm	Base thickness	t_{base} = **350** mm
Height of retaining wall	h_{wall} = **3350** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **350** 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

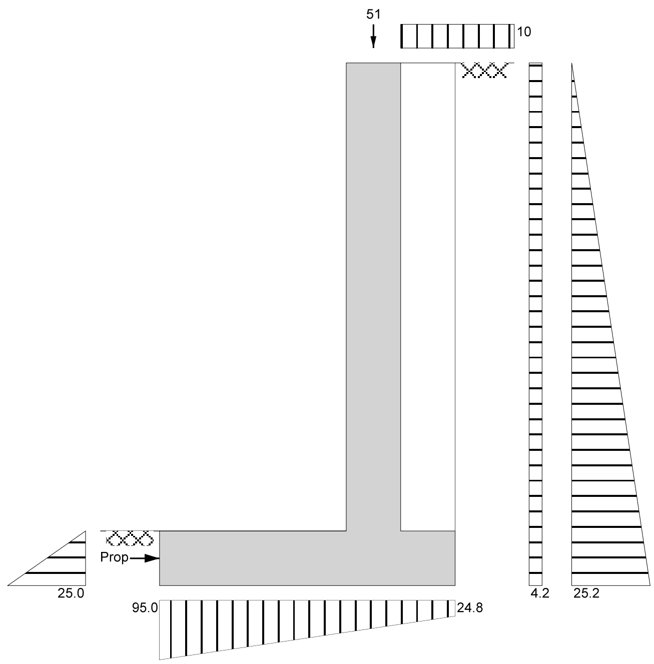
Density of wall construction	γ_{wall} = **23.6** kN/m^3	Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of soil surface	β = **0.0** deg	Effective height at back of wall	h_{eff} = **3350** mm
Mobilisation factor	M = **1.5**		
Moist density	γ_m = **18.0** kN/m^3	Saturated density	γ_s = **21.0** 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 = **10.0** kN/m^2		
Vertical dead load	W_{dead} = **35.0** kN/m	Vertical live load	W_{live} = **16.0** 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} = **1375** 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} = **20.2** kN/m

Check bearing pressure

Total vertical reaction	R = **113.9** kN/m	Distance to reaction	x_{bar} = **765** mm
Eccentricity of reaction	e = **185** mm		

Reaction acts within middle third of base

Bearing pressure at toe	p_{toe} = **95.0** kN/m^2	Bearing pressure at heel	p_{heel} = **24.8** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

<u>RETAINING WALL DESIGN (BS 8002:1994)</u>

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} = **20.2** kN/m

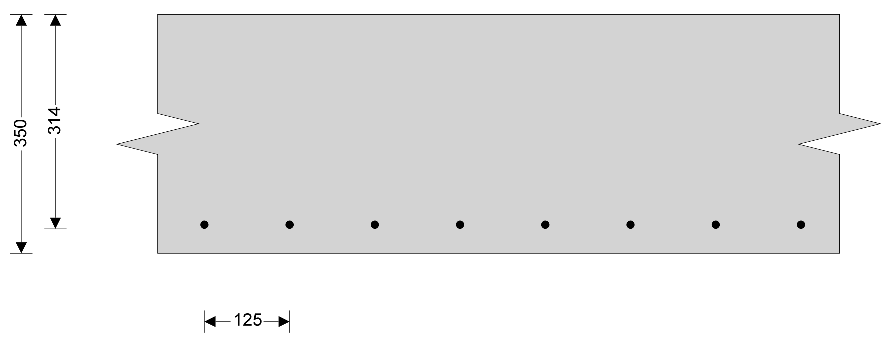
<u>Design of reinforced concrete retaining wall toe (BS 8002:1994)</u>

Material properties

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

Base details

Minimum reinforcement k = **0.13** % Cover in toe c_{toe} = **30** mm



Design of retaining wall toe

Shear at heel V_{toe} = **113.1** kN/m Moment at heel M_{toe} = **96.6** kNm/m

Compression reinforcement is not required

Check toe in bending

Reinforcement provided **12 mm dia.bars @ 125 mm centres**

Area required $A_{s\_toe\_req}$ = **744.8** mm^2/m Area provided $A_{s\_toe\_prov}$ = **905** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress v_{toe} = **0.360** N/mm^2 Allowable shear stress v_{adm} = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_toe} = **0.496** N/mm^2

v_{toe} < v_{c_toe} - No shear reinforcement required

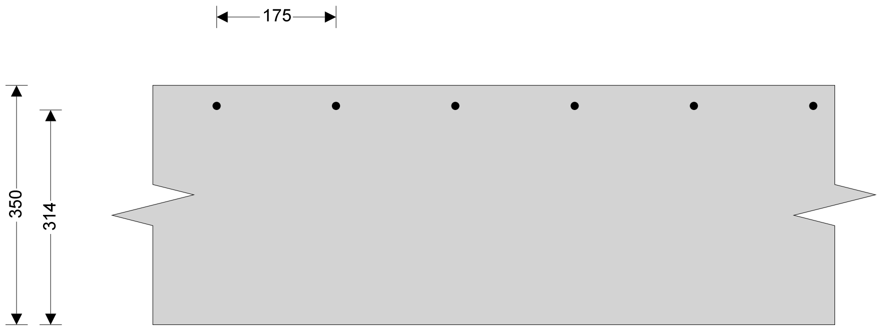
<u>Design of reinforced concrete retaining wall heel (BS 8002:1994)</u>

Material properties

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

Base details

Minimum reinforcement k = **0.13** % Cover in heel c_{heel} = **30** mm



Design of retaining wall heel

Shear at heel V_{heel} = **21.4** kN/m Moment at heel M_{heel} = **7.0** kNm/m

Compression reinforcement is not required

Check heel in bending

Reinforcement provided **12 mm dia.bars @ 175 mm centres**

Area required $A_{s\_heel\_req}$ = **455.0** mm^2/m Area provided $A_{s\_heel\_prov}$ = **646** mm^2/m

PASS - Reinforcement provided at the retaining wall heel is adequate

Check shear resistance at heel

Design shear stress v_{heel} = **0.068** N/mm^2 Allowable shear stress v_{adm} = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_heel} = **0.537** N/mm^2

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

<u>Design of reinforced concrete retaining wall stem (BS 8002:1994)</u>

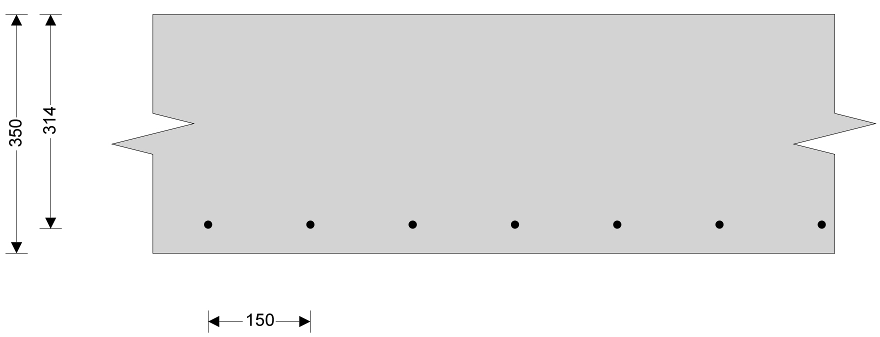
Material properties

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

Wall details

Minimum reinforcement k = **0.13** %

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



Design of retaining wall stem

Shear at base of stem V_{stem} = **36.5** kN/m Moment at base of stem M_{stem} = **89.4** kNm/m

Compression reinforcement is not required

Check wall stem in bending

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

Area required $A_{s\_stem\_req}$ = **689.1** mm^2/m Area provided $A_{s\_stem\_prov}$ = **754** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

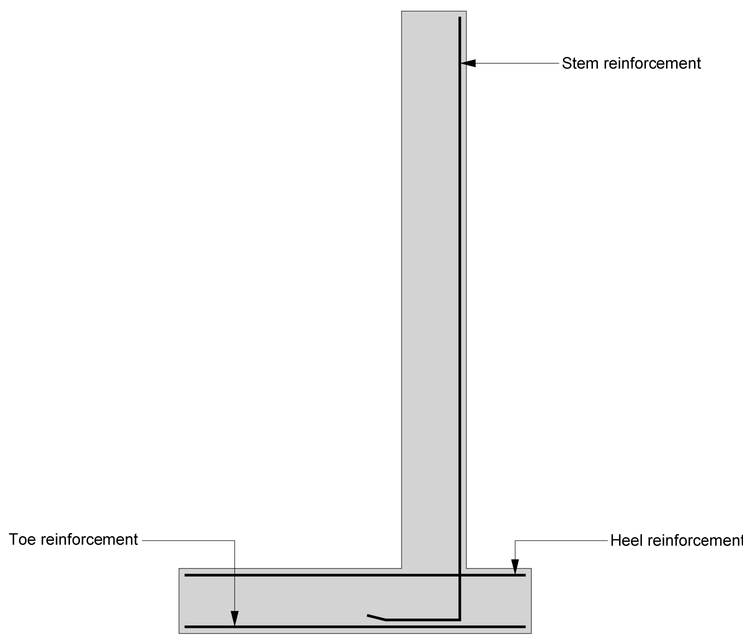
Design shear stress v_{stem} = **0.116** N/mm^2 Allowable shear stress v_{adm} = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.467** N/mm^2

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

<u>Indicative retaining wall reinforcement diagram</u>



Toe bars - 12 mm dia.@ 125 mm centres - (905 mm^2/m)
Heel bars - 12 mm dia.@ 175 mm centres - (646 mm^2/m)
Stem bars - 12 mm dia.@ 150 mm centres - (754 mm^2/m)

WALL 1 (PERMANENT CONDITION)

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06

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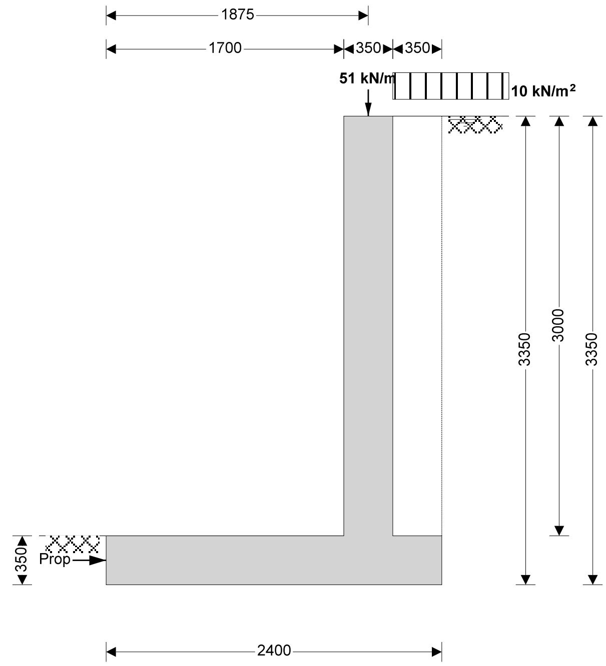


Wall details

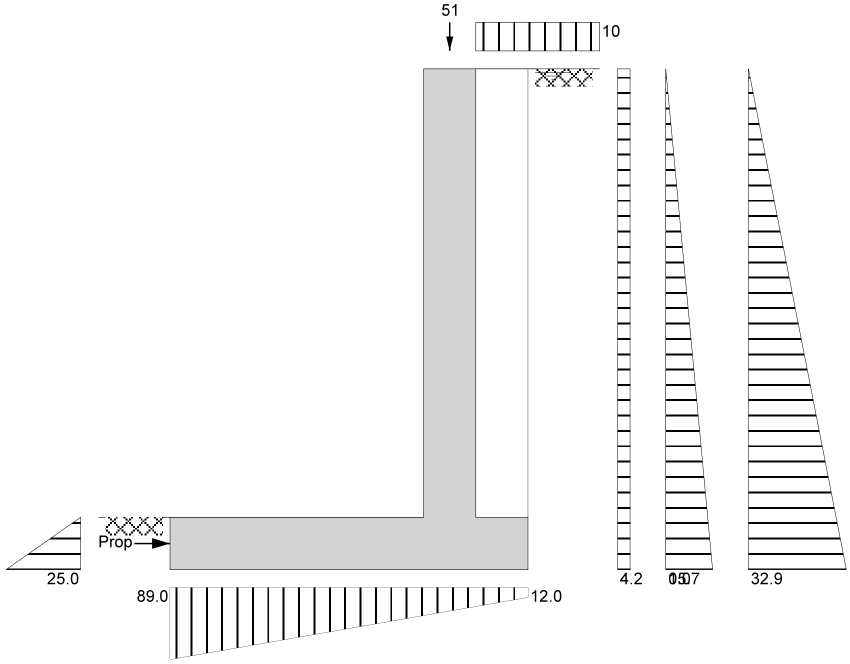
Retaining wall type	**Cantilever**		
Height of wall stem	h_{stem} = **3000** mm	Wall stem thickness	t_{wall} = **350** mm
Length of toe	l_{toe} = **1700** mm	Length of heel	l_{heel} = **350** mm
Overall length of base	l_{base} = **2400** mm	Base thickness	t_{base} = **350** mm
Height of retaining wall	h_{wall} = **3350** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **350** mm
Position of downstand	l_{ds} = **1400** 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 = **10.0** kN/m^2		
Vertical dead load	W_{dead} = **35.0** kN/m	Vertical live load	W_{live} = **16.0** 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} = **1875** 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} = **56.8** kN/m

Check bearing pressure

Total vertical reaction R = **121.2** kN/m Distance to reaction x_{bar} = **895** mm

Eccentricity of reaction e = **305** mm

Reaction acts within middle third of base

Bearing pressure at toe p_{toe} = **89.0** kN/m^2 Bearing pressure at heel p_{heel} = **12.0** 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} = **56.8** kN/m

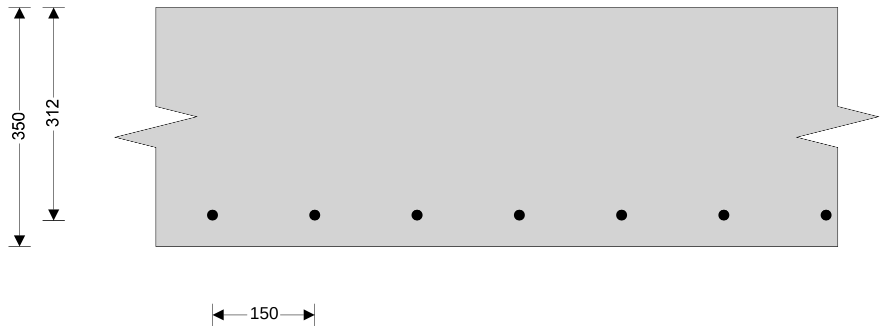
Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

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

Base details

Minimum reinforcement k = **0.13** % Cover in toe c_{toe} = **30** mm



Design of retaining wall toe

Shear at heel V_{toe} = **130.9** kN/m Moment at heel M_{toe} = **153.8** kNm/m

Compression reinforcement is not required

Check toe in bending

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

Area required $A_{s\_toe\_req}$ = **1196.2** mm^2/m Area provided $A_{s\_toe\_prov}$ = **1340** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress v_{toe} = **0.420** N/mm^2 Allowable shear stress v_{adm} = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_toe} = **0.568** N/mm^2

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

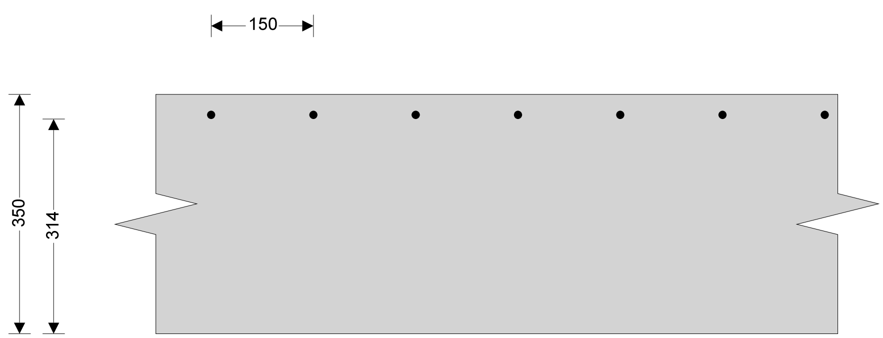
<u>Design of reinforced concrete retaining wall heel (BS 8002:1994)</u>

Material properties

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

Base details

Minimum reinforcement k = **0.13** % Cover in heel c_{heel} = **30** mm



Design of retaining wall heel

Shear at heel V_{heel} = **31.9** kN/m Moment at heel M_{heel} = **11.0** kNm/m

Compression reinforcement is not required

Check heel in bending

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

Area required $A_{s\_heel\_req}$ = **455.0** mm^2/m Area provided $A_{s\_heel\_prov}$ = **754** mm^2/m

PASS - Reinforcement provided at the retaining wall heel is adequate

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Check shear resistance at heel

Design shear stress v_{heel} = **0.102** N/mm^2 Allowable shear stress v_{adm} = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_heel} = **0.449** N/mm^2

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

<u>Design of reinforced concrete retaining wall stem (BS 8002:1994)</u>

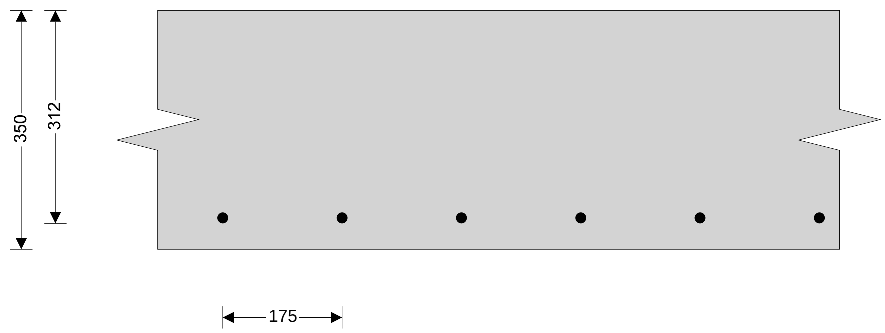
Material properties

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

Wall details

Minimum reinforcement k = **0.13** %

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



Design of retaining wall stem

Shear at base of stem V_{stem} = **29.1** kN/m Moment at base of stem M_{stem} = **125.0** kNm/m

Compression reinforcement is not required

Check wall stem in bending

Reinforcement provided **16 mm dia.bars @ 175 mm centres**

Area required $A_{s\_stem\_req}$ = **969.1** mm^2/m Area provided $A_{s\_stem\_prov}$ = **1149** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

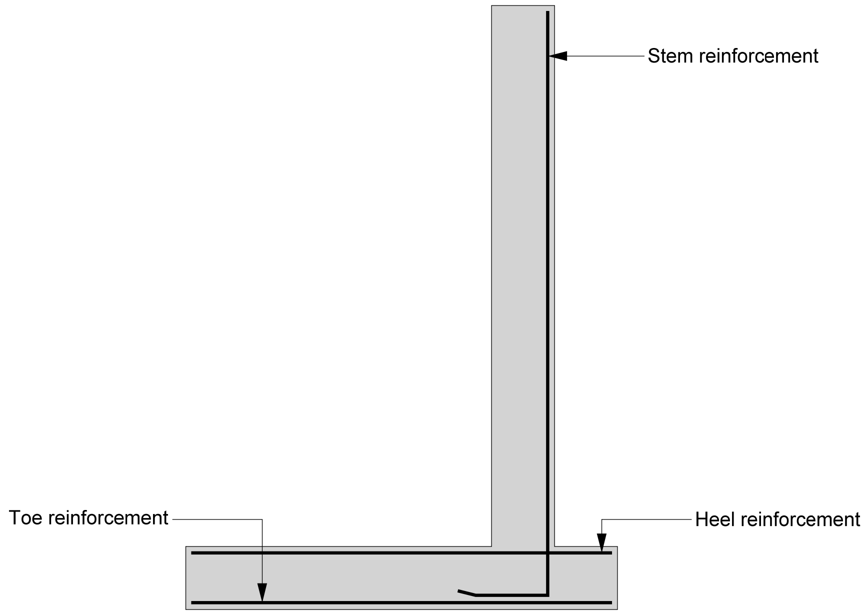
Design shear stress v_{stem} = **0.093** N/mm^2 Allowable shear stress v_{adm} = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.539** N/mm^2

