Appendix C

Ground Investigation & Basement Impact Assessment report

geotechnical and environmental consultants

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Basement Impact Assessment and Ground Investigation Report

The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX

On behalf of John Fitzpatrick c/o Momentum Structural Engineers

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EXECUTIVE SUMMARY	
Site Background	The site comprised a 635m2 rectangular shaped plot of land, with a south-east to north-west orientation, located along the north-western side of Lyndhurst Road. The site was located within Hampstead, a mainly residential suburb within the inner north-west London Borough of Camden. The existing property comprised a semi-detached, two-storey residential dwelling. Soft landscaped areas and hardstanding areas are noted to the front and rear of the property.
Proposed Development	Following the demolition of the existing property, the proposed development involves the construction of a lightweight property with a RC basement, whilst underpinning of existing adjacent walls will be undertaken. The basement is to require an excavation of ~2.80 – 3.20m of soil (from existing levels of 11.54 – 11.94m AOD), to the proposed basement formation level of 8.74m AOD, which allows a 200m basement slab to proposed basement floor level at 8.94m AOD. A external garden annex building will also be constructed at the rear extent of the site. No lower ground floor level was anticipated below the annex. The amount of hardstanding across the entire site was anticipated to increase.
Screened in Risks	• Perched Water and Groundwater • Seasonal Soil Moisture and Volume Change Potential • Pressure Induced Settlement and Heave • Retaining Wall Design • Instability During Excavation • Ground Movement and Nearby Assets • Sub-Surface Concrete in Aggressive Ground Conditions • Surface Water Flooding and Site Drainage • Groundwater Flooding and Flow • Sewer Flooding • Nearby London Underground Tunnel
Site Investigation	Site works were between 5th and 13th December 2023 and comprised the drilling of 3No. Windowless Sampler Boreholes (WS1 – WS3) to 5.00m bgl. In-situ strength testing (SPTs) was undertaken at 1.00m intervals. Combined groundwater and ground-gas monitoring standpipes were installed to 5.00m bgl within WS1 and WS2, with a response zone between 1.00 – 5.00m bgl. Additionally, the hand excavation of 5No. Foundation Exposures (TP1 – TP5) was undertaken.
Encountered Ground Conditions	The ground conditions encountered within the trial holes constructed on the site did generally conform to that anticipated from examination of the geology map. A capping of Made Ground was noted to overlie the Claygate Member before the London Clay Formation. No groundwater strikes were noted during the site investigation. It should be noted that groundwater strikes may have been obscured by the drilling processes. Groundwater monitoring was undertaken on two occasions to date, where standing water was noted at 4.80m bgl within WS1, whilst WS2 was dry.
Foundation Design	**Main Property** Foundations should be taken through any Topsoil/Made Ground before founding onto competent, moisture stable soils. Therefore, the proposed foundation depths at 8.74m AOD/2.26m bgl at WS1/3.46m bgl at WS2, and would be founded on the interface between the Claygate Member and London Clay Formation, both of which were considered suitable bearing stratums. Foundations at this depth can be designed based on a presumed allowable bearing capacity of 100kN/m^2. This is based on trial hole records, the results of in-situ testing, inspection of samples recovered, and referral to BS 8004:2015, Code of Practice for Foundations. **External Annex** Foundations should be taken through any Topsoil/Made Ground before founding onto competent, moisture stable soils. Given the underlying Claygate Member had volume change

	potential, fresh roots should also be bypassed by 300mm. Based on the maximum volume change potential found within the Claygate Member (high volume change potential), a absolute minimum foundation depth of 1.00m bgl was proposed. Therefore, the proposed foundation depth for the external annex should be at 1.00m bgl/11.80m AOD at WS3. Foundations at this depth can be designed based on a presumed allowable bearing capacity of 120kN/m^2. This might be limited by settlements, however due to basement, heave is predominant. This is based on trial hole records, the results of in-situ testing, inspection of samples recovered, and referral to BS 8004:2015, Code of Practice for Foundations.
Heave Assessment	A maximum amount of heave of 24.20mm was noted following the mass excavation of the basement void (Model 3), and was noted to be the maximum amount of heave during the construction phases. Once constructed, the maximum amount of heave increased from 16.80mm for short term conditions (Model 4), to 24.60.00mm for long term conditions (Model 5); therefore, the highest risk of movement will likely occur during the construction of the basement and later through long-term heave of the constructed basement. It should be noted that the effect of the demolition load is conservative and the amount of heave may be lower than calculated.
Ground Movement Assessment	All walls were assessed as having Category 0 (Negligible) or Category 1 (Very Slight) damage. Given the length of the roads, the deflection expected was not anticipated to cause damage; however, monitoring is recommended as good practice. The southern extreme of the Overground London Underground line is ~10.00m north of the northern boundary of the site, approximately 20.00m away from the northern extreme of the proposed basement. Ground movement relating to the underpinned basement is to extend 4 x underpin depth, which equates to 14.00m. As the tunnel is further away from the basement than 14.00m, no ground movement is to be expected at the location of the tunnel and therefore there will not be any affect.

1.0 INTRODUCTION

1.1 General

Ground and Water Limited were instructed by John Fitzpatrick c/o Momentum Structural Engineers on the 15th November 2023 to conduct a Basement Impact Assessment and Ground Investigation Report on the site referred to as The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX. The scope of the investigation was detailed within the Ground and Water Limited fee proposal (reference: GW-2378Rev3).

1.2 Aims of the Investigation

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

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

The proposed development includes a basement. A Basement Impact Assessment, including screening and detailed comment on surface water flooding/management or combined flooding (sourced from SFRA or similar sources) was part of the remit of the report. The requirements of the following reports were reviewed with respect to this project:

- Camden Local Plan, 2017, by Camden Council;
- The requirements of the Camden Planning Guidance: Basements (2018)
- The Preliminary Flood Risk Assessment, 2011, by Halcrow, Greater London Authority and Camden Council
- The Preliminary Flood Risk Assessment Addendum, 2017, by Camden Council
- The Strategic Flood Risk Assessment, 2014, by URS Infrastructure and Environment UK Ltd.
- The Surface Water Management Plan, 2011, by Halcrow, Greater London Authority and Camden Council
- Managing Flood Risk in Camden: The London Borough of Camden Flood Risk Management Strategy, by Camden Council.
- The Camden Geological, Hydrogeological and Hydrological Study: Guidance for Subterranean Development, 2010, by ARUP;
- The History of Lost Rivers in Camden: The Historical Study of the Kilburn and Tyburn, produced by UCL and LBC on 26th March 2010
- The History of the River Fleet, produced by The UCL River Fleet Restoration Team on 27th March 2009.

In addition, a Ground Movement Assessment for the impact of the proposed development on surrounding properties and assets was in the remit of the report.

A full scale Environmental Desk Study and Contamination Assessment including a gas risk assessment were not part of the remit of this report; however Included within the fee proposal was an allowance to undertake chemical laboratory testing on soil samples recovered from the site to enable recommendations for the safe redevelopment of the site and the protection of site workers, end-users and the public from any potential contamination identified.

The techniques adopted for the investigation were chosen considering the requirements of the client, anticipated ground conditions, 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.

1.4 Technical Glossary

Generic technical terms can be viewed within the glossary provided within Appendix B.

2.0 SITE SETTING

2.1 Site Location

The site comprised a 274m^2 rectangular shaped plot of land, with a south-east to north-west orientation, located along the north-western side of Lyndhurst Road. The site was located within Hampstead, a mainly residential suburb within the inner north-west London Borough of Camden. A Site Location Plan is provided within Figure 1.

2.2 Site Description

The existing property comprised a semi-detached, two-storey residential dwelling. Soft landscaped areas and hardstanding areas are noted to the front and rear of the property. A topographic survey of the site was undertaken and can be viewed within Figure 2. An aerial view of the site has also been provided within Figure 3.

The existing levels across the site can be viewed in the following table, as well as the adjoining property which was also scoped into the topographic survey (shown in Figure 2).

Summary of Existing Elevations		
Property	**Level**	**Existing Elevations**
The site (The Cottage, 10 Lyndhurst Road)	Front Garden Level of The Cottage, 10 Lyndhurst Road	~11.00 – 11.50m AOD in the vegetated area, lowering to ~10.90m AOD in hardstanding areas
	Rear Garden Level of The Cottage, 10 Lyndhurst Road	~11.90 – 13.00m AOD
Adjoining Property (10A – 10D Lyndhurst Road)	Front Garden Level of 10 Lyndhurst Road	~10.80 – 11.00m AOD
	Western Front Lightwell of 10 Lyndhurst Road	~8.80m AOD
	Eastern Front Lightwell of 10 Lyndhurst Road	~9.60m AOD
	Rear Lightwell of 10 Lyndhurst Road	~8.70 – 8.80m AOD
	Rear Garden Level of 10 Lyndhurst Road	~11.90 – 12.90m AOD

A existing plan view of the ground floor level can be viewed within Figure 4, with a plan view of the existing basement level within Figure 5. A section view of the existing property can be viewed within Figure 6.

The site was situated on a southerly facing slope, decreasing in elevation from Hampstead Heath in the north-east (~137.00m AOD), generally towards the River Thames to the south (~0.00m AOD). The site was not situated within an area where a natural or man-made slope of >7° was present. A contour map has been provided within Figure 7.

2.3 Historical Map Review

The site formed part of a larger undeveloped area at the earliest available map (1871), with Lyndhurst Road noted immediately south of the site and scattered properties noted within the site environs. By the 1896 historical map, a structure was noted across the site, relating to The Cottage 10 Lyndhurst Road. By the 1952 historical map, the London Underground tunnel was noted to the north of the site. No other significant changes were noted from the earliest map to current day.

Historical maps, obtained from GroundSure, can be viewed within Appendix C. It should be noted that the site boundary also includes the adjoining property, 10A – 10D Lyndhurst Road.

2.4 Subterranean Developments

The southern extreme of the Overground London Underground line is ~10.00m north of the northern boundary of the site, as shown by Figure 8, and approximately 20.00m from the closest part of the basement development. No other London Underground tunnels were noted within close proximity to the site. No railway cuttings were noted within a 250m radius of the site. The site is not in close proximity to any National Rail lines. The site was considered to be not sufficiently close to underground transport services, in order for these to affect the property and there are no approved proposals for any TfL services in the vicinity that would affect the development.

2.5 Nearby Assets

The properties along Lyndhurst Road were noted were mainly 3-to-4 storey, semi-detached and detached residential properties, with lower ground floor levels ~2.00m below the properties ground level. Properties to the west and north generally had ground floors at higher elevation than the site, whereas to the south and east, properties generally had ground floors at lower elevation.

Across Lyndhurst Road, numbers 19, 20 and 21, as well as their walls, gate piers and former lodge were categorised as Grade II listed buildings.

2.6 Proposed Development

Following the demolition of the existing property, the proposed development involves the construction of a lightweight property with a RC basement, whilst underpinning of existing adjacent walls will be undertaken. The basement is to require an excavation of ~2.80 – 3.20m of soil (from existing levels of 11.54 – 11.94m AOD), to the proposed basement formation level of 8.74m AOD, which allows a 200m basement slab to proposed basement floor level at 8.94m AOD.

A external garden annex building will also be constructed at the rear extent of the site. No lower ground floor level was anticipated below the annex.

The amount of hardstanding across the entire site was anticipated to increase.

The proposed development fell within Geotechnical Design Category 2 in accordance with Eurocode 7. Plan views of the proposed development can be viewed within Figures 9 and 10 with a cross-section of the proposed development provided within Figure 11.

2.7 Geology

The BGS Geological Map for the area revealed that the site was underlain by the Claygate Member, underlain by the London Clay Formation bedrock. No superficial deposits, outcrops of other bedrock deposits or areas of Made/Worked Ground were noted within close proximity of the site.

2.8 Hydrogeology and Hydrology

The DEFRA online maps indicated that the site was located on Secondary (A) Bedrock Aquifer associated with the Claygate Member, underlain by Unproductive Bedrock Strata associated with the London Clay Formation. No designation was given to superficial deposits due to their likely absence.

From analysis of hydrogeological and topographical maps, the groundwater table was anticipated to be encountered at shallow to moderate depth within the Claygate Member, capping the impermeable London Clay Formation. Perched water was also likely to be found within the Made Ground, especially after periods of intense or prolonged rainfall. It was considered that the groundwater was flowing southwards, towards the River Wey and in alignment with local topography.

No surface water features were noted within a 250m radius of the site. The nearest surface water feature was observed to be the Hampstead Heath Pond 1, noted ~620m north-west of the site, as shown by Figure 12. The Grand Union Canal was noted ~2.10km south-east of the site. The easterly flowing River Thames was noted ~6.10km south-east of the site. No old rivers were noted in close proximity to the site, as shown by Figure 13.

2.9 BGS Borehole Records

A BGS borehole record in similar geology ~190m north-east of the site (ref.: TQ28NE7) noted a 300mm thick layer of macadam to be underlain by clay for the remaining depth of the borehole (15.85m bgl) No groundwater observations were noted on the record.

Another BGS borehole record in similar geology ~320m north-west of the site (ref.: TQ28NE304), noted Made Ground to 2.13m bgl, underlain by London Clay to 109.73m bgl, before Lambeth Group (formerly Woolwich and Reading Beds), Thanet Sand and Upper Chalk soils. No groundwater observations were noted on the record.

2.10 Flooding

A summary of the risk of various flooding types has been summarised in the following table.

Summary of Flood Risk			
Type of Flooding	**Figure Reference**	**On-site Flood Risk**	**Maximum Nearby Flood Risk**
Rivers and Seas	Figure 14	Flood Zone 1	Flood Zone 1
Flood Defences	Figure 14	None	None
Reservoir	Figure 15	No risk	No risk
Surface Water Flooding	Figure 16	Very Low risk	Very Low to Low risk
Groundwater and Throughflow Flooding	Figure 17	Not in an area where there are permeable superficial deposits/increased susceptibility to shallow groundwater; however, four groundwater flooding incidents were noted nearby in similar geology.	
Sewer Flooding	Figures 18 and 19	The site was in a post code area where there were 0 records of internal sewer flood incidents and 1 record of external sewer flood incidents.	
Critical Drainage Areas/Local Flood Risk Areas	Figures 20 and 21	Within the Counter's Creek Catchment CDA. Not within any other CDAs or LFRAs.	

2.11 Radon

A review of the freely available UK Health Security Agency radon database, UK Radon, indicated that the site was located within a 1km grid square, where the maximum radon potential of <1% was recorded. The neighbouring 1km grid square was noted to have a maximum radon potential of <1%. Basic radon protection measures are required in areas where more than 3% of houses are at or above the Action Level. The site was in an area where a risk assessment was not required. As the site is a basement however, it is considered to be a vulnerable structure and upgrading waterproofing to include some radon protection is recommended.

2.12 Unexploded Ordnance Review

A review of the data available on www.zeticauxo.com/ revealed the site was located within the London high-risk area associated with unexploded ordnance (UXO). The London area is further separated into 25No. categories based on bombing densities, where green is indicated for areas having <10 bombs dropped per km^2 and red is indicated for areas having >150 bombs dropped per km^2. The site is situated within the red area, near the high risk side the spectrum.

It is recommended that a Preliminary UXO report is purchased for the site to better assess the UXO risk.

3.0 BASEMENT IMPACT ASSESSMENT

A scoping and screening assessment was undertaken for the proposed development based on the supplementary planning document (SPD) for the London Borough of Camden. This stage should identify any areas of concern and therefore focus efforts on further investigation.

3.1 Stage 1: Screening

The screening questions/fields for three distinct topics (surface water/flooding, groundwater, and stability) have been summarised within this section of the report.

Questions relating to surface water and flooding, as well as discussion and conclusions, can be viewed within the following table.

Surface Water and Flooding Screening Flowchart		
Question	**Discussion**	**Conclusion**
Question 1: Is the site within the catchment of the pond chains of Hampstead Heath?	The site was not located within the catchment of any of the pond chains of Hampstead Heath.	No further action required.
Question 2: As part of the of the proposed site drainage, will surface water flows be materially changed from the existing route?	The amount of hardstanding across the entire site was anticipated to increase; therefore, the route of surface water flows may be affected.	**Take forward to scoping.**
Question 3: Will the proposed basement development result in a change to the hard surfaces/paved external areas?	The amount of hardstanding across the entire site was anticipated to increase	**Take forward to scoping.**
Question 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?	The amount of hardstanding across the entire site was anticipated to increase; therefore, that surface water inflows may be affected.	**Take forward to scoping.**
Question 5: Will the proposed basement result in a change to the surface water being received by adjacent properties or downstream watercourses?	The amount of hardstanding across the entire site was anticipated to increase; therefore, the amount of surface water expected upstream and downstream at adjacent properties may change.	**Take forward to scoping.**
Question 6: Is the site in an area identified to have surface water flood risk according to either the Local Flood Risk Management Strategy or the Strategic Flood Risk Assessment or is it at risk from flooding, for example, because the basement is below the static water level of a nearby surface water feature?	The site fell within a Flood Zone 1, not benefitting from flood defences or flood storage areas. There was no risk of reservoir flooding on-site. The site and surrounding area was at very low risk of surface water flooding. As the site was underlain by a Secondary (A) Aquifer, underlain by Unproductive Strata, there was considered to be a risk of groundwater flooding; however, this is considered to be low as a result of the anticipated cohesive nature of the soils.	**Take forward to scoping.**

Questions relating to groundwater, as well as discussion and conclusions, can be viewed within the following table.

Subterranean (Groundwater) Screening Flowchart		
Question	**Discussion**	**Conclusion**
Question 1a: Is the site located directly above an aquifer?	The site was located within a Secondary (A) Aquifer comprising the Claygate Member, underlain by Unproductive Strata associated with the London Clay Formation. The strata, however, are expected to mainly include cohesive soils.	**Take forward to scoping.**
Question 1b: Will the proposed basement extend beneath the water table surface?	It was anticipated that groundwater was perched on top of the London Clay Formation, within the Claygate Member. Perched water was also likely to be found within the Made Ground and underlying strata where silty/sandy/gravelly bands are noted, especially after periods of intense or prolonged rainfall. Based on the proposed basement depth, it was possible that some water may be encountered.	**Take forward to scoping.**
Question 2: Is the site within 100 m of a watercourse, well (used/disused) or potential spring line?	No surface water features or watercourses were noted within a 250m radius of the site.	No further action required.
Question 3: Is the site within the catchment of the pond chains on Hampstead Heath?	The site was not located within the catchment of any of the pond chains of Hampstead Heath.	No further action required.
Question 4: Will the proposed basement development result in a change in the proportion of hard surfaced/paved areas?	The amount of hardstanding across the entire site was anticipated to increase	**Take forward to scoping.**
Question 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)?	The amount of hardstanding across the entire site was anticipated to increase; therefore, the amount of surface water draining into the ground may be subject to change.	**Take forward to scoping.**
Question 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 or spring line?	No surface water features were noted within a 250m radius of the site. The nearest surface water feature was observed to be the Hampstead Heath Pond 1, noted ~620m north-west of the site. The Grand Union Canal was noted ~2.10km south-east of the site. The easterly flowing River Thames was noted ~6.10km south-east of the site. No old rivers were noted in close proximity to the site.	No further action required.

Questions relating to ground stability, as well as discussion and conclusions, can be viewed within the following table.

