Basement Impact Assessment

Site 5 Templewood Avenue
London NW3 7UY

Client Shirley Stone
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Our Ref BIA/6162 R1.1

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Foreword

This report has been prepared in accordance with the scope and terms agreed with the Client, and the resources available, using all reasonable professional skill and care. The report is for the exclusive use of the client and shall not be relied upon by any third party without explicit written agreement from Chelmer Site Investigation Laboratories Ltd.

This report is specific to the proposed site use or development, as appropriate, and as described in the report; Chelmer Site Investigation Laboratories Ltd accept no liability for any use of the report or its contents for any purpose other than the development or proposed site use described herein.

This assessment has involved consideration, using normal professional skill and care, of the findings of ground investigation data obtained from the Client and other sources. Ground investigations involve sampling a very small proportion of the ground of interest as a result of which it is inevitable that variations in ground conditions, including groundwater, will remain unrecorded around and between the exploratory hole locations; groundwater levels/pressures will also vary seasonally and with other man-induced influences; no liability can be accepted for any adverse consequences of such variations.

This report must be read in its entirety in order to obtain a full understanding of our recommendations and conclusions.

Report Status: **FINAL**		
Role	**By**	**Signature**
Lead author:	Keith Gabriel MSc DIC CGeol FGS UK Registered Ground Engineering Adviser	
Slope/ground stability aspects approved by:	Mike Summersgill MSc CEng MICE C.WEM FCIWEM	
Subterranean (Groundwater) flow aspects approved by:	Keith Gabriel MSc DIC CGeol FGS	
Surface flow and flooding aspects approved by:	Mike Summersgill MSc CEng MICE C.WEM FCIWEM	

1.0 INTRODUCTION

1.1 This Basement Impact Assessment has been prepared in support of a planning application to be submitted to the London Borough of Camden (LBC) for the construction of a basement beneath No.5 Templewood Avenue, NW3 7UY. Details of the proposed basement are given in Section 3. The assessment is in accordance with the requirements of the London Borough of Camden (LBC) Development Policy DP27 in relation to basement construction, and follows the requirements set out in LBC's guidance document CPG4 'Basements and Lightwells' (July 2015).

1.2 This assessment has been prepared by Keith Gabriel, a Chartered Geologist with an MSc degree in Engineering Geology (who has specialised in slope stability and hydrogeology), and Mike Summersgill, a Chartered Civil Engineer and Chartered Water and Environmental Manager with an MSc degree in Soil Mechanics (geotechnical and hydrology specialist). Both authors have previously undertaken assessments of basements in several London Boroughs.

1.3 A preliminary site inspection (walk-over survey) of the house was undertaken on Friday 12th February 2016. Photos from that visit are presented in Appendix A. Desk study data have been collected from various sources including borehole records (Appendix B) and geological data, environmental data and historic maps from Groundsure which are presented in Appendices E, F and G. Relevant information from the desk study and site inspections is presented in Sections 2–6, followed by the Basement Impact Assessment in accordance with CPG4 Stages 1–4 in Sections 7–10 respectively. The factual report on the ground investigation is included in Appendix C and the findings are summarised in Section 9.

1.4 The following site-specific documents in relation to the proposed basement and planning application have been considered:

Survey Solutions / Brod Wight Architects:

<u>Existing</u>

- Drg No. 1046-S01 Site Plan
- Drg No. 1046-S02 Basement Floor Plan
- Drg No. 1046-S03 Ground Floor Plan
- Drg No. 1046-S04 First Floor Plan
- Drg No. 1046-S05 Second & Third Floor Plans
- Drg No. 1046-S07 Front Elevation
- Drg No. 1046-S08 Rear Elevation
- Drg No. 1046-S09 North East Side Elevation
- Drg No. 1046-S10 South West Side Elevation
- Drg No. 1046-S11 Section A-A
- Drg No. 1046-S12 Section B-B
- Drg No. 1046-S13 Section C-C

<u>Proposed</u>

- Drg No. 1046-AP01 Site Plan
- Drg No. 1046-AP02 Basement Floor Plan
- Drg No. 1046-AP03 Ground Floor Plan
- Drg No. 1046-AP04 First Floor Plan
- Drg No. 1046-AP05 Second & Third Floor Plans
- Drg No. 1046-AP07 Front Elevation
- Drg No. 1046-AP08 Rear Elevation

- Drg No. 1046-AP09 North East Side Elevation
- Drg No. 1046-AP10 South West Side Elevation
- Drg No. 1046-AP11 Section A-A
- Drg No. 1046-AP12 Section B-B (North Wing)
- Drg No. 1046-AP13 Section B-B (Main House)

Elliott Wood Partnership LLP (Consulting Engineers)

- Drg No. 2150493 SK001 rev.P1 Typical Underpin Detail
- Drg No. 2150493 S-90 rev.P4 Lower Ground Plan
- Drg No. 2150493 S-100 rev.P4 Ground Floor Plan
- Drg No. 2150493 S-110 rev.P4 First Floor Plan
- Loading Mark-up on Draft S-90 P3 Plan.

This report should be read in conjunction with all the documents and drawings listed above.

1.5 Instructions to prepare this Basement Impact Assessment (BIA) were confirmed by purchase order No. PO5598.

2.0 THE PROPERTY AND TOPOGRAPHIC SETTING

2.1 No.5 Templewood Avenue is a large, three-/four-storey, detached house, situated within the Redington Frognal Conservation area in the London Borough of Camden. It has a small original(?) two-storey projection on the north-east side and a single-storey extension at the rear. The property has been divided into separate flats, the configuration of which has changed over the years. Templewood Avenue is located between Branch Hill to the east, and Redington Road to the west/south, and can be accessed at its northern and southern ends where it joins West Heath Road and Redington Road respectively. As shown in Figure 1, No.5 is situated on the north-west side of Templewood Avenue, opposite the junction with Templewood Gardens, between No's 3 & 5a Templewood Avenue, which are located to the south-west and north-east respectively (see Photo 1 in Appendix A). To the north-west, the property is bounded by the rear garden of No.58a Redington Road, and a block of garages. In Figure 1, the site of No.4 Templewood Avenue is shown in white, because it was/is undergoing significant alterations (see paragraph 2.8).