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

<u>Indicative retaining wall reinforcement diagram</u>

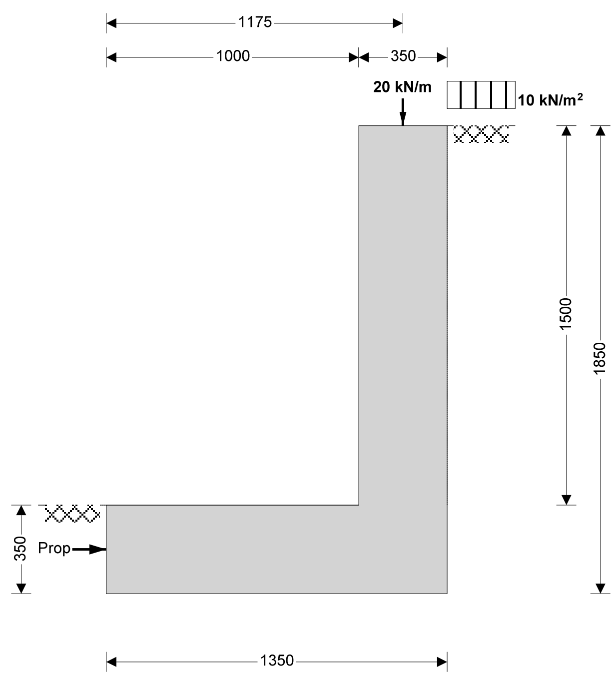


Toe bars - 16 mm dia.@ 150 mm centres - (1340 mm^2/m)
Heel bars - 12 mm dia.@ 150 mm centres - (754 mm^2/m)
Stem bars - 16 mm dia.@ 175 mm centres - (1149 mm^2/m)

WALL 2 (TEMPORARY CONDITION)

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



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Wall details

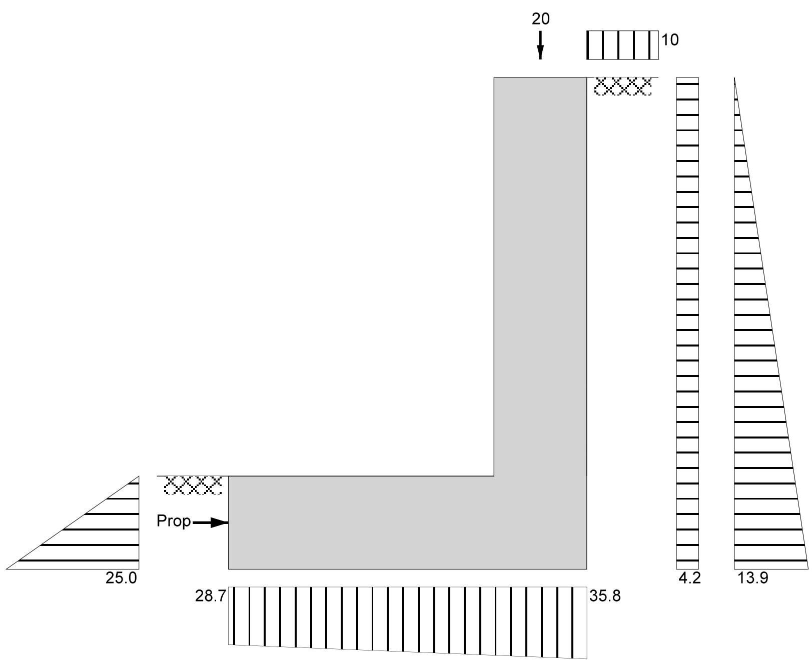
Retaining wall type	**Cantilever**		
Height of wall stem	h_{stem} = **1500** mm	Wall stem thickness	t_{wall} = **350** mm
Length of toe	l_{toe} = **1000** mm	Length of heel	l_{heel} = **0** mm
Overall length of base	l_{base} = **1350** mm	Base thickness	t_{base} = **350** mm
Height of retaining wall	h_{wall} = **1850** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **350** 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} = **1850** 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 = **10.0** kN/m^2		
Vertical dead load	W_{dead} = **16.0** kN/m	Vertical live load	W_{live} = **4.0** 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} = **1175** 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} = **3.0** kN/m

Check bearing pressure

Total vertical reaction	R = **43.5** kN/m	Distance to reaction	x_{bar} = **700** mm
Eccentricity of reaction	e = **25** mm		

Reaction acts within middle third of base

Bearing pressure at toe	p_{toe} = **28.7** kN/m^2	Bearing pressure at heel	p_{heel} = **35.8** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

CROFT STRUCTURAL ENGINEERS

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} = **3.0** kN/m

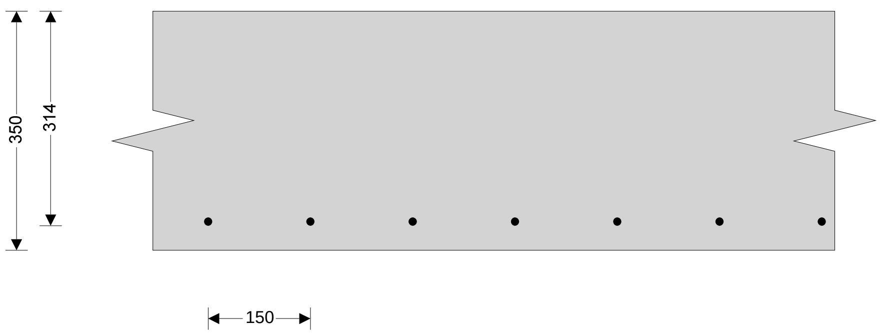
Design of reinforced concrete retaining wall toe (BS 8002:1994)

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} = **30** mm



Design of retaining wall toe

Shear at heel V_{toe} = **41.7** kN/m Moment at heel M_{toe} = **32.0** kNm/m

Compression reinforcement is not required

Check toe in bending

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

Area required $A_{s\_toe\_req}$ = **455.0** mm^2/m Area provided $A_{s\_toe\_prov}$ = **754** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress v_{toe} = **0.133** 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} = **0.488** N/mm^2

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

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

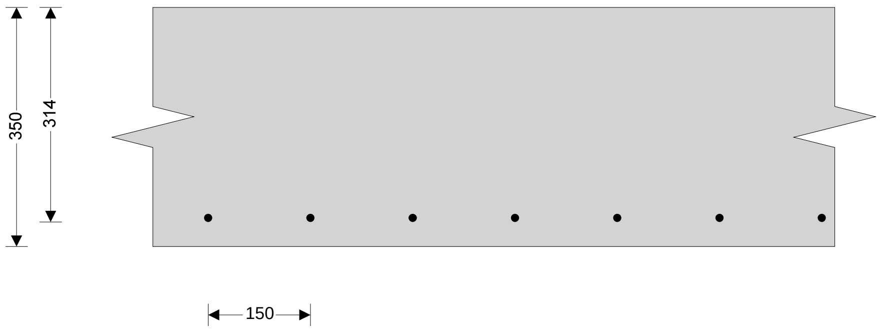
Material properties

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



Design of retaining wall stem

Shear at base of stem V_{stem} = **12.7** kN/m Moment at base of stem M_{stem} = **24.4** kNm/m

Compression reinforcement is not required

Check wall stem in bending

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

Area required $A_{s\_stem\_req}$ = **455.0** mm^2/m Area provided $A_{s\_stem\_prov}$ = **754** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

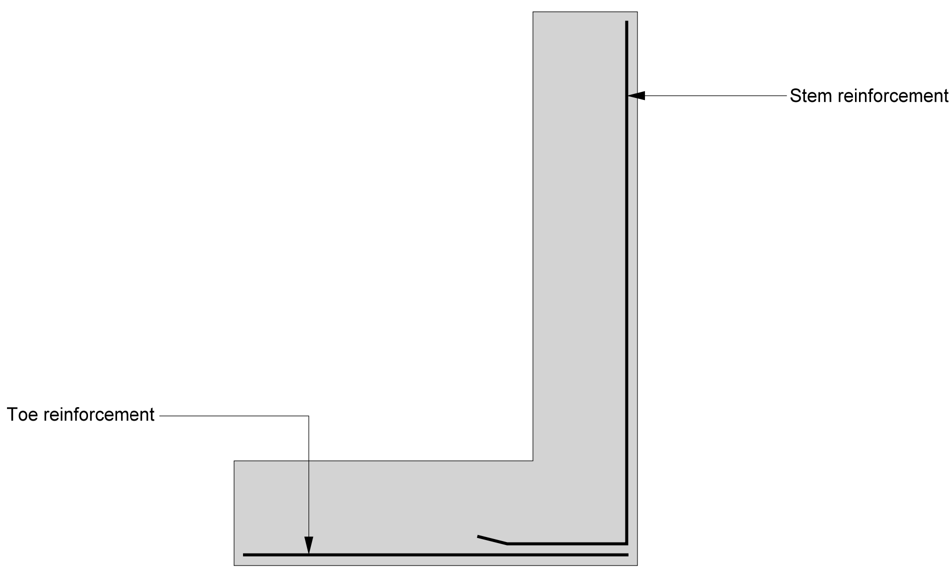
Design shear stress v_{stem} = **0.041** N/mm^2 Allowable shear stress v_{adm} = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.488** N/mm^2

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

<u>Indicative retaining wall reinforcement diagram</u>



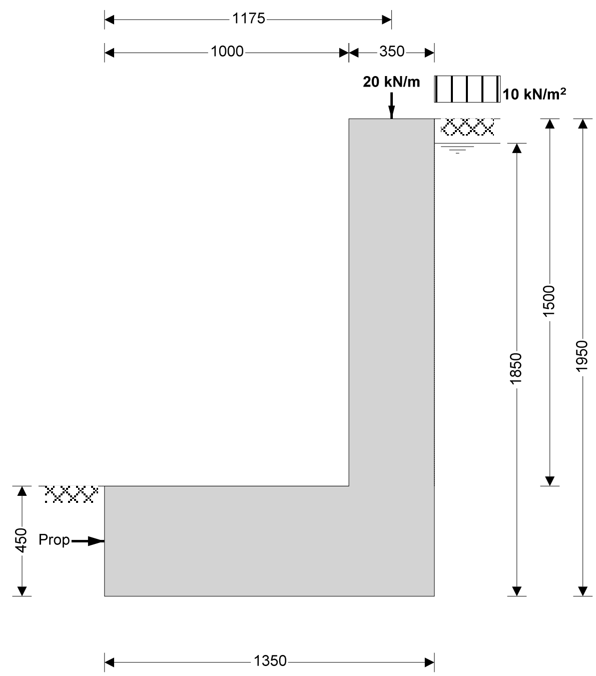
Toe bars - 12 mm dia.@ 150 mm centres - (754 mm^2/m)

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

WALL 2 (PERMANENT CONDITION)

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} = **1500** mm	Wall stem thickness	t_{wall} = **350** mm
Length of toe	l_{toe} = **1000** mm	Length of heel	l_{heel} = **0** mm
Overall length of base	l_{base} = **1350** mm	Base thickness	t_{base} = **450** mm
Height of retaining wall	h_{wall} = **1950** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **450** 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} = **1850** 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} = **1950** 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

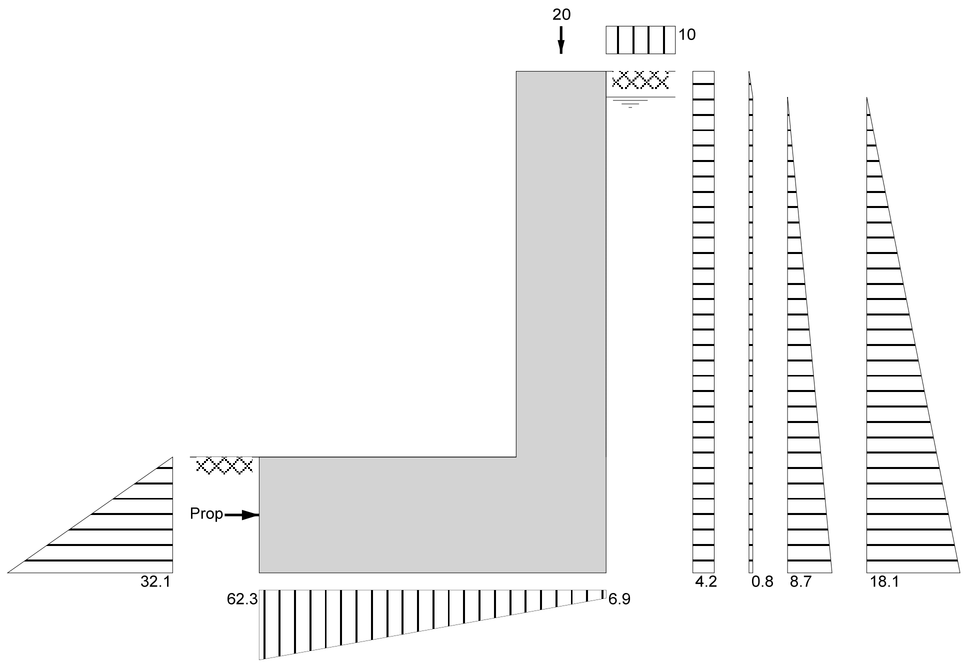
Using Coulomb theory

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

Loading details

Surcharge load	Surcharge = **10.0** kN/m^2		
Vertical dead load	W_{dead} = **16.0** kN/m	Vertical live load	W_{live} = **4.0** 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} = **1175** mm | Height of horizontal load h_{load} = **0** mm



Loads shown in kN/m, pressures shown in kN/m²

Calculate propping force

Propping force F_{prop} = **12.8** kN/m

Check bearing pressure

Total vertical reaction R = **46.7** kN/m | Distance to reaction x_{bar} = **495** mm

Eccentricity of reaction e = **180** mm

Reaction acts within middle third of base

Bearing pressure at toe p_{toe} = **62.3** kN/m^2 | Bearing pressure at heel p_{heel} = **6.9** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

<u>RETAINING WALL DESIGN (BS 8002:1994)</u>

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} = **12.8** kN/m

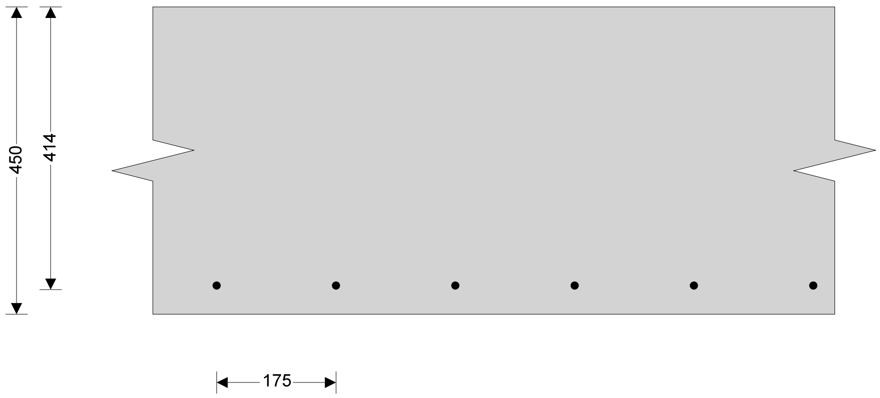
<u>Design of reinforced concrete retaining wall toe (BS 8002:1994)</u>

Material properties

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

Base details

Minimum reinforcement k = **0.13** % | Cover in toe c_{toe} = **30** mm



Design of retaining wall toe

Shear at heel V_{toe} = **51.3** kN/m Moment at heel M_{toe} = **48.5** kNm/m

Compression reinforcement is not required

Check toe in bending

Reinforcement provided **12 mm dia.bars @ 175 mm centres**

Area required $A_{s\_toe\_req}$ = **585.0** mm^2/m Area provided $A_{s\_toe\_prov}$ = **646** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress v_{toe} = **0.124** N/mm^2 Allowable shear stress v_{adm} = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_toe} = **0.381** N/mm^2

v_{toe} < v_{c_toe} - No shear reinforcement required

<u>**Design of reinforced concrete retaining wall stem (BS 8002:1994)**</u>

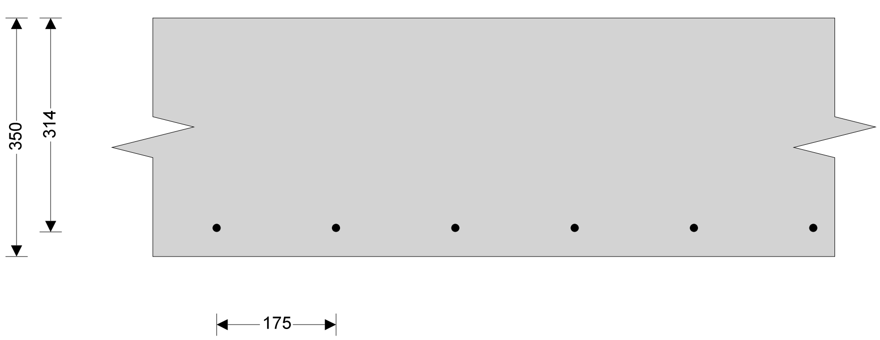
Material properties

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

Wall details

Minimum reinforcement k = **0.13** %

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



Design of retaining wall stem

Shear at base of stem V_{stem} = **8.5** kN/m Moment at base of stem M_{stem} = **25.9** kNm/m

Compression reinforcement is not required

Check wall stem in bending

Reinforcement provided **12 mm dia.bars @ 175 mm centres**

Area required $A_{s\_stem\_req}$ = **455.0** mm^2/m | Area provided $A_{s\_stem\_prov}$ = **646** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress v_{stem} = **0.027** N/mm^2 | Allowable shear stress v_{adm} = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.443** N/mm^2

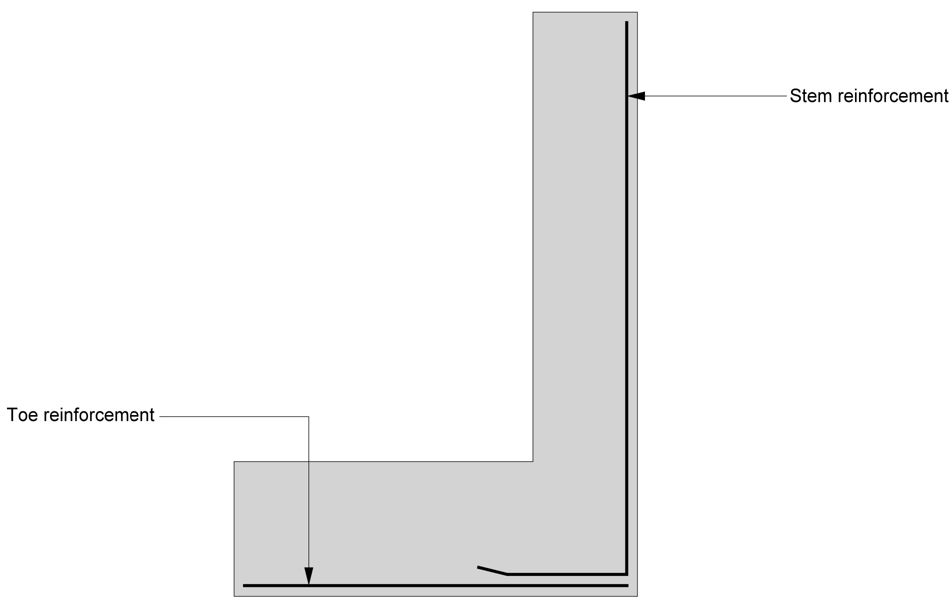
v_{stem} < v_{c_stem} - No shear reinforcement required

Check retaining wall deflection

Max span/depth ratio $ratio_{max}$ = **14.00** | Actual span/depth ratio $ratio_{act}$ = **4.78**

PASS - Span to depth ratio is acceptable

Indicative retaining wall reinforcement diagram



Toe bars - 12 mm dia.@ 175 mm centres - (646 mm^2/m)

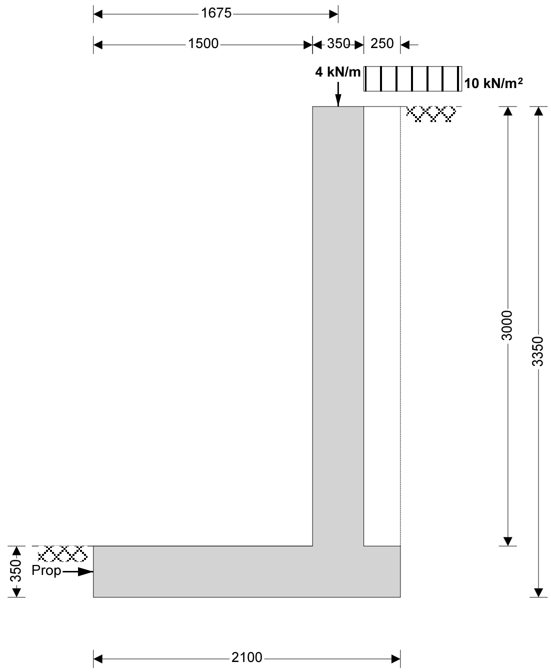
Stem bars - 12 mm dia.@ 175 mm centres - (646 mm^2/m)

WALL 3 (TEMPORARY CONDITION)