Stability Screening Flowchart		
Question	**Discussion**	**Conclusion**
Question 1: Does the existing site include slopes, natural or manmade, greater than 7°?	The site was not situated within an area where a natural or man-made slope of >7° was present.	No further action required
Question 2: Will the proposed re-profiling of landscaping at the site change slopes at the property boundary to more than 7°?	Following relevelling of the site to allow for the proposed development, no slopes greater than 7° were likely.	No further action required
Question 3: Does the development neighbour land, including railway cuttings and the like, with a slope greater than 7°?	The site was situated on a southerly facing slope, decreasing in elevation from Hampstead Heath in the north-east (~137.00m AOD), generally towards the River Thames to the south (~0.00m AOD). The site was not situated within an area where a natural or man-made slope of >7° was present.	No further action required

Stability Screening Flowchart		
Question	**Discussion**	**Conclusion**
Question 4: Is the site within a wider hillside setting in which the general slope is greater than 7°?	The site was situated on a southerly facing slope, decreasing in elevation from Hampstead Heath in the north-east (~137.00m AOD), generally towards the River Thames to the south (~0.00m AOD). The site was not situated within an area where a natural or man-made slope of >7° was present.	No further action required
Question 5: Is the London Clay the shallowest strata at the site?	No, the site was located on the Claygate Member of the London Clay Formation underlain by the London Clay Formation. However, stability issues may arise through excavation of both strata.	**Take forward to scoping.**
Question 6: Will any trees 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?	At the time of reporting, January 2024, it was assumed that no trees were to be removed as part of the proposed development.	No further action required.
Question 7: Is there a history of seasonal shrink-swell subsidence in the local area and/or evidence of such effects at the site?	Anticipated geology considered the presence of mainly cohesive soils of Claygate Member, underlain by the London Clay Formation, both of which are likely to have medium to high volume change potential, and therefore would be subject to subsidence due to shrinkage-swelling. No evidence of potential movement/damage was known by Ground and Water Limited or noticed on site.	**Take forward to scoping.**
Question 8: Is the site within 100 m of a watercourse or potential spring line?	No surface water features or watercourses were noted within a 250m radius of the site.	Take forward to scoping.
Question 9: Is the site within an area of previously worked ground?	The site was not noted to be within a known area of Made Ground. Some Made Ground is expected as a result of the construction activities on site.	No further action required.
Question 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?	A study of the aquifer maps on the Environment Agency website revealed the site to be located within a Secondary (A) Aquifer comprising the Claygate Member, underlain by Unproductive Strata associated with the London Clay Formation. The soils of the Claygate Member, however, are expected to mainly comprise cohesive soils. It was anticipated that groundwater was perched on top of the London Clay Formation, within the Claygate Member. Perched water was also likely to be found within the Made Ground and underlying strata where silty/sandy/gravelly bands are noted, especially after periods of intense or prolonged rainfall. It was possible that the basement may encounter some perched water, however, this was not expected to be significant.	**Take forward to scoping.**
Question 11: Is the site within 50m of the Hampstead Heath ponds?	The site was not within 50m of the catchment of any of the Hampstead Heath ponds	No further action required.
Question 12: Is the site within 5m of a highway or pedestrian right of way?	The pavement of Lyndhurst Road was immediately south of the site.	**Take forward to scoping.**
Question 13: Will the proposed basement significantly increase the differential depth of foundations relative to neighbouring properties?	The properties along Lyndhurst Road were noted were mainly 3-to-4 storey, semi-detached and detached residential properties, with lower ground floor levels ~2.00m below the properties ground level. Properties to the west and north generally had ground floors at higher elevation than the site, whereas to the south and east, properties generally had ground floors at lower elevation.	**Take forward to scoping.**

Stability Screening Flowchart		
Question	**Discussion**	**Conclusion**
Question 14: Is the site over (or within the exclusion zone of) any tunnels, e.g. railway lines?	The southern extreme of the Overground London Underground line is ~10.00m north of the northern boundary of the site and approximately 20.00m from the closest part of the basement development. No other London Underground tunnels were noted within close proximity to the site. No railway cuttings were noted within a 250m radius of the site. The site is not in close proximity to any National Rail lines.	**Take forward to scoping.**

3.2 Stage 2: Scoping

There are areas of concerns that the Screening process have highlighted.

- **Perched Water and Groundwater:** It was anticipated that groundwater was perched on top of the London Clay Formation, within the Claygate Member, with perched water possibly at shallower depths. Given the proposed basement depth, it was likely that the basement may encounter perched water/groundwater during construction. **This is to be taken forward for further assessment through a ground investigation and the installation of a monitoring well.**

- **Seasonal Soil Moisture and Volume Change Potential:** Anticipated geology considered the presence of mainly cohesive soils of Claygate Member of the London Clay Formation, underlain by the London Clay Formation, both of which are likely to be subject to subsidence due to shrinkage-swelling. **The depth and volume change potential of the underlying soils should be investigated.**

- **Pressure Induced Settlement and Heave:** Given the overburden pressure release following excavation of soil, as well as the loading of retaining wall foundations, the pressure across the basement is likely to cause differential settlement and heave. **Regarding the bulk basement construction, care will need to be taken to ensure that the slab is protected through accommodating heave (primarily) and any seasonal if applicable.**

- **Retaining Wall Design:** Given the design of basements, retaining walls should be appropriately designed to withstand the horizontal pressure of adjacent strata. **Retaining walls should be appropriately designed**.

- **Instability During Excavation:** Stability issues may arise during the excavation through natural soils and Made Ground. **Measures to be undertaken throughout excavation and construction will be discussed within this report, and more specifically the construction method statement.**

- **Ground Movement and Nearby Assets:** The properties along Lyndhurst Road were noted were mainly 3-to-4 storey, semi-detached and detached residential properties, with lower ground floor levels ~2.00m below the properties ground level. Properties to the west and north generally had ground floors at higher elevation than the site, whereas to the south and east, properties generally had ground floors at lower elevation. Differential foundation depths would cause potential damage to the walls of nearby buildings, due to soil displacement following the excavation/installation of the basement. This may also cause damage to nearby roads, pavements and utilities. **A Ground Movement Assessment (GMA) is required to assess the soil displacement and damage to nearby buildings, roads, pavements and utilities.**

- **Sub-Surface Concrete in Aggressive Ground Conditions:** Concrete may corrode if unsuitable concrete is used. A suitable concrete class should be used for all sub-surface concrete used for all foundations, based on the levels of sulphates and the pH within the ground it is being constructed on/through. **Testing in accordance with BRE Special Digest is required to be undertaken and a concrete specification is to be provided.**

- **Surface Water Flooding and Site Drainage:** Data from the Environment Agency website indicated that the site, and the majority of the surrounding area, was at very low risk of surface water flooding. The amount of hardstanding is likely to increase following the construction of the proposed development, leading to less areas for surface water to infiltrate into the ground. **The effect the proposed development will have on surface water flooding and the requirements to prevent surface water flooding and site drainage is to be discussed further within this report.**

- **Groundwater Flooding and Flow:** As the site was underlain by a Secondary (A) Aquifer, underlain by Unproductive Strata, there was considered to be a risk of groundwater flooding; however, this is considered to be low as a result of the anticipated cohesive nature of the soils. **A groundwater monitoring well should be installed as part of the site investigation, as well as groundwater dip measurements following the site works, to investigate groundwater levels.**

- **Sewer Flooding:** Given their subterranean position, basements can be susceptible to flooding from sewers. The site was in a post code area where there were 0 records of internal sewer flood incidents and 1 record of external sewer flood incidents. A CDA/Policy Area (Group3_008) (multiple CDAs linked together to provide a planning policy tool for the end users) has been drawn to match the Counters Creek catchment as almost all flooding issues spanning this area are interlinked due to the sewer network. **The effect the basement will have on the risk of sewer flooding and the requirements to prevent sewer flooding is to be discussed further within this report.**

- **Nearby London Underground Tunnel:** The southern extreme of the Overground London Underground line is ~10.00m north of the northern boundary of the site approximately 20.00m away from the northern extreme of the proposed basement. **Further comment should be made on the potential risk to the tunnel from ground movement.**

A site-specific ground investigation has been undertaken to inform design, including provision of information on the existing foundations. The results of this investigation and subsequent engineering considerations are provided within this report.

The submission of a drainage scheme will likely be required. It is understood this will form part of the overall Structural Scheme and will be included in the Structural Engineers report.

A qualified arboriculturist should be consulted for advice on the impact of nearby trees to the construction of the basement.

4.0 SITE WORKS

4.1 Scope of Works

Site works were between 5th and 13th December 2023 and comprised the drilling of 3No. Windowless Sampler Boreholes (WS1 – WS3) to 5.00m bgl. In-situ strength testing (SPTs) was undertaken at 1.00m intervals. Combined groundwater and ground-gas monitoring standpipes were installed to 5.00m bgl within WS1 and WS2, with a response zone between 1.00 – 5.00m bgl. Additionally, the hand excavation of 5No. Foundation Exposures (TP1 – TP5) was undertaken. A trial hole location plan has been provided within Figure 22.

Combined Ground-gas and Groundwater Monitoring Well Construction						
Trial Hole	**Type of Installation**	**Depth of Installation (m bgl)**	**Thickness of slotted piping with gravel filter pack (m)**	**Depth of plain piping with bentonite seal (m bgl)**	**Response Zone (m bgl)**	**Piping internal diameter (mm)**
WS1	Standpipe	5.00	4.00	1.00	1.00 – 5.00	50
WS2	Standpipe	5.00	4.00	1.00	1.00 – 5.00	50

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, the exploratory position was relocated away from these areas.

As a further precautionary measure, the borehole was hand excavated to 1.00m below the local ground level (bgl) and scanned with a Cable Avoidance Tool (CAT scanner) to minimise the risk to services.

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

4.2 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. A programme of chemical laboratory testing, scheduled by Ground and Water Limited and carried out by an accredited chemical testing laboratory, was undertaken on soils samples recovered from the trial holes.

5.0 ENCOUNTERED GROUND CONDITIONS

5.1 Soil Conditions

The trial holes were logged by a Ground and Water Limited representative, 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 conform to that anticipated from examination of the geology map. A capping of Made Ground was noted to overlie the Claygate Member before the London Clay Formation.

The succession of conditions and description of soils encountered in the trial holes in descending order is tabulated below.

Summary of Strata Encountered			
Strata	Top Depth (m bgl)	Base Depth (m bgl)	Thickness (m)
GRASS OVER MADE GROUND/MADE GROUND: Dark brown gravelly sandy silty CLAY. Sand was fine to coarse. Gravel was fine to coarse, sub-angular to sub-rounded flint and brick. ***(WS1 – WS3)***	11.10 – 12.80	10.80 – 12.20	0.30 – 0.60
CLAYGATE MEMBER: Brown/orange mottled slightly sandy silty CLAY. ***(WS1 – WS3)***	10.80 – 12.80	7.90 – 9.90	2.30 – 2.90
LONDON CLAY FORMATION: Grey/brown/orange mottled silty CLAY. ***(WS1 – WS3)***	7.90 – 9.90	<6.10 – <7.80	>1.80 – >2.10

For details of the composition of the soils encountered at particular points, reference must be made to the individual trial hole logs within Appendix D of this report. A trial hole location plan can also be viewed within Figure 22.

5.2 Foundation Exposures

The hand excavation of Foundation Exposures (TP1 – TP5) was undertaken across the site. A tabulated summary showing the depth and width of each foundation can be viewed below, as well as the bearing stratum. Diagrams of each foundation exposure can be viewed within Figures 23 – 30.

Summary of Foundations Encountered			
Trial Hole	Depth of Foundation (m bgl)	Width at the Base of Foundation (mm)	Bearing Stratum
TP1a (party wall)	0.74	120	London Clay Formation
TP1b (front facade)	0.56	400	London Clay Formation
TP1c (garden vaults)	>2.50	80	London Clay Formation
TP2	2.75	250	London Clay Formation
TP3 (south-east wall)	0.52	230	London Clay Formation
TP3 (west wall)	0.39	120	London Clay Formation
TP4	0.41	100	Made Ground
TP5	0.39	120	Made Ground

5.3 Roots Encountered

Fresh roots were noted to 0.30m bgl within TP2, 0.50m bgl within WS3 and TP1a/b/c, as well 1.00m bgl within WS1 and WS3. Fresh roots were also noted to the full depth of TP4 (0.45m bgl) and TP5 (0.40m bgl). No fresh roots were encountered when excavating TP3. It should be noted that the accuracy of determining the depth of root penetration through narrow diameter boreholes is considered low. It should be noted that 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.

5.4 Groundwater Conditions

No groundwater strikes were noted during the site investigation. It should be noted that groundwater strikes may have been obscured by the drilling processes.

Changes in groundwater level occur for a number of reasons including seasonal effects and variations in drainage. The investigation was undertaken in December 2023 when groundwater levels are likely to be rising from their annual minimum (lowest elevation). Exact groundwater levels may only be determined through long term measurements from monitoring wells installed on-site.

Groundwater monitoring was undertaken on two occasions to date. The results can be seen tabulated below.

Groundwater Observations			
Date	**Trial Hole**	**Water Level (m bgl)**	**Final Well Depth (m bgl)**
21/12/2023	WS1	4.80	5.00
	WS2	Not monitored	Not monitored
15/01/2024	WS1	4.80	5.00
	WS2	Dry	5.00

5.5 Obstructions

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

6.0 IN-SITU AND LABORATORY TESTING

6.1 In-Situ Strength Testing

Standard Penetration Tests (SPTs) and Super Heavy Dynamic Probes (SHDPs) were undertaken as part of the site investigation. The results of the SPT's have not been amended to consider hammer efficiency, rod lengths and overburden pressure in accordance with Eurocode 7. The test results are presented on the borehole logs within Appendix D. An interpretation of the in-situ geotechnical testing results is given in the table below. It must be noted that field measurements of undrained shear strength (Cu) are dependent on a number of variables including disturbance of sample, method of investigation and also the size of specimen or test zone.

Interpretation of In-situ Geotechnical Testing Results				
Strata	**Strata (m AOD)**	**SPT "N" Value**	**Equivalent Undrained Shear Strength (Cu)**	**Cohesive Soil Type (Cu)**
Claygate Member	WS1/10.80 – 7.90 WS2/11.50 – 8.80 WS3/12.20 – 9.90	11 – 19	45 – 95	Medium to High
London Clay Formation	WS1/7.90 – ~7.10 WS3/9.90 – <7.80	9 – 16	45 – 80	Medium
Soft London Clay Formation	WS1/~7.10 – <6.10 WS2/8.80 – <6.90	7 – 8	35 – 40	Low

6.2 Geotechnical Laboratory Testing

A programme of geotechnical laboratory testing, scheduled by Ground and Water Limited and carried out by an accredited geotechnical testing laboratory was undertaken on samples recovered. 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	BS1377:2016:Part 2:Clauses 3.2, 4.3 & 5	5
Water Soluble Sulphate and pH Test	BS1377:2018:Part 3:Clause 5	1
BRE Special Digest 1 Tests	BRE Special Digest 1 "Concrete in Aggressive Ground (BRE, 2005).	2

6.2.1 Atterberg Limit Testing

A précis of Atterberg limit testing undertaken can be seen tabulated below. The test results are presented within Appendix E.

Atterberg Limit Tests Results Summary							
Stratum	**Moisture Content (%)**	**Passing 425 µm sieve (%)**	**Modified PI (%)**	**Soil Class**	**Consistency Index (Ic)**	**Volume Change Potential**	
						BRE	**NHBC**
Claygate Member	22 – 31	100	36 – 41	CH	Stiff	Medium to High	Medium to High
London Clay Formation	23 – 30	100	36 – 39	CH	Stiff	Medium	Medium
• **NP – Non-plastic** • **BRE Volume Change Potential refers to BRE Digest 240 (based on Atterberg results)**							

Atterberg Limit Tests Results Summary							
Stratum	Moisture Content (%)	Passing 425 µm sieve (%)	Modified PI (%)	Soil Class	Consistency Index (Ic)	Volume Change Potential	
						BRE	NHBC
• Soil Classification based on British Soil Classification System. • Consistency Index (Ic) based on BS EN IS0 14688-2:2018.							

6.2.2 Moisture Deficit Assessment

The results of the Atterberg Limit tests were analysed to determine the Liquidity Index of the samples, to give an indication as to whether the samples recovered showed a moisture deficit as well assessing their degree of consolidation. Liquid Limit analyses was undertaken to assess whether there were any potentially significant moisture deficits within the samples tested. Potentially significant moisture deficits were noted in the following samples:

- WS2/0.80m bgl, caused by a combination of lithology and root uptake
- WS3/3.50m bgl, caused by lithology.

6.3 Chemical Laboratory Testing

An un-targeted set of samples (3No. Made Ground) were submitted to the accredited chemical laboratory for analysis. The results can be viewed in Appendix F.

Based on the proposed development, the results of the chemical laboratory testing were compared to the Generic Assessment Criteria (GAC) for a 'Residential without homegrown produce' land-use scenario, as this was considered the most appropriate land-use scenario.

Elevated levels of lead were found within all samples; therefore, a Full Contamination Assessment is recommended, which was not within the scope of this report.

7.0 ENGINEERING CONSIDERATIONS

7.1 Soil Characteristics, Foundation Considerations and Bearing Capacities

A summary of the soil characteristics following the intrusive site investigation and laboratory testing and the relevant foundation considerations has been provided below. The following information from the ground investigation was considered pertinent to the design of foundations.

- Foundations should be taken through any Made Ground and either into, or onto a suitable underlying natural stratum of adequate bearing characteristics.
- Foundations should be designed in accordance with soils of high volume change potential.
- The design and construction of the basement and associated structural elements would need to take into account the volume change potential of the respective soils. Special foundation precautions may be required to prevent possible future shrinkage/swelling of the soils affecting the integrity of the faces of foundations and/or structural element (underfloor void diameter/compressible material/void formers etc).
- For the shallow footings of the annex, given the presence of cohesive soils at shallow depth, special foundation precautions may be required to prevent possible future shrinkage/swelling of the soils affecting the integrity of the faces of foundations (underfloor void diameter/compressible material/void formers etc). A compressible layer must be provided to accommodate potential seasonal movement, based on NHBC guidance.
- Due to the shallow soils having volume change potential, a suspended slab is required.
- The loads of proposed foundations should not exceed the allowable bearing capacity of the soils they are founding upon.
- Foundations must not be placed within fresh root penetrated and/or desiccated soils with volume change potential. It is recommended that foundations are taken at least 300mm into non-fresh root penetrated strata if the soils have volume change potential, or into soils of no volume change potential.
- The influence of trees on or surrounding the site will need to be taken into account in final design (NHBC Standards Chapter 4. 2) (tree rings).
- Any water ingress must be prevented from entering foundation trenches and excavations must be kept dry and either concreted or blinded as soon after excavation as possible. If water were allowed to accumulate within the excavation for even a short period of time, an increase in heave occur. The shear strength will also be reduced, resulting in lower bearing capacities, resulting in increased settlements. Instability issues may arise within the foundation trenches, in case of perched water being present.
- Final designs for the foundations should be carried out by a suitably qualified Engineer based on the findings of this investigation and with reference to the anticipated loadings, serviceability requirements for the structure and the developments proximity to former, present, and proposed trees.

Main Property

Foundations should be taken through any Topsoil/Made Ground before founding onto competent, moisture stable soils. Therefore, the proposed foundation depths at 8.74m AOD/2.36m bgl at WS1/3.46m bgl at WS2 and would be founded on the interface between the Claygate Member and London Clay Formation, both of which were considered suitable bearing stratums. Foundations at this depth can be designed based on an allowable bearing capacity of 100kN/m^2. This is based on trial hole records, the results of in-situ testing, inspection of samples recovered, and referral to BS 8004:2015, Code of Practice for Foundations.