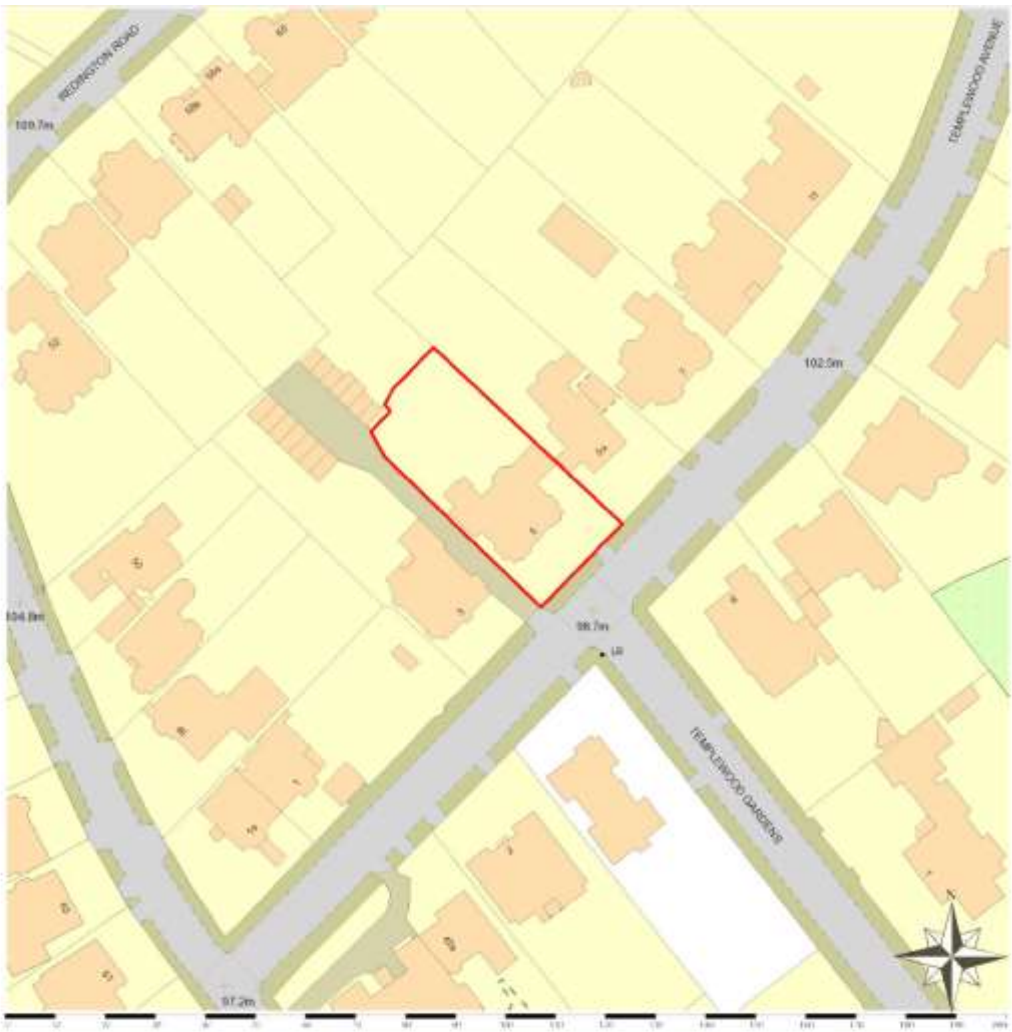


Figure 1: Extract from 1:1,250 OS map (not to scale) with the site outlined in red

2.2 Externally, at the front of the house, there is a large parking/amenity area which is mostly surfaced with concrete/asphalt, and falls southwards away from the front of the house (Photos 1-3). Along the north-eastern boundary with No.5a Templewood Avenue there is a narrow raised border, which widens at the east corner of the site around a large oak tree. There are other planting areas either side of the front entrance to No.5. A low brick wall separates this parking/amenity area from the footway (except at its two access points with the Templewood Avenue carriageway), behind which is a hedge, and to the north-east and south-west, this parking/amenity area is bounded by brick walls.

2.3 To the rear of the house, there is a large patio area within which are several small planting areas (Photo 5). This patio is separated from the rear wall of the house by a U-shaped recessed drainage channel. Upslope of this patio is an area which has been mostly laid to lawn (Photo 4), and also includes numerous planting/soft landscaped areas, as well as a small rockery and wooden terrace, located to the rear of the single-storey rear projection. Overall, the rear garden falls relatively steeply towards the rear of the house; however it has been terraced, to form two broadly level areas. Wooden fences separate the rear garden from the adjoining rear gardens to No.5a Templewood Avenue and No.58a Redington Road, and the access road to the west of the house, although adjacent to the single-storey rear projection, the wooden fence panels sit on top of low brick walls. To the rear of the single-storey rear projection is a large Sycamore tree, which is to be removed.

2.4 No evidence was seen of major crack damage to the superstructure of No.5 Templewood Avenue, however within the rear garden, immediately to the rear of the single-storey projection, the low retaining wall which separates part of the rear patio from a planting area appeared to be failing, and is leaning towards the house (Photo 6). At the front of the property, the south-western brick boundary wall which separates the front parking/amenity area to No.5 from the access road to the garages shows some crack damage, some of which may have been caused by an impact with a vehicle (lorry/van?). On the opposite side of the front parking/amenity area, the brick boundary wall which separates No's 5/5a Templewood Avenue also shows significant crack damage, which is thought to have been cause by tree root activity, given the close proximity of a large Oak tree (trunk and damaged wall visible in Photo 3).

2.5 Reference to the historic OS map dated 1870 and the 1871 Town Plan (see Appendix G), shows the plot of No.5 Templewood Avenue within a field, labelled No.91. With the exception of West Heath Road to the north, little of the surrounding road network had been constructed prior to this date, and the surrounding area consisted primarily of farmland. Substantial development of the area began with the construction of Redington Road, between publication of the 1871 and 1893 OS maps, and continued between publication of the 1896 and 1915 OS maps, with the construction of the Templewood Avenue carriageway, and almost all of the properties on Templewood Avenue, including No.5. The adjacent No.5a Templewood Avenue was built at a much later date, between publication of the 1953 and 1970 OS maps.

2.6 No.5 Templewood Avenue is situated on a broadly south-facing slope, between two weakly developed valleys associated with the alignment of two former tributaries to the river Westbourne. The contours on the 1:25,000 scale Ordnance Survey (OS) map indicate an overall slope angle within the immediate vicinity of the site of approximately 4.1° (measured between the 100m contour, which crosses the southern part of the site, and the 105m contour just to the north of the site). Considerable variation in the slope angle can be found both above and below the site, with measured slope angles ranging from 7.0° (between the 110m and 120m contours) to approximately 2.4° (between the 100m and 95m contours). This results in an overall concave slope profile with a decreasing gradient towards its base. Figure 16 of the Camden GHHS (Camden Geological, Hydrogeological and Hydrological Study by Arup, November 2010) suggests that there are slopes >7° within the southern part of the site, and a very localized area which slopes at an angle >10°, immediately to the south-west of the site – see extract presented in Figure 3. Slopes >7° are probably present in both the rear garden and the front amenity area to No.5, while the >10° slope is probably located within the access drive to the garages, located alongside No.5.

2.7 Using the spot heights on Figure 1, a slope angle of around 0.8° towards the south-west was calculated for the Templewood Avenue carriageway, between No.5 and the junction of Templewood Avenue with Redington Road. A much steeper slope angle of around 3.1° was calculated for the carriageway upslope of No.5, between No's 5 and 11 Templewood Avenue.