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06

CROFT STRUCTURAL ENGINEERS



Wall details

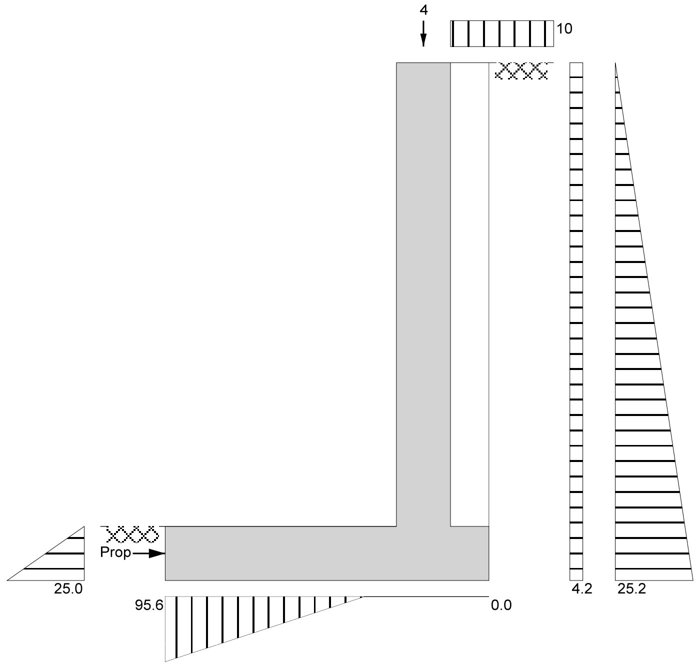
Retaining wall type	**Cantilever**		
Height of wall stem	h_{stem} = **3000** mm	Wall stem thickness	t_{wall} = **350** mm
Length of toe	l_{toe} = **1500** mm	Length of heel	l_{heel} = **250** mm
Overall length of base	l_{base} = **2100** mm	Base thickness	t_{base} = **350** mm
Height of retaining wall	h_{wall} = **3350** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **350** mm
Position of downstand	l_{ds} = **800** 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 = **10.0** kN/m^2		
Vertical dead load	W_{dead} = **1.0** kN/m	Vertical live load	W_{live} = **2.5** 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} = **1675** 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} = **32.9** kN/m

Check bearing pressure

Total vertical reaction R = **61.6** kN/m Distance to reaction x_{bar} = **430** mm

Eccentricity of reaction e = **620** mm

Reaction acts outside middle third of base

Bearing pressure at toe p_{toe} = **95.6** kN/m^2 Bearing pressure at heel p_{heel} = **0.0** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

<u>RETAINING WALL DESIGN (BS 8002:1994)</u>

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} = **32.9** kN/m

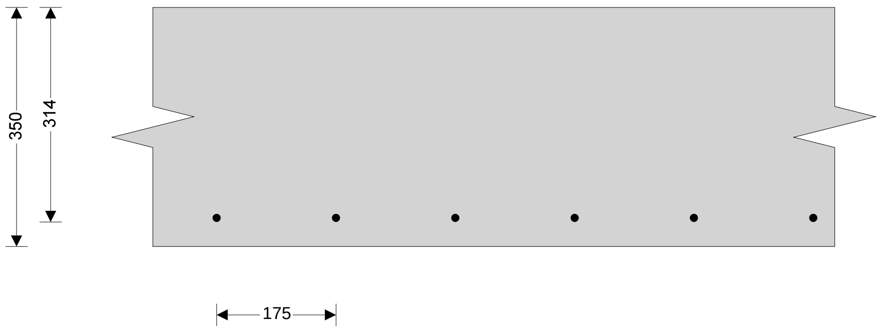
<u>Design of reinforced concrete retaining wall toe (BS 8002:1994)</u>

Material properties

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

Base details

Minimum reinforcement k = **0.13** % Cover in toe c_{toe} = **30** mm



Design of retaining wall toe

Shear at heel V_{toe} = **17.3** kN/m Moment at heel M_{toe} = **16.2** kNm/m

Compression reinforcement is not required

Check toe in bending

Reinforcement provided **12 mm dia.bars @ 175 mm centres**

Area required $A_{s\_toe\_req}$ = **455.0** mm^2/m Area provided $A_{s\_toe\_prov}$ = **646** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress v_{toe} = **0.056** N/mm^2 Allowable shear stress v_{adm} = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_toe} = **0.539** N/mm^2

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

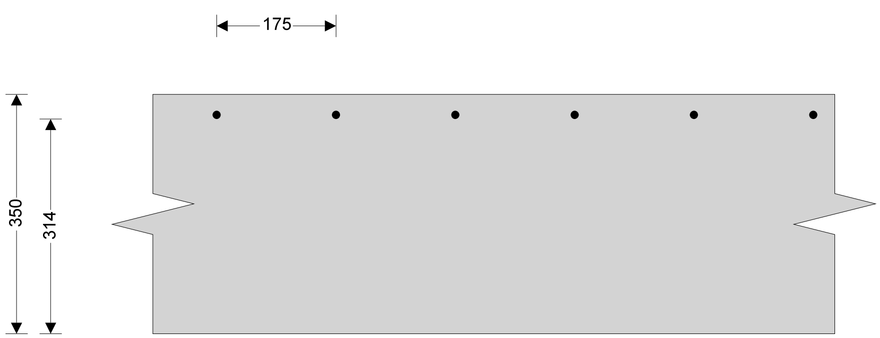
<u>Design of reinforced concrete retaining wall heel (BS 8002:1994)</u>

Material properties

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

Base details

Minimum reinforcement k = **0.13** % Cover in heel c_{heel} = **30** mm



Design of retaining wall heel

Shear at heel V_{heel} = **25.8** kN/m Moment at heel M_{heel} = **7.9** kNm/m

Compression reinforcement is not required

Check heel in bending

Reinforcement provided **12 mm dia.bars @ 175 mm centres**

Area required $A_{s\_heel\_req}$ = **455.0** mm^2/m Area provided $A_{s\_heel\_prov}$ = **646** mm^2/m

PASS - Reinforcement provided at the retaining wall heel is adequate

Check shear resistance at heel

Design shear stress v_{heel} = **0.082** N/mm^2 Allowable shear stress v_{adm} = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_heel} = **0.467** N/mm^2

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

<u>Design of reinforced concrete retaining wall stem (BS 8002:1994)</u>

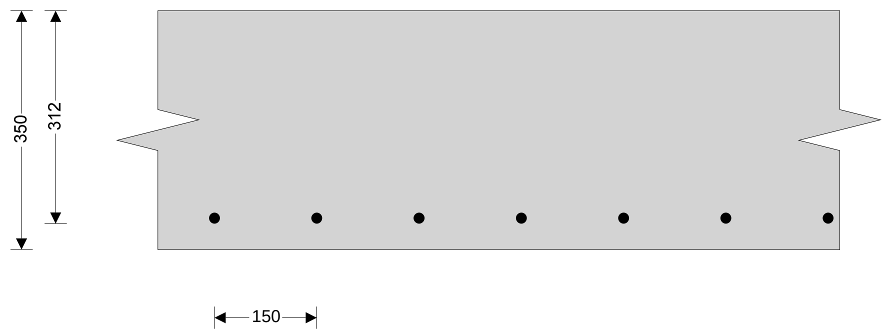
Material properties

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

Wall details

Minimum reinforcement k = **0.13** %

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



Design of retaining wall stem

Shear at base of stem V_{stem} = **13.0** kN/m Moment at base of stem M_{stem} = **126.1** kNm/m

Compression reinforcement is not required

Check wall stem in bending

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

Area required $A_{s\_stem\_req}$ = **977.8** mm^2/m Area provided $A_{s\_stem\_prov}$ = **1340** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress v_{stem} = **0.042** N/mm^2 Allowable shear stress v_{adm} = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.568** N/mm^2

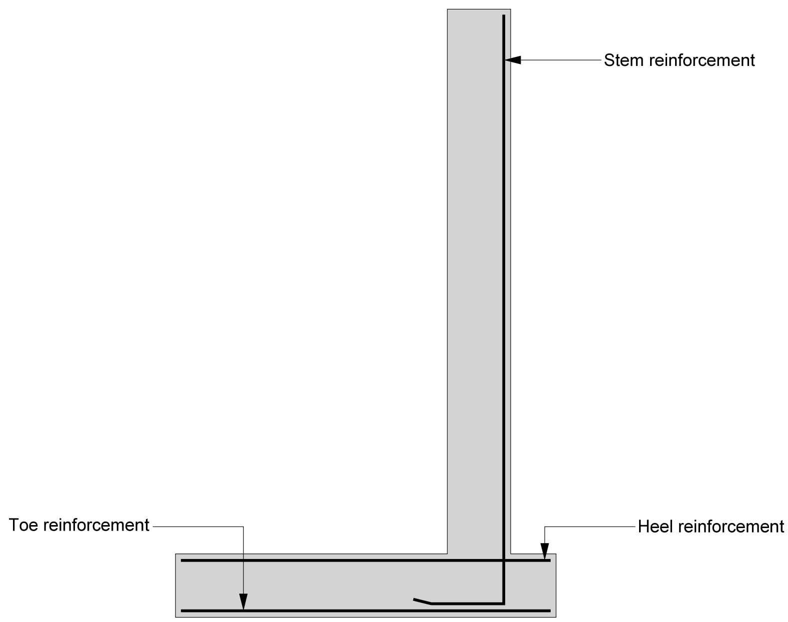
$v_{stem} < v_{c\_stem}$ - No shear reinforcement required

Check retaining wall deflection

Max span/depth ratio $ratio_{max}$ = **10.06** Actual span/depth ratio $ratio_{act}$ = **9.62**

PASS - Span to depth ratio is acceptable

<u>Indicative retaining wall reinforcement diagram</u>

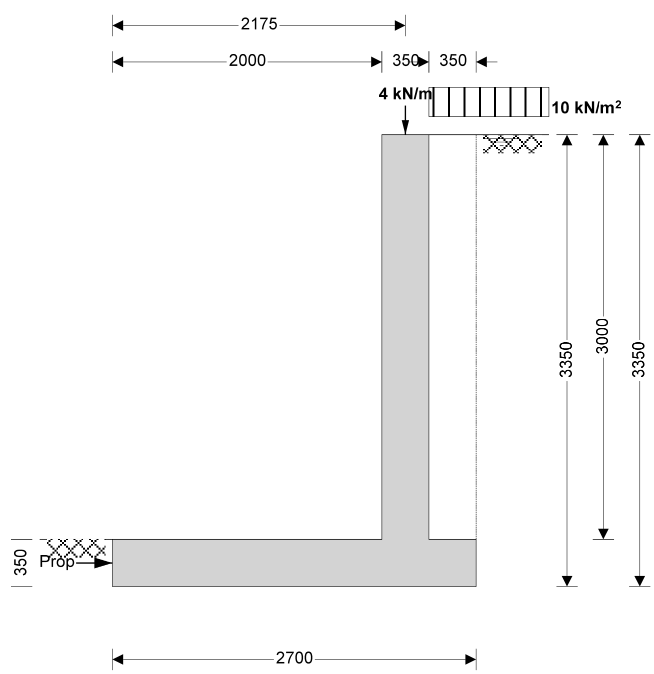


Toe bars - 12 mm dia.@ 175 mm centres - (646 mm^2/m)
Heel bars - 12 mm dia.@ 175 mm centres - (646 mm^2/m)
Stem bars - 16 mm dia.@ 150 mm centres - (1340 mm^2/m)

WALL 3 (PERMANENT CONDITION)

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



CROFT STRUCTURAL ENGINEERS

Wall details

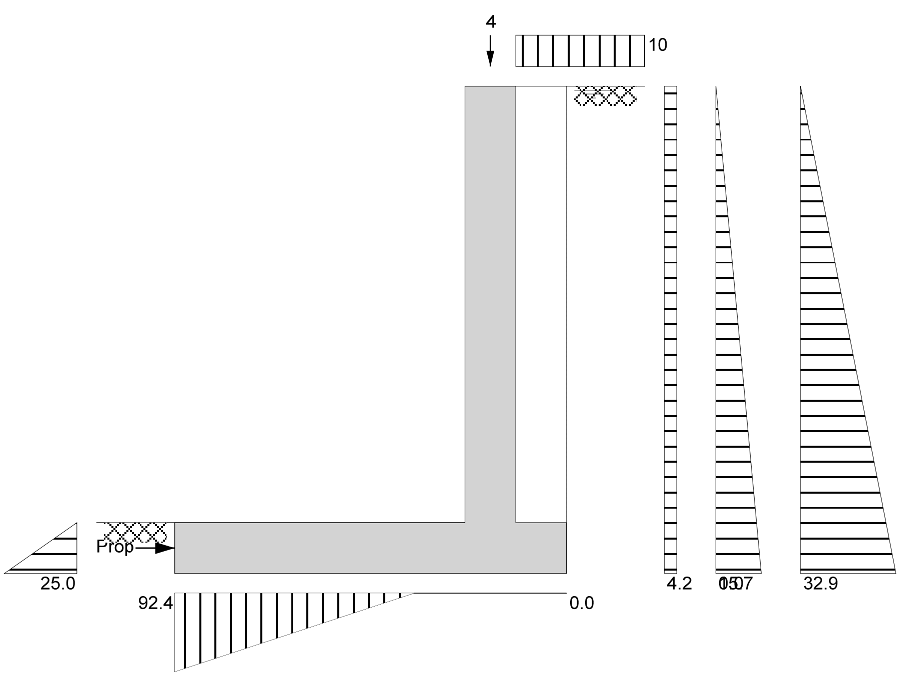
Retaining wall type	**Cantilever**		
Height of wall stem	h_{stem} = **3000** mm	Wall stem thickness	t_{wall} = **350** mm
Length of toe	l_{toe} = **2000** mm	Length of heel	l_{heel} = **350** mm
Overall length of base	l_{base} = **2700** mm	Base thickness	t_{base} = **350** mm
Height of retaining wall	h_{wall} = **3350** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **350** mm
Position of downstand	l_{ds} = **1650** 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 = **10.0** kN/m^2		
Vertical dead load	W_{dead} = **1.0** kN/m	Vertical live load	W_{live} = **2.5** 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} = **2175** 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} = **67.4** kN/m

Check bearing pressure

Total vertical reaction R = **76.1** kN/m Distance to reaction x_{bar} = **549** mm

Eccentricity of reaction e = **801** mm

Reaction acts outside middle third of base

Bearing pressure at toe p_{toe} = **92.4** 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} = **67.4** kN/m

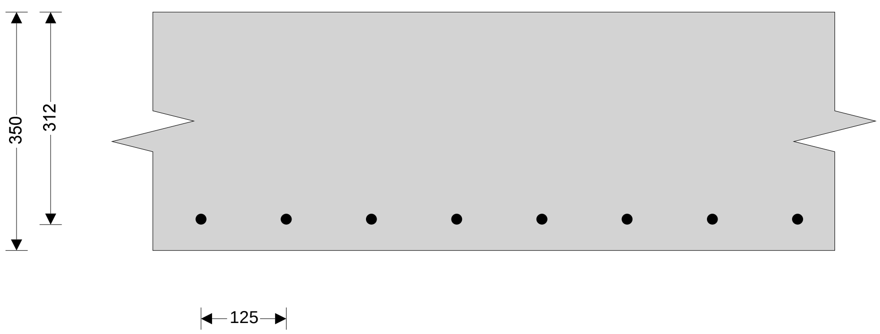
Design of reinforced concrete retaining wall toe (BS 8002:1994)

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} = **30** mm



Design of retaining wall toe

Shear at heel V_{toe} = **84.7** kN/m Moment at heel M_{toe} = **150.4** kNm/m

Compression reinforcement is not required

Check toe in bending

Reinforcement provided **16 mm dia.bars @ 125 mm centres**

Area required $A_{s\_toe\_req}$ = **1166.1** mm^2/m Area provided $A_{s\_toe\_prov}$ = **1608** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress v_{toe} = **0.271** 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} = **0.631** N/mm^2

v_{toe} < v_{c_toe} - No shear reinforcement required

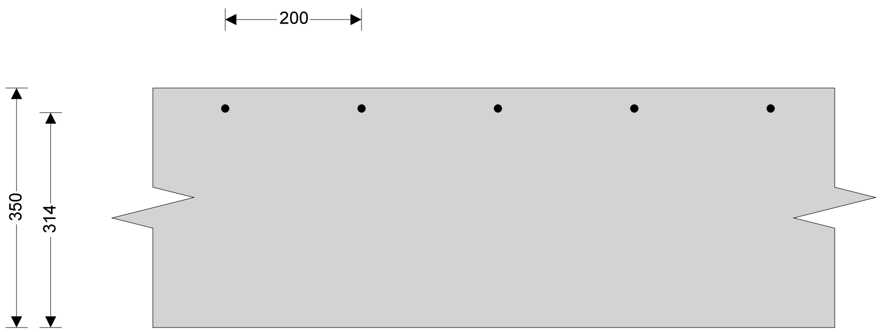
Design of reinforced concrete retaining wall heel (BS 8002:1994)

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 heel c_{heel} = **30** mm



Design of retaining wall heel

Shear at heel V_{heel} = **40.5** kN/m Moment at heel M_{heel} = **14.4** kNm/m

Compression reinforcement is not required

Check heel in bending

Reinforcement provided **12 mm dia.bars @ 200 mm centres**

Area required $A_{s\_heel\_req}$ = **455.0** mm^2/m Area provided $A_{s\_heel\_prov}$ = **565** mm^2/m

PASS - Reinforcement provided at the retaining wall heel is adequate

Check shear resistance at heel

Design shear stress v_{heel} = **0.129** 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_heel} = **0.443** N/mm^2

v_{heel} < v_{c_heel} - No shear reinforcement required

<u>Design of reinforced concrete retaining wall stem (BS 8002:1994)</u>

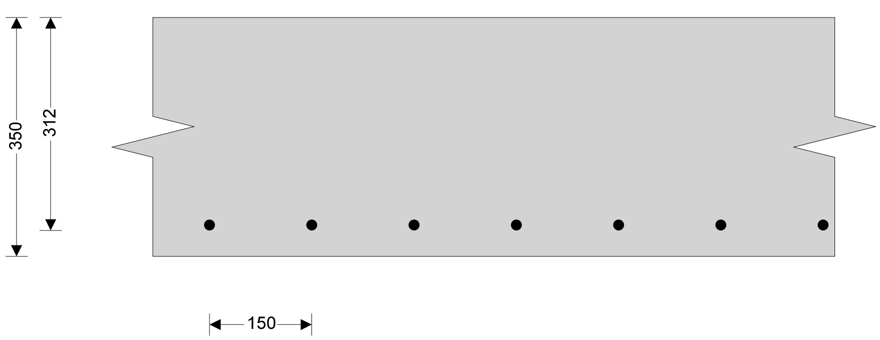
Material properties

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



Design of retaining wall stem

Shear at base of stem V_{stem} = **14.3** kN/m Moment at base of stem M_{stem} = **125.0** kNm/m

Compression reinforcement is not required

Check wall stem in bending

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

Area required $A_{s\_stem\_req}$ = **969.1** mm^2/m Area provided $A_{s\_stem\_prov}$ = **1340** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

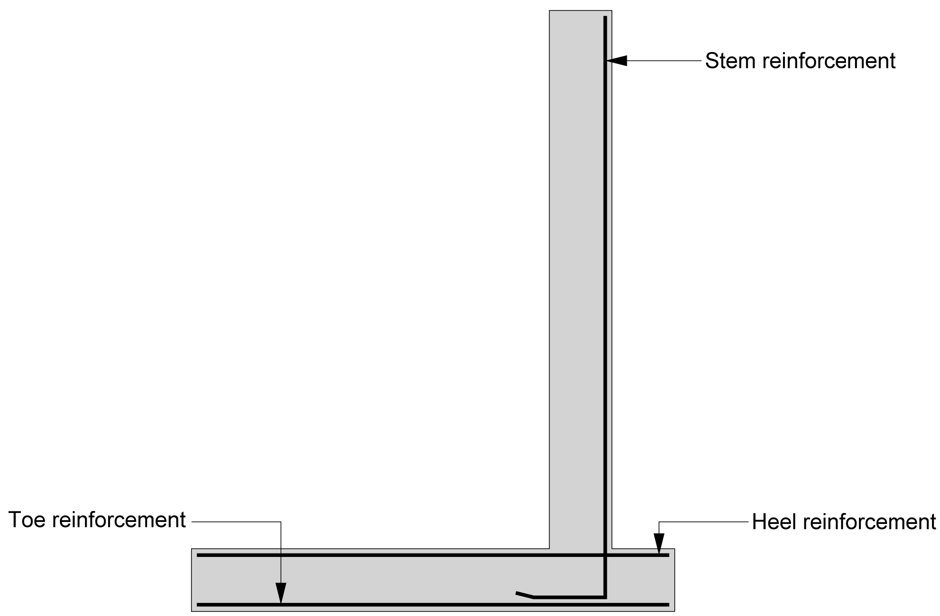
Design shear stress v_{stem} = **0.046** N/mm^2 Allowable shear stress v_{adm} = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.594** N/mm^2

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

<u>Indicative retaining wall reinforcement diagram</u>



Toe bars - 16 mm dia.@ 125 mm centres - (1608 mm^2/m)