External Annex

Foundations should be taken through any Topsoil/Made Ground before founding onto competent, moisture stable soils. Given the underlying Claygate Member had volume change potential, fresh roots should also be bypassed by 300mm. Based on the maximum volume change potential found within the Claygate Member (high volume change potential), an absolute minimum foundation depth of 1.00m bgl was proposed. Therefore, the proposed foundation depth for the external annex should be at 1.00m bgl/11.80m AOD at WS3. Foundations at this depth can be designed based on an allowable bearing capacity of 120kN/m^2. This is based on trial hole records, the results of in-situ testing, inspection of samples recovered, and referral to BS 8004:2015, Code of Practice for Foundations.

7.2 Geotechnical Analysis

This section of the report states suitable geotechnical parameters for the soils encountered as well as analysis the bearing capacity of the soils. A settlement/heave analysis was also undertaken following the construction of the proposed development using Pdisp from Oasys.

7.2.1 Geotechnical Parameters for Modelling

Following a literature review from well documents publications, the short-term and long-term Young's Modulus (E short term and E') has been produced. The parameters, shown below, were used when undertaking the settlement/heave analysis within Pdisp. The soil profiles were based on WS1, which was considered the most conservative.

Summary of Geotechnical Parameters								
Geological Strata	Depth (m AOD)		Short-term Young's Modulus (Eu short term) (kPa)		Long-term, Young's Modulus (E' long term) (kPa)		Poisson's Ratio	
	Top	Base	Top	Base	Top	Base	ST	LT
Made Ground	11.10	10.80	10,000.000	10,000.000	10,000.000	10,000.000	0.45	0.20
Claygate Member	10.80	7.90	17,979.200	46,130.080	13484.400	34,597.560	0.45	0.20
London Clay Formation	7.90	-2.10	15,100.400	21,376.400	11,325.300	16,032.300	0.45	0.20
Inferred London Clay Formation	-2.10	-10.85	52,000.000	80,000.000	39,000.000	60,000.000	0.45	0.20
	-10.85	-75.00	80,000.000	80,000.000	60,000.000	60,000.000	0.45	0.20

Made Ground

Made Ground was modelled between 11.10 – 10.80m AOD.

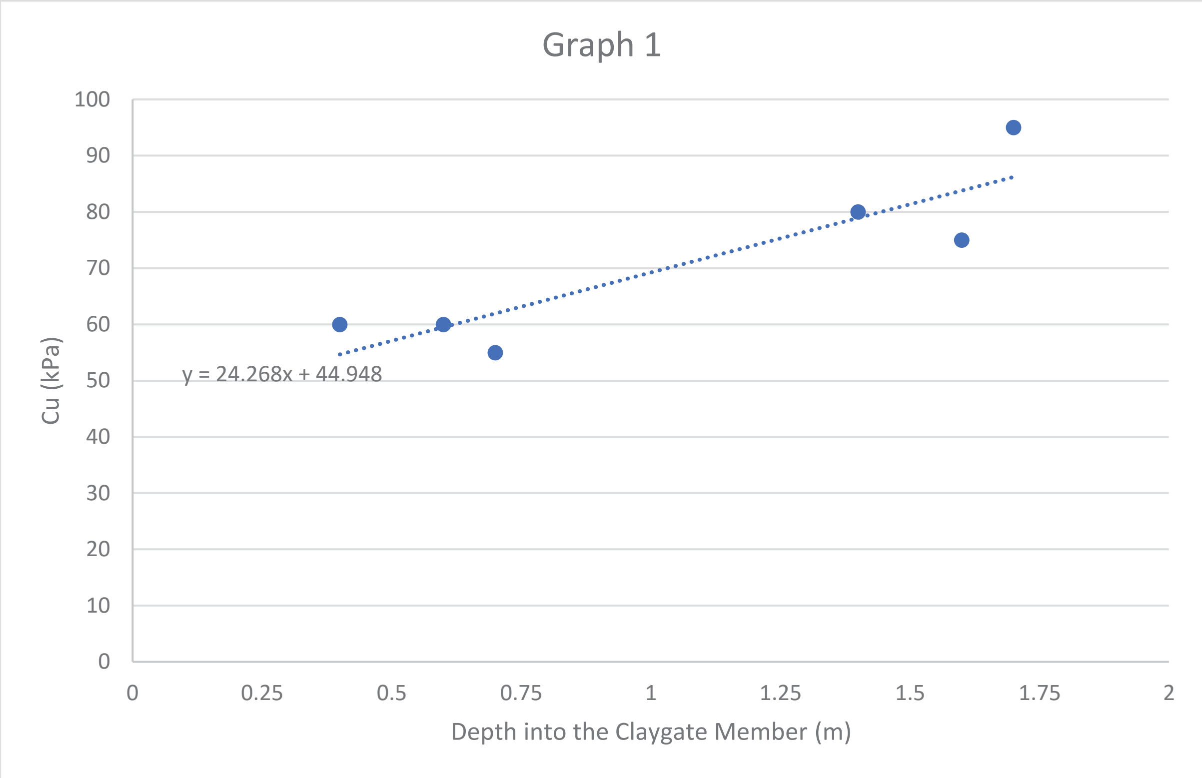
A short-term and long-term Young Modulus (Eu and E') of 10MPa was suitable and on the conservative side, regarding Made Ground encountered on site. A Poisson's Ratio of 0.45 was considered suitable for these soils, given their variable nature.

Claygate Member

Cohesive soils of the Claygate Member were modelled between 10.80 – 7.90m AOD.

Where SPT "N" Values were undertaken, the Cu could be calculated by multiplying by 5, as stated by Stroud (1974).

A graphical plot of how the undrained shear strength increases with depth into the Claygate Member is shown within Graph 1. The trendline within Graph 1 stated the following equation: **Cu = (24.268 x depth into the CM) + 44.948.**



The relationship between Eu and Cu is generally dependent on strain levels. For small strains, for cohesive soils with PI of 30 – 50%, a ratio of 300 – 600 can be adopted based on well documented publications (*Jamiolkowski et al 1979*). This is also reflected for the London Clay Formation, after extensive research, within graphs depicting strains and Eu/Cu ratios included in *"Burland JB, Standing, JR, and Jardine, FM (2001) Building response to tunnelling, case studies from construction of the Jubilee Line Extension CIRIA Special Publication 200".* A Eu to Cu ratio of 400 has then been applied.

A Poisson's Ratio of 0.45 was considered suitable for these soils, given their cohesive nature.

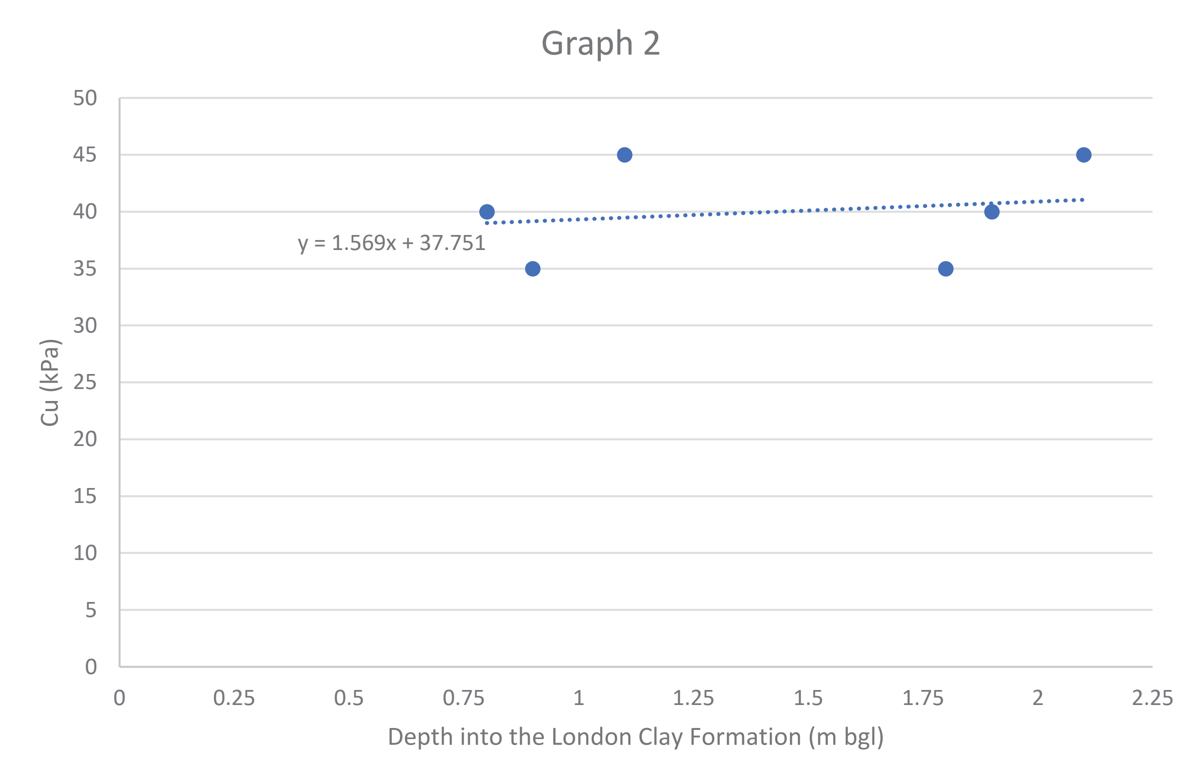
London Clay Formation

Cohesive soils of the London Clay Formation were encountered from 7.90 – 6.10m AOD during site works, and inferred to ~50.00m BOD based on BGS borehole records.

Where SPT "N" Values were undertaken, the Cu could be calculated by multiplying by 5, as stated by Stroud (1974).

Where the London Clay Formation was encountered, all SPTs were used to make a Cu vs depth plot. A graphical plot of how the undrained shear strength increases with depth into the Claygate Member is shown within Graph 2. The trendline within Graph 2 stated the following equation: **Undrained shear**

strength = (1.569 x depth into the London Clay Formation) + 37.751. This formula was used for the first 10.00m of the London Clay Formation.



Where the London Clay Formation was inferred, a design line was taken from "*Burland JB, Standing, JR, and Jardine, FM (2001) Building response to tunnelling, case studies from construction of the Jubilee Line Extension CIRIA Special Publication 200*". The equation was **undrained shear strength = (depth into the LCF x 8) + 50**. This formula was used for the remaining depth of the London Clay Formation.

It is considered likely that once a Cu of 200kPa is found within the London Clay Formation, it is unlikely that it will increase further. At a depth of 18.75m into the London Clay Formation, a value of 200kPa was achieved and therefore from this elevation (i.e. 10.85m BOD), the Eu/Cu values of the London Clay Formation did not change.

The relationship between Eu and Cu is generally dependent on strain levels. For small strains, for cohesive soils with PI of 30 – 50%, a ratio of 300 – 600 can be adopted based on well documented publications (*Jamiolkowski et al 1979*). This is also reflected for the London Clay Formation, after extensive research, within graphs depicting strains and Eu/Cu ratios included in *"Burland JB, Standing, JR, and Jardine, FM (2001) Building response to tunnelling, case studies from construction of the Jubilee Line Extension CIRIA Special Publication 200".* A Eu to Cu ratio of 400 has then been applied.

A Poisson's Ratio of 0.45 was considered suitable for these soils, given their cohesive nature.

Long-Term Conditions

For Made Ground, it was considered suitable for E' and Eu to be equal, given that these soils are more permeable and to limit the level of anticipated Young Modulus at a representative value. A ratio of E' to Eu of ~0.75 was considered a sensible approach for this stage in the design, for cohesive soils of the

Claygate Member and the London Clay Formation.

A long term Poisson's Ratio of 0.20 was applied for all soils.

7.2.2 Settlement/Heave Analysis

Analyses of vertical ground movements, using the Mindlin analysis method within Pdisp software, was undertaken to assess the potential movements resulting from changes of net vertical pressure changes. Geotechnical parameters noted in the previous section of this report were used for the model. A rigid boundary at depth was considered at 50.00m BOD, for calculation purposes. The inputs and outputs of this analysis can be viewed within Appendix G.

Five representative stages of construction, in terms of the change in vertical pressure, have been modelled. These were considered to adequately approximate the movements rising from the basement construction.

- **Stage 1:** Excavation of the retaining wall voids, with short-term conditions, as well as the demolition load removal;
- **Stage 2:** The construction of the retaining walls, with short-term conditions. Please note that the previous overburden pressure release relating to the excavation of the retaining wall voids was not applied, as the concrete weight was anticipated to counter this;
- **Stage 3:** All previous loads/load removals, as well as loads associated with the mass excavation of the basement footprint, with short-term conditions;
- **Stage 4:** All previous loads/load removals, as well as loads associated with the construction of the basement slab, with short term conditions. The basement is fully constructed from this stage onwards;
- **Stage 5:** All previous loads/load removals, for long-term conditions.

The existing property on-site was to be demolished. An average load removal of 50kN/m^2 was modelled at ground floor level to replicate the overburden pressure release of the demolition. This load removal was modelled at 10.90m AOD.

Given the overall rectangular shape of the basement, the excavation was based on a rectangle using the maximum length and width of the basement. This was considered conservative and will ensure accurate results.

The overburden pressure release following the excavation and removal of soils was based on a specific weight of soil of 19kN/m. Based on a proposed founding depth of 8.74m AOD, and a maximum excavation thickness of 3.20m, an overburden pressure release of 60.80kN/m^2. The overburden pressure release was modelled at 8.74m AOD.

Retaining wall loads were not finalised by the structural engineer at the time of writing, but were to be limited to a maximum load of 100kN/m^2; therefore, for models throughout the construction of the basement (i.e. Stage 2 and Stage 3), a load of 30kN/m^2 was applied, increasing to 100kN/m^2 when fully constructed (i.e. Stage 4 onwards). This was selected in order not to underestimate the heave and overestimate any settlement. These loads were modelled at 8.74m AOD and noted to extend 1.00m towards the centre of the basement.

The load of the basement slab was considered to be 10kN/m^2 and modelled at 8.74m AOD.

A tabulated summary of all applied loads, at each stage/model, can be viewed below.

Summary of Net Bearing Pressure Changes for PDisp Analysis					
Description	Level (m AOD)	Applied Load (+ive)/ Load Removal (-ive) (kN/m^2)			
		Stage 1	Stage 2	Stage 3	Stage 4 and 5
Demolition of Existing Building	10.90	-50.00	-50.00	-50.00	-50.00
Excavation of Retaining Wall Voids	8.74	-60.80			
Construction of Retaining Walls	8.74		30.00	30.00	100.00
Mass Excavation Void	8.74			-60.80	-60.80
Construction of Basement Slabs	8.74				10.00

The method stated above was considered to comprise a comprehensive and reasonably conservative approach, in order to estimate the maximum potential heave and settlements.

A tabulated summary concluding the amount of soil displacement shown at 8.74m AOD within the contour plots can be viewed below. It should be noted that the soil displacement between models are not cumulative values; therefore, the amount of soil displacement between models should not be added together as each model shows each construction stage individually.

Settlement/Heave Analysis	
Model	Soil Displacement
Model 1	14.50mm maximum heave. No settlement.
Model 2	9.00mm maximum heave. 1.59mm maximum settlement
Model 3	24.20mm maximum heave. No settlement.
Model 4	16.80mm maximum heave. 7.20mm settlement.
Model 5	24.60mm maximum heave. 9.48mm settlement.
Diagrammatic representation can be viewed within Appendix G. Please note that the above figures should not be added together (or be superimposed) and that they represent anticipated movements at different accumulated stages of construction, in order to approach and test all expected combinations of loading regimes (models).	

A maximum amount of heave of 24.20mm was noted following the mass excavation of the basement void (Model 3), and was noted to be the maximum amount of heave during the construction phases. Once constructed, the maximum amount of heave increased from 16.80mm for short term conditions (Model 4), to 24.60.00mm for long term conditions (Model 5); therefore, the highest risk of movement will likely occur during the construction of the basement and later through long-term heave of the constructed basement. It should be noted that the effect of the demolition load is conservative and the amount of heave may be lower than calculated.

7.2.3 Additional Comments

Regarding the bulk basement construction, care will need to be taken to ensure that the slab is protected through accommodating heave. Heave protection measures will need to be incorporated.

The final design of the basement structure will also need to take into account environmental factors, such as ventilation/protection against ground gas; however, investigation into this was not included within the scope of this report.

Final designs for the foundations should be carried out by a suitably qualified Engineer based on the findings of this investigation and with reference to the anticipated loadings, serviceability requirements for the foundations. A Structural Engineer will also need to review the anticipated ground movements and assess their potential impact on the existing structure and neighbouring properties. It must be noted that finalised construction will aid the structural stability of the neighbouring party walls, reducing the risk of the seasonal movements noted during the structural works.

7.3 Retaining Walls, Excavations and Stability

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

Appropriate propping and support should be incorporated during construction of the basement.

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 the soils encountered with an appropriate angle of shear resistance (Φ') and effective cohesion (C') for the ground conditions encountered, regarding long-term considerations, as well using an appropriate undrained shear strength Cu for short-term considerations.

The overlying Made Ground needs to be considered in the design of the basement. A conservative value of Cu will need to be considered.

Based on the ground conditions encountered within the boreholes the following parameters tabulated below could be used in the design of retaining walls, for a long-term consideration. These have been designed based on the in-situ strength testing profile recorded, results of geotechnical classification tests and reference to literature.

Retaining Wall/Basement Design Parameters					
Strata	**Unit Volume Weight (kN/m³)**	**Cohesion Intercept (c') (kPa)**	**Angle of Shearing Resistance (°)**	**Ka (Rankine)**	**Kp (Rankine)**
Made Ground	~19	0	12	0.66	1.52
Natural Cohesive Soils	~20 – 22	0	24	0.42	2.37

It should be noted that the Ka and Kp values presented in the table, are shown for guidance and they are derived from the Rankine theory for soil pressures. The values for angles of internal friction provided are considered to be characteristic values of the soils encountered.

According to C760, a design method (e.g. EC7) should be adopted and followed through the whole design process. In addition, the following considerations should be considered during the design process:

- Appropriate consideration of groundwater levels.
- Surcharge pressure equivalent to the pressures of any adjacent buildings.
- Surcharge pressures from potential piling work platforms and heavy plant traffic.

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.

Ground Instability Recommendations

No significant instability issues related with soils are expected and no instability issues were observed during the ground investigation. Specific measures should be included in a competent Construction Method Statement for the works on this site by the structural engineer and the contractor. If instability is noted, the following could be applied for good workmanship and mitigation of any risk. It should be noted that these are indicative.

- Where soft/loose spots are encountered, trench sheets should be left in. Alternatively, a back prop with precast lintels or sacrificial boards should be installed. If the soil support to the ends of the lintels is insufficient, brace the ends of the PC lintels with 150x150 C24 timbers and prop with Acrows diagonally back to the ground.
- Where voids are present, trench sheeting with 75mm diameter holes should be installed, to allow the concrete to flow behind the trench sheeting thereby filling any voids encountered in soils behind.
- Prior to casting, a layer of DPM should be installed between trench sheeting (or PC lintels) and new concrete. The lintels should be cut into the soil by 150mm either side of the pin. A site stock of a minimum of 10 lintels should be present to prevent delays due to ordering.

7.4 Ground Movement Analysis

The ground movement assessment and resulting damage assessment can be viewed within this section.

7.4.1 Models

A model was created to assess soil displacement and the resulting damage to surrounding assets. The inputs and outputs of this analyses can be viewed within Appendix H.

The walls of surrounding properties were modelled at the existing lower ground floor level (10.90 m AOD/along the z-axis).