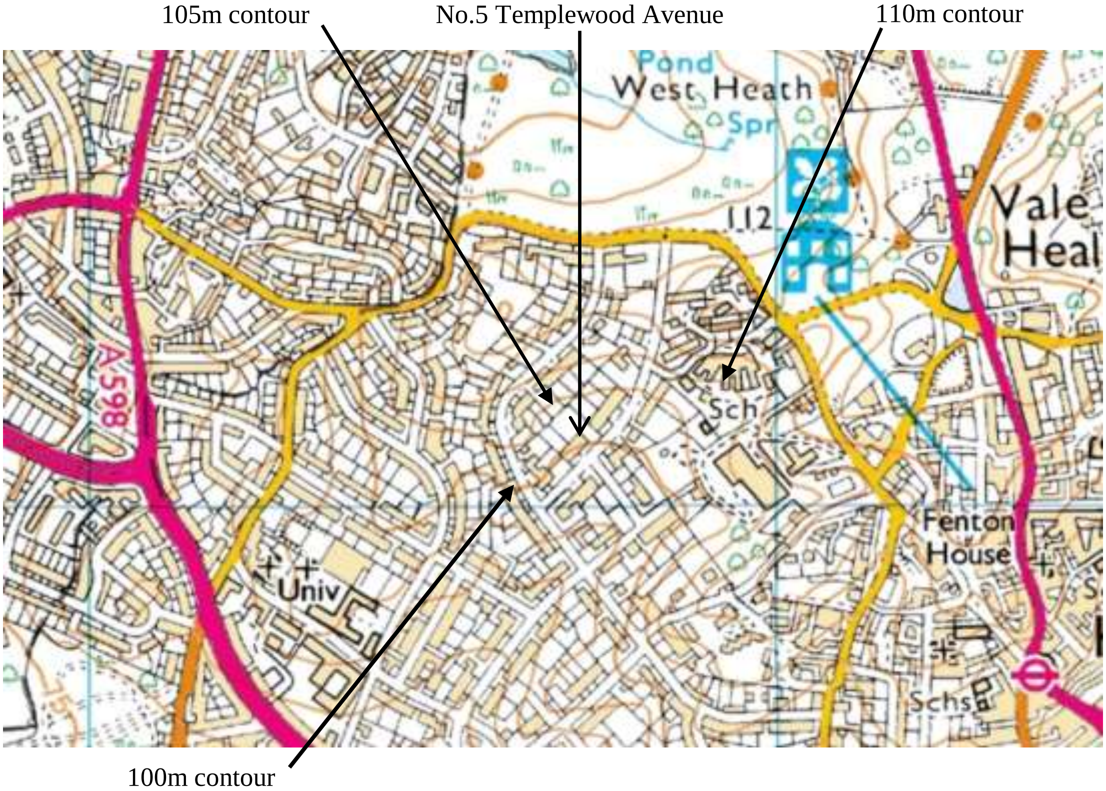


Figure 2: Enlarged extract from 1:25,000 Ordnance Survey map showing site location.

2.8 The bomb map for Hampstead shows that no hits were recorded within close proximity to the site, however three hits were recorded to the east of the site. These hits were high explosive or incendiary bombs, which are estimated to have landed approximately at the site of No.12 Templewood Avenue, just south of No.12 within a field, and within the site of Spedan Tower (now Grange Gardens). The OS maps however do not show any major changes to the pattern of housing after WWII in the area concerned.

2.9 The London County Council Bomb Damage Map for this area indicates that the closest property to No.5 which is recorded as having suffered bomb damage is No.6 Templewood Avenue, located just to the south-east, which suffered damage recorded as "Seriously damaged, doubtful if repairable". This is somewhat inconsistent with the bomb map for Hampstead, which shows that the hit nearest to the site was further to the north-east (see paragraph 2.4 above).

2.10 A search of planning applications on LBC's planning website found a number of applications for the construction of basements beneath houses, extensions to existing basements, and the construction of new houses with basements in the vicinity of No.5, including:

- **No.4 Templewood Avenue:** Application (2011/1710/P) involving the "*Excavation and enlargement of existing basement to provide a new swimming pool, gym, utility spaces and associated light wells; erection of a ground floor rear extension, new terraces at ground and first floor levels, new replacement roof, works to chimneys, new dormer windows, new entrance gates and associated external alterations and landscaping to single dwelling house (Class C3) following works of demolition to dwelling*" was granted planning permission on 20th October 2011. A Basement Impact Assessment (BIA) by Arup was found on the website, which includes the results of a site-specific ground investigation.
- **No.6 Templewood Avenue:** Application (2012/1976/P) involving the "*Excavation of basement with associated lightwells, replacement of single storey garage and rear garden summer house, addition of two new dormer windows to north east roof slope, alterations to existing fenestration and new hard and soft landscaping, all associated with the use as residential dwelling (Class C3)*" was granted planning permission subject to a Section 106 legal agreement on 12th October 2012. A BIA was found, including two on-site borehole logs.
- **No.8 Templewood Avenue:** Application (2011/4966/P) involving "*Enlargement of basement for uses ancillary to existing flat (Class C3)*" was refused planning permission on 8th December 2011. A soil investigation report was found including one borehole.
- **No.9 Templewood Avenue:** Application (2012/6873/P) involving "*Alteration and extension of ground floor and basement flat including enlargement of existing basement, light wells at rear of property, alterations and extension to rear elevation at ground floor level, demolition and reconstruction of existing side extension, alterations to front entrance, and landscaping works*" was granted planning permission subject to section106 legal agreement on 1st August 2013. A basement impact assessment was found, including two site-specific borehole logs.
- **No.11 Templewood Avenue:** Application (2011/5127/P) involving the "*Enlargement of basement including creation of two rear lightwells, erection of extensions at rear ground and part first floor level, erection of dormer in rear roofslope, installation of rooflights, alterations to front boundary wall and installation of 2 x air condenser units with acoustic enclosure in rear garden*" was granted planning permission subject to Section 106 legal agreement on 7th December 2011. A basement impact assessment was found, including three site-specific borehole logs.
- **No.14 Templewood Avenue:** Application (2013/6912/P) involving the "Excavation works to provide single basement floor level, side and rear extensions at ground floor level, extension and alterations to coach house and other external alterations, removal of car port and erection of cycle store, associated landscaping, and conversion from five self-contained flats to a dwelling house (Class C3)" was granted planning permission on 12th November 2013. The basement impact assessment by ourselves is available online, including three site-specific borehole logs.

Other applications relating to the construction of basements were also found further away from the site, including No's 12 & 17 Templewood Avenue. Only the application relating to No 17 Templewood Avenue included ground investigation results.

3.0 PROPOSED BASEMENT

3.1 The drawings by Brod Wight Architects (as listed in paragraph 1.4) show that the proposed works include:

- A single-storey basement beneath the full footprint of the house. This basement will include a swimming pool beneath the north-eastern part of the house, which will also extend beneath part of the rear garden.
- Three lightwells:
  - in front of the two-storey projection on the north-east side of the house,
  - alongside the front end of the left (south-west) flank wall of the house,
  - to the rear of the single-storey rear projection and alongside the swimming pool area. This lightwell will provide access to the side passageway via a flight of steps.
- The extension of the existing two-storey projection on the east side of the house. This projection will be demolished and rebuilt in the same form, but with a larger footprint, which extends further to the rear of the house, broadly in line with the rear wall of the existing single-storey rear projection on the west side of the house.
- An entirely separate single car lift in the forecourt area (internal dimensions: 3.1m by 6.1m, by 4.2m deep).

3.2 The structural drawings by Elliott Wood Partnership show that the scheme will comprise a contiguous bored pile wall (BPW), which will form the proposed rear wall of the basement, along with its side walls where the proposed basement extends beyond the footprint of the existing house, and reinforced concrete (RC) underpinning beneath the existing walls to No.5. The proposed swimming pool with consist of a separate RC concrete box within the basement structure, with steel columns bearing onto the perimeter walls of this swimming pool.

3.3 Elliott Wood Partnerships' 'Typical underpin detail' (Drg No. 2150493 SK001 rev.P1) shows a thickness of 0.35m for the basement slab, which when combined with an assumed thickness of 150mm for insulation, cavity drainage and floor structure, gives a founding level (formation) for the proposed upper part of the basement of around 4.3m below the existing ground floor level (101.40m AOD), and around 7.1m below the existing ground floor level for the proposed pool area. For the underpin bases, a thickness of 0.55m is given (also shown in Elliott Wood Partnerships' 'Typical underpin detail'), therefore the founding level of the underpin bases will be around 4.5m below existing ground floor level for the upper part of the basement (around **97.0m AOD**), and around 7.3m below existing ground floor level for the pool area (around **94.2m AOD**).

3.4 The depths of excavation required are expected to vary considerably, from around **3.3m** within the footprint of the main part of the house, where there is a 1.0m deep crawl space, up to around **7.8m** for the swimming pool area where it extends beneath part of the rear garden (all as tabulated below).