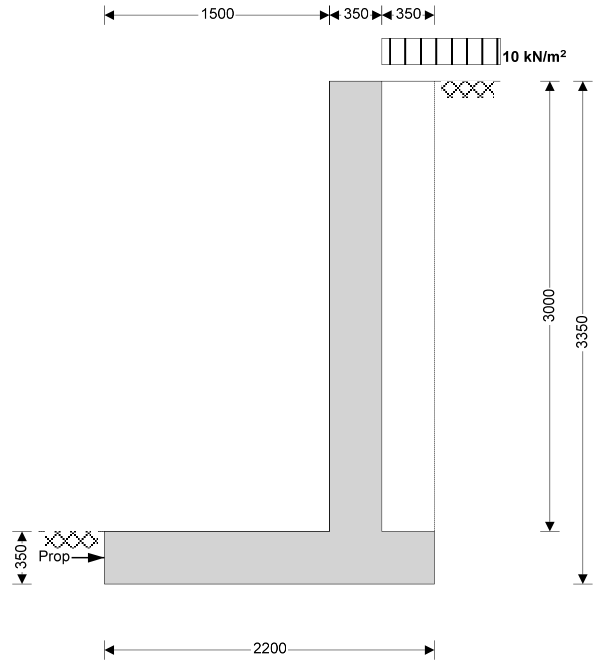
Heel bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)

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

WALL 4

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

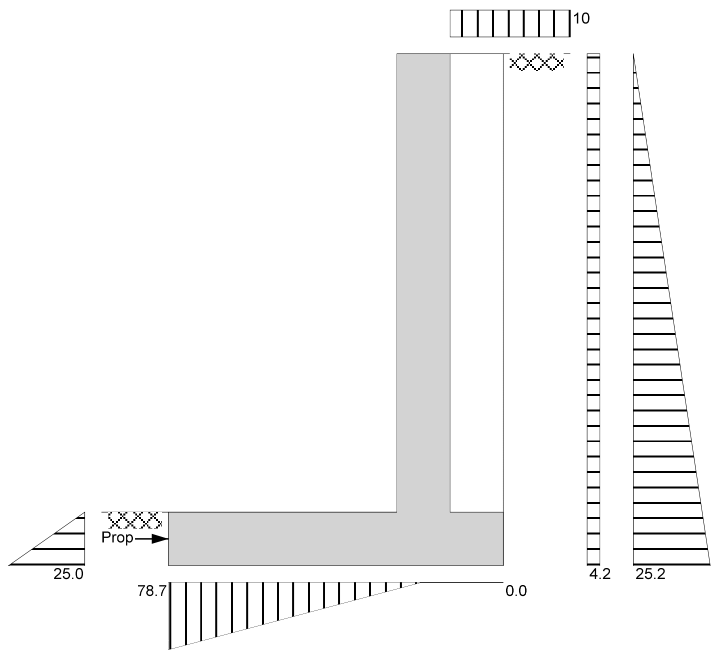
Retaining wall type	**Cantilever**		
Height of wall stem	h_{stem} = **3000** mm	Wall stem thickness	t_{wall} = **350** mm
Length of toe	l_{toe} = **1500** mm	Length of heel	l_{heel} = **350** mm
Overall length of base	l_{base} = **2200** mm	Base thickness	t_{base} = **350** mm
Height of retaining wall	h_{wall} = **3350** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **350** 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 = **10.0** kN/m^2		
Vertical dead load	W_{dead} = **0.0** kN/m	Vertical live load	W_{live} = **0.0** 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} = **0** mm	Height of horizontal load	h_{load} = **0** mm



Loads shown in kN/m, pressures shown in kN/m²

Calculate propping force

Propping force F_{prop} = **31.1** kN/m

Check bearing pressure

Total vertical reaction R = **65.4** kN/m Distance to reaction x_{bar} = **553** mm

Eccentricity of reaction e = **547** mm

Reaction acts outside middle third of base

Bearing pressure at toe p_{toe} = **78.7** kN/m^2 Bearing pressure at heel p_{heel} = **0.0** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

<u>RETAINING WALL DESIGN (BS 8002:1994)</u>

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} = **31.1** kN/m

<u>Design of reinforced concrete retaining wall toe (BS 8002:1994)</u>

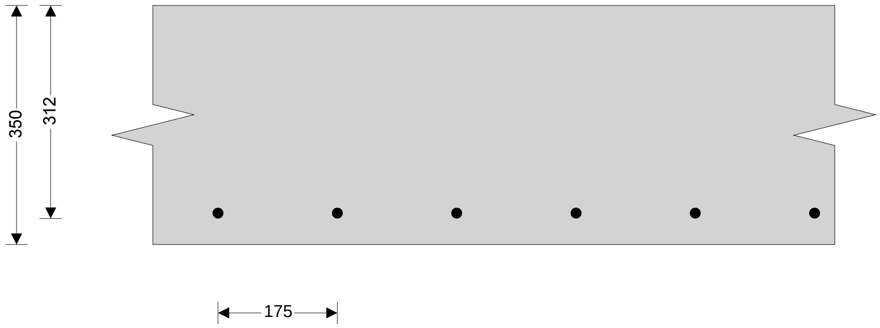
Material properties

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} = **30** mm

CROFT STRUCTURAL ENGINEERS



Design of retaining wall toe

Shear at heel V_{toe} = **74.8** kN/m Moment at heel M_{toe} = **133.3** kNm/m

Compression reinforcement is not required

Check toe in bending

Reinforcement provided **16 mm dia.bars @ 175 mm centres**

Area required $A_{s\_toe\_req}$ = **1034.1** mm^2/m Area provided $A_{s\_toe\_prov}$ = **1149** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress v_{toe} = **0.240** 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} = **0.564** N/mm^2

v_{toe} < v_{c_toe} - No shear reinforcement required

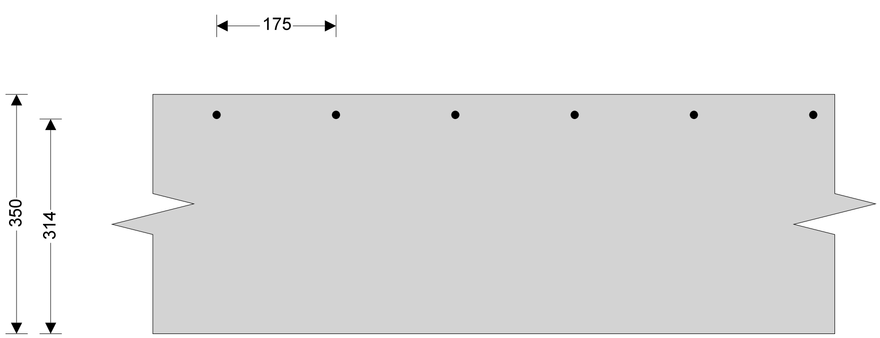
<u>Design of reinforced concrete retaining wall heel (BS 8002:1994)</u>

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 heel c_{heel} = **30** mm



Design of retaining wall heel

Shear at heel V_{heel} = **36.1** kN/m Moment at heel M_{heel} = **12.8** kNm/m

Compression reinforcement is not required

Check heel in bending

Reinforcement provided **12 mm dia.bars @ 175 mm centres**

Area required $A_{s\_heel\_req}$ = **455.0** mm^2/m Area provided $A_{s\_heel\_prov}$ = **646** mm^2/m

PASS - Reinforcement provided at the retaining wall heel is adequate

Check shear resistance at heel

Design shear stress v_{heel} = **0.115** 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_heel} = **0.464** N/mm^2

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

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

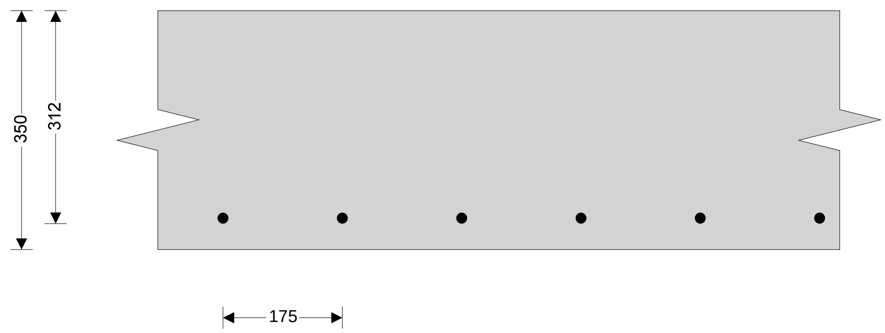
Material properties

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



Design of retaining wall stem

Shear at base of stem V_{stem} = **15.4** kN/m Moment at base of stem M_{stem} = **126.1** kNm/m

Compression reinforcement is not required

Check wall stem in bending

Reinforcement provided **16 mm dia.bars @ 175 mm centres**

Area required $A_{s\_stem\_req}$ = **977.8** mm^2/m Area provided $A_{s\_stem\_prov}$ = **1149** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

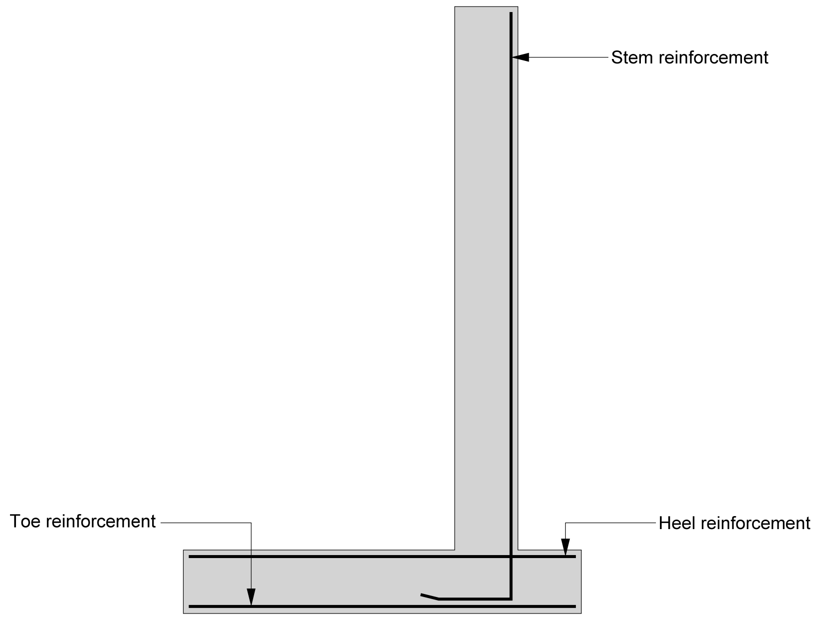
Design shear stress v_{stem} = **0.049** N/mm^2 Allowable shear stress v_{adm} = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.564** N/mm^2

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

Indicative retaining wall reinforcement diagram

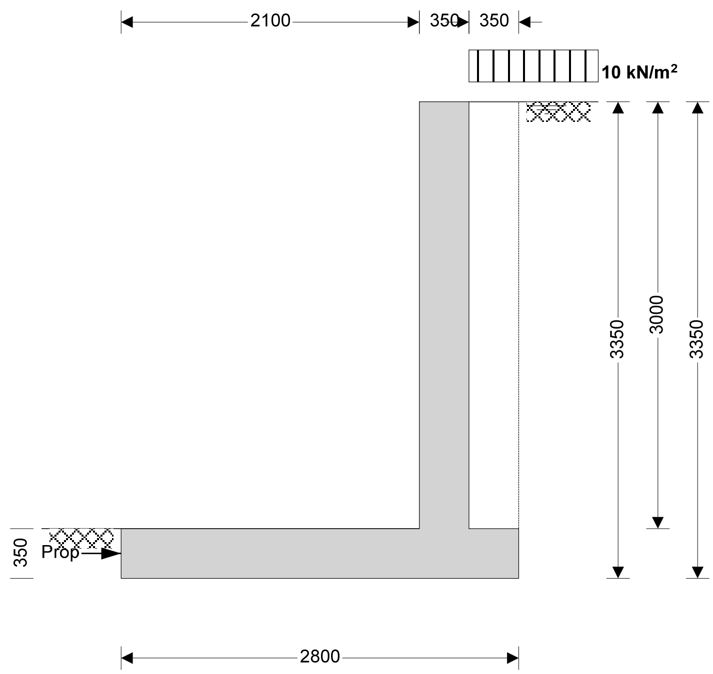


Toe bars - 16 mm dia.@ 175 mm centres - (1149 mm^2/m)
Heel bars - 12 mm dia.@ 175 mm centres - (646 mm^2/m)
Stem bars - 16 mm dia.@ 175 mm centres - (1149 mm^2/m)

WALL 4 (PERMANENT CONDITION)

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

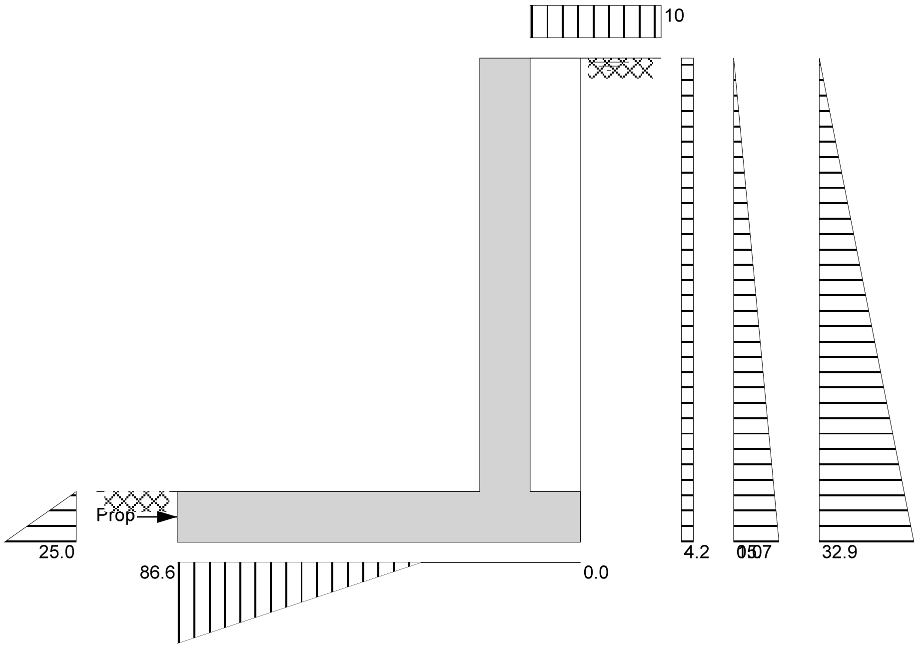
Retaining wall type	**Cantilever**		
Height of wall stem	h_{stem} = **3000** mm	Wall stem thickness	t_{wall} = **350** mm
Length of toe	l_{toe} = **2100** mm	Length of heel	l_{heel} = **350** mm
Overall length of base	l_{base} = **2800** mm	Base thickness	t_{base} = **350** mm
Height of retaining wall	h_{wall} = **3350** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **350** mm
Position of downstand	l_{ds} = **1750** 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 = **10.0** kN/m^2		
Vertical dead load	W_{dead} = **0.0** kN/m	Vertical live load	W_{live} = **0.0** 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} = **0** mm	Height of horizontal load	h_{load} = **0** mm



Loads shown in kN/m, pressures shown in kN/m²

Calculate propping force

Propping force	F_{prop} = **67.4** kN/m		

Check bearing pressure

Total vertical reaction	R = **73.5** kN/m	Distance to reaction	x_{bar} = **565** mm
Eccentricity of reaction	e = **835** mm		

Reaction acts outside middle third of base

Bearing pressure at toe	p_{toe} = **86.6** kN/m²	Bearing pressure at heel	p_{heel} = **0.0** kN/m²

PASS - Maximum bearing pressure is less than allowable bearing pressure

<u>RETAINING WALL DESIGN (BS 8002:1994)</u>

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} = **67.4** kN/m		

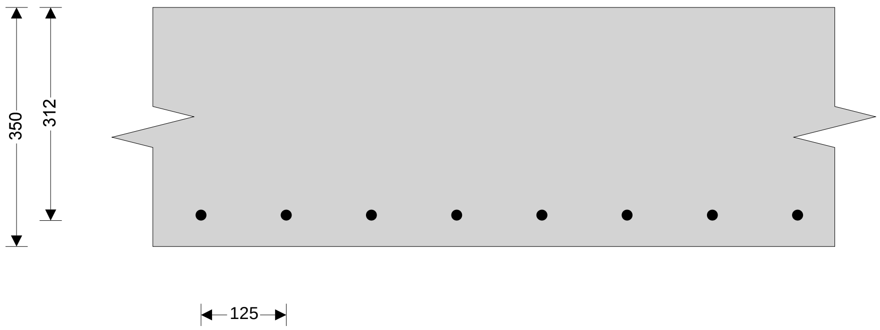
<u>Design of reinforced concrete retaining wall toe (BS 8002:1994)</u>

Material properties

Strength of concrete	f_{cu} = **40** N/mm²	Strength of reinforcement	f_y = **500** N/mm²

Base details

Minimum reinforcement	k = **0.13** %	Cover in toe	c_{toe} = **30** mm



Design of retaining wall toe

Shear at heel V_{toe} = **79.3** kN/m Moment at heel M_{toe} = **182.6** kNm/m

Compression reinforcement is not required

Check toe in bending

Reinforcement provided **16 mm dia.bars @ 125 mm centres**

Area required $A_{s\_toe\_req}$ = **1423.9** mm^2/m Area provided $A_{s\_toe\_prov}$ = **1608** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress v_{toe} = **0.254** 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} = **0.631** N/mm^2

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

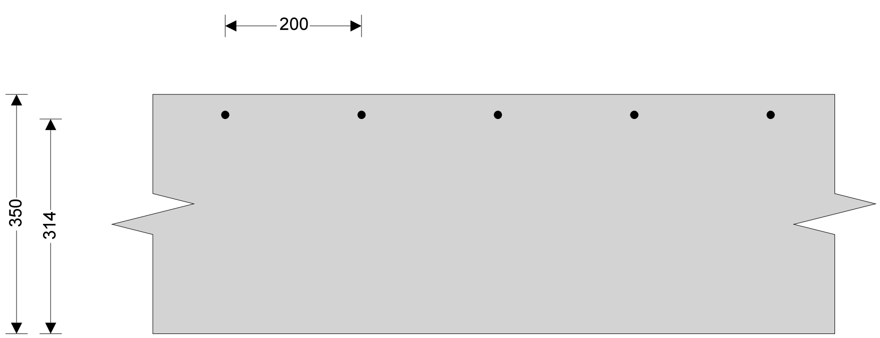
Design of reinforced concrete retaining wall heel (BS 8002:1994)

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 heel c_{heel} = **30** mm



Design of retaining wall heel

Shear at heel V_{heel} = **40.5** kN/m Moment at heel M_{heel} = **14.4** kNm/m

Compression reinforcement is not required

Check heel in bending

Reinforcement provided **12 mm dia.bars @ 200 mm centres**

Area required $A_{s\_heel\_req}$ = **455.0** mm^2/m Area provided $A_{s\_heel\_prov}$ = **565** mm^2/m

PASS - Reinforcement provided at the retaining wall heel is adequate

Check shear resistance at heel

Design shear stress v_{heel} = **0.129** 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_heel} = **0.443** N/mm^2

v_{heel} < v_{c_heel} - No shear reinforcement required

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

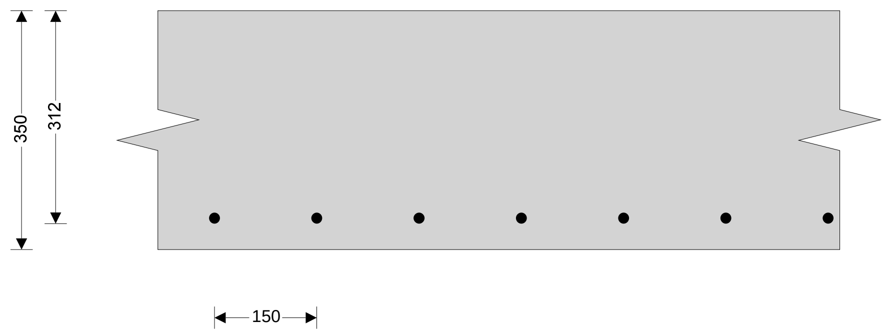
Material properties

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



Design of retaining wall stem

Shear at base of stem V_{stem} = **10.2** kN/m Moment at base of stem M_{stem} = **150.8** kNm/m

Compression reinforcement is not required

Check wall stem in bending

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

Area required $A_{s\_stem\_req}$ = **1169.9** mm^2/m Area provided $A_{s\_stem\_prov}$ = **1340** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

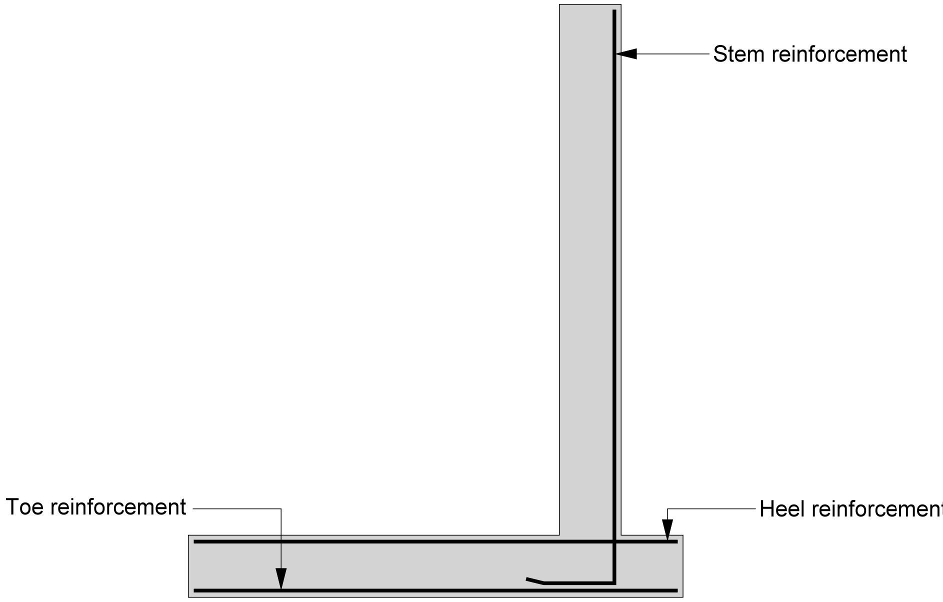
Design shear stress v_{stem} = **0.033** N/mm^2 Allowable shear stress v_{adm} = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.594** N/mm^2

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

Indicative retaining wall reinforcement diagram



Toe bars - 16 mm dia.@ 125 mm centres - (1608 mm^2/m)
Heel bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)
Stem bars - 16 mm dia.@ 150 mm centres - (1340 mm^2/m)

Appendix C

Basement Method Statement

106 Savernake Road

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 for 106 Savernake Road 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. Prior to construction, the excavations for the basement retaining walls will be propped. This will include propping at high level which will remain in place as sacrificial propping when the concrete is cast.