One excavation was modelled for 3.20m to create an overall excavation similar to that of the basement, based on the maximum length and width of the proposed basement.

It is conventionally considered that given the ground conditions and good workmanship, the amount of structural movement of underpinned walls can be expected to reach a maximum of ~5.00mm per stage of underpinning. Horizontal wall movements are also expected to occur due to yielding and these permit movement of the soils behind the underpinned wall during the basement excavation. As a conservative estimation, the magnitude of the horizontal movement of the underpinned wall was assumed to be equal to the vertical movement of the wall. It should be noted that these movements are essentially dependent on the ground conditions on-site.

Ground movement can be expected to gradually decrease with distance from the basement development. In the absence of underpinning data, the empirical data set out in CIRIA C760 was used to provide a approximation to the following assumptions of linear decay:

- For vertical movement, the assumption of zero movement at a distance equal to 2 times the underpinning depth was used
- For horizontal movement, the assumption of zero movement at a distance equal to 4 times the underpinning depth was used

As loads from the proposed newbuild may also have an effect on ground movement, the vertical movements relating to the positive loads only from Pdisp Model 4 were imported into the model also. The 'positive loads only' approach will ensure conservatism in the modelling/approach for the movements away from the basement.

A tabulated summary showing which curves and Pdisp results were used can be viewed below.

Summary of XDisp Analysis Assessments			
Model	**PDisp Results**	**Vertical Ground Movement**	**Horizontal Ground Movement**
1	Stage 4 Vertical Displacements	A linear curve where 5mm was noted at the excavation face, reducing to 0mm at a distance of 2 x underpinning depth	A linear curve where 5mm was noted at the excavation face, reducing to 0mm at a distance of 4 x underpinning depth

7.4.2 Analyses

Once the analysis was undertaken, Xdisp segments areas of hogging, sagging and negligible movement along each wall, and gave each segment a category of damage; however, as the wall was thought to act as one structurally, these segments were combined and a damage category for the wall as a whole was given.

The following parameters have been used to inform all assessment. The following parameters have been used to inform assessment.

- Given limitations of the software, a conservative assessment was undertaken assuming that all properties and levels were relative to the ground level.
- The method of basement construction is understood to be traditional underpinning;
- A high wall stiffness should be considered;
- In the permanent case the wall should be propped at high level;

In terms of damage assessment, the widely accepted Burland et al, 1977 method was used for combined segments along structural features.

It was considered that the construction design from the structural engineer will account for any damage resulting from predicted soil displacement on the actual building, as advised in this report.

7.4.3 Results

All walls were assessed as having Category 0 (Negligible) or Category 1 (Very Slight) damage.

It is likely that the front and rear walls along Lyndhurst Road may act structurally as one wall, potentially lessening structural damage.

During the site walkover, there was evidence of lightwells/lower ground floors within the neighbouring properties. These sub-surface structures may offer rigidity to the surrounding ground and reduce ground movement resulting from the excavation and construction of the basement, lessening structural damage on overlying walls.

7.4.4 Assessment of Roads and Utilities

Given the length of the roads, the deflection expected was not anticipated to cause damage; however, monitoring is recommended as good practice.

7.4.5 London Underground Tunnel

The southern extreme of the Overground London Underground line is ~10.00m north of the northern boundary of the site approximately 20.00m away from the northern extreme of the proposed basement.

Ground movement relating to the underpinned basement is to extend 4 x underpin depth, which equates to 14.00m. As the tunnel is further away from the basement than 14.00m, no ground movement is to be expected at the location of the tunnel and therefore there will not be any effect.

7.4.6 Additional Comments

Should the following precautions be included in the Construction Method Statement, as well as best practice and good construction techniques are utilised by a reputable contractor, then ground movements due to underpinning will be limited. In the permanent case the wall will/should always be propped at high level. **It is recommended that monitoring is undertaken as good practice.**

It will be important that the building contractor is closely supervised and is experienced in this type of construction. It will be critical to prevent exposed faces from collapse or significant ground loss into the new excavation and temporary face support should be maintained where practicable. The adequacy of temporary support will be critical in limiting ground movements. A number of factors will assist in limiting ground movements:

- Most ground movement will occur during excavation and construction so the adequacy of temporary support will be critical in limiting ground movements;
- The speed of propping and support is key to limiting ground movements;
- Good workmanship will contribute to minimising ground movements;
- The assessment assumes the wall is in competent clay;
- Larger movements will be expected where soft soils are encountered at, above and below formation;
- The adequacy of temporary support will be critical in limiting ground movements;

CIRIA C760 advises that ground movements are influenced by the quality of workmanship. The party wall act will apply to this development and will re-enforce good workmanship. The act provides an effective mechanism for ensuring that structural integrity of the neighbouring property is maintained throughout the construction phase. Examples of this can be viewed below.

- Ensuring that adequate propping is in place at all times during construction;
- Minimise deterioration of the central soil mass by the use of blinding/covering with a waterproof membrane;
- Installation of the first (stiff) support quickly and early in the construction sequence for each excavation panel;
- Control dewatering to minimise fines removal and drawdown;
- Avoid overbreak.
- Avoid leaving ground unsupported.

7.5 Structural Monitoring

As stated within the previous section, it is recommended that structural monitoring is undertaken to ensure the movements remain within acceptable limits and to enable mitigation to be effectively implemented in the event of trigger values for movement being exceeded.

The final extent of the structural monitoring will be a matter for the agreement with the neighbours as part of the Party Wall Agreements.

Monitoring positions should be located at the front and rear elevations of the neighbouring properties. The targets should be set at both a low and high level and a minimum of four targets should be installed at each elevation (two targets near party wall and two targets at the far end of the elevation). Precise survey equipment should be used to record all vertical and horizontal components of movement (in three perpendicular dimensions) to a minimum accuracy of 1mm.

Before any excavation or construction works commence, monitoring over a period of at least a month should be undertaken in order to establish a baseline situation and record any seasonal movement trends that may also affect measurements during the development.

During all underpinning works and basement excavation works, monitoring should be undertaken daily at the start and end of every work shift. At other times, monitoring should be undertaken weekly to cover a period prior to commencement of any works and ceasing after completion of the works, by agreement of all interested parties.

7.6 Sub-Surface Concrete Design

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'* considering the pH of the soils. For the classification given below, the "mobile" and "natural" case was adopted given the geology encountered and the residential use of the site.

Made Ground

The water soluble sulphate concentration was <4mg/l, with a pH range of 7.6 – 8.2. According to BRE Special Digest 1, 2005, 'Concrete in Aggressive Ground' a Sulphate Design Class of DS-1 could be used for sub-surface concrete in contact with the Made Ground. Table C1 of the Digest indicated an ACEC (Aggressive Chemical Environment for Concrete) classification of AC-1.

Claygate Member

According to Box C6 of BRE Special Digest 1, 2005, 'Concrete in Aggressive Ground' the London Clay

Formation of which the Claygate Member is a member of fell within a list of UK geological formations known to contain pyrite. It was therefore required to consider the levels of total potential sulphate in the classification process.

The water soluble sulphate concentration ranged between <4 – 54.7mg/l, with a pH range of 7.83 – 7.97. The total potential sulphate (3 x total sulphur) was 0.2877%. According to BRE Special Digest 1, 2005, 'Concrete in Aggressive Ground' a Sulphate Design Class of DS-2 could be used for sub-surface concrete in contact with the Claygate Member. Table C1 of the Digest indicated an ACEC (Aggressive Chemical Environment for Concrete) classification of AC-2.

London Clay Formation

According to Box C6 of BRE Special Digest 1, 2005, 'Concrete in Aggressive Ground' the London Clay Formation fell within a list of UK geological formations known to contain pyrite. It was therefore required to consider the levels of total potential sulphate in the classification process.

The water soluble sulphate concentration was 74.4mg/l, with a pH of 7.55. The total potential sulphate (3 x total sulphur) was 0.1392%. According to BRE Special Digest 1, 2005, 'Concrete in Aggressive Ground' a Sulphate Design Class of DS-1 could be used for sub-surface concrete in contact with the London Clay Formation. Table C1 of the Digest indicated an ACEC (Aggressive Chemical Environment for Concrete) classification of AC-1.

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 considered should ferruginous staining/pyrite nodules be encountered within the London Clay Formation.

7.7 Hydrogeological Effects, Flooding and Surface Water Disposal

Basements have potential to greatly impact hydrological and hydrogeological regimes. Numerous comments and considerations reflecting on the relationship between the basement and groundwater/surface water have been discussed below.

7.7.1 Basement Construction

If the construction works take place during the winter months, when the groundwater level is expected to be at its higher elevation, water could accumulate thus dewatering could be required to facilitate the construction and prevent the base of the excavation blowing before the slab was cast. **The lower ground floors must be suitably tanked to prevent ingress of groundwater and also surface water run-off. A dewatering or permitting grout contingency plan should be included within the Construction Method Statement and considered in the final design. As there will be potential for groundwater to collect behind the retaining walls, the basement should be waterproofed and designed to withstand hydrostatic pressures in accordance with BS8102:2009: Code of Practice for the Protection of Below Ground Structures against Water from the Ground.**

Should groundwater/perched water be encountered across the site, dewatering from sumps introduced into the floor of the excavation may be required. Consideration could be given to creating a coffer dam using contiguous piled or sheet piled walls to aid construction below the perched water table if groundwater becomes a significant issue. **The advice of a reputable dewatering company should be sought.**

7.7.2 Site Drainage

The majority of new developments are encouraged to use Sustainable Urban Drainage Systems (SUDS) to manage surface water drainage. This ensures that any volumes and peak flow rates of surface water leaving a developed site are no greater than the rates prior to the proposed development unless specific off-site arrangements are made and result in the same effect.

The principles of SUDS and the requirements of the London Plan Policy 5.13 Sustainable Drainage should be applied to reduce the risk of flooding from surface water ponding and collection associated with the construction of the basement.

In accordance with the London Plan Policy 5.13 Sustainable Drainage the surface water run-off should be managed as close to its source as possible in line with the following drainage hierarchy.

- Rainwater use as a resource (for example rainwater harvesting, blue roofs for irrigation)
- Rainwater infiltration to ground at or close to source
- Rainwater attenuation in green infrastructure features for gradual release (for example green roofs, rain gardens)
- Rainwater discharge direct to a watercourse (unless not appropriate)
- Controlled rainwater discharge to a surface water sewer or drain
- Controlled rainwater discharge to a combined sewer.

Drainage should be designed and implemented in ways that deliver other policy objectives of this Plan, including water use efficiency and quality, biodiversity, amenity and recreation.

Soakage testing in accordance with BRE365 was beyond the scope of this investigation.

Any soakaways should be located sufficiently away from buildings and infrastructure, in order to prevent undermining of foundations. Additional drainage may be considered should significant amounts of water be encountered.

Consultation with the Environment Agency must be sought regarding any use that may have an impact on groundwater resources, abstractions and surface water features/watercourses.

7.7.3 Additional Comments

The site itself has the potential to flood from groundwater, due to a Secondary Aquifer underlain by Unproductive Strata; however, given the cohesive nature of the soils, the amount of groundwater was likely to be limited. Perched water may be encountered within the Made Ground and the underlying geological formations, especially after periods of prolonged or intense rainfall. **This should be considered in final design.**

Due to the relatively low permeability rates of the cohesive soils, groundwater is more likely to flow through the more permeable capping soils (i.e. Made Ground) and to some extent, the Claygate Member. As the proposed basement will bypass the shallow surface soils and extend into the Claygate Member, there will be a lateral barrier for perched water flow; however, given the limited quantity of the perched water anticipated and encountered, no adverse effect on groundwater flow was anticipated.

Given their subterranean position, lower ground floors can be susceptible to flooding from sewers. In order to minimise the risk of sewer flooding to the development, all subterranean development must be connected to the sewerage network, installed with a positively pumped non-return valve device.

Consultation with the Environment Agency must be sought regarding any use that may have an impact on groundwater resources, abstractions and surface water features/watercourses.

7.8 Discovery Strategy

A full contamination assessment was beyond the scope of this investigation, where targeted sampling was not undertaken. There may be areas of contamination that have not been identified during the course of the intrusive investigation (e.g. underground storage tanks). Such occurrences may be discovered during the 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 and then the Local Authority will need to be informed.

7.9 Waste Disposal

The excavation of foundations and soils 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 (WM3) 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.

Following the investigation, 6No. samples (3No. Made Ground, 2No. Claygate Member and 1No. London Clay Formation) were submitted to the analytical laboratory to undergo a suite of testing for contamination testing, as discussed in the previous sections. Sampling depths were chosen to reflect the receptor of concern, human health, and typically comprised a surface or near surface sample and periodically to 1.00m bgl. Any horizon where olfactory or visual evidence of contamination was present was also sampled.

Based on a risk phase analysis of the chemical laboratory test results, in accordance with EC Hazardous Waste Directive and undertaken by Ground and Water Limited, all soil samples encountered on-site were NON-HAZARDOUS, other than a sample of Made Ground from WS1 at 0.20m bgl, where high levels of lead and zinc caused the soil to be exotoxic. The results of the assessment are given within

Appendix I.

It is important to note that whilst we consider our in-house assessment tool to be an accurate interpretation of the requirements of WM3, therefore producing an initial classification in accordance with the guidance, this method classifies soils as either non-hazardous or hazardous and landfill operators have their own assessment tools and can often come to different conclusions. As a result, some landfill operators could refuse to take apparently suitable waste. It is recommended that the receiving landfill views the results of this assessment and the chemical laboratory results to determine their own classification.

In addition to the samples described above, 1 sample of Made Ground was scheduled to undergo Waste Acceptance Criteria (WAC) testing with single batch leachate. The results indicated that the sample complied with the stable non-reactive hazardous waste in non-hazardous landfill. The results can be seen in Appendix F. The locations of these samples can be seen on Figure 22.

It should be noted that the receiving landfill may see the presence of Made Ground within the soils to be indicative of a non-hazardous classification despite the presence of the WAC testing certificates indicating that it is inert. Therefore, it should be considered that all Made Ground is classified as Non-hazardous, subject to finalising with the receiving landfill.

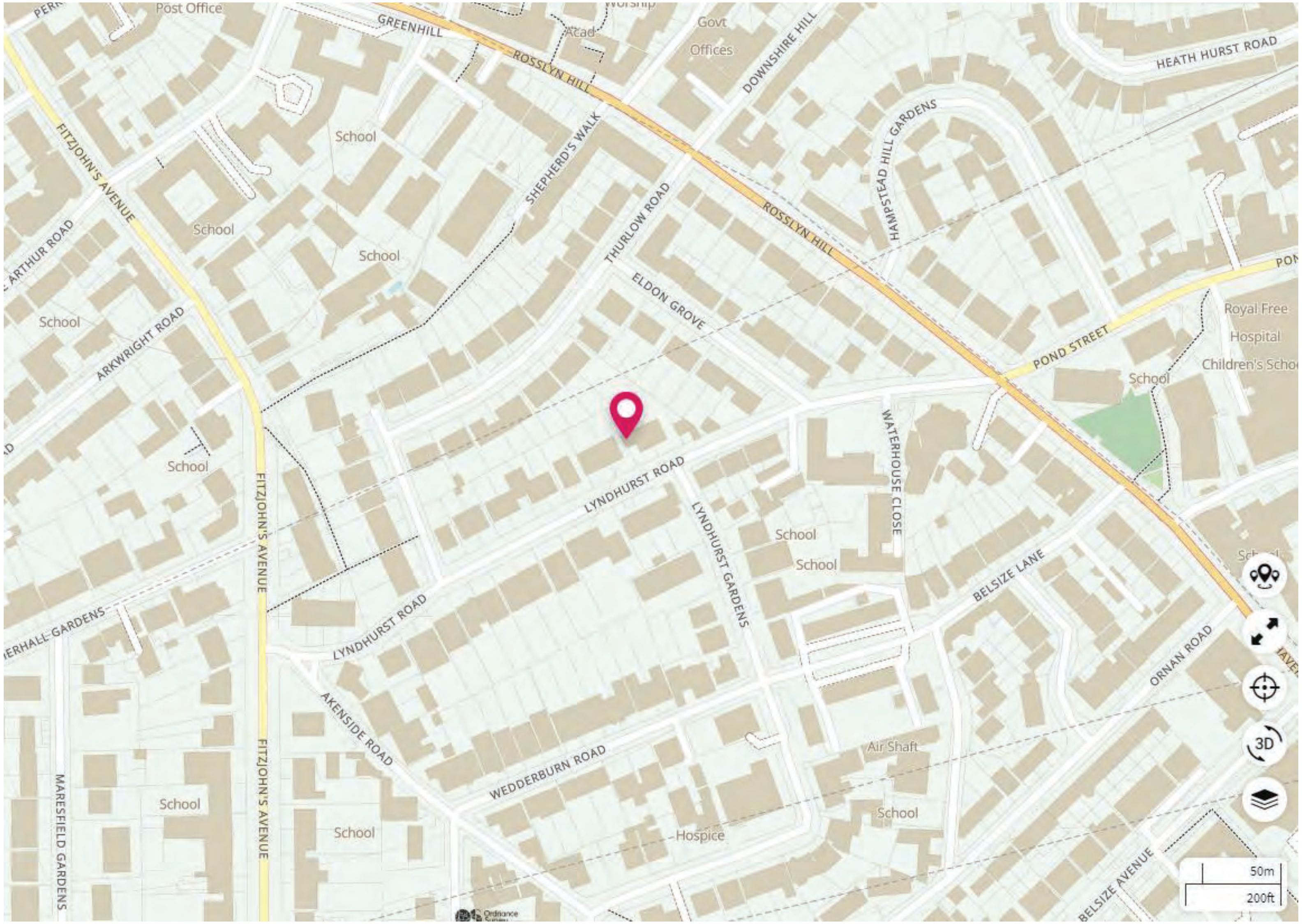
Where contaminated soils are to be removed, they should be placed on an impermeable membrane (visqueen or similar) to ensure that no cross-contamination of soils occurs.

7.10 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 would be 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.