Table 1: Existing and proposed levels, and depths of excavation

Existing Location	Proposed Location	Existing GL/FFL (m AOD)	Prop'd GL/FFL (m AOD)	Formation level – Basement Slab / Underpins (m AOD)	Excavation Depths – Basement Slab / Underpins (m)
Single-storey extension on NW corner of house	Basement	101.7	97.7	97.2 / 97.0	4.5 / 4.7
Rear patio	Basement	101.2	97.7	97.2 / 97.0	4.0 / 4.2
	Pool		94.9	94.4 / 94.2	6.7 / 7.0
Raised rear garden	Basement	102.0 (average)	97.7	97.2 / 97.0	5.2 / 5.0
	Pool		94.9	94.4 / 94.2	7.6 / 7.8
All other areas (allows for 1.0m crawl space below Gr.Fl.)	Basement	100.5	97.7	97.2 / 97.0	3.3 / 3.5
	Pool		94.9	94.4 / 94.2	6.1 / 6.3

4.0 GEOLOGICAL SETTING

4.1 Mapping by the British Geological Survey (BGS) indicates that the site is underlain by the Claygate Member. Just to the north of the site, the Claygate Member is mapped as being overlain by the Bagshot Formation, and on the 1920's geological map the Bagshot Formation is shown extending into No.5's rear garden, although past experience in this area has found the Bagshot Formation to be less extensive than has been mapped. Figure 3 shows an extract from Figure 16 of the Camden GHHS (Camden Geological, Hydrogeological and Hydrological Study by Arup, November 2010) which illustrates the geology of the north-west Hampstead area.

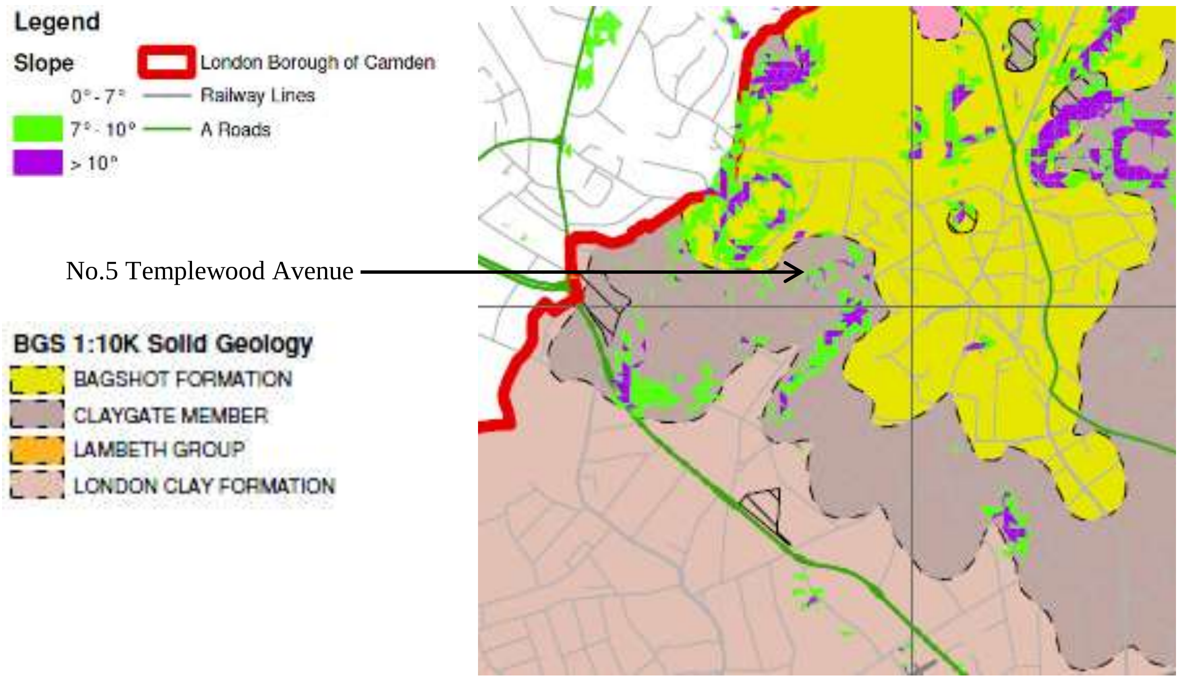


Figure 3: Extract from Figure 16 of the Camden GHHS (Arup, 2010) showing geology and slope angles >7° and >10°.

4.2 In urban parts of London, these natural strata are typically overlain by Made Ground. A thin superficial layer of natural, locally-derived re-worked soils called Head deposits may also be present (because these are not mapped by the British Geological Survey where they are expected to be less than 1.0m thick). In the areas which have been excavated, some or all of these deposits may have been removed.

4.3 The Claygate Member forms the uppermost unit of the London Clay Formation and is described in the relevant BGS memoir (Ellison et al, 2004) as "*alternating beds of clayey silt, very silty clay, sandy silt and glauconitic silty fine sand. Beds are generally 1 to 5m thick, although the boundaries are generally diffuse as a result of bioturbation*". The Claygate Member was 16.0m thick in the Hampstead Heath borehole (located to the NE of the site of present interest, near the top of the Heath) where the Claygate Member occurred between the levels of 93.71m and 109.71m AOD).

4.4 The London Clay beneath the Claygate Member is well documented as being a firm to very stiff over-consolidated clay which is typically of high or very high plasticity and high volume change potential. As a result it undergoes considerable volume changes in response to variations in its natural moisture content (the clay shrinks on drying and swells on subsequent rehydration). These changes can occur seasonally, in response to normal climatic variations, to depths of up to 1.50m and to much greater depths in the presence of the trees whose roots abstract moisture from the clay. The clay will also swell when unloaded by excavations such as those required for the construction of basements. The more silty and sandy clays of the Claygate Member generally have somewhat lower plasticities.

4.5 The Bagshot Formation which overlies the Claygate Member to the north of the site is described by the BGS as "*pale yellow-brown to pale grey or white, locally orange or crimson, fine- to coarse-grained sand that is frequently micaceous and locally clayey, with sparse glauconite and sparse seams of gravel*". The base of the Bagshot Formation is marked by an erosional surface, with a basal fine gravelly sand developed in places.

4.6 The results of the BGS classifications of six natural ground subsidence/stability hazards are presented in the Groundsure GeoInsight report (see Appendix D, Section 4); most indicated "Negligible" or "Very low" hazard ratings with the exception of 'Shrink – Swell Clay' for which a 'Moderate' hazard rating was given, which reflects the outcrop of the Claygate Member at surface. Additionally, a "Low" hazard rating was indicated for 'Running Sand' to the north of the site, reflecting the outcrop of the Bagshot Formation at surface in that area.

4.7 A search of the BGS borehole database was undertaken for information on previous ground investigations and any wells in the vicinity of the site. Few relevant BGS boreholes were identified close to the site, however a number of boreholes within very close proximity to the site were found during the search of planning applications (see paragraph 2.6). The locations of a selection of these boreholes are presented on the location plan in Appendix B, and the strata depths are summarised in Table 2. For full strata descriptions, reference should be made to the logs in Appendix B. A total of three boreholes (BH1-BH3) were drilled as part of a ground investigation at No.11 Templewood Avenue, to the north-east of No.5. The boreholes display broadly similar information, therefore in Table 1 only the highest and lowest values recorded in those boreholes are presented, giving the range of depths and levels recorded.