1.7. The cantilever pins are designed to be inherently stable during the construction stage without temporary propping to the head. However, propping at high level should be installed to increase the safety margin during construction and to keep associated ground movements to a minimum. 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.

1.8. A soil investigation has been undertaken. The soil conditions are London Clay Formation: Brown with blue grey mottling, slightly silty clay. Refer Soil Investigation (SI) report attached.

1.9. 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.

1.10. There is ground water seepage found at 2.3m below ground in the bore hole taken at site. After a month's time the reading in the bore hole indicated a ground level of 1.65 m below ground level. Refer SI report attached.

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. Water seepage is observed in the bore hole taken at site. Hence local dewatering is expected at 1.65 m below ground level.

2.4.1. Place a bore hole to the front of the property down to a depth of 6m
2.4.2. Pump water away from site.

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

3. Basement Sequencing

3.1. Begin by casting the mass concrete pad foundations for the steel columns in existing cellar location as shown in the drawing SL-10.

3.2. Excavate the front light-well to basement formation level.

3.2.1. Needle and prop the front bay wall above the excavation.

3.3. Place inclined conveyor from excavated light well to external ground level.

3.4. Excavate and prop pit for Pin No. 1 (pin numbers are as referenced on Drawing SL-10). Follow the steps described in Section 4.

3.4.1. Ensure the front wall above is suitably propped before the excavation.

3.5. Place rebar and cast concrete for retaining wall for Pin No. 1, following the steps in Section 4.

3.6. To allow (below ground) access to construct Pin No 3, partially excavate soil (and prop) in the location of Pin No. 5

3.6.1. At this stage, do not excavate below the party wall for Pin 5.

3.7. Excavate and prop pit for Pin No. 3, and then construct retaining wall underpin (following the steps described in Section 4).

3.8. Repeat the above steps for the remaining underpins around the perimeter, following the sequence shown on drawing SL-10.

3.8.1. Ensure that load-bearing walls above are suitably needled and propped before excavating below them.

3.8.2. Prop the existing ground floor structure as the excavation progresses.

3.8.3. Excavation for the next numbered sequential sections of underpinning shall not commence until at least 8 hours after drypacking of previous works. Excavation of adjacent pin to not commence until 48 hours after drypacking. (24hours possible due to inclusion of Conbextra 100 cement accelerator to dry pack mix).

3.8.4.Erect steel columns as required

3.8.5.Steelwork to support Ground floor to be inserted as works progress.

3.9. 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.

3.10. Provide water proofing to retaining walls as required.

4. Underpinning and Cantilevered Walls

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

4.2. Provide propping to floor where necessary.

4.3. Excavate first section of retaining wall (no more than 1000mm wide). Where excavation is greater than 1.0m wide provide temporary propping to sides of excavation to prevent earth collapse (Health and Safety). A 1000mm width wall has a lower risk of collapse to the heel face.

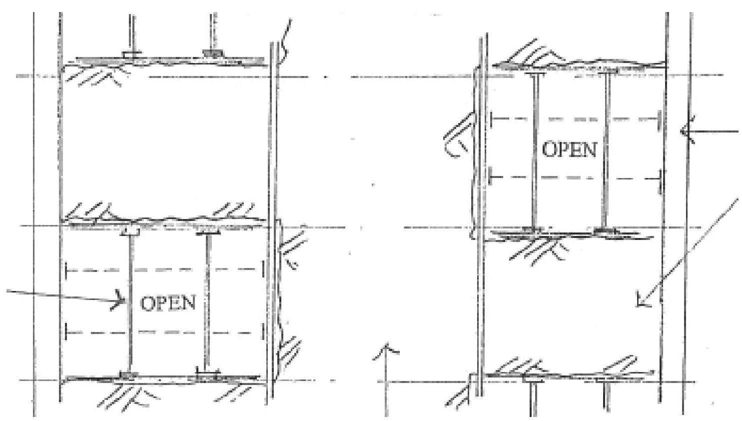


Figure 1 – Schematic Plan view of Soil Propping

Figure 2 Propping

Figure 3 Excavation of Pin

Figure 4 Completed Wall

4.4. Back propping 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 as the excavation progresses.

4.4.1. <u>High level propping should be included.</u>

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

4.4.3. 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.)

4.4.4. 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.

4.4.5. 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.

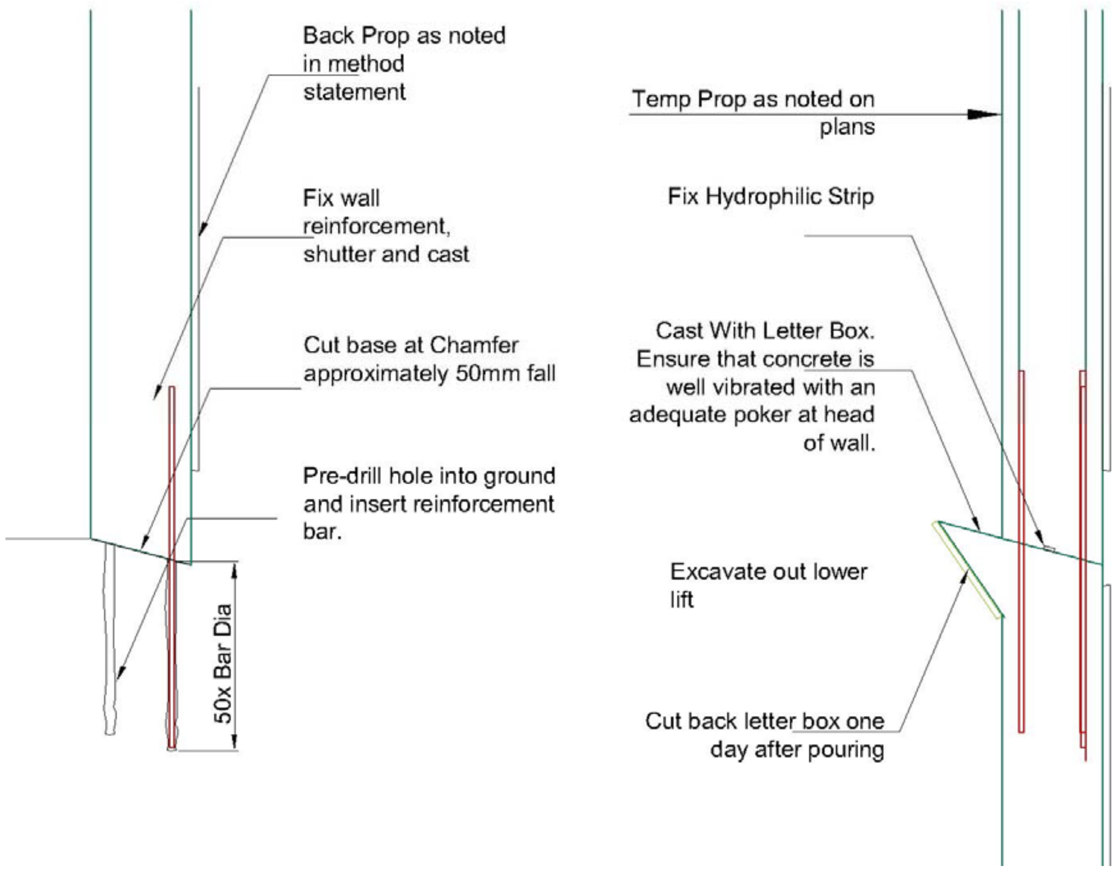
4.6. Underpins can be completed in Segmental lifts (eg: top section of wall followed by bottom section of wall).

Crofts recommendation is that walls with high vertical loads or susceptible to settlement, and all party walls, should be completed as first pin top first pin bottom, next pin top next pin bottom. **<u>We do not recommend</u>** for such conditions that all the top sections for every pin followed by all the lower pins are completed; such a sequencing can result in the existing wall being left on a narrower section than the original footing for too long resulting in settlement.

4.6.1.Place reinforcement for retaining wall segmental lift

4.6.1.1. At lift sections reinforcement needs to be driven in. This is to be completed by pre drilling holes and inserting the reinforcement into the predrilled hole.

4.6.1.2. Underside of the wall to be cast with chamfer to allow concrete for lower lift to be cast and no packing to be required.



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

4.8. 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.

4.9. Clear underside of existing footing.

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

4.11. Place blinding.

4.12. Place reinforcement for retaining wall base, heel (wherever it is present –as shown in drawing) & toe. Site supervisor to inspect and sign off works for proceeding to next stage.

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

4.14. Ensure that Concrete is of sufficient strength, check engineers specifications

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

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

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

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

4.19. 24 hours after pouring the concrete pin the gap shall be filled using a dry pack mortar. Ram in drypack between retaining wall and existing masonry.

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

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

4.22. 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 pad stones built into the masonry walls (refer to Structural Engineer's details for pad stone & beam sizes)

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

6. Supporting existing walls above basement excavation

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

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

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

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

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

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

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

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

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

7. Approval

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

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

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

8. Trench sheet design and temporary prop Calculations

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

Trench sheets should be placed at centres to deal with the ground. It is expected that the soil between the trench sheeting will arch. Looser soil will required tighter centres. It is typical for underpins to be placed at 1200c/c, in this condition the highest load on a trench sheet is when 2 no’s trench sheets are used. It is for this design that these calculations have been provided.

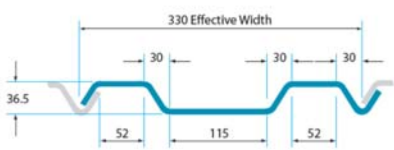
Soil and ground conditions are variable. Typically one finds that in the temporary condition clays are more stable and the C_u (cohesive) values in clay reduce the risk of collapse. It is this cohesive nature that allows clays to be cut into a vertical slope. For these calculations weak sand and gravels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

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

STANDARD LAP TRENCH SHEETING

STANDARD LAP

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



Technical Information

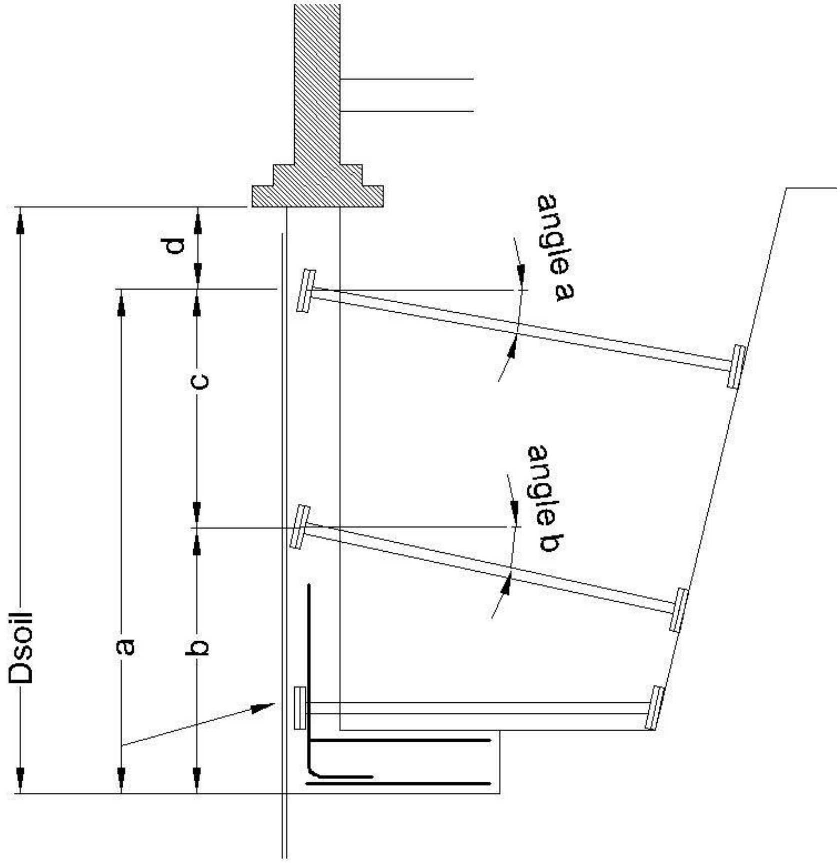
Effective width per sheet (mm)	330
Thickness (mm)	3.4
Depth (mm)	35
Weight per linear metre (kg/m)	10.8
Weight per m² (kg)	32.9
Section modulus per metre width (cm³)	48.3
Section modulus per sheet (cm³)	15.9
I value per metre width (cm⁴)	81.7
I value per sheet (cm⁴)	26.9
Total rolled metres per tonne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

Sxx = 15.9 cm^3

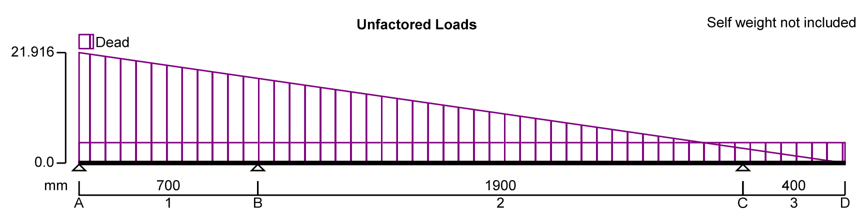
py = 275N/mm^2

Ixx = 26.9cm^4

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



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



CONTINUOUS BEAM ANALYSIS - INPUT

BEAM DETAILS

Number of spans = **3**

Material Properties:

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

Support Conditions:

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

CROFT STRUCTURAL ENGINEERS

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

Span Definitions:

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

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

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

LOADING DETAILS

Beam Loads:

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

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

LOAD COMBINATIONS

Load combination 1

Span 1 1×Dead

Span 2 1×Dead

Span 3 1×Dead

CONTINUOUS BEAM ANALYSIS - RESULTS

Unfactored support reactions

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

Support Reactions - Combination Summary

Support A Max react = **-1.4** kN Min react = **-1.4** kN Max mom = **0.0** kNm Min mom = **0.0** kNm

Support B Max react = **-32.8** kN Min react = **-32.8** kN Max mom = **0.0** kNm Min mom = **0.0** kNm

Support C Max react = **-10.8** kN Min react = **-10.8** kN Max mom = **0.0** kNm Min mom = **0.0** kNm

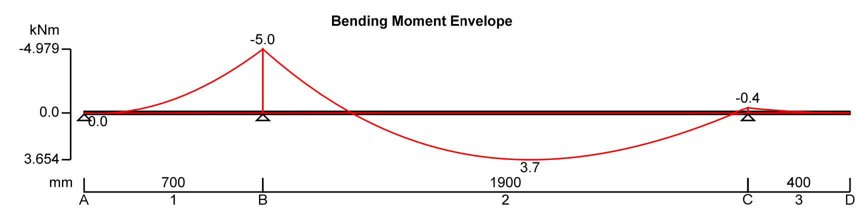
Support D Max react = **0.0** kN Min react = **0.0** kN Max mom = **0.0** kNm Min mom = **0.0** kNm

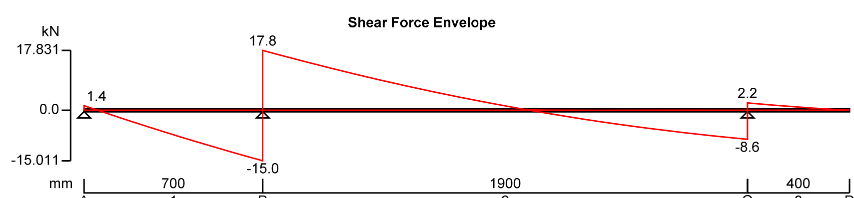
Beam Max/Min results - Combination Summary

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

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

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





Number of sheets Nos = 2

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

SRU4.0

Safe working loads for Acrow Props — loads given in kN

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

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

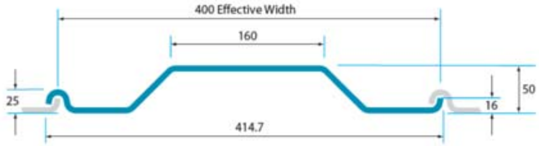
Any Acro Prop is accetpable

CROFT STRUCTURAL ENGINEERS

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

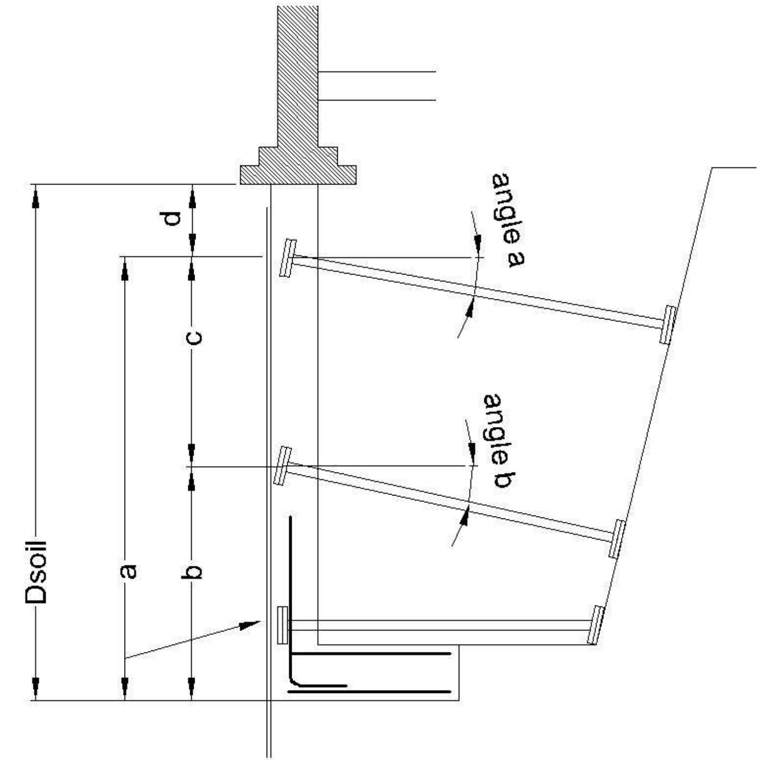
Effective width per sheet (mm)	400
Thickness (mm)	6.0
Depth (mm)	50
Weight per linear metre (kg/m)	21.90
Weight per m² (kg)	55.2
Section modulus per metre width (cm³)	101
Section modulus per sheet (cm³)	40.34
I value per metre width (cm⁴)	250
I value per sheet (cm⁴)	101
Total rolled metres per tonne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

$Sxx = 48.3cm^3$

$py = 275N/mm^2$

$Ixx = 26.9cm^4$

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

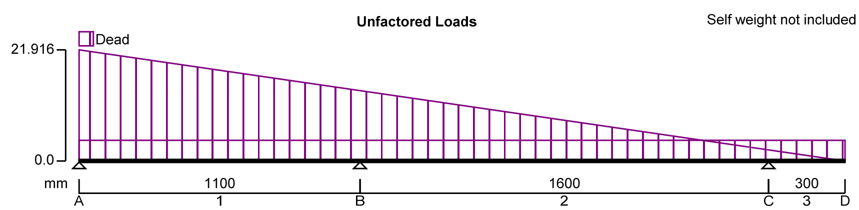


Length a a = **2.700** m

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

Span 3 Length = **300** mm Cross-sectional area = **17806** mm^2 Moment of inertia = **269.×10^3** mm^4