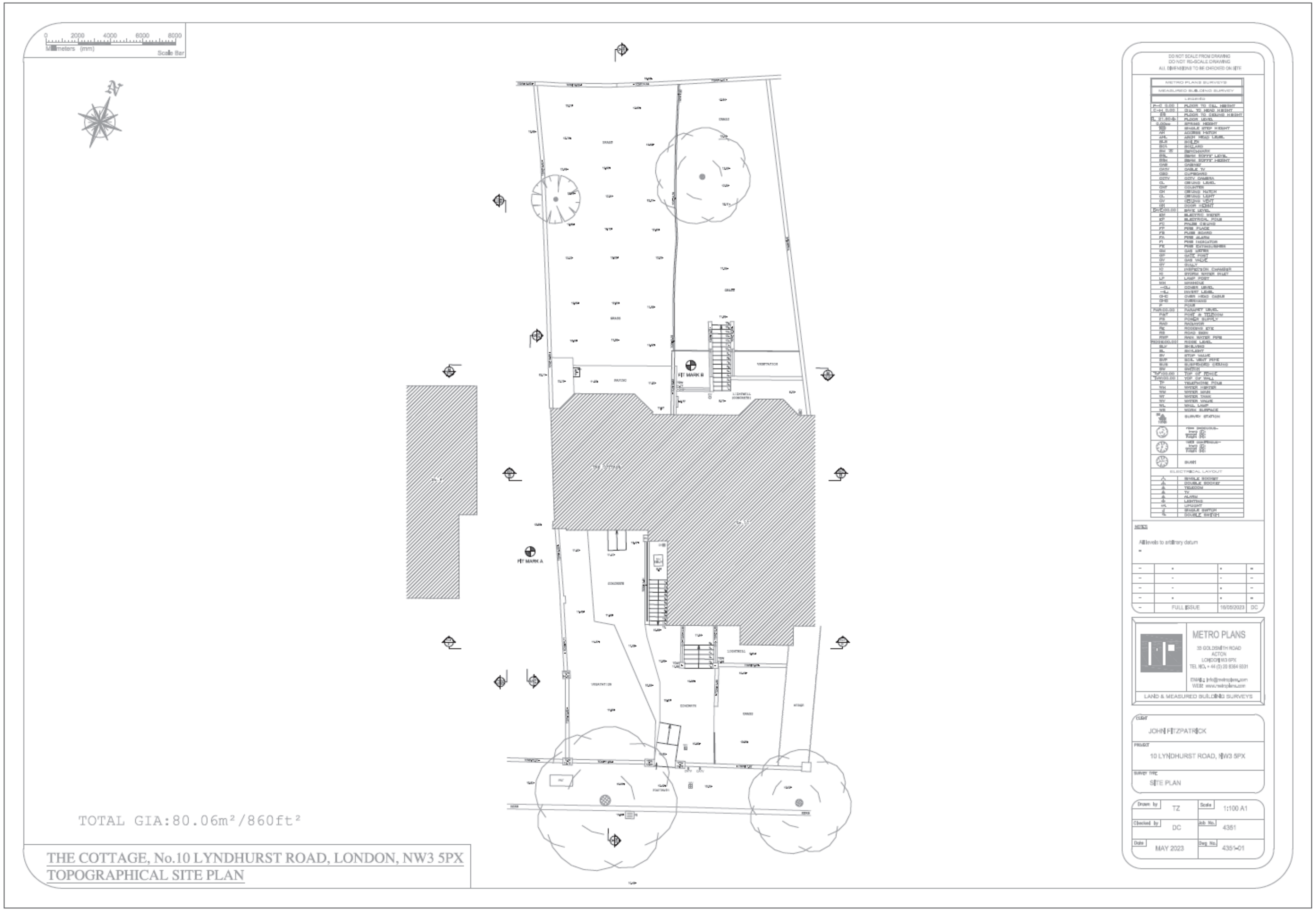


Site Location

Not to Scale

The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX	
John Fitzpatrick c/o Momentum Structural Engineers	**January 2024**
Figure 1: Site Location Plan	**GWPR5717**

ground&water



Not to Scale

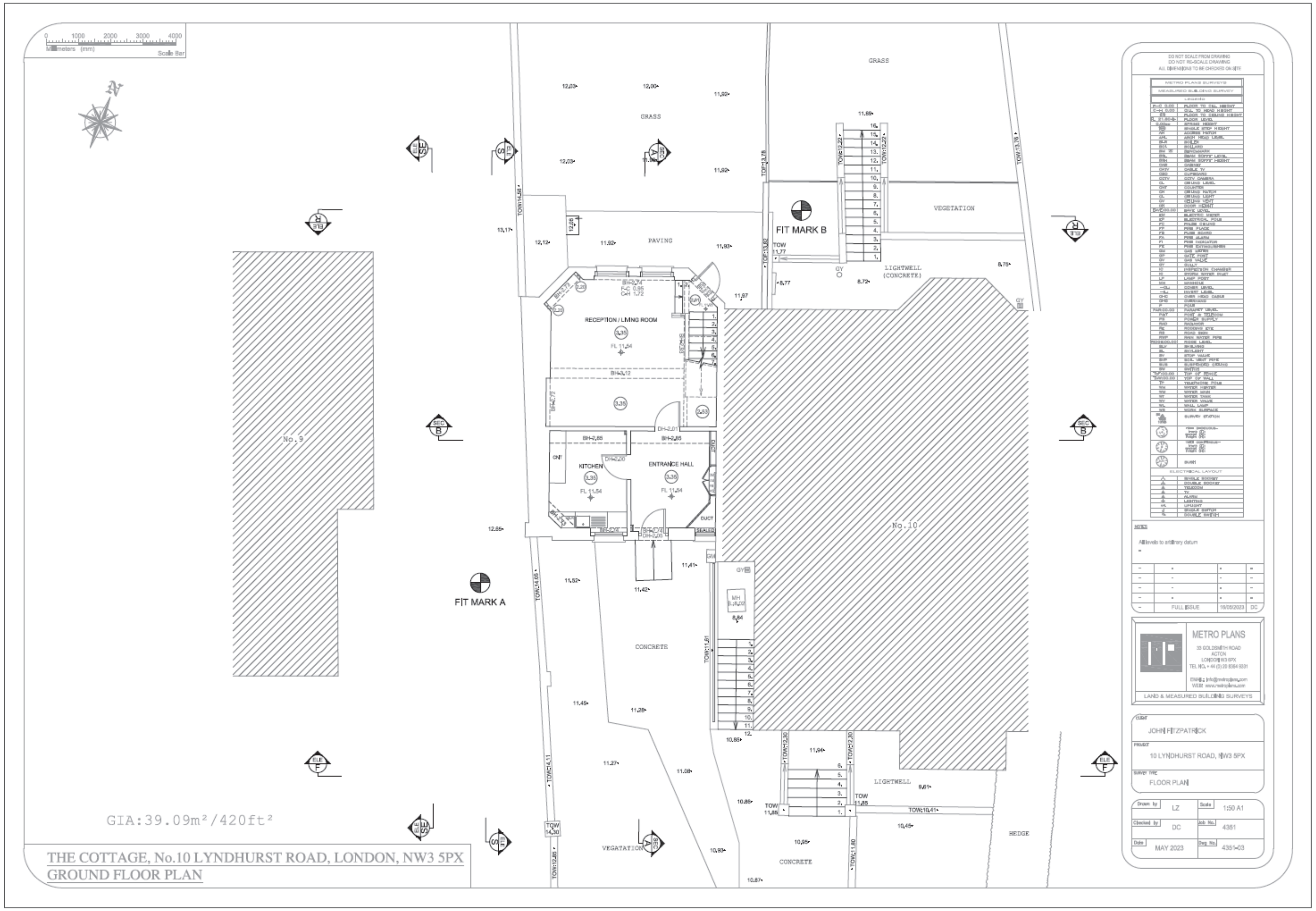
The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX

John Fitzpatrick c/o Momentum Structural Engineers	January 2024
Figure 2: Topographic Survey	GWPR5717

Site boundary

Not to Scale

The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX		ground&water
John Fitzpatrick c/o Momentum Structural Engineers	January 2024	
Figure 3: Aerial View of the Site	GWPR5717	



Not to Scale

The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX	
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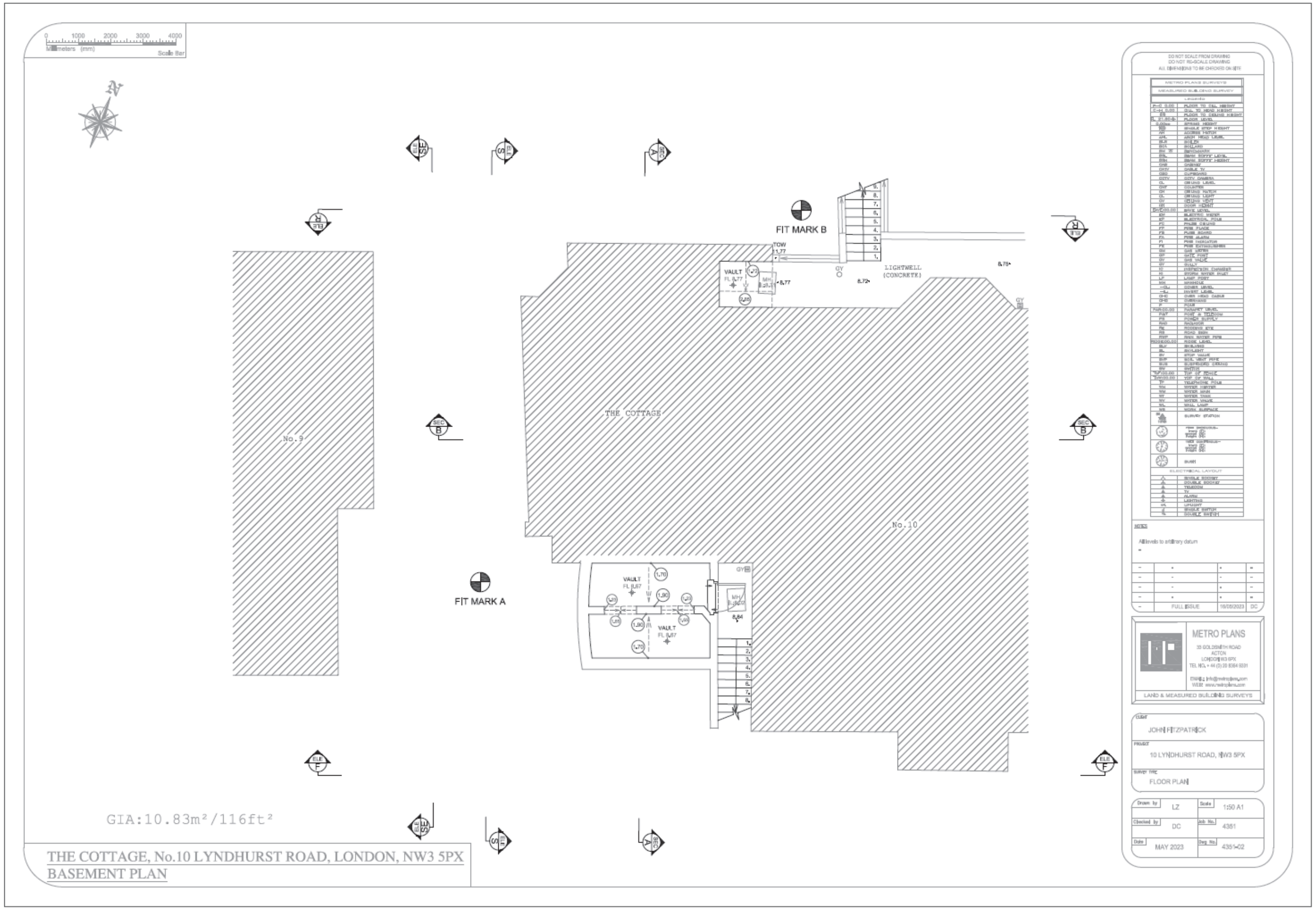
Figure 4: Existing Plan View – Ground Floor	GWPR5717

ground&water



Not to Scale

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Figure 5: Existing Plan View – Basement	GWPR5717

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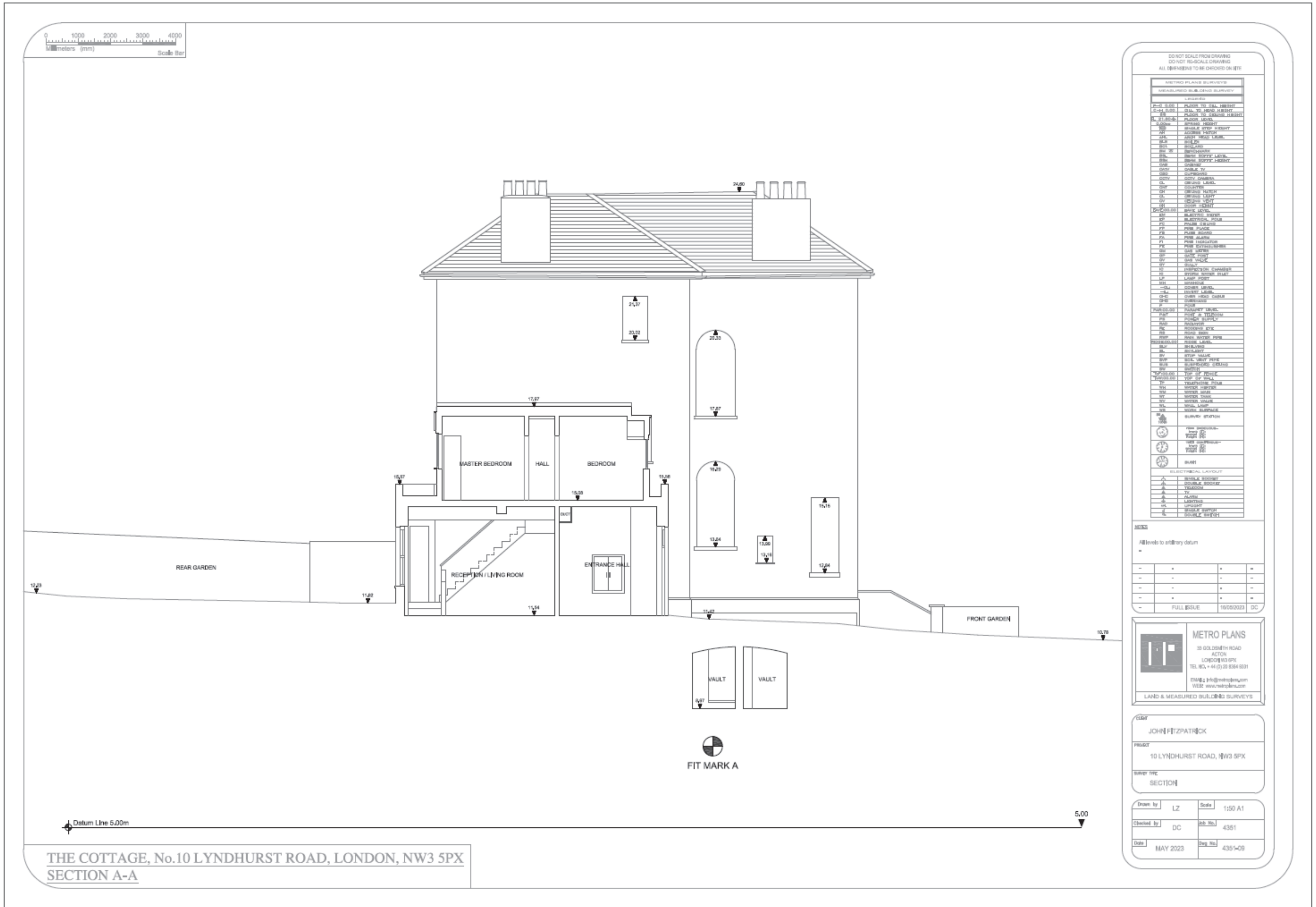

Not to Scale

The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX

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Figure 6: Existing Section View	GWPR5717

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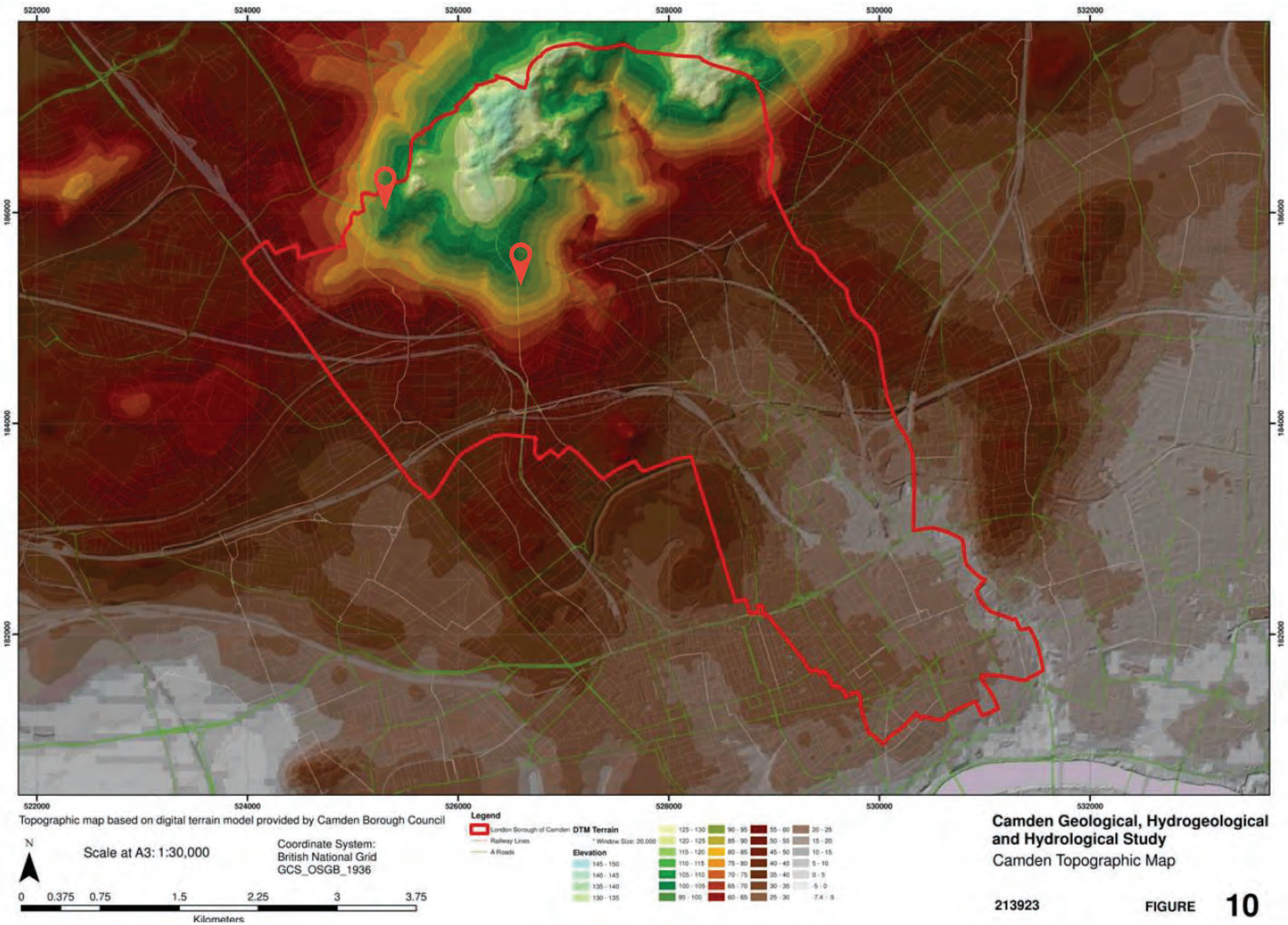

Site Location

Not to Scale

The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX	
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Figure 7: Topographical Map	**GWPR5717**

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Site boundary

Location of Tunnel

Not to Scale

The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX	
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Figure 8: Location of Overground London Underground Tunnel	**GWPR5717**