Table 2: Summary of BGS and other Boreholes - Depths/levels to base of strata					
Strata (abbreviated descriptions)	**No.11 Templewood Avenue BH1-BH3**		**No.6 Templewood Avenue BH1/BH2**	**No.8 Templewood Avenue BH1**	**No.4 Templewood Avenue BH1/BH2**
Approx GL (m AOD)	Depth (m)	**Level 106.24-104.52**	Depth (m)	Depth (m)	Depth (m)
Made Ground	0.15-2.85	**106.09-102.16**	1.60/	1.40	0.35/0.35
Mottled silty sandy CLAY with gravel and roots (Head?)	-	**-**	1.80/3.50	2.50	-
Soft to firm, brown, slightly sandy, silty CLAY with occn'l silt and fine sand partings (Weathered Claygate Mbr)	5.00-5.45	**100.79-99.52**	6.00/5.00	3.80	1.80/1.75
Concretionary Limestone	-	**-**	6.10/-	-	-
Soft to stiff, grey, sandy CLAY with pockets of silt and closely spaced partings of fine sand (Claygate Mbr)	14.70-19.60	**91.24-84.92**	>10.00/11.00	6.00	12.00/13.00
Stiff to very stiff, greyish brown, slightly sandy silty CLAY (London Clay Fm)	>25.00	**<79.52**	-/>20.00		>20.00

5.0 HYDROLOGICAL SETTING (SURFACE WATER)

No.5 Templewood Avenue

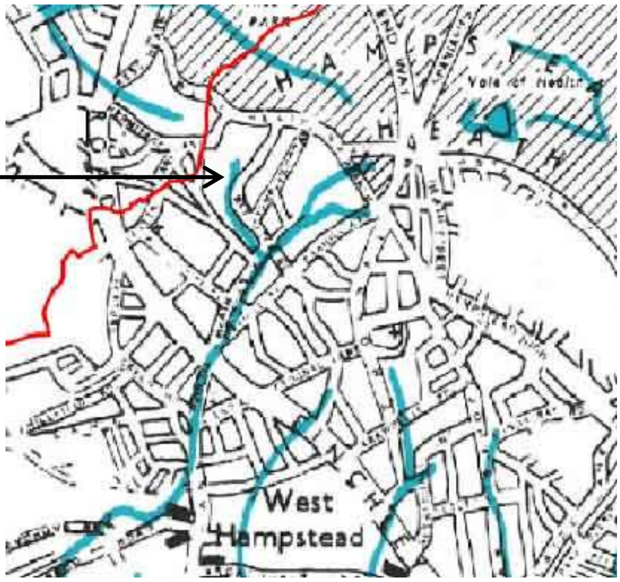


Figure 4: Extract from Figure 11 of the Camden GHHS (Arup, 2010) showing former watercourses, based on Barton (1992).

5.1 As shown in Figure 4, the site lies between two tributaries to the river Westbourne, one of the 'lost' rivers of London, most of which now run in dedicated culverts or the sewer system. The locations of both of these tributaries are also illustrated by the contours in Figure 2, which reveal weakly developed valleys to the west and south-east of the site, and both of these tributaries can also be seen on the 1870 & 1871 historic OS maps, which clearly show streams flowing at these location. These tributaries were most likely culverted when the area was developed.

5.2 Figure 14 of the Camden GHHS (Arup, 2010) shows that the site is not within the catchment of any of the Hampstead Heath Pond Chains, of which the Golders Hill Chain is the nearest.

5.3 The gentle fall of the footway away from the property, together with the south-westwards fall of Templewood Avenue are likely to prevent surface water on the carriageway from reaching the house under all conceivable conditions. Providing that they are well maintained, the brick boundary walls which separate the front parking/amenity area from the adjacent front parking/amenity area to No.5a, and the access road to the garages along the south-west boundary of the site will prevent run-off from the adjoining land. Thus, the surface water catchment for the front parking/amenity area is likely to be limited to direct rainfall and run-off from the rear garden. The front parking/amenity area is predominantly surfaced with asphalt, so infiltration will be limited or nil in most of this area, although infiltration will occur in any of the flower beds (see Photos 1-3) and soft landscaped areas (though that would be nil when the ground is saturated or frozen). Further upslope, the rear garden and the access path on the south-west side of the house is separated from the access road to the garages by a wooden fence and a low concrete kerb; the latter, together with the steep fall of this access road, should prevent most surface water run-off from the driveway entering No.5's site.

5.4 The rear garden to No.5 is bounded by wooden fences, thus it is likely to receive some excess overland run-off from the adjoining gardens upslope, though that may be minimal if the sands of the Bagshot Formation are present at surface immediately upslope of the site. The relative level of the rear gardens to No's 5 & 5a are not known (dense vegetation along that boundary prevented viewing of No.5a's rear garden), though the general slope profile suggests No.5a's is likely to be slightly higher, so it is possible that some surface water may run-off from No.5a's garden into No.5's under storm conditions. No.5's rear lawn has been terraced, and overall falls relatively steeply towards the patio area at the rear of the house, thus any run-off is likely to reach the drainage channel along the rear wall of the house. Infiltration is likely to be limited or nil in the paved patio area to the rear of the house, although infiltration will occur within the lawn, as well as the flower beds and soft landscaped areas (see Photos 4-6).

5.5 Figure 5 shows that both Templewood Avenue and Templewood Gardens were subject to surface water flooding in 2002 but not in the 1975 floods. The implications of those historical events are addressed in Section 10.8. While the whole length of the road is recorded as having flooded, the floods generally affected only a short length of these roads; in the case of Templewood Avenue that was possibly at/near its junction with Templewood Gardens, on the south-east side of the carriageway, immediately to the south of the site.

Figure 5: Extract from Figure 15 of the Camden GHHS (Arup, 2010) showing roads which flooded in 1975 (light blue), in 2002 (dark blue), and 'Areas with potential to be at risk of surface water flooding' (wide light blue bands).

5.6 Maps on the website of the Environment Agency (EA) show that the site lies within Flood Zone 1, which is defined as areas where flooding from rivers and the sea is very unlikely, with less than a 0.1 per cent (1 in 1000) chance of such flooding occurring each year. The EA's website also shows that this area does not fall within an area at risk of flooding from reservoirs.

5.7 The following hydrological data for the site has been obtained from the Groundsure EnviroInsight report (see Appendix F), including:

- The closest river (or more specifically "Detailed River Network entries") is the Tertiary grade upper waters of the Golders Hill pond chain, 442m to the north of the site. This feeds into the 'Leg of Mutton Pond', located 496m north of the site (App.F, Section 6.10). As No.5 is in a different catchment, this river is irrelevant to the proposed basement.
- There are no surface water features within 250m of the site (App.F, Section 6.11).
- There are no surface water abstraction licences within 2000m of the site (App.F, Section 6.4).
- There are no flood defences, no areas benefitting from flood defences, and no flood storage areas within 250m of the site (App.F, Sections 7.4, 7.5 & 7.6).

Figure 6: Extract from the Environment Agency's 'Risk of Flooding from Surface Water'.

5.8 Further modelling of surface water flooding has been undertaken by the Environment Agency and was published on its website in January 2014. An extract from their model is presented in Figure 6. While this map identifies four levels of risk (high, medium, low and very low) it is understood that it is based at least in part on depths of flooding and flow paths. This modelling shows a 'Very Low' risk of flooding (the lowest category for the national background level of risk) for No.5 and the adjacent properties on the north-west side of Templewood Avenue. Broadly opposite the site however, two localized areas at a 'Medium' to 'High' risk of flooding from surface water are shown within the sites of No's 6 & 10 Templewood Avenue, along with an area at a 'Medium' risk of flooding, which extends from these localized areas, down Templewood Gardens, and then south-westwards down Redington Gardens & Heath Drive.