LOADING DETAILS

Beam Loads:

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

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

LOAD COMBINATIONS

Load combination 1

Span 1 1×Dead

Span 2 1×Dead

Span 3 1×Dead

CONTINUOUS BEAM ANALYSIS - RESULTS

Support Reactions - Combination Summary

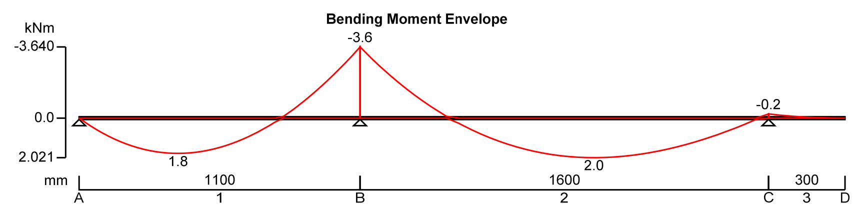
Support A	Max react = **-9.5** kN	Min react = **-9.5** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support B	Max react = **-28.0** kN	Min react = **-28.0** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support C	Max react = **-7.5** kN	Min react = **-7.5** kN	Max mom = **0.0** kNm	Min mom = **0.0**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

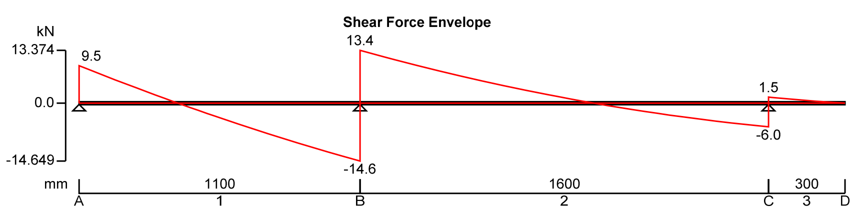
Beam Max/Min results - Combination Summary

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

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

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





Number of sheets Nos = 2

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

CROFT STRUCTURAL ENGINEERS

SRU 4.0

Safe working loads for Acrow Props — loads given in kN

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

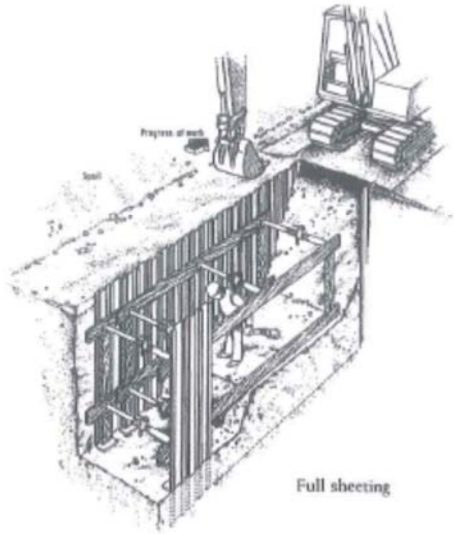
Shear V = (14.6kN + 13.4kN) /2 = 14.000kN

Any Acro Prop is accetpable

Sheeting requirements

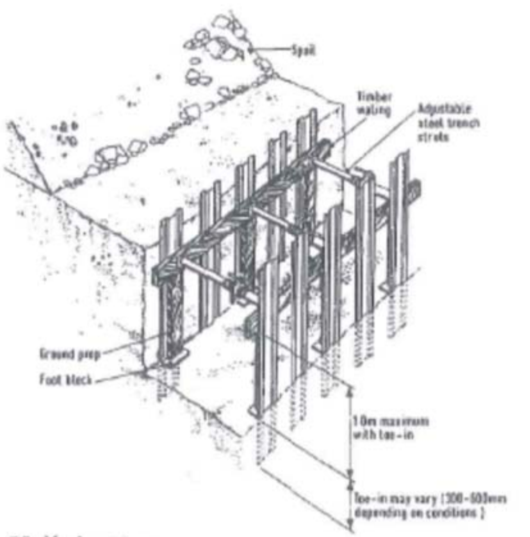
Ground Type	Trench Depth, D: less than 1.2m[1]	1.2 to 3m	3 to 4.5m	4.5 to 6m
Sands and gravels Silt Soft Clay High compressibility Peat	Close, ½, ¼, ⅛ or nil	Close	Close	Close
Firm/stiff Clay Low compressibility Peat	¼, ⅛ or nil	½ or ¼	½ or ¼	Close or ½
Rock[2]	From ½ for incompetent rock to nil for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

Sheeting requirements



Full sheeting

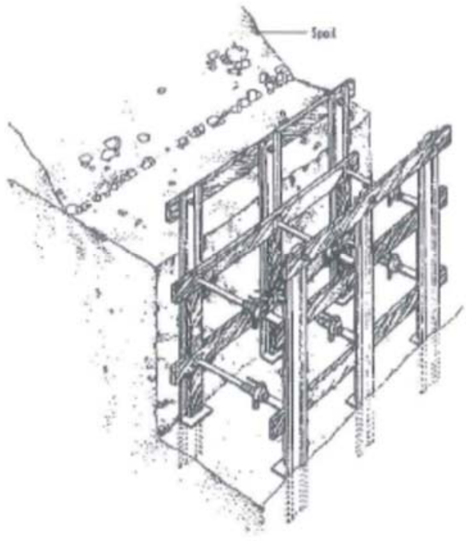
Sheeting requirements



Half sheeting
shown for 1.5 m deep trench

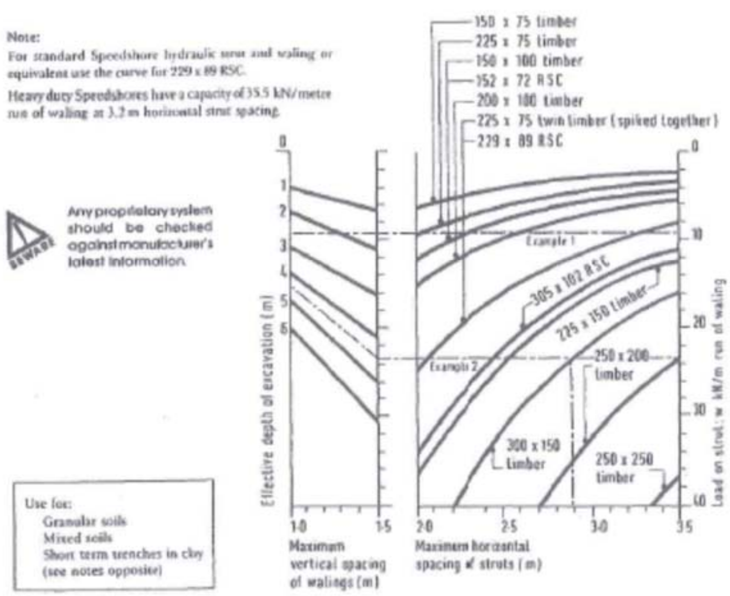
CROFT STRUCTURAL ENGINEERS

Sheeting requirements



Quarter sheeting

Design to CIRIA 97

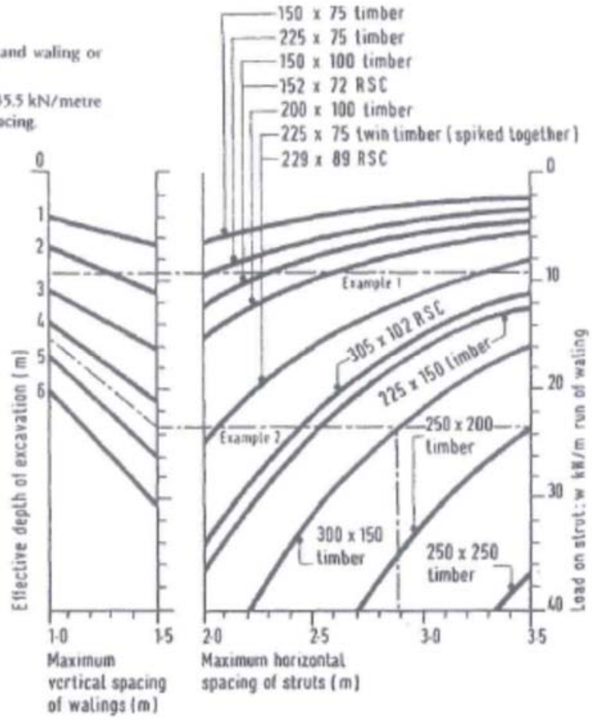


CROFT STRUCTURAL ENGINEERS

Note:
For standard Speedshore hydraulic strut and waling or equivalent use the curve for 229 x 89 RSC.
Heavy duty Speedshores have a capacity of 35.5 kN/metre run of waling at 3.2 m horizontal strut spacing.

BEWARE

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



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

Appendix D

Soil Investigation Report

Ground and Water Limited
Unit 2, The Long Barn,
Norton Farm,
Selborne Road,
Alton
GU34 3NB

Tel: 0333 600 1221
E-mail: enquiries@groundandwater.co.uk/
francis.williams@groundandwater.co.uk

Stage 1 Screening Report – Camden Geological, hydrogeological and hydrological study: Guidance for subterranean development

CLIENT	Advantage Basements & Cellar Company Limited c/o Croft Structural Engineers Limited
SITE ADDRESS	106 Savenake Road, London, NW3 2JR
REPORT REFERENCE	GWPR1123/GIR/January 2015

SURFACE FLOW AND FLOODING SCREENING FLOWCHART:

1. Is the site within the catchment of the pond chains on Hamsptead Heath?	No.
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?	Client to confirm. Percentage of hardstanding not considered likely to increase significantly.
3. Will the proposed basement development result in a change in the proportion of hard surfaced/paved external areas?	Client to confirm. Percentage of hardstanding not considered likely to increase significantly.
4. Will the proposed basement result in changes to the profile of the inflows (instantaneous and long-term) of surface water being received by adjacent properties or downsteam watercourses?	No.
5. Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream watercourses.	No.

Prepared By:	Verified By:
	F.T. Williams
Roger Foord BA (Hons) MSc DIC FGS MSoBRA	Francis Williams M.Geol. (Hons) FGS CEnv AGS MSoBRA

ground&water
geotechnical and environmental consultants

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CLIENT	Advantage Basements & Cellar Company Limited c/o Croft Structural Engineers Limited
SITE ADDRESS	106 Savenake Road, London, NW3 2JR
REPORT REFERENCE	GWPR1123/GIR/January 2015

SUBTERRANEAN (GROUNDWATER) FLOW:

1a. Is the site directly above an aquifer?	No. Site overlies Unproductive Strata of the London Clay Formation.
1b. Will the proposed basement extend beneath the water table surface?	No. However perched water migrating through London Clay Formation and Made Ground likely.
2. Is the site within 100m of a watercourse, well (used/disused) or potential spring line?	No.
3. Is the site within the catchment of the pond chains on Hampstead Heath?	No.
4. Will the proposed basement development result in a change in the area of hard surfaces/paved areas?	Client to confirm. Percentage of hardstanding not considered likely to increase significantly
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)?	Client to confirm. Percentage of hardstanding not considered likely to increase significantly
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 any local pond (not just the pond chains on Hampstead Heath) or spring line?	No.

Prepared By:	Verified By:
	F. T. Williams
Roger Foord BA (Hons) MSc DIC FGS MSoBRA	Francis Williams M.Geol. (Hons) FGS CEnv AGS MSoBRA

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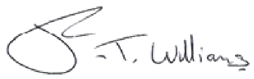
Stage 1 Screening Report – Camden Geological, hydrogeological and hydrological study: Guidance for subterranean development

CLIENT	Advantage Basements & Cellar Company Limited c/o Croft Structural Engineers Limited
SITE ADDRESS	106 Savenake Road, London, NW3 2JR
REPORT REFERENCE	GWPR1123/GIR/January 2015

SLOPE STABILITY SCREENING CHART:

1. Does the existing site include slopes, natural or manmade greater than 7°?	No.
2. Will the proposed re-profiling of landscaping at the site change slopes at the boundary to more than 7°?	No.
3. Does the development neighbour land, including railway cuttings and the like, with a slope greater than 7°?	No.
4. Is the site within a wide hillside setting in which the general slope is greater than 7°?	No.
5. Is the London Clay the shallowest strata at the site?	Yes – Scoping and mitigation measures required.
6. Will any tree/s be felled as part of the proposed development and/or are any works proposed within any tree protection zones where trees are to be retained.	No.
7. Is there a history of seasonal shrink-swell subsidence in the local area, and/or evidence of such effects at the site?	No. Shrink-swell likely in London Clay Formation. Scoping and mitigation measures required.
8. Is the site within 100m of a watercourse or a potential spring line?	No.
9. Is the site within an area of previously worked ground?	No.
10. Is the site within an aquifer? Is so, will the proposed basement extent beneath the water table such that dewatering may be required during construction?	No. Site overlies Unproductive Strata of the London Clay Formation.
11. Is the site within 50m of the Hampstead Heath Ponds?	No.
12. Is the site within 5m of a highway or pedestrian right of way?	Yes. Scoping and mitigation measures required.

13. Will the proposed basement significantly increase differential depths of foundations relative to neighbouring properties?	Yes. Scoping and mitigation measures required.
14. Is the site over (or within the exclusion zone of) any tunnels, e.g. railway lines?	No.

Prepared By:	Verified By:
	
Roger Foord BA (Hons) MSc DIC FGS MSoBRA	Francis Williams M.Geol. (Hons) FGS CEnv AGS MSoBRA

GROUND INVESTIGATION REPORT

for the site at

106 SAVENAKE ROAD, LONDON NW3 2JR

on behalf of

ADVANTAGE BASEMENT & CELLAR COMPANY LTD
C/O CROFT STRUCTURAL ENGINEERS

Report Reference: GWPR1123/GIR/JANUARY 2015		Status: FINAL
Issue:	Prepared By:	Verified By:
V1.01 JANUARY 2015		
	Mathias Gabrat BEng MSc. Geotechnical/Geo-environmental Engineer	Francis Williams M.Geol. (Hons) FGS CEnv AGS MSoBRA Director
File Reference: Ground and Water/Project Files/ GWPR1123 106 Savenake Road, London		

CONTENTS

FIGURES

APPENDICES

1.0 INTRODUCTION

1.1 General

Ground and Water Limited were instructed by the Advantage Basement and Cellar Company Ltd, c/o Croft Structural Engineers Limited, on the 11th December 2014 to undertake a Ground Investigation on a site at 106 Savenake Road, London NW3 2JR. The scope of the investigation was detailed within the Ground and Water Limited fee proposal ref: GWQ2310, dated 10th December 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.

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

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.

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 ~320m^2 in area and orientated in a north-east to south-west direction. The site was located ~60m east/south-east of Savenake Road's junction with Rona Road, with a railway line to the north-east. The site was located in the Gospal Oak area of the north-west London, on the south-eastern edge of Hampstead Heath. The site was located within the London Borough of Camden.

Topographic survey points on Savenake Road, to the south-west of the site, indicated a site level of between 44.4m - 47.7m Above Ordnance Datum (AOD).

The national grid reference for the centre of the site was approximately TQ 28124 85695. A site location plan is given within Figure 1. A plan showing the site area is given within Figure 2.

2.2 Site Description

The site was occupied by a semi-detached three storey brick built residential property with roof accommodation and paved front garden. Access to the rear garden was via a <0.80m wide pathway down the north-western side of the property, following two steps up to the front garden. The property had an existing cellar. An aerial view of the site is given within Figure 3.

Residential houses were noted to the west, south and the immediate east of the site. A school was noted further east with a railway line to the north. Gospel Oak Train Station was noted further north-east. The sites environs were noted to gently slope to the south-east.

2.3 Proposed Development

At the time of reporting, January 2015, the proposed development is understood to comprise the construction of a basement beneath footprint of the existing property, through deepening and extension of the existing cellar. The basement slab is anticipated to be formed at ~3.00 -3.50m below ground level (bgl). The basement area is estimated at ~10.0m by 6.20m bgl. A plan view of the proposed development can be seen in Figure 4 and a section view in Figure 5.

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.

2.4 Geology

The geology map of the British Geological Survey of Great Britain for the North London area (Sheet 256) revealed the site to be situated on the London Clay Formation.

The geology map and Figure 3 of the Camden Geological, Hydrogeological and Hydrological Study indicated that Worked Ground was noted ~200m south-east of the site.

London Clay Formation

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

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

A 2.0m deep BGS borehole ~60m south of the site revealed ~0.20m of Made Ground to overlie firm to stiff fissured silty clays. Groundwater was struck at 1.80m 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).

Figure 17 of the Camden Geological, Hydrogeological and Hydrological Study indicated that 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 revealed that the site was located on an **Unproductive Strata** comprising the bedrock of the London Clay Formation. No designation was given for any superficial deposits due to their likely absence.

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.

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.

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.

The nearest surface water feature to the site was a swimming pool located ~100m north-east of the site.

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

From analysis of hydrogeological and topographical maps groundwater was anticipated to be encountered at moderate depth (4-6m below existing ground level (bgl)) and it was considered that the groundwater was flowing in a south-easterly direction in alignment with the local topography.

Examination of the Environment Agency records showed that the site was not situated within a floodplain or flood warning area.

2.7 Radon

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

3.0 FIELDWORK

3.1 Scope of Works

Fieldwork was undertaken on the 13th December 2014 and comprised the drilling of one window sampler borehole (WS1) to a depth of 6.00m bgl and the hand excavation of two trial pit foundation exposures (TP/FE1 and TP/FE2).

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

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

Combined Bio-gas and Groundwater Monitoring Well Construction				
Trial Hole	Depth of Installation (m bgl)	Thickness of slotted piping with gravel filter pack (m)	Depth of plain piping with bentonite seal (m bgl)	Piping external diameter (mm)
WS1	5.00	4.00	1.00	19

The approximate locations of the trial holes can be seen within Figure 6.

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 did generally conformed to that anticipated from examination of the geology map. A capping of Made Ground was noted to overlie the London Clay Formation.

The ground conditions encountered during the investigation are described in this section. For more complete information about the Made Ground and 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 6.

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

Made Ground
London Clay Formation

Made Ground

Made Ground was encountered beneath a 0.20m thick patio slab and sub-base in BH1, and from ground level within the trial pit foundation exposures, for the full depth of each of the trial pit foundation exposures, a maximum depth of 0.80m bgl, and to a proved depth of 1.50m bgl in BH1. The Made Ground generally comprised a light to dark brown, locally black or orange brown, sandy gravelly clay. The sand was fine to coarse grained and the gravel was rare to abundant, fine to coarse, sub-rounded to angular flint, brick and concrete

Within BH1 from 0.20 – 0.60m bgl the Made Ground was described as a black slightly sandy, very clayey, gravel. The sand was fine to coarse grained and the gravel was abundant, fine to coarse, sub-rounded to angular brick, flint and concrete.

London Clay Formation

Soils described as London Clay Formation comprising brown, with blue grey mottling, slightly silty clay were encountered underlying the Made Ground in BH1. The deposits were proved to a depth of 6.00m 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 (Bedroom Wall)

Trial pit foundation exposure TP/FE1 (Bedroom Wall) was hand excavated adjacent to the existing ground floor bedroom, close to the centre of the north-western flank wall of the property. The exact location of the trial hole can be seen in Figure 6 and a section drawing of the foundation encountered during TP/FE1 (Bedroom Wall) can be seen in Figure 7.

The foundation layout encountered consisted of a brick wall to ground level. From ground level to a depth of 0.20m bgl a brick wall was noted. Two brick steps out (both 0.04m in width) from the property were then noted each comprising a single course of bricks (0.08m in thickness) were noted. The brick steps were noted to rest upon concrete footing which stepped out by 0.06m and was 0.28m thick. The foundation was noted to rest upon soils described as Made Ground comprising a light brown slightly gravelly clay at 0.64m bgl. The ground conditions encountered directly surrounding the foundation are shown in Figure 7.

TP/FE1 (Kitchen Wall)

Trial pit foundation exposure TP/FE1 (Kitchen Wall) was hand excavated adjacent to the existing kitchen, close to the centre of the north-western flank wall of the property. The exact location of the trial hole can be seen in Figure 6 and a section drawing of the foundation encountered during TP/FE1 (Kitchen Wall) can be seen in Figure 8.

The foundation layout encountered consisted of a brick wall to ground level. From ground level to a depth of 0.20m bgl a brick wall was noted. Two brick steps out (both 0.04m in width) from the property were then noted each comprising a single course of bricks (0.08m in thickness) were noted. The brick steps were noted to rest upon concrete footing which stepped out by 0.07m and was 0.27m thick. The foundation was noted to rest upon soils described as Made Ground comprising a light brown slightly gravelly clay at 0.63m bgl. The ground conditions encountered directly surrounding the foundation are shown in Figure 8.