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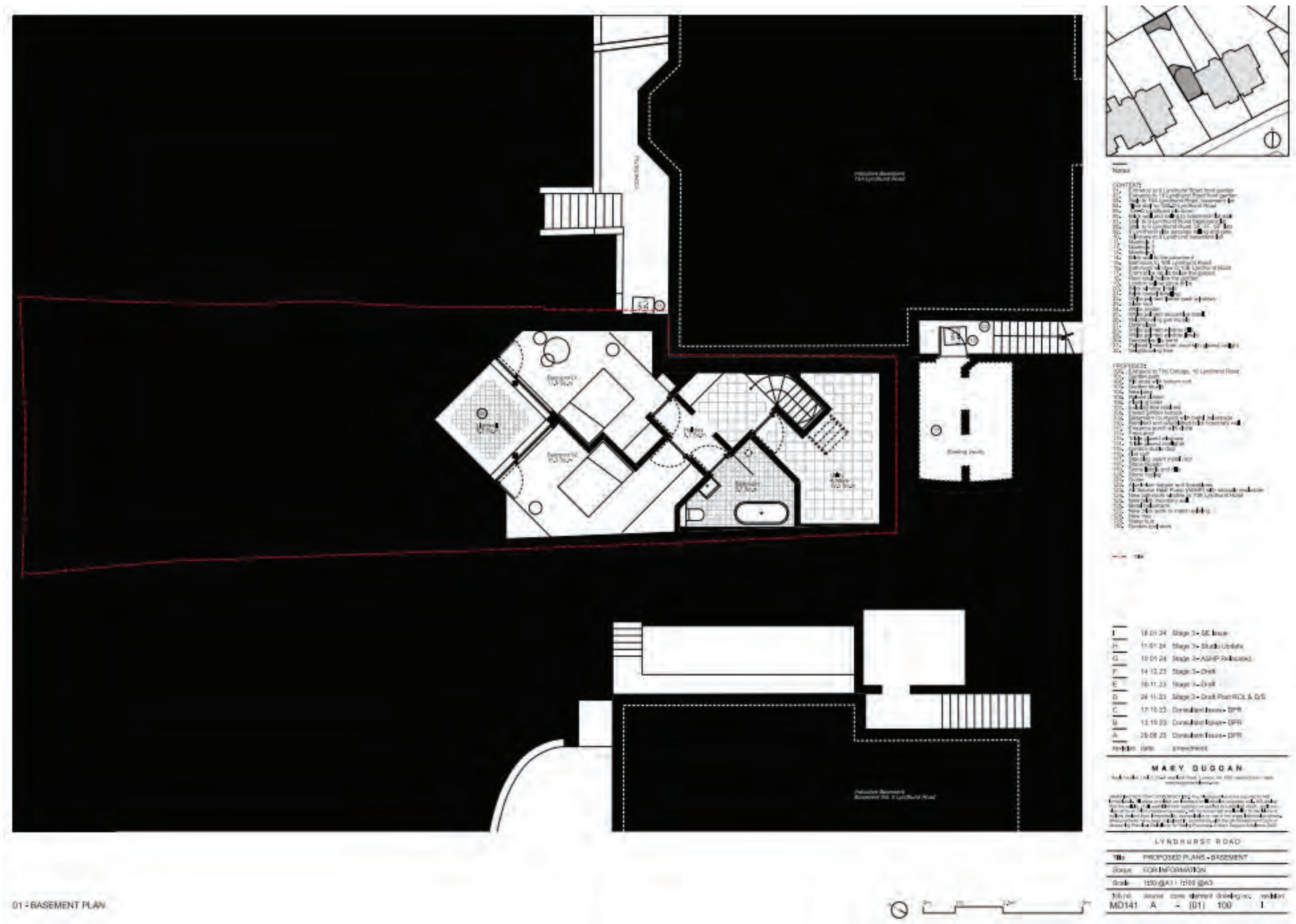


Not to Scale

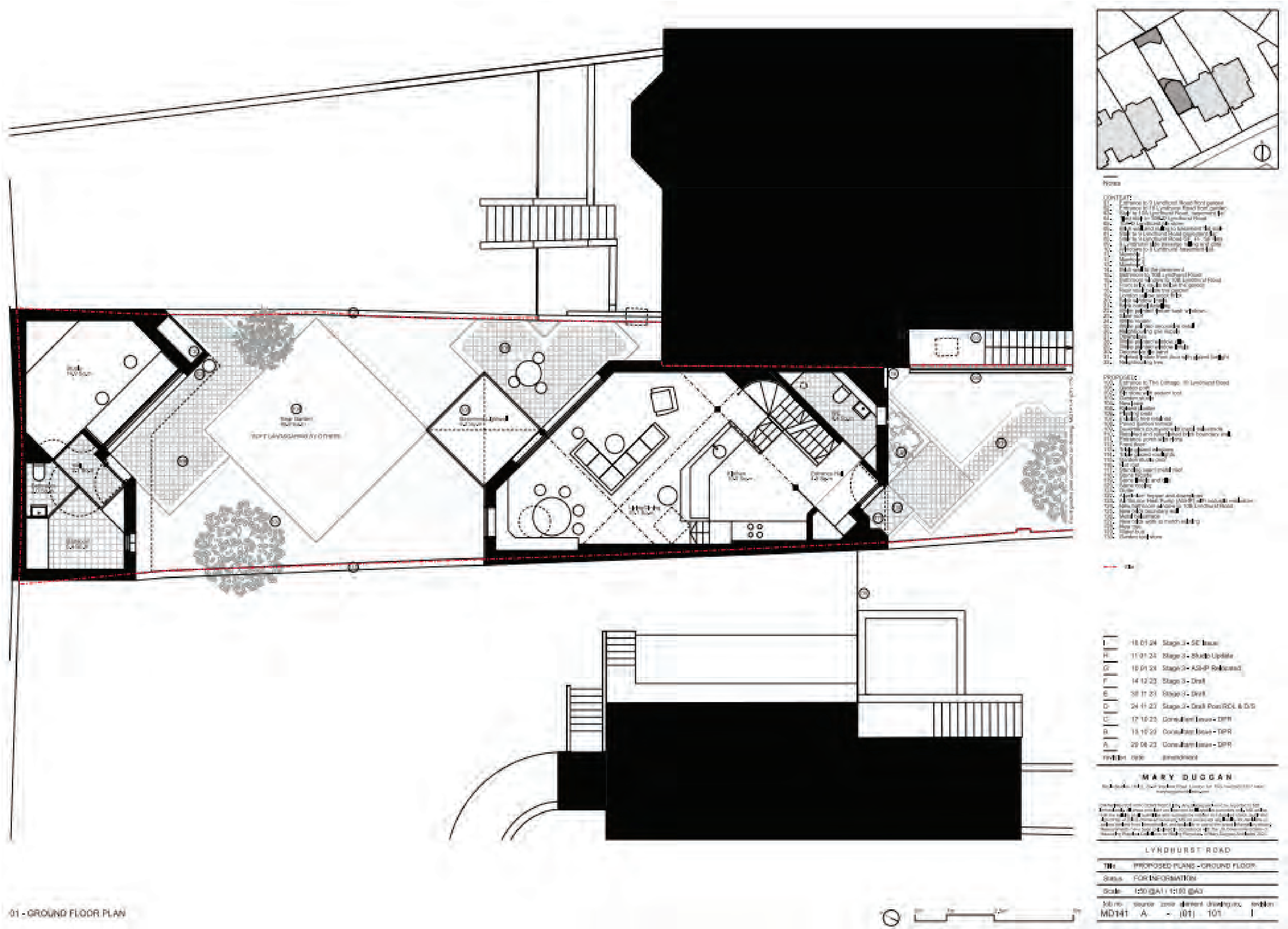
The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX

John Fitzpatrick c/o Momentum Structural Engineers

January 2024

Figure 9: Proposed Basement Floor Plan Views

GWPR5717



Not to Scale

The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX

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Figure 10: Proposed Ground Floor Plan View	GWPR5717

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Figure 11: Proposed Section View	**GWPR5717**

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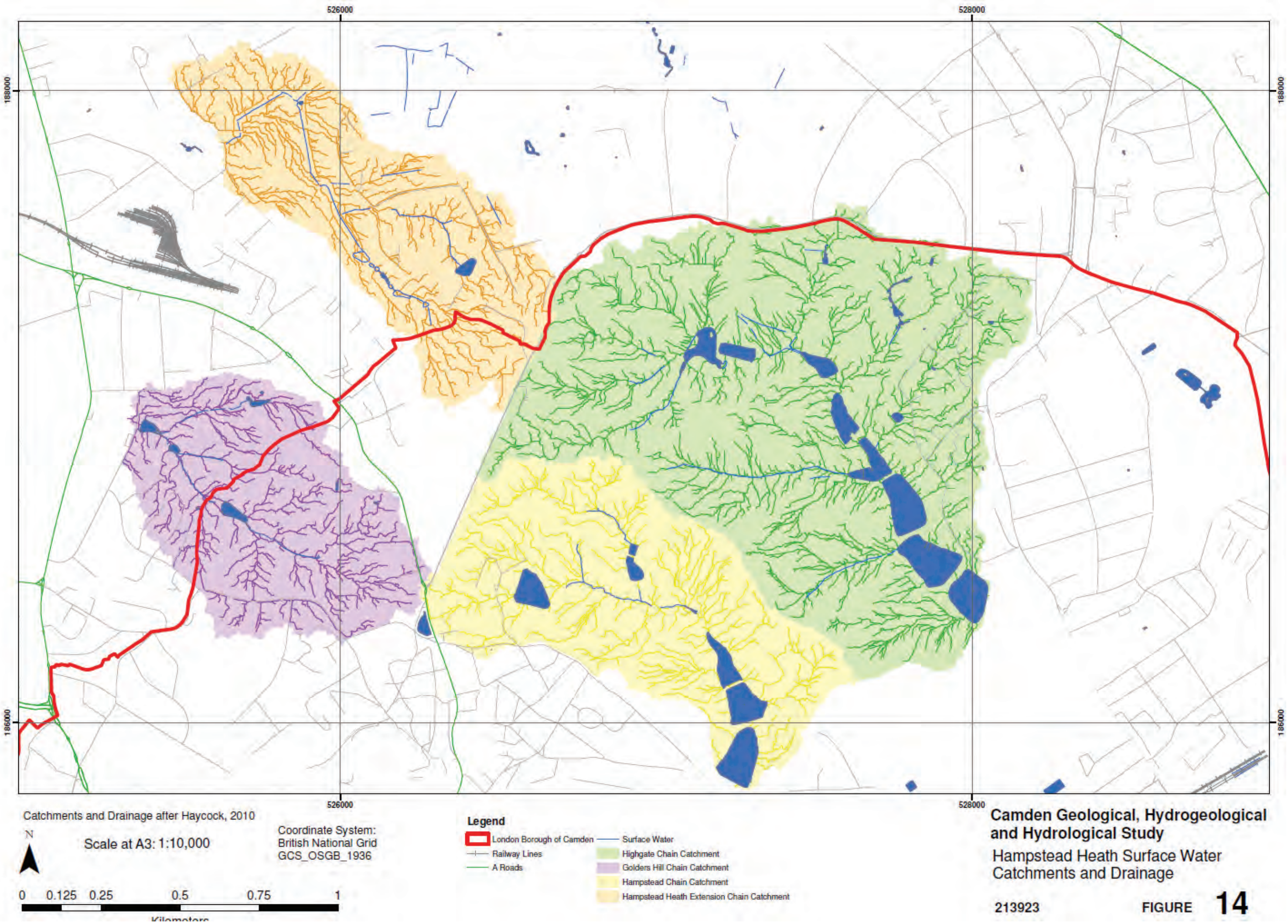

Not to Scale

The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX	
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Figure 12: Hampstead Heath Pond Drainage Basins	GWPR5717

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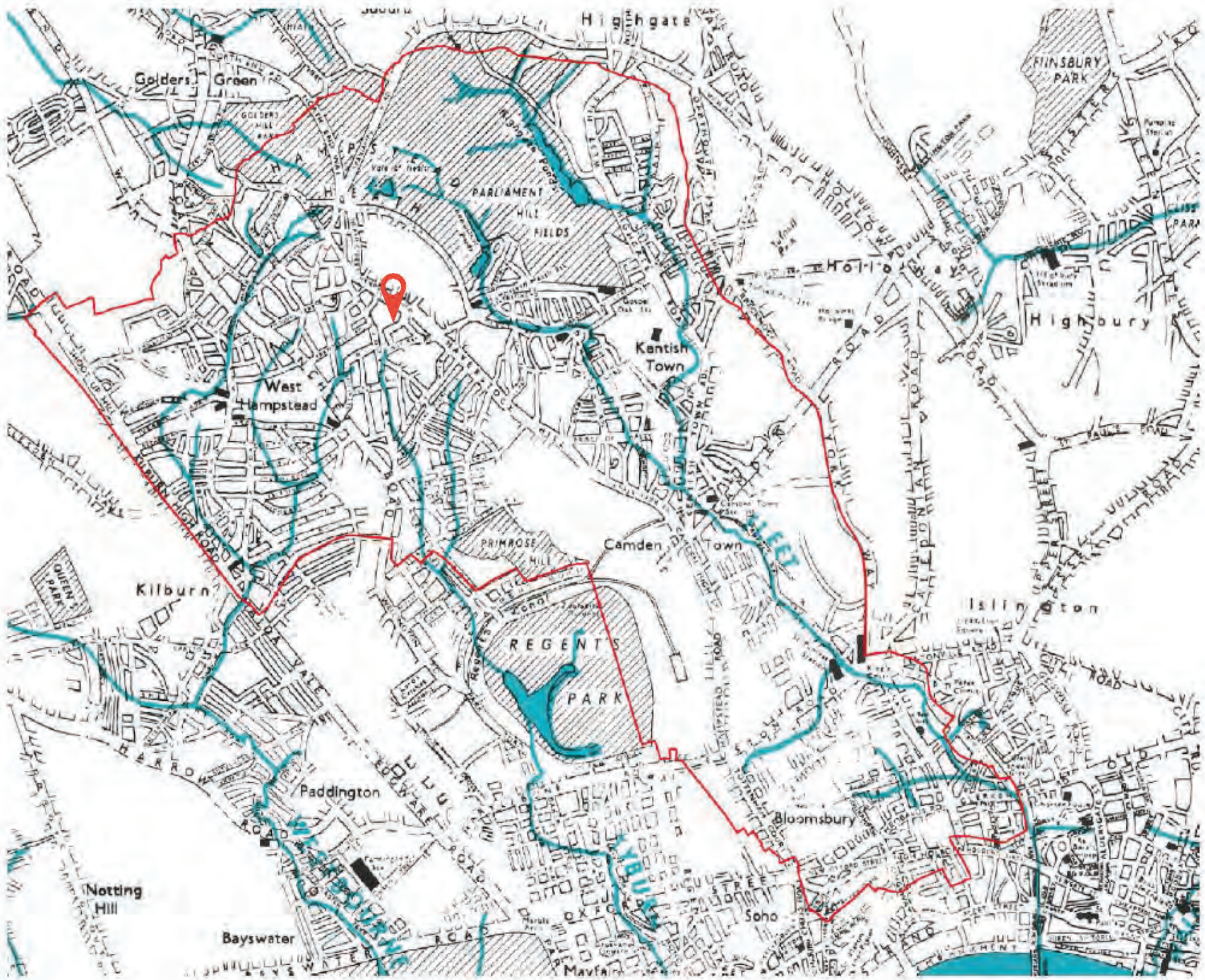

Source – Barton, Lost Rivers of London

Camden Geological, Hydrogeological and Hydrological Study
Watercourses

213923

FIGURE 11

Site Location

Not to Scale

The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX	
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Figure 13: Underground Streams and Diverted Spring Lines	GWPR5717

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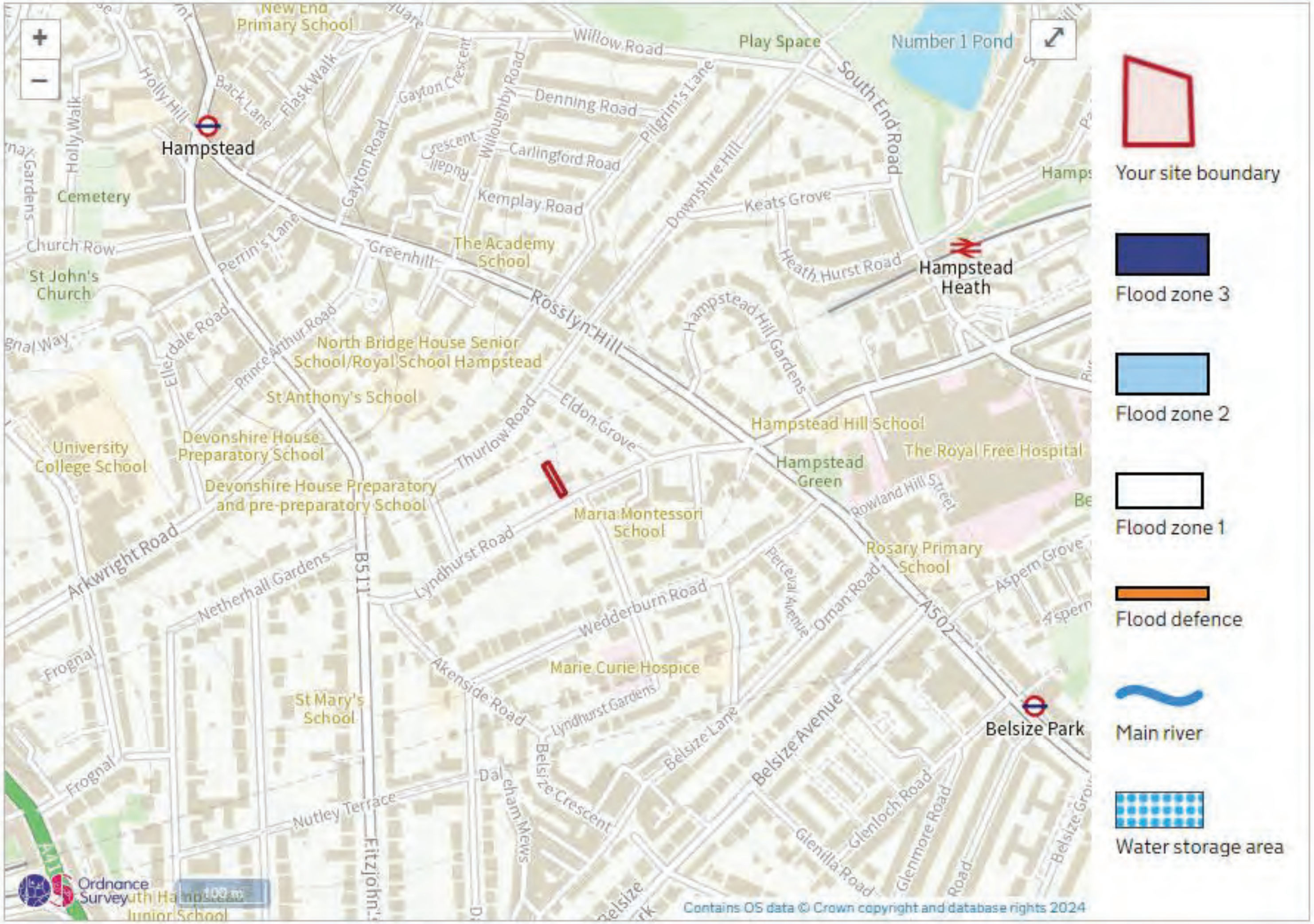

Not to Scale

The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX	
John Fitzpatrick c/o Momentum Structural Engineers	**January 2024**
Figure 14: Flood Map For Planning	**GWPR5717**

Flood risk

Extent of flooding

Location

NW3 5PX

Maximum extent of flooding from reservoirs:

when river levels are normal when there is also flooding from rivers

Site Location

Not to Scale

The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX	
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Figure 15: Reservoir Flood Risk Map	GWPR5717