5.9 More recently, surface water flood modelling has been undertaken by URS as part of a Strategic Flood Risk Assessment for the London Borough of Camden, and was published in July 2014; an extract from their model is presented in Figure 7. As per the Environment Agency's modelling, this map identifies the same four levels of risk (high, medium, low and very low). Similar to the Environment Agency's modelling (Figure 6), this modelling shows two localized areas at a 'Medium' to 'High' risk of flooding from surface water within the sites of No's 6 & 10 Templewood Avenue, along with an area at a 'Low' risk of flooding from surface water, which extends from these areas, down Templewood Gardens, Redington Gardens and Heath Drive.

5.10 Figure 7 also shows that Templewood Avenue falls within the Group3_010 Critical Drainage Area identified within the Strategic Flood Risk Assessment (2014).

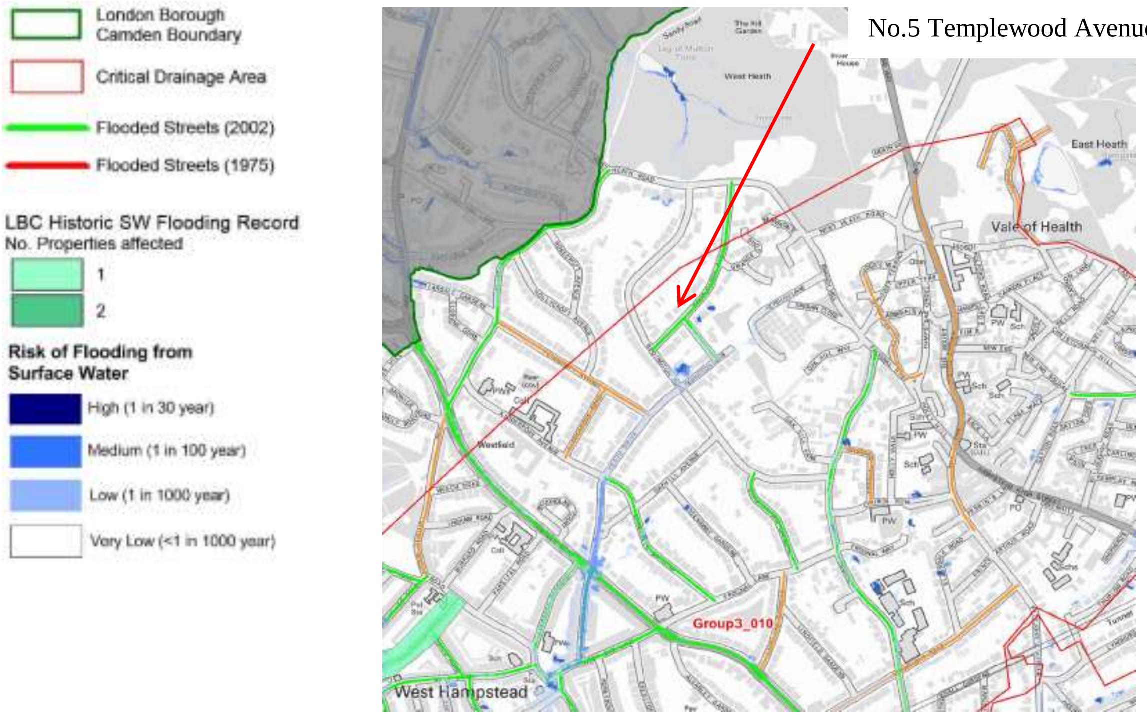


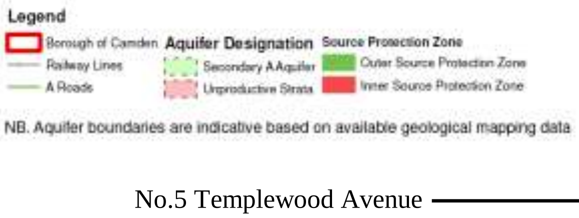
Figure 7: Extract from Figure 3v of the Camden Strategic Flood Risk Assessment (SFRA) (URS, July 2014) showing risk of flooding from surface water.

5.11 The implications from these flood models are discussed in Section 10.7.

5.12 Figures 5a & 5b of the Camden Strategic Flood Risk Assessment present historic records of internal and external sewer flooding respectively, based on Thames Water's DG5 Flood Register. These figures show that, when the Camden Strategic Flood Risk Assessment was written (July 2014), none of the properties within this postcode were recorded by Thames Water as having been affected by internal or external sewer flooding in the previous 10 years.

6.0 HYDROGEOLOGICAL SETTING (GROUNDWATER)

6.1 Both the Claygate Member which underlies the site, and the Bagshot Formation which is mapped at surface just to the north of the site are classified by the Environment Agency as a superficial 'Secondary A Aquifer', whereas the underlying London Clay is an 'Unproductive Stratum' as indicated by Figure 8.



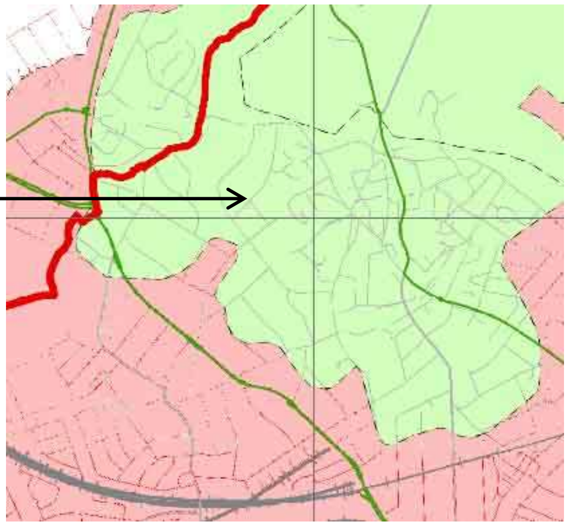
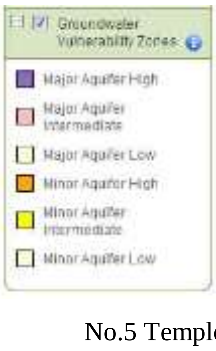


Figure 8: Extract from Figure 8 of the Camden GHHS (Arup, 2010) showing aquifer designations.

6.2 The Chalk Principal Aquifer which occurs at depth beneath the London Clay is not considered relevant to the proposed basement so is not considered further.

6.3 Under the old groundwater vulnerability classification scheme, which now applies only to superficial soils, the site is classed as 'Minor Aquifer High' groundwater vulnerability, as shown in Figure 9.



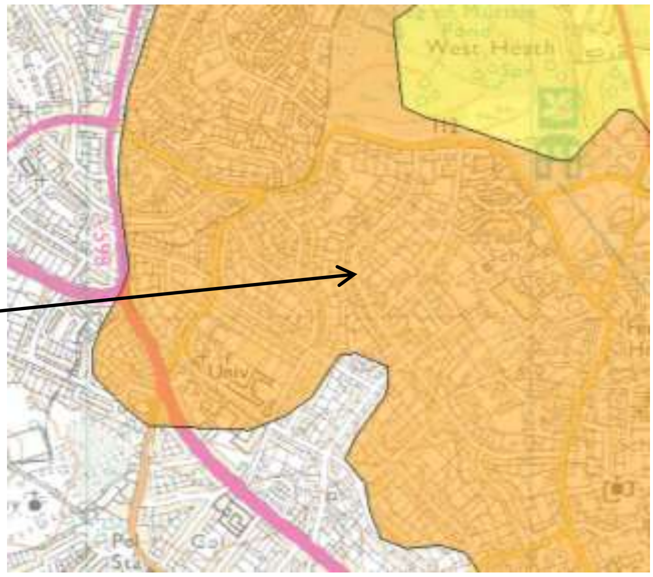


Figure 9: Extract from Environment Agency's map of Groundwater Vulnerability Zones and SPZs
(Red = Zone I, Green = Zone II).