TP/FE2

Trial pit foundation exposure TP/FE2 was hand excavated adjacent to the bay window at the front of the property. The exact location of the trial hole can be seen in Figure 6 and a section drawing of the foundation encountered during TP/FE1 (Kitchen Wall) can be seen in Figure 9.

The foundation layout encountered consisted of a brick wall to ground level. From ground level to a depth of 0.20m bgl a brick wall was noted. Three brick steps out (all 0.05m in width) from the property were then noted each comprising a single course of bricks (0.06 – 0.08m in thickness) were noted. The brick steps were noted to rest upon a concrete footing which stepped out by 0.04m and was 0.17m thick. The foundation was noted to rest upon soils described as Made Ground comprising a brown to orange brown slightly gravelly silty clay at 0.57m bgl. The ground conditions encountered directly surrounding the foundation are shown in Figure 9.

4.3 Roots Encountered

Fresh roots were noted to 2.00m bgl within BH1. No roots were observed within the trial pit foundation exposures.

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 encountered at 2.30m bgl within BH1 and likely represents perched groundwater within the London Clay Formation. The remaining trial holes were dry.

A return visit to monitor the groundwater level within the well installed in BH1 on the 9th January 2015 revealed a standing groundwater level of 1.65m bgl, with the well being recorded as 4.80m deep.

Given the seepage of groundwater in BH1 at 2.30m bgl, the standing water level in BH1 is likely to represent perched groundwater migrating and collecting within a standpipe installed within the impermeable soils of the London Clay Formation.

Changes in groundwater level occur for a number of reasons including seasonal effects and variations in drainage. Exact groundwater levels may only be determined through long term measurements from monitoring wells installed on-site. The investigation was undertaken in December 2014 and January 2015, when groundwater levels are close to 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 LABORATORY GEOTECHNICAL TESTING

5.1 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 Made Ground 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	3
Moisture Content Tests	BS1377:1990:Part 2:Clauses 3.2	3
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).	2

5.1.1 Atterberg Limit Tests

A précis of Atterberg Limit Tests undertaken on three samples of 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**
London Clay Formation	34 - 39	99 – 100	45.54 – 53.00	CV	Stiff	High	High

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

5.1.2 Comparison of Soil's Moisture Content with Index Properties

5.1.2.1 Liquidity Index Analyses

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

The results are tabulated overpage. 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**
London Clay Formation **BH1/2.00m bgl** (Brown CLAY with rare fine gravel)	34	29	45.54	0.11	Heavily Overconsolidated.
London Clay Formation **BH1/3.00m bgl** (Brown CLAY with rare fine gravel)	39	30	47.52	0.19	Heavily Overconsolidated.
London Clay Formation **BH1/4.00m bgl** (Brown CLAY with blue grey veins and scattered selenite)	37	28	53.00	0.17	Heavily Overconsolidated.

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

5.1.2.2 Liquid Limit

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

Moisture Content vs. Liquid Limit				
Strata/Trial Hole/Depth/Soil Description	**Moisture Content (MC) (%)**	**Liquid Limit (LL) (%)**	**40% Liquid Limit (LL)**	**Result**
London Clay Formation **BH1/2.00m bgl** (Brown CLAY with rare fine gravel)	34	75	30.0	MC > 0.4 x LL (No significant moisture deficit)
London Clay Formation **BH1/3.00m bgl** (Brown CLAY with rare fine gravel)	39	78	31.2	MC > 0.4 x LL (No significant moisture deficit)
London Clay Formation **BH1/4.00m bgl** (Brown CLAY with blue grey veins and scattered selenite)	37	81	32.4	MC > 0.4 x LL (No significant moisture deficit)

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

5.1.3 Moisture Content Profiling

A moisture content versus depth plot for BH1 can be seen within Figure 10. Figure 10 showed natural minor variations in moisture content with depth and no evidence for a possible moisture deficit.

5.1.4 BRE Special Digest 1

In accordance with BRE Special Digest 1 ‘Concrete in Aggressive Ground’ (BRE, 2005) one sample of Made Ground (BH1/1.50m bgl) and one sample of the London Clay Formation

(BH1/5.50m bgl) were scheduled for laboratory analysis to determine parameters for concrete specification.

Summary of Results of BRE Special Digest Testing			
Determinand	**Unit**	**Minimum**	**Maximum**
pH	-	7.1	7.3
Ammonium as NH_4	mg/kg	5.1	6.4
Sulphur	mg/kg	<200	2639
Chloride (water soluble)	mg/kg	10	55
Magnesium (water soluble)	g/l	0.0220	0.1580
Nitrate (water soluble)	mg/kg	12	27
Sulphate (water soluble)	g/l	0.10	1.89
Sulphate (total)	mg/kg	496	7877

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 proved to a depth of 1.50m bgl within BH1. The base of the Made Ground was not proved within TP/FE1 – TP/FE2 due to their termination at shallow depth (0.80m bgl).

  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 London Clay Formation comprising brown, with blue grey mottling, slightly silty clay were encountered underlying the Made Ground in BH1. The deposits were proved to a depth of 6.00m bgl.

  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 deposits to be stiff. Geotechnical analyses revealed the soils to be heavily overconsolidated with no moisture deficit.

  The soils of the London Clay Formation are heavily overconsolidated cohesive soils and are therefore likely to be a suitable stratum bearing stratum for the proposed development. 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.

- Fresh roots were noted to 2.00m bgl within BH1. No roots were observed within the trial pit foundation exposures.

- A groundwater seepage was encountered at 2.30m bgl within BH1 and likely represents perched groundwater within the London Clay Formation. The remaining trial holes were dry. A return visit to monitor the groundwater level within the well installed in BH1 on the 9th January 2015 revealed a standing groundwater level of 1.65m bgl, with the well being recorded as 4.80m deep.

  It was considered likely that the groundwater levels noted represented perched water migrating through the London Clay Formation and ponding within the well due to the cohesive, impermeable, nature of the deposits.

6.2 Basement Foundations

At the time of reporting, January 2015, the proposed development is understood to comprise the construction of a basement beneath footprint of the existing property, through deepening and extension of the existing cellar. The basement slab is anticipated to be formed at ~3.00 -3.50m below ground level (bgl). The basement area is estimated at ~10.0m by 6.20m bgl. A plan view of the proposed development can be seen in Figure 4 and a section view in Figure 5.

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.

Fresh roots were noted to 2.00m bgl within BH1. No roots were observed within the trial pit foundation exposures.

A foundation depth of ~3.00 – 3.50m bgl will be beyond the depth of Made Ground and root penetration noted within the investigation.

It is considered likely the proposed basement will be constructed with load bearing concrete retaining walls with semi-ground bearing concrete floors. Foundations constructed on the cohesive soils of the London Clay Formation at 3.0 – 3.5m bgl could be designed based on a presumed allowable bearing capacity of 100 - 125kN/m^2. This is based on the trial hole records, a 5m long by 1m wide foundation, a maximum settlement of 25mm and reference to BS8485.

A bearing capacity of less than 60kN/m^2 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.

A groundwater seepage was encountered at 2.30m bgl within BH1 and likely represents perched groundwater within the London Clay Formation. The remaining trial holes were dry. A return visit to monitor the groundwater level within the well installed in BH1 on the 9th January 2015 revealed a standing groundwater level of 1.65m bgl, with the well being recorded as 4.80m deep.

Given the seepage of groundwater in BH1 at 2.30m bgl, the standing water level in BH1 is likely to represent perched groundwater migrating and collecting within a standpipe installed within the impermeable soils of the London Clay Formation.

Therefore perched water is likely to be encountered during the construction of the basement. Dewatering is likely to 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

Given the ground conditions encountered, a piled foundation scheme was considered unlikely to be required at this site.

6.4 Basement Excavations & Stability

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

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

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

Based on the ground conditions encountered within the boreholes the following parameters could be used in the design of retaining walls. These have been designed based on the 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
London Clay Formation	~15	0	24	0.42	2.37

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

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

6.5 Hydrogeological Effects

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

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

A groundwater seepage was encountered at 2.30m bgl within BH1 and likely represents perched groundwater within the London Clay Formation. The remaining trial holes were dry. A return visit to monitor the groundwater level within the well installed in BH1 on the 9th January 2015 revealed a standing groundwater level of 1.65m bgl, with the well being recorded as 4.80m deep.

Given the seepage of groundwater in BH1 at 2.30m bgl and the standing water level in BH1 is likely to represent perched groundwater migrating and collecting within a standpipe installed within the impermeable soils of the London Clay Formation.

Based on the above it is considered likely that perched water will be encountered during basement construction, but the basement will not be constructed below the groundwater table. In relation to the basement, once constructed, additional drainage should be considered as 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 Made Ground and London Clay Formation, from both the geotechnical and chemical laboratory testing, fell into Class DS-1, DS-3 and DS-4 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-4. For the classification given, the "mobile" and "natural" case was adopted given the presence of the perched water within the cohesive London Clay Formation and use of the site. The sulphate concentration in the samples ranged from 100-3250mg/l with a pH range of 7.1-7.3. The total sulphate concentration ranged from 0.05 - 0.79%.

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

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

6.7 Surface Water Disposal

Infiltration tests were beyond the scope of the investigation.

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

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

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

6.8 Discovery Strategy

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

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

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

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

6.9 Waste Disposal

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

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

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

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

Once 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.

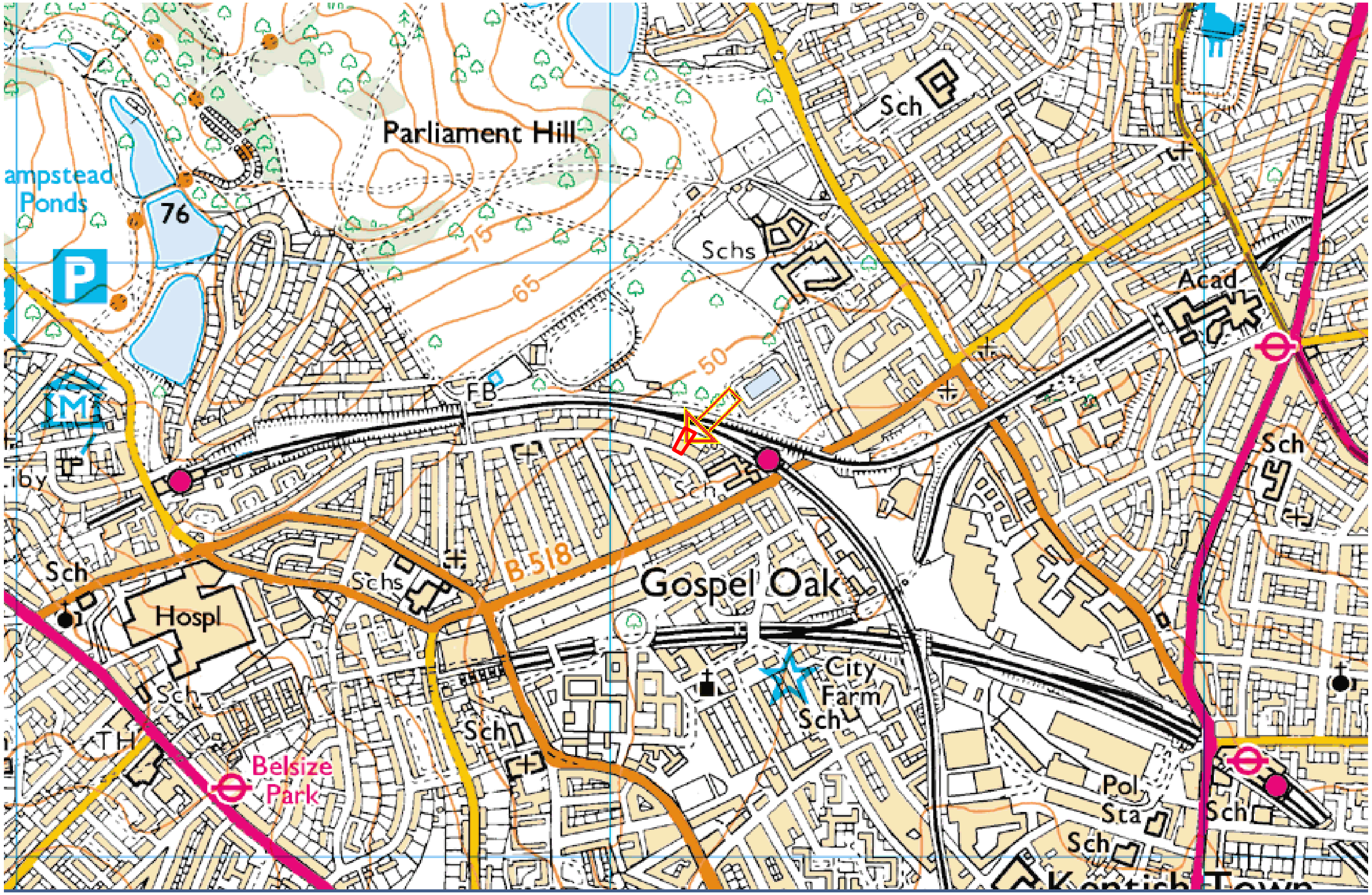
6.11 Duty of Care

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.

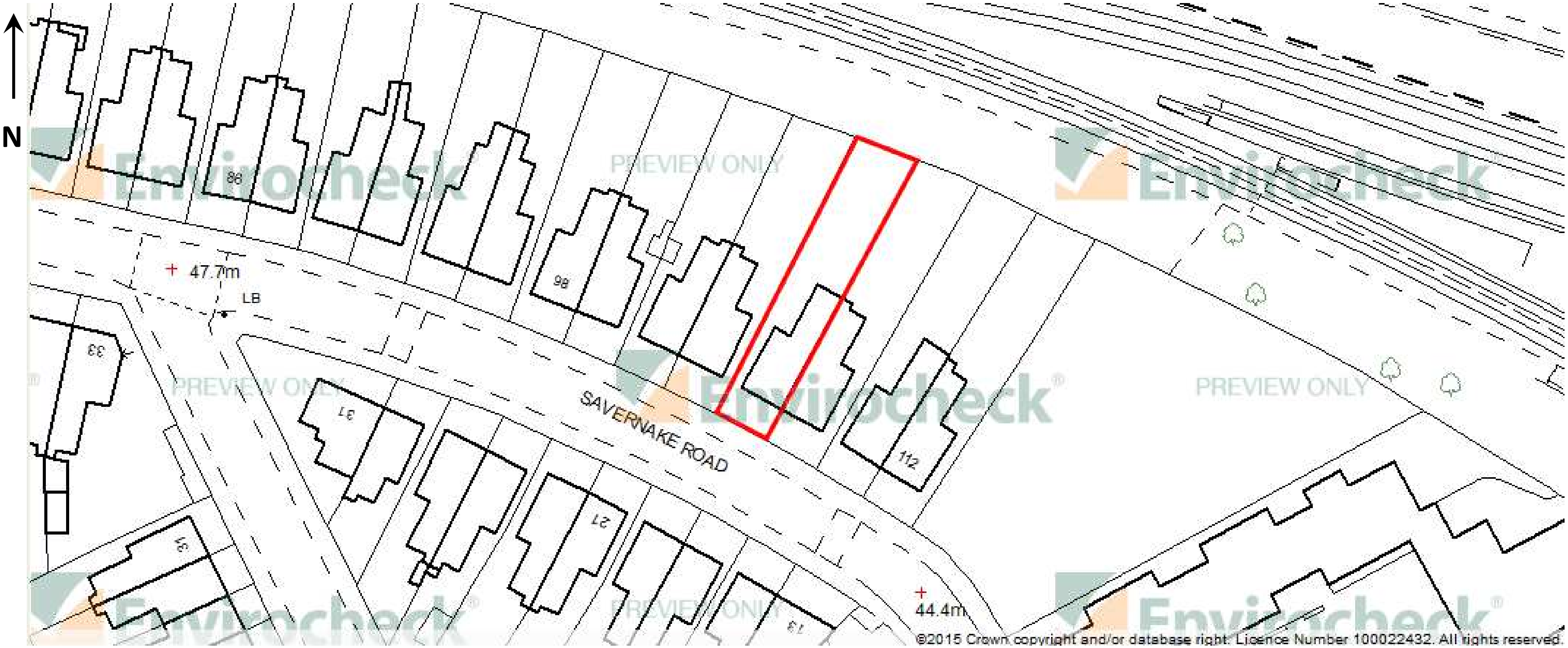
N



APPROXIMATE SITE BOUNDARY

NOT TO SCALE

Project: 106 Savenake Road, London NW3 2JR		**Figure 1** ground&water
Client: Advantage Basements & Cellar Company Limited	**Date:** January 2015	
Site Location Plan	**Ref:** GWPR1123	



APPROXIMATE SITE BOUNDARY

NOT TO SCALE

Project: 106 Savenake Road, London NW3 2JR		**Figure 2**
Client: Advantage Basement & Cellar Company Limited	**Date:** January 2015	ground&water
Site Development Area	**Ref:** GWPR1123	

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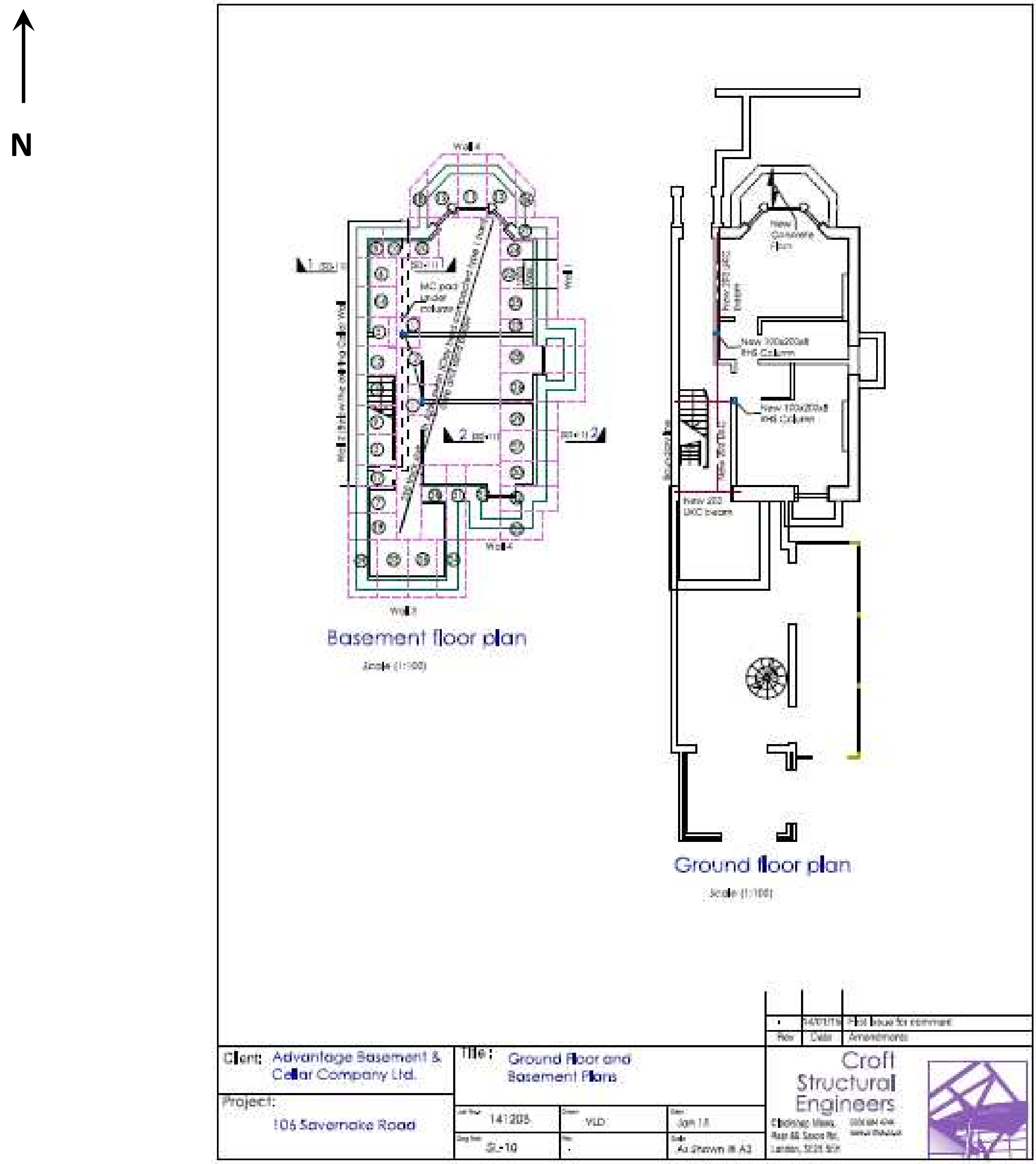
APPROXIMATE SITE BOUNDARY

NOT TO SCALE

Project: 106 Savenake Road, London NW3 2JR		Figure 3
Client: Advantage Basement & Cellar Company Limited	Date: January 2015	ground&water
Aerial View of the Site	Ref: GWPR1123	