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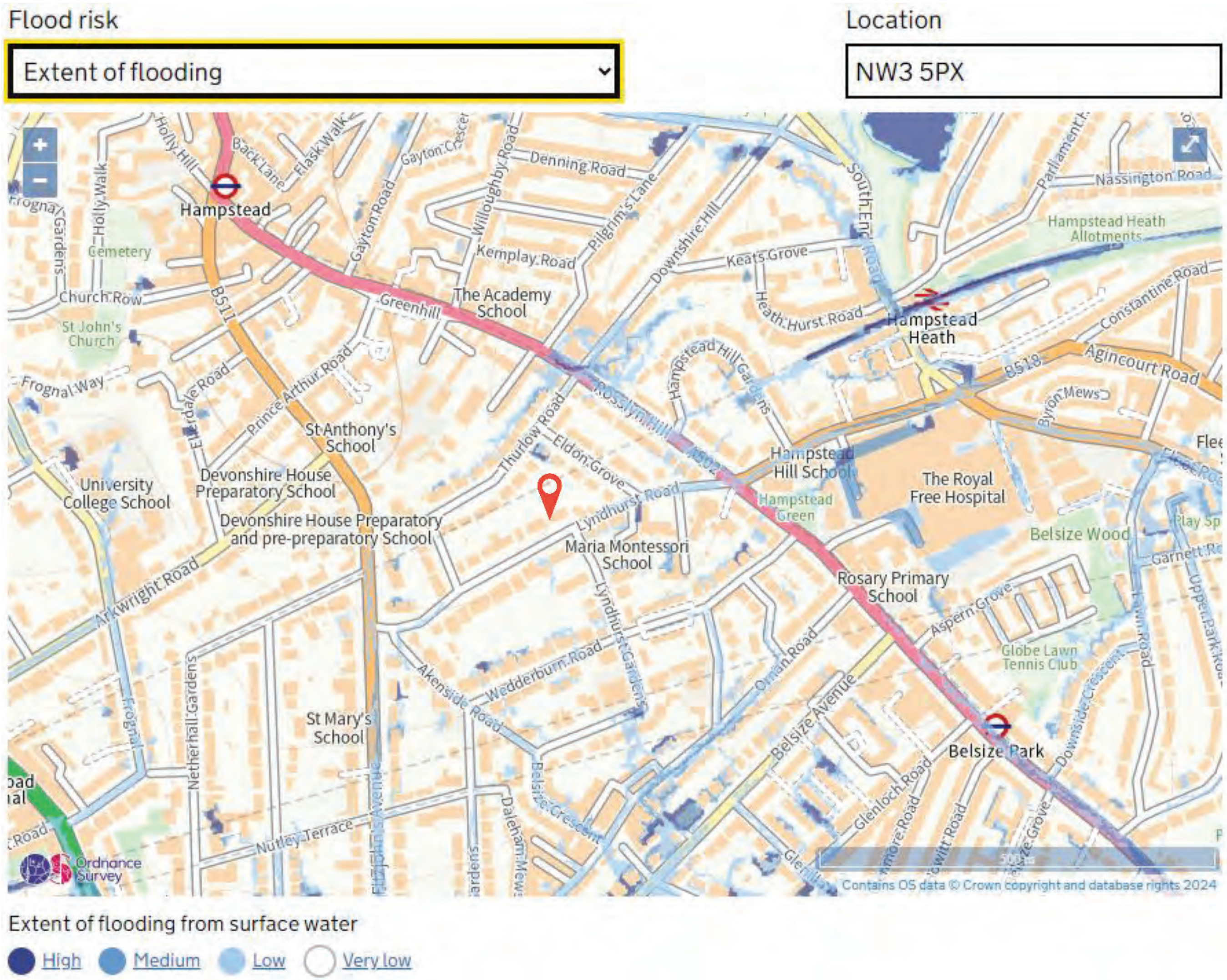

Site Location

Not to Scale

The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX	
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Figure 16: Surface Water Flooding Map	**GWPR5717**

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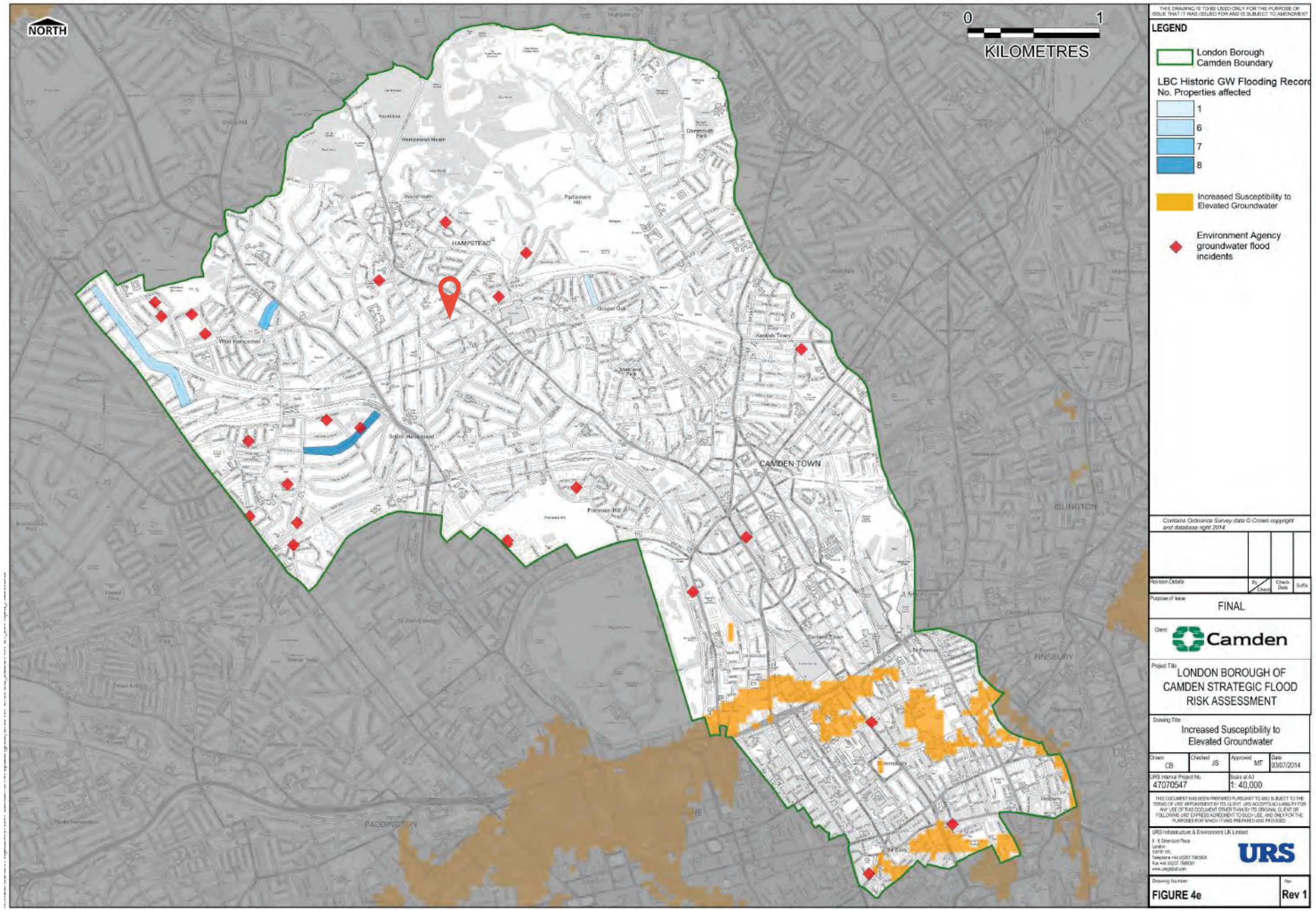

Not to Scale

The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX	
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Figure 17: Groundwater Flood Risk Map	GWPR5717

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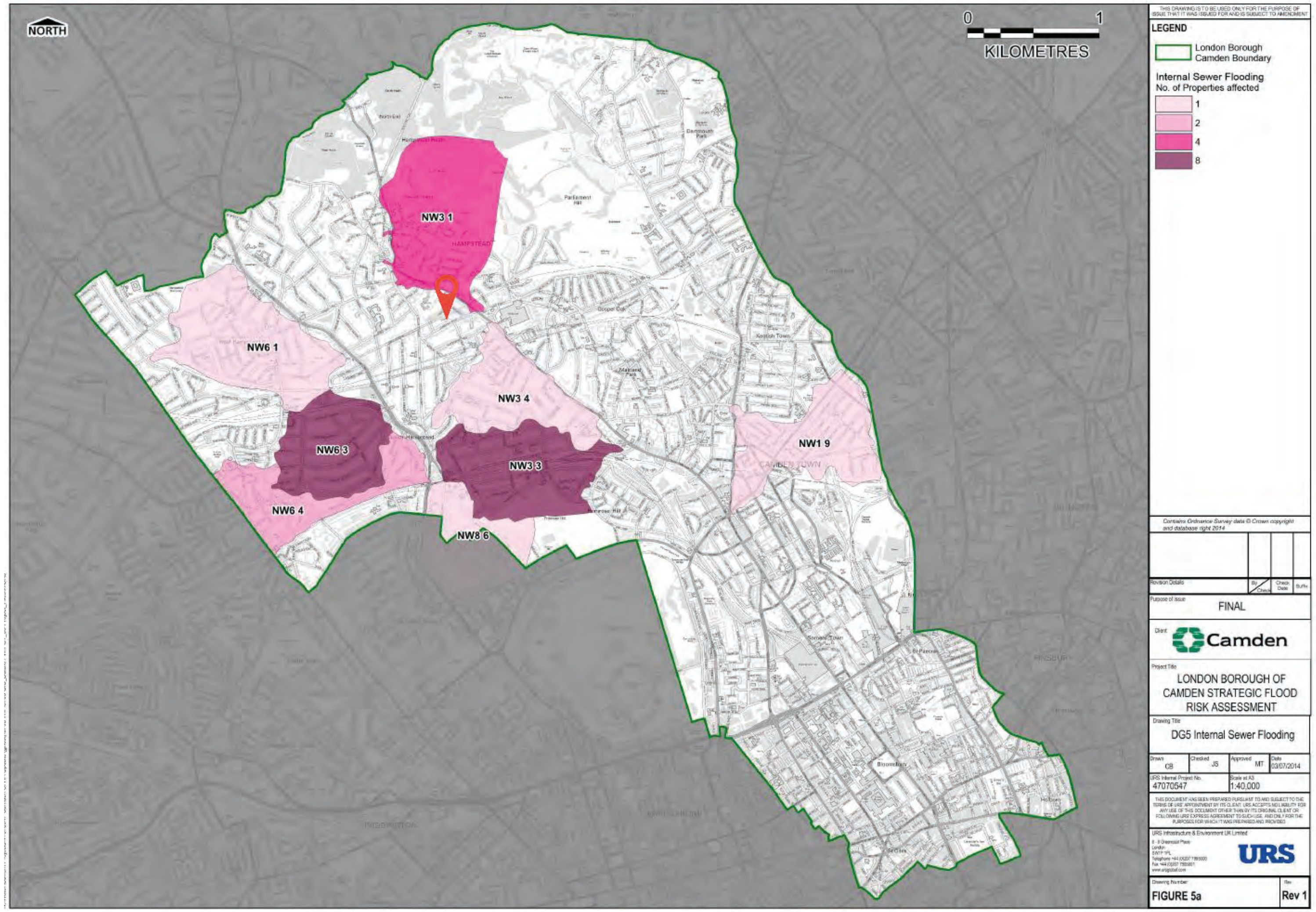

Site Location

Not to Scale

The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX	
John Fitzpatrick c/o Momentum Structural Engineers	**January 2024**
Figure 18: Internal Sewer Flooding Map	**GWPR5717**

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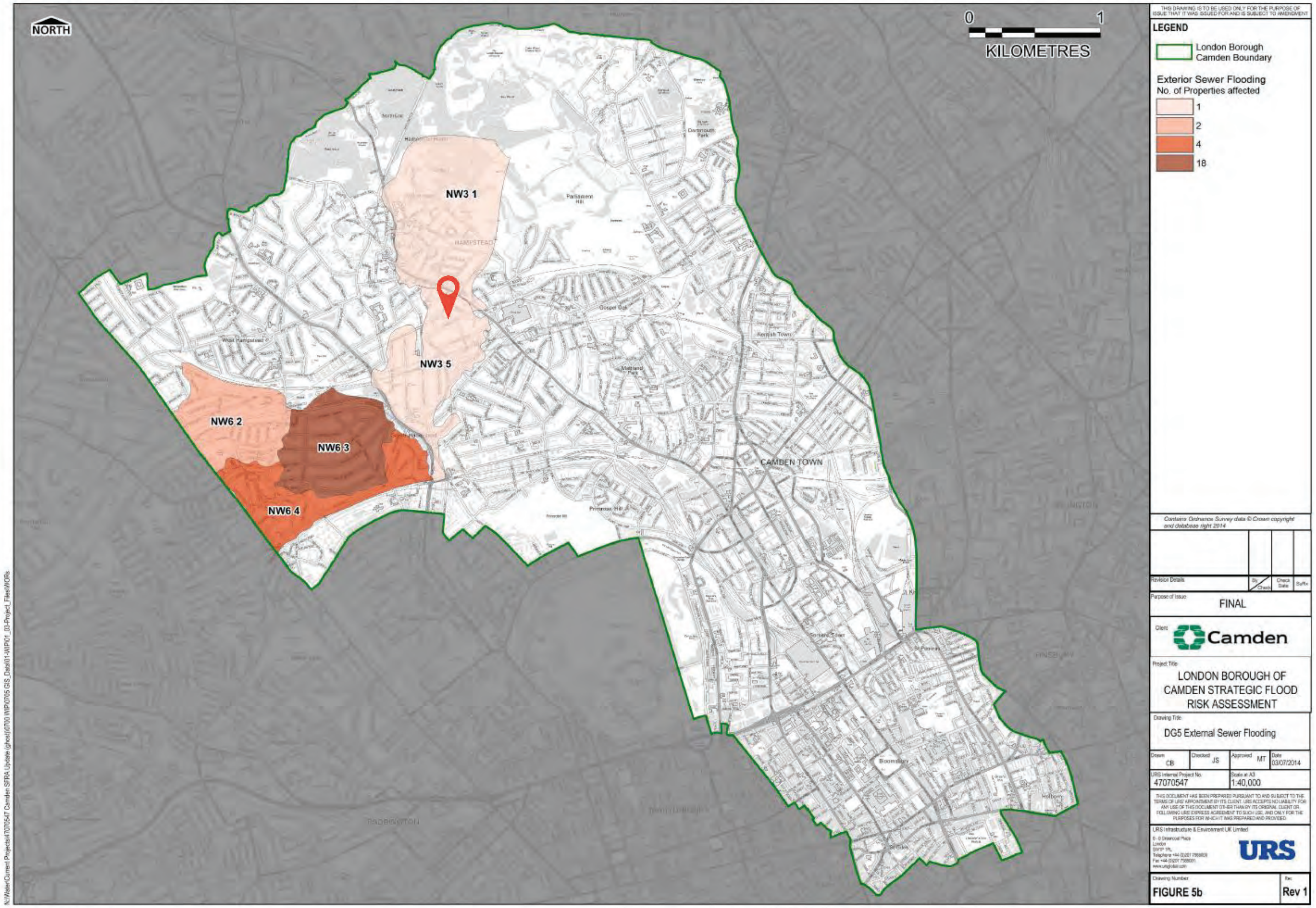

Site Location

Not to Scale

The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX	
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Figure 19: External Sewer Flooding Map	**GWPR5717**

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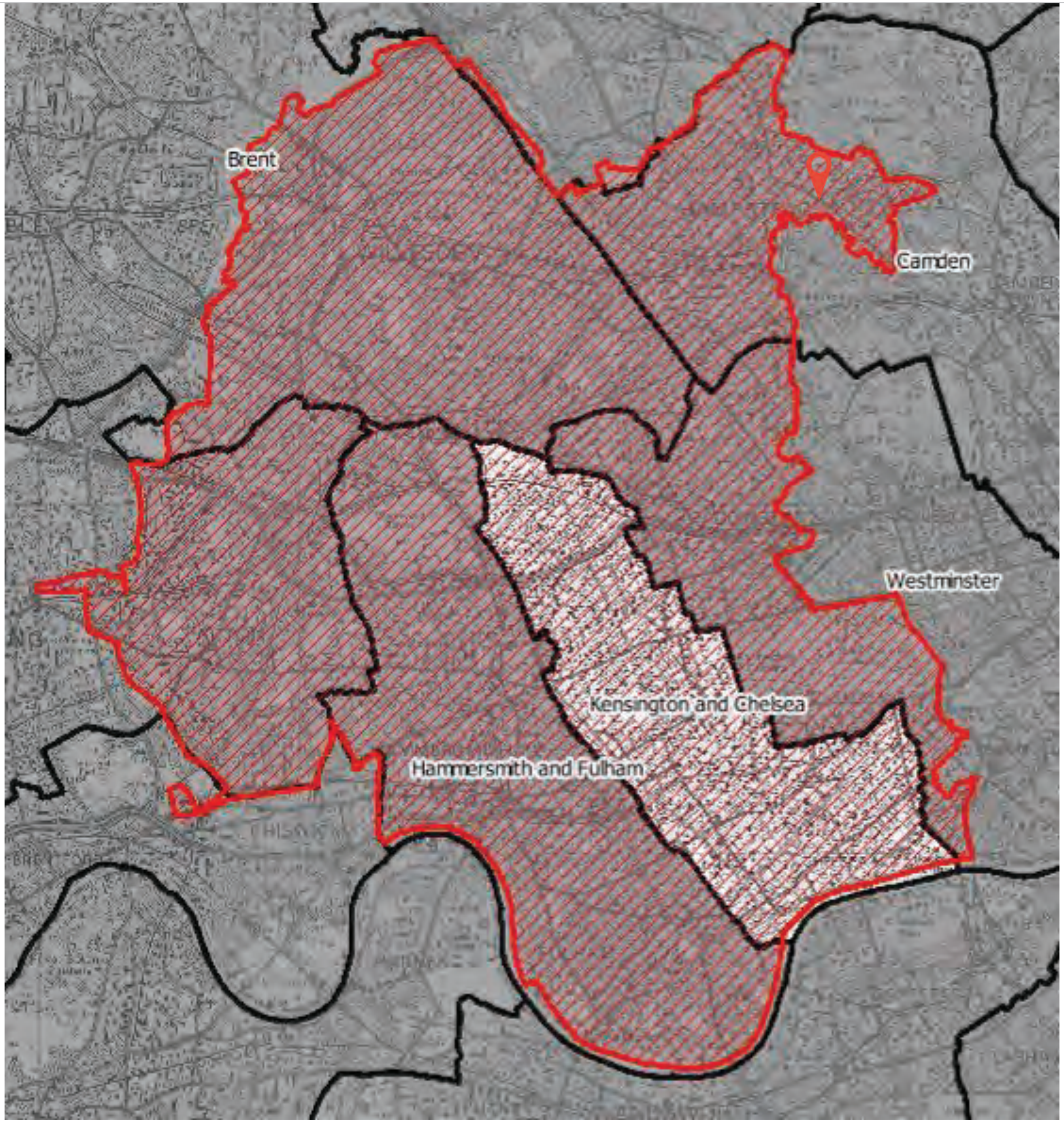