6.4 Although no discrete beds of silty sand or sandy silt were recorded by the site-specific ground investigation within the Claygate Member, the groundwater seepages and strikes indicate that they are probably present. Those beds would generally be expected to be water-bearing and where these are laterally continuous they can give rise to moderate water entries into excavations. The clay and silty clay beds would also be expected to be saturated, with water pressures controlled by the water levels/ pressures in adjacent silt/sand beds, by tree root activity or by the influence of man-made changes such as utility trenches (which can act as either land drains or as sources of water and high groundwater pressures). Boreholes drilled through low permeability layers can also homogenise groundwater pressures between permeable layers, if they are not adequately sealed. Natural groundwater flow rates, if any, in the silt/sand horizons within the Claygate Member are typically low. Variations in groundwater levels and pressures will occur seasonally and with other man-induced influences.

6.5 Local perched groundwater may occur near surface in Made Ground, and possibly also in any Head deposits which overlie the Claygate Member, in at least the winter and early spring seasons.

6.6 While the London Clay Formation is classified as an 'Unproductive Stratum' it can still be water-bearing. The water pressures within the clay in the depths of current interest are likely to be hydrostatic, which means they increase linearly with depth, except where they are modified by tree root activity or the influence of man-made changes such as utility trenches. As in the Claygate Member, any silt or sand partings, laminations or thicker beds are likely to contain free groundwater and, where these are laterally continuous, they can give rise to moderate water entries into excavations. In most cases there will be only very limited or no natural flow in these silt/sand horizons.

6.7 The presence of the Bagshot Formation (located just to the north of the site), which is predominantly composed of sands, along with the interbedded sands, silts and clays of the underlying Claygate Member, gives rise to various springs in the headwater valleys of the various rivers which drain from Hampstead Heath, as shown most clearly on the 2002 1:10,000 scale OS map in Appendix G. While no springs are recorded on the historic Ordnance Survey maps in the vicinity of Templewood Avenue, the streams visible on the 1870 map were clearly fed by springs, and these have since (historically) been collected and channelled into drains/culverts before the area was developed.

6.8 Environment Agency groundwater flooding incidents were presented on Figure 4e of the Camden Strategic Flood Risk Assessment. The closest incident to Templewood Avenue was recorded to the south-east, near the junction between Church Row and Holly Walk, broadly at the mapped interface between the Claygate Member and the overlying Bagshot Formation. Figure 4e also shows that Templewood Avenue is not in an area of Increased Susceptibility to Elevated Groundwater (coloured orange).

6.9 The groundwater catchment areas upslope of No.5 are likely to differ for each of the main stratigraphic units:

- Made Ground: The catchment for any perched groundwater in the Made Ground is probably limited to No.5's own front parking/amenity area and rear garden, as well as any immediately adjoining areas of Made Ground, except where the trenches for drains and other services provide greater interconnection.
- Claygate Member: The catchment for the Claygate Member will comprise recharge from the overlying soils in the vicinity of the site plus from a much wider area, determined by the lateral degree of interconnection between the sand horizons in the Claygate Member.

6.10 Other hydrogeological data obtained from the Groundsure EnviroInsight report (Appendix F) include:

- There are no groundwater abstraction licences within 2000m of the site (see App.F, Section 6.3).
- There are no abstraction licences for potable water within 2000m of the site (App.F, Section 6.5).
- There are no Source Protection Zones (SPZ) within 500m of the site (App.F, Sections 6.6 & 6.7, and Figure 7). The nearest is around 2km to the south of the site, so is irrelevant to the current issue.
- For an area within 50m of No.5 the BGS has classified the susceptibility to groundwater flooding as '**Limited Potential**', at a 'Low' confidence level (App.F, Sections 7.7 and 7.8). Such groundwater flooding is defined as "*the emergence of groundwater at the ground surface or the rising of groundwater into man-made ground under conditions where the normal range of groundwater levels is exceeded*".

6.11 Groundwater information gleaned from the desk study is summarised in Table 3 below.

Table 3: Summary of groundwater records from desk study boreholes

Location	**Groundwater Seepage**	**Groundwater Strike**	**Standing level:** 20 mins after strike, or on completion	**Highest Standing Level during monitoring**
	m bgl (m AOD)	m bgl (m AOD)	m bgl (m AOD)	m bgl (m AOD)
No.11 TA (BH1)	Constant below 5.00 (101.24)	-	-	5.25 (100.99)
No.11 TA (BH2)	-	4.60[1] (100.41) 5.60[1] (99.41)	4.45 (100.56) 5.15 (99.86)	3.44 (101.57)-
No.11 TA (BH3)	Constant[2]	5.50[1] (99.02) 9.00[1] (95.52)	-	3.15 (101.37)
No.6 TA (BH1)	4.00	6.00	6.00	1.56
No.6 TA (BH2)	3.20, 5.00 & 9.70	10.50 not sealed - base BH @ 20m	10.30	1.78
No.8 TA (BH1)	1.40	4.40	5.70	-
No.4 TA (BH1)	-	6.00, 8.00, 10.00, 13.50 & 18.00	5.90, 6.00 & 4.50 (from first 3 strikes)	2.61
No.4 TA (BH2)	-	8.00, 14.00 & 18.00	6.00 from first strike	2.35

Notes:
TA = Templewood Avenue
1. Recorded as seepage on borehole log.
2. 'Constant water seepage throughout drilling' recorded on borehole log.

6.12 Details of what was found by the site-specific ground investigation in December 2015 are presented in Section 9.

7.0 STAGE 1- SCREENING

7.1 The screening has been undertaken in accordance with the three screening flowcharts presented in LBC's CPG4 guidance document. Information to assist with answering these screening questions has been obtained from various sources including the site-specific ground investigation, the Camden geological, hydrogeological and hydrological study (Arup, 2010), historic maps and data obtained from Groundsure (see Appendices E, F & G) and other sources as referenced.

7.2 Subterranean (groundwater) flow screening flowchart:

Question		Response, with justification of 'No' answers	Clauses where considered further
1a	Is the site located directly above an aquifer?	Yes	Carried forward to Scoping: 8.2, Section 10.2
1b	Will the proposed basement extend beneath the water table surface?	Yes	Carried forward to Scoping: 8.2, Sections 10.2 & 10.3
2	Is the site within 100m of a watercourse?	No – Though one of the former Westbourne tributaries which is thought to have been culverted in the 1800's is located approx. 140m to the south-west of the proposed basement.	5.1, Figure 4 & 5.7
3	Is the site within the catchment of the pond chains on Hampstead Heath?	No – As shown on Figure 14 of the Camden GHHS, the site is approximately 175m south of the Golders Hill Pond Chain catchment.	5.7
4	Will the proposed basement development result in a change in the proportion of hard surfaced/ paved areas?	Yes – because of the basement, extended patio and rear lightwell will extend beyond the rear patio, below the rear garden; although there are likely to be no changes in the proportion of hard surfaced/ paved areas within the front parking/ amenity area.	Carried forward to Scoping: 8.2, Section 10.2
5	As part of the site drainage, will more surface water (eg: rainfall and run-off) than at present be discharged to the ground (eg: via soakaways and/or SUDS)?	No – increased discharge of surface water to the ground would not be appropriate owing to high groundwater.	
6	Is the lowest point of the proposed excavation (allowing for any drainage and foundation space under the basement floor) close to, or lower than, the mean water level in any local pond (not just the pond chains on Hampstead Heath) or spring line?	Yes – There are no surface water features within 250m of the site, but a spring line does occur in places at the Bagshot Fm – Claygate member interface.	Carried forward to Scoping: 4.1, 6.8, 8.2, Section 10.2

7.3 Slope/ground stability screening flowchart:

Question		Response, with justification of 'No' answers	Clauses where considered further
1	Does the existing site include slopes, natural or man-made, greater than 7°? (approximately 1 in 8)	Yes – Gradients within the site are predominantly gentle, however parts of the rear garden and front parking/ amenity area appear to have been identified as >7°.	Carried forward to Scoping: 2.6, Figure 3, 8.3, Section 10.4
2	Will the proposed re-profiling of landscaping at site change slopes at the property boundary to more than 7°?	No – The limited re-profiling of landscaping is either not alongside the boundary, or will require construction of a retaining wall.	
3	Does the development neighbour land, including railway cuttings and the like, with a slope greater than 7°?	Yes – At least part of the access road up to the garages to the north-west of the site is probably >7°, and parts have been identified as >10°.	Carried forward to Scoping: 2.6, 8.3, Section 10.4
4	Is the site in a wider hillside setting in which the general slope is greater than 7°?	No – The general slope angle is <7°.	2.6, 2.7 and Figures 2 & 3
5	Is the London Clay the shallowest strata at the site?	No – Site is underlain by the Claygate Member.	4.1
6	Will any tree/s be felled as part of the proposed development and/or are any works proposed within any tree root protection zones where trees are to be retained?	Yes. A large Sycamore tree immediately to the rear of the existing single-storey rear projection will be felled as part of the proposed works. All other trees are to be retained.	Carried forward to Scoping: 8.3, Section 10.4
7	Is there a history of seasonal shrink/swell subsidence in the local area, and/or evidence of such effects at the site?	No – Only very minor crack damage was evident to No.5, which could all have been from other causes.	
8	Is the site within 100m of a watercourse or potential spring line?	Yes – A potential spring line. The former Westbourne tributary is located approx. 140m to the south-west of the proposed basement, but is thought to have been culverted in the 1800's.	Carried forward to Scoping: 8.3, Section 10.4
9	Is the site within an area of previously worked ground?	No – The closest area of worked ground is approximately 159m north-east of the site. See maps on pages 8 & 15 of the GeoInsight report (in App. E).	4.1
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?	Yes and Yes	Carried forward to Scoping: 8.3, Section 10.4
11	Is the site within 50m of the Hampstead Heath ponds?	No – Site is approx 520m from nearest Hampstead Pond Chain (Leg of Mutton Pond) and 560m from a pond to the east of the site.	5.6

12	Is the site within 5m of a highway or a pedestrian right of way?	No – the proposed basement will be >9m from the Templewood Avenue boundary.	
13	Will the proposed basement substantially increase the differential depth of foundations relative to neighbouring properties?	Yes – Neither the adjacent No.5a nor No.3 Templewood Avenue are known to have modern basements, although No.3 is located >4.5m away, which will limit any potential impact on its footings.	Carried forward to Scoping: 8.3, Section 10.4
14	Is the site over or within the exclusion zone of any tunnels, eg railway lines.	No – Re railway tunnels. Unknown re other tunnels.	Carried forward to Scoping: 8.3, 10.1.3

7.4 Surface flow and flooding screening flowchart:

Question		**Response, with justification of 'No' answers**	**Clauses where considered further**
1	Is the site within the catchment of the pond chains on Hampstead Heath?	No – As shown on Figure 14 of the Camden GHHS.	
2	As part of the proposed site drainage, will surface water flows (eg volume of rainfall and peak run-off) be materially changed from the existing route?	No – The surface water flow routes will not change, with the exception of the need to pump water from lightwells.	
3	Will the proposed basement development result in a change in the proportion of hard surfaced / paved external areas?	Yes	Carried forward to Scoping: 3.1 , 8.4 & Section 10.7
4	Will the proposed basement result in changes to the profile of the inflows (instantaneous and long-term) of surface water being received by the adjacent properties or downstream watercourses?	No – There will be no or insignificant change in run-off to adjacent properties. The historic natural watercourse downslope of the property has been culverted since the 1800's.	5.3, 5.4
5	Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream watercourses?	No – The surfaces generating the run-off will be the same/ similar and no significant run-off to adjacent properties is anticipated	5.3, 5.4
6	Is the site in an area known to be at risk from surface water flooding, such as South Hampstead, West Hampstead, Gospel Oak and King's Cross, or is it at risk from flooding, for example because the proposed basement is below the static water level of a nearby surface water feature?	Yes/No – Templewood Avenue did flood in 1975, and this may have been broadly opposite the site, but on the other side of the carriageway; however surface water flood modelling by the Environment Agency indicates a 'Very Low' flood risk (the lowest) for this property and its immediate surrounds.	Carried forward to Scoping: 5.5, 5.8 & Figure 6. Section 10.7.

7.5 Non-technical Summary – Stage 1:

The screening exercise in accordance with CPG4 has identified thirteen issues which need to be taken forward to Scoping (Stage 2); four related to groundwater, seven are related to ground stability and two are related to flooding potential.

8.0 STAGE 2 - SCOPING

8.1 The scoping stage is required to identify the potential impacts from the aspects of the proposed basement which have been shown by the screening process to need further investigation. A conceptual ground model is usually compiled at the scoping stage; however, because the ground investigation has already been undertaken for this project, the conceptual ground model including the findings of the ground investigation is described under Stage 4 (see Section 10.1).

8.2 Subterranean (groundwater) flow scoping:

Issue (= Screening Question)		Potential impact and actions
1a	Is the site located directly above an aquifer?	**Potential impact**: Infiltration could be reduced. **Action:** Ground investigation required, then review.
1b	Will the proposed basement extend beneath the water table surface?	The anticipated groundwater regime is described in Section 6, Hydrogeological Setting. **Potential impact**: Local restriction of groundwater flows (perched groundwater or below groundwater table). **Action:** Ground investigation required, then review.
4	Will the proposed basement development result in a change in the proportion of hard surfaced/ paved areas?	**Potential impact:** Increased hard surfacing would decrease infiltration of surface water into the ground. Reduced hard surfacing above an aquifer, while generally beneficial in promoting recharge, might lead to local groundwater flooding elsewhere. **Action:** Review potential impacts of proposed changes, including appropriate types of SuDS for use as site-specific mitigation when relevant.
6	Is the lowest point of the proposed excavation (allowing for any drainage and foundation space under the basement floor) close to, or lower than, the mean water level in any local pond (not just the pond chains on Hampstead Heath) or spring line?	**Potential impact:** Risk of inundation of the basement excavations. Temporary dewatering and/or the permanent works might lower the pond water level or cause a spring to run dry. **Action:** Review potential impacts in relation to the site's hydrogeology; recommend appropriate site-specific mitigation when relevant.

8.3 Slope/ground stability scoping:

Issue (= Screening Question)		Potential impact and actions
1	Does the existing site include slopes, natural or man-made, greater than 7°? (approximately 1 in 8)	**Potential impact**: Clay slopes may be only marginally stable owing to past solifluction, excavations or placement of fill material. Increases in groundwater levels/pressures could cause slope failure. **Action:** Additional support, both temporary and permanent