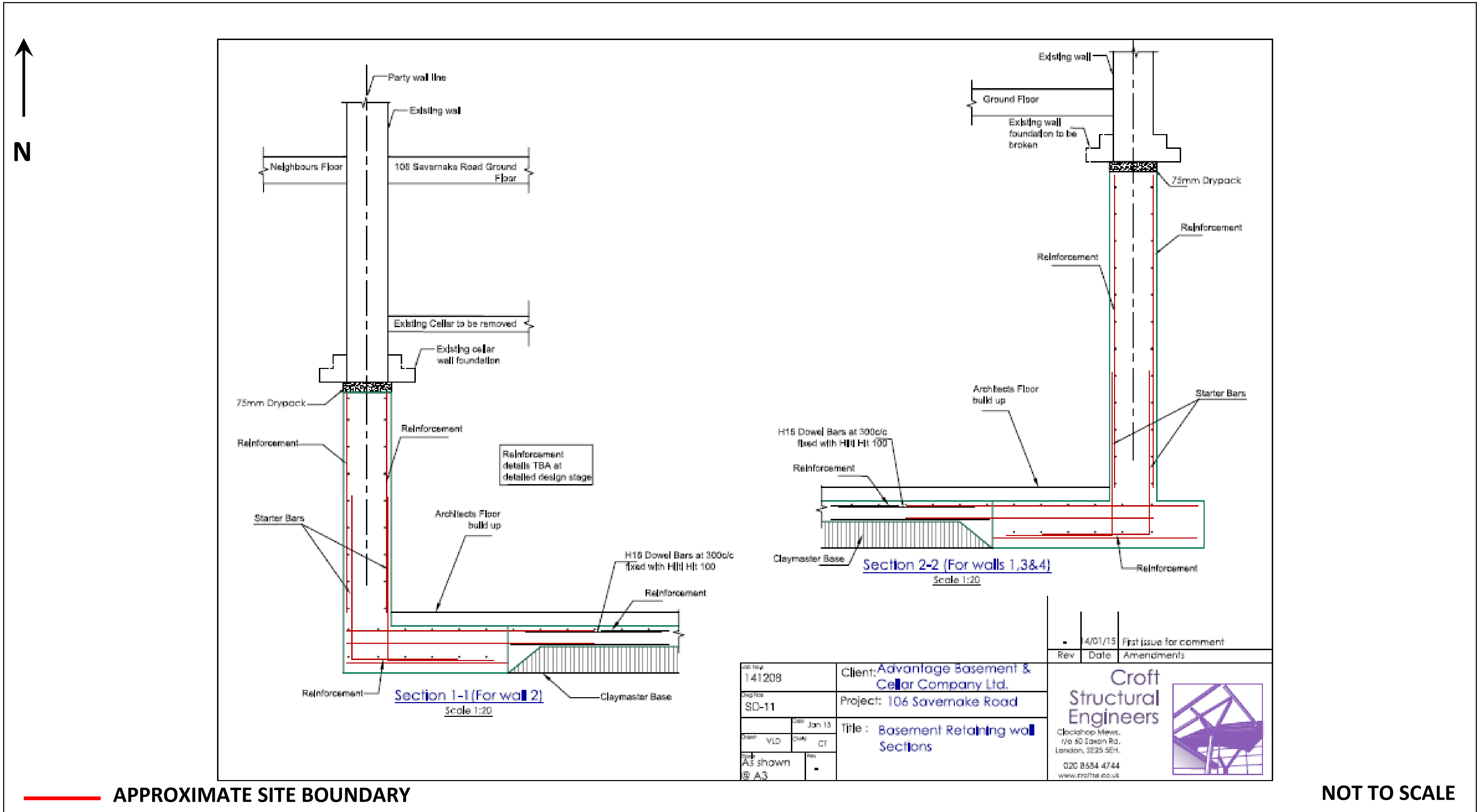


APPROXIMATE SITE BOUNDARY

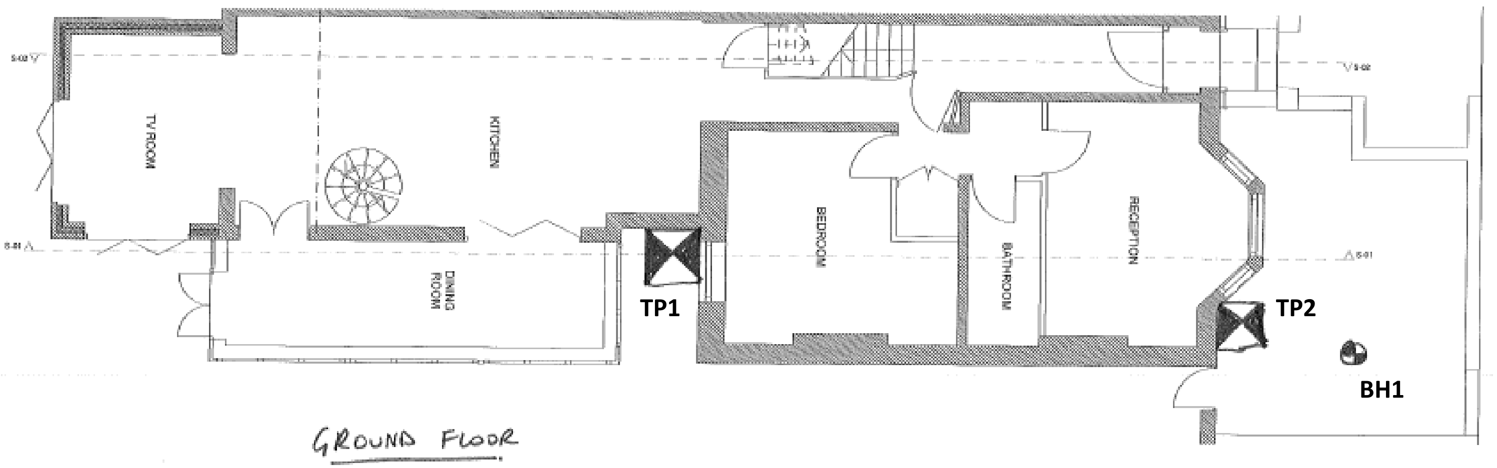
NOT TO SCALE

Project: 106 Savenake Road, London NW3 2JR	**Figure 4**
Client: Advantage Basement & Cellar Company Limited	**Date:** January 2015
Proposed Development – Plan View	**Ref:** GWPR1123

ground&water

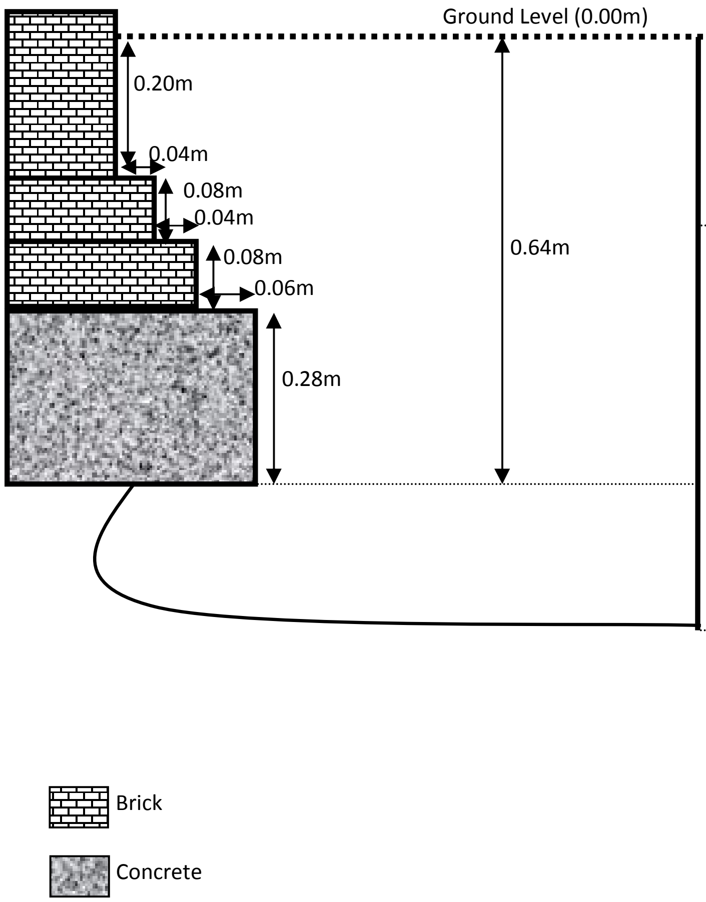


Project: 106 Savenake Road, London NW3 2JR	**Figure 5**
Client: Advantage Basement & Cellar Company Limited — **Date:** January 2015	ground&water
Proposed Development – Section View — **Ref:** GWPR1123	



NOT TO SCALE

Project: 106 Savenake Road, London NW3 2JR		**Figure 6**
Client: Advantage Basements & Cellar Company Limited	**Date:** January 2015	ground&water
Trial Hole Location Plan	**Ref:** GWPR1123	

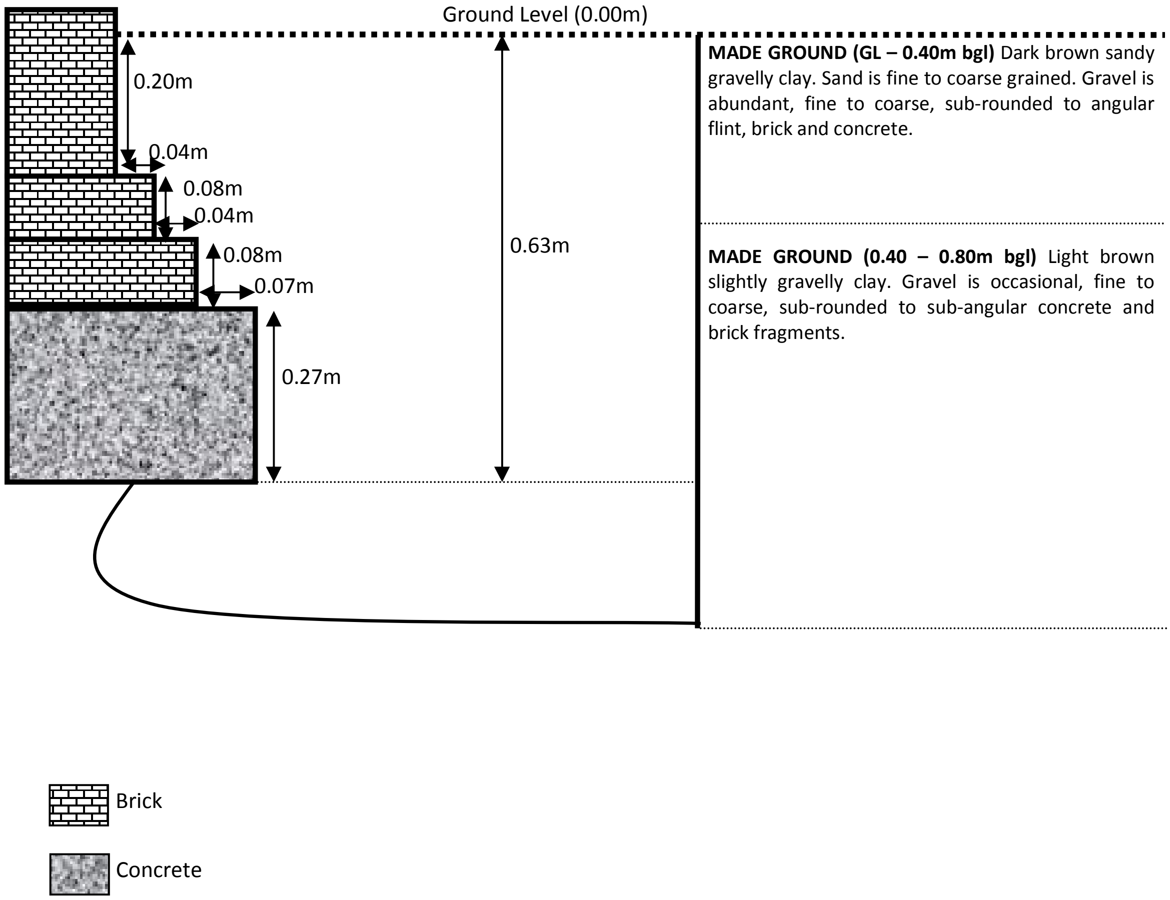


MADE GROUND (GL – 0.40m bgl) Dark brown sandy gravelly clay. Sand is fine to coarse grained. Gravel is abundant, fine to coarse, sub-rounded to angular flint, brick and concrete.

MADE GROUND (0.40 – 0.80m bgl) Light brown slightly gravelly clay. Gravel is occasional, fine to coarse, sub-rounded to sub-angular concrete and brick fragments.

NOTE: NOT TO SCALE

Project: **106 Savenake Road London NW3 2JR**		**Figure 6** ground&water
Client: **Advantage Basement & Cellar Company Limited**	**Date:** **January 2015**	
Section Drawing: Foundation Exposure TP/FE1A Bedroom Wall	**Ref:** **GWPR1123**	



NOTE: NOT TO SCALE

Project: 106 Savenake Road London NW3 2JR

Client: Advantage Basement & Cellar Company Limited

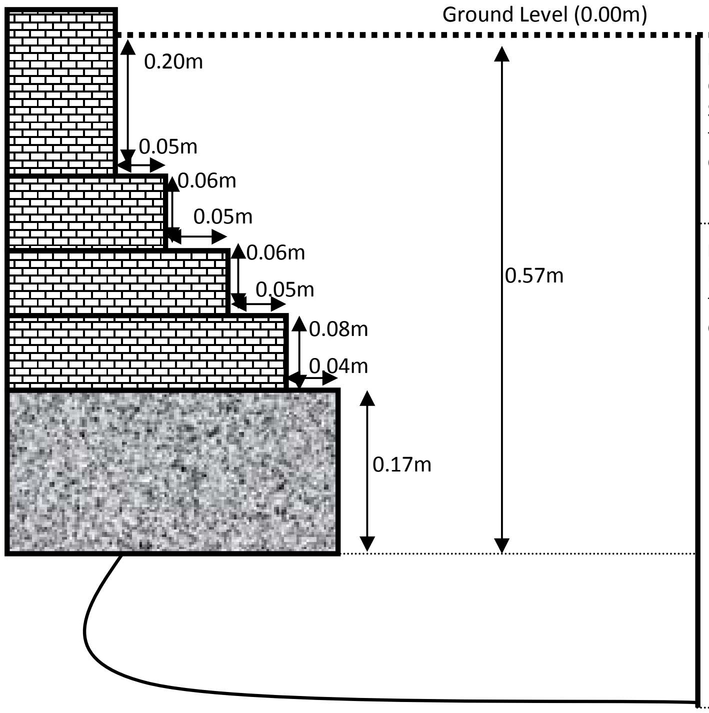
Date: January 2015

Section Drawing: Foundation Exposure TP/FE1B Kitchen Wall

Ref: GWPR1123

Figure 8

ground&water



MADE GROUND (GL – 0.40m bgl) Light brown to dark brown, locally black, slightly sandy gravelly clay. Sand is fine to coarse grained. Gravel is occasional, fine to coarse, sub-rounded to angular brick and concrete.

MADE GROUND (0.40 – 0.80m bgl) Brown to orange brown slightly gravelly silty clay. Gravel is rare, fine to medium, sub-rounded to sub-angular brick and concrete.

Brick

Concrete

NOTE: NOT TO SCALE

Project: **106 Savenake Road London NW3 2JR**		**Figure 9** ground&water
Client: **Advantage Basements & Cellar Company Limited**	**Date:** **January 2015**	
Section Drawing: Foundation Exposure TP/FE2	**Ref:** **GWPR1123**	

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 106 Savenake Road, London NW3 2JR.

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: 106 Savenake Road,	Project No.: GWPR1123	Co-ords: -	Hole Type: WS
Location: London NW3 2JR		Level: -	Scale: 1:50
Client: Advantage Basements & Cellar Company Ltd		Dates: 13/12/2014	Logged By: DM

Samples & In Situ Testing

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

Depth (m)	Level (m AOD)	Stratum Description
0.20		PATIO SLAB AND SUB-BASE
0.60		MADE GROUND: Black slightly sandy very clayey gravel. Sand is fine to coarse grained. Gravel is abundant, fine to coarse, sub-rounded to angular brick, flint and concrete.
1.50		MADE GROUND: Brown slightly gravelly clay. Gravel is fine to medium, sub-rounded to sub-angular brick, concrete and flint.
6.00		LONDON CLAY FORMATION: Brown, with blue grey mottling, slightly silty CLAY.

End of Borehole at 6.00 m

Remarks: Roots noted to 2.00m bgl
Groundwater seepage at 2.30m bgl.

APPENDIX C
Geotechnical Laboratory Test Results

Project Name:	106 Savenake Road, London NW3 2JR		Samples Received:	18/12/2014
			Project Started:	19/12/2014
Client:	Ground and Water Ltd		Testing Started:	09/01/2015
Project No:	GWPR1123	Our job/report no: 18066	Date Reported:	12/01/2015

K4 SOILS

Borehole No:	Sample No:	Depth (m)	Description	Moisture content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing 0.425 mm (%)	Remarks
BH1	-	2.00	Brown CLAY with rare fine gravel	34	75	29	46	99	
BH1	-	2.50	Brown CLAY	35					
BH1	-	3.00	Brown CLAY with rare fine gravel	39	78	30	48	99	
BH1	-	4.00	Brown CLAY with blue grey veins and scattered selenite	37	81	28	53	100	
BH1	-	4.50	Brown CLAY with blue grey veins and scattered selenite	37					
BH1	-	5.00	Brown CLAY with blue grey veins and scattered selenite	36					

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: 12/01/2015

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

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

Project Name:	106 Savenake Road, London NW3 2JR		K4 SOILS
Client:	Ground and Water Ltd	Project no: GWPR1123	
		Our job no: 18066	

Borehole No:	Sample No:	Depth m	Description	pH	Sulphate content (g/l)
BH1	-	3.00	Brown CLAY with rare fine gravel	7.1	3.25

Date	Summary of Test Results	Checked and Approved
12/01/2015	BS 1377 : Part 3 :Clause 5 : 1990	Initials : kp
	Determination of sulphate content of soil and ground water : gravimetric method	

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: 14-27439

Site Reference: 106 Savenake Road, London NW3 2JR

Project / Job Ref: GWPR1123

Order No: None Supplied

Sample Receipt Date: 18/12/2014

Sample Scheduled Date: 18/12/2014

Report Issue Number: 1

Reporting Date: 24/12/2014

Authorised by:

Russell Jarvis
Director
On behalf of QTS Environmental Ltd

Authorised by:

Kevin Old
Director
On behalf of QTS Environmental Ltd

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

4480

Soil Analysis Certificate							
QTS Environmental Report No: 14-27439	**Date Sampled**	13/12/14	13/12/14				
Ground & Water Ltd	**Time Sampled**	None Supplied	None Supplied				
Site Reference: 106 Savenake Road, London NW3	**TP / BH No**	BH1	BH1				
Project / Job Ref: GWPR1123	**Additional Refs**	None Supplied	None Supplied				
Order No: None Supplied	**Depth (m)**	1.50	5.50				
Reporting Date: 24/12/2014	**QTSE Sample No**	129870	129871				

Determinand	**Unit**	**RL**	**Accreditation**					
pH	pH Units	N/a	**MCERTS**	7.1	7.3			
Total Sulphate as SO_4	mg/kg	< 200	NONE	496	7877			
W/S Sulphate as SO4 (2:1)	g/l	< 0.01	**MCERTS**	0.10	1.89			
Total Sulphur	mg/kg	< 200	NONE	< 200	2639			
Ammonium as NH_4	mg/kg	< 0.5	NONE	5.1	6.4			
W/S Chloride (2:1)	mg/kg	< 1	**MCERTS**	10	55			
Water Soluble Nitrate (2:1) as NO_3	mg/kg	< 3	**MCERTS**	27	12			
W/S Magnesium	g/l	< 0.0001	NONE	0.0220	0.1580			

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: 14-27439
Ground & Water Ltd
Site Reference: 106 Savenake Road, London NW3 2JR
Project / Job Ref: GWPR1123
Order No: None Supplied
Reporting Date: 24/12/2014

QTSE Sample No	TP / BH No	Additional Refs	Depth (m)	Moisture Content (%)	Sample Matrix Description
129870	BH1	None Supplied	1.50	21.2	Light brown clay with gravel
129871	BH1	None Supplied	5.50	19.6	Light brown clay with gravel

Moisture content is part of procedure E003 & is not an accredited test
Insufficient Sample I/S
Unsuitable Sample U/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 - Methodology & Miscellaneous Information
QTS Environmental Report No: 14-27439
Ground & Water Ltd
Site Reference: 106 Savenake Road, London NW3 2JR
Project / Job Ref: GWPR1123
Order No: None Supplied
Reporting Date: 24/12/2014

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