Site Location

Not to Scale

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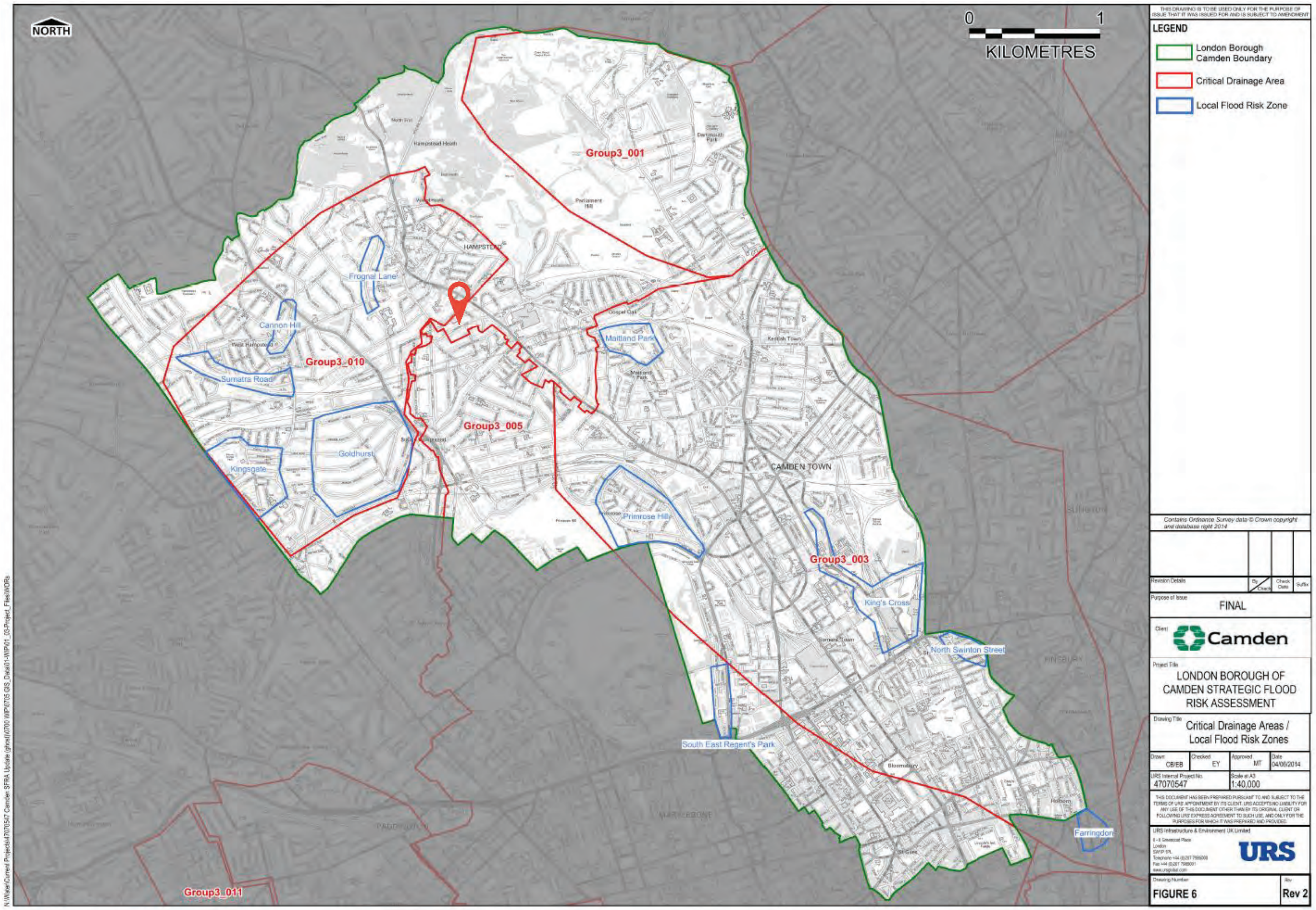
The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX	
John Fitzpatrick c/o Momentum Structural Engineers	January 2024
Figure 20: Counter’s Creek Critical Drainage Area	GWPR5717



Site Location

Not to Scale

The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX	
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Figure 21: Critical Drainage Area Map	GWPR5717

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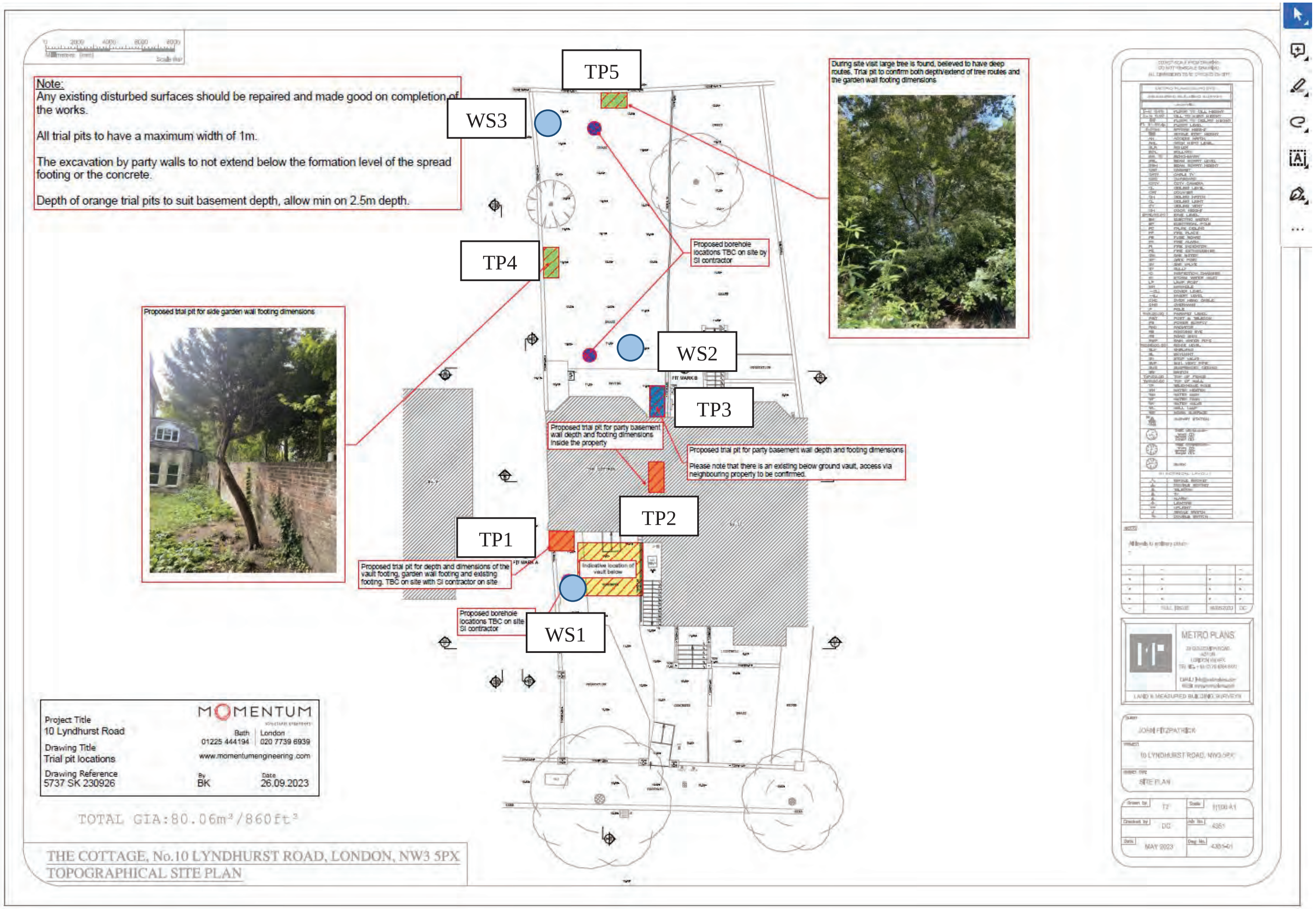

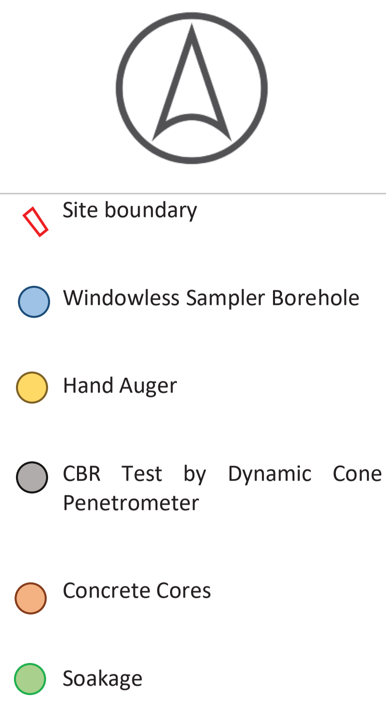

Not to Scale

The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX	
John Fitzpatrick c/o Momentum Structural Engineers	**January 2024**
Figure 22: Trial Hole Location Plan	**GWPR5717**

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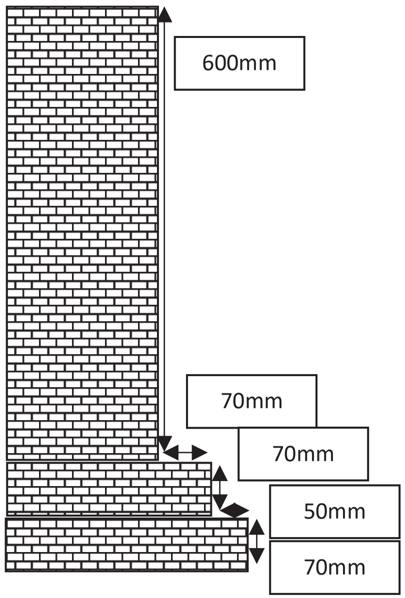
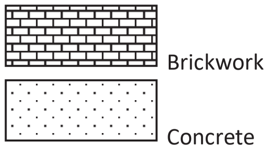


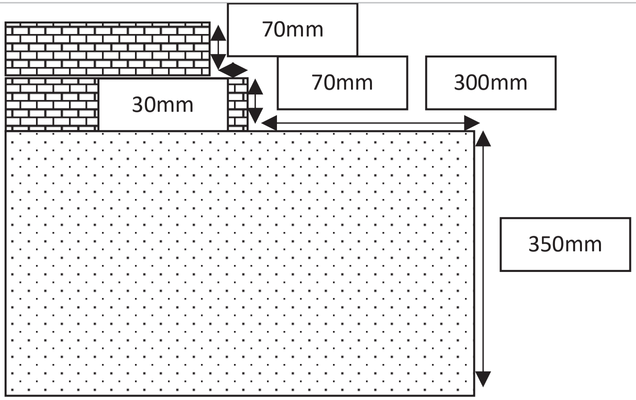
The base of the foundation was proven to 0.74m bgl, onto the London Clay Formation. Fresh roots were noted to 0.50m bgl.

Legend



The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX	
John Fitzpatrick c/o Momentum Structural Engineers	**January 2024**
Figure 23: Foundation Exposure TP1a (party wall)	**GWPR5717**

ground&water



The base of the foundation was proven to 0.56m bgl, onto the London Clay Formation. Fresh roots were noted to 0.50m bgl.

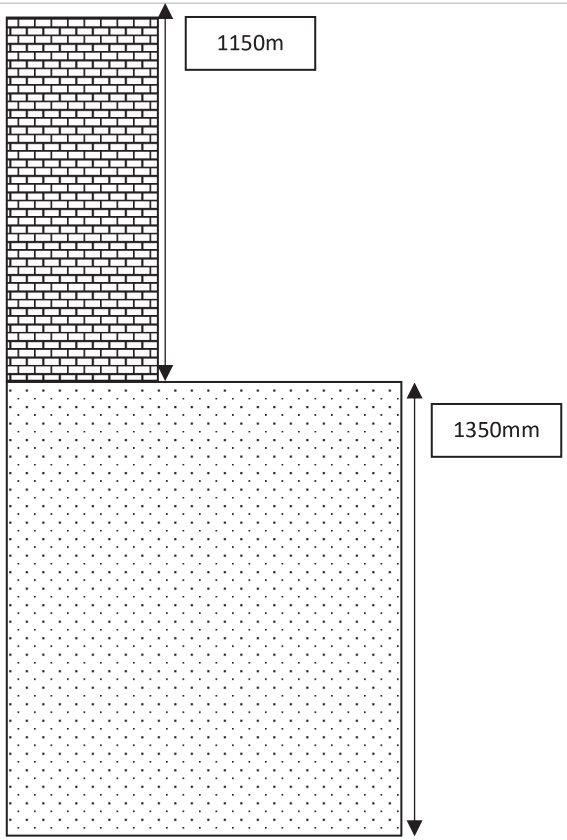
Legend

Brickwork

Concrete

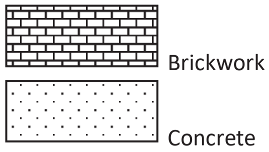
The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX	
John Fitzpatrick c/o Momentum Structural Engineers	**January 2024**
Figure 24: Foundation Exposure TP1b (front façade)	**GWPR5717**

ground&water



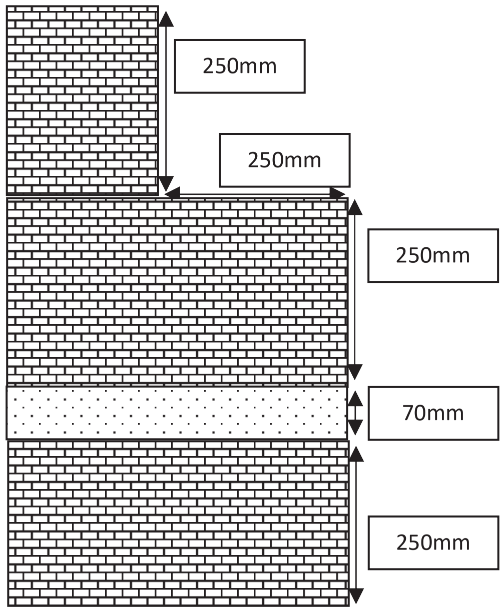
The base of the foundation was >2.50m bgl and not proven. Fresh roots were noted to 0.50m bgl.

Legend



The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX	
John Fitzpatrick c/o Momentum Structural Engineers	**January 2024**
Figure 25: Foundation Exposure TP1c (garden vault)	**GWPR5717**

ground&water



The base of the foundation was proven to 2.75m bgl, onto the London Clay Formation. Fresh roots were noted to 0.30m bgl.

Legend

Brickwork

Concrete

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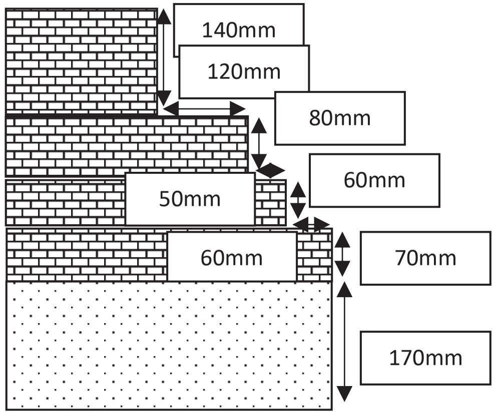
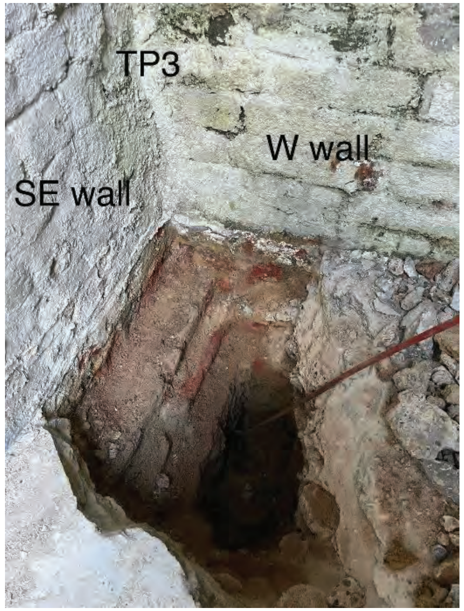
John Fitzpatrick c/o Momentum Structural Engineers	**January 2024**
Figure 26: Foundation Exposure TP2	**GWPR5717**

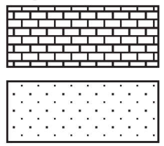
ground&water



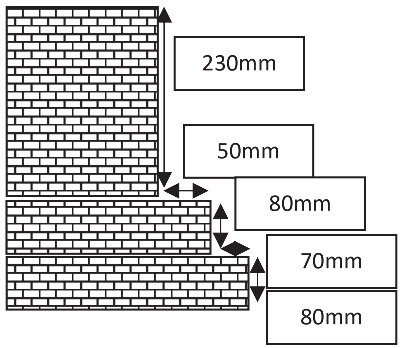


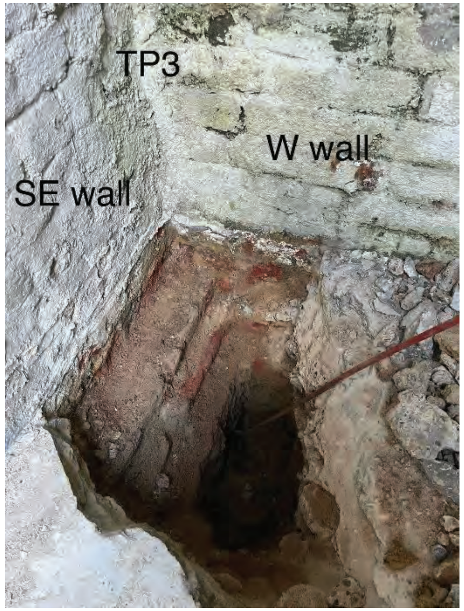
The base of the foundation was proven to 0.52m bgl, onto the London Clay Formation. No fresh roots were encountered.

Legend



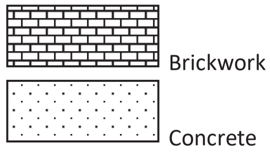
The Cottage, 10 Lyndhurst Road, Hampstead, London NW3 5PX	
John Fitzpatrick c/o Momentum Structural Engineers	**January 2024**
Figure 27: Foundation Exposure TP3 (south-east wall)	**GWPR5717**





The base of the foundation was proven to 0.39m bgl, onto the London Clay Formation. No fresh roots were encountered.

Legend



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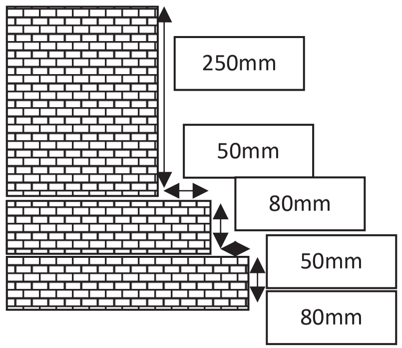
John Fitzpatrick c/o Momentum Structural Engineers	**January 2024**
Figure 28: Foundation Exposure TP3 (west wall)	**GWPR5717**

ground&water



The base of the foundation was proven to 0.41m bgl, onto the Made Ground. Fresh roots were noted for the full depth of TP4 (0.45m bgl)

Legend

Brickwork

Concrete

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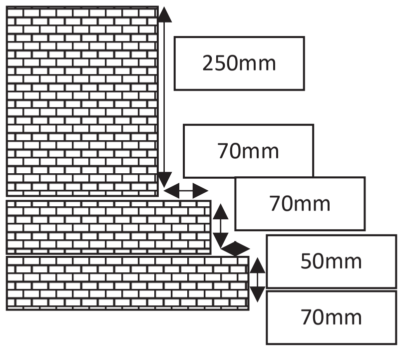
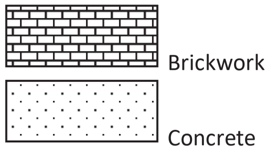
John Fitzpatrick c/o Momentum Structural Engineers	**January 2024**
Figure 29: Foundation Exposure TP4	**GWPR5717**



The base of the foundation was proven to 0.39m bgl, onto the Made Ground. Fresh roots were noted for the full depth of TP5 (0.40m bgl)

Legend



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John Fitzpatrick c/o Momentum Structural Engineers	**January 2024**
Figure 30: Foundation Exposure TP5	**GWPR5717**

ground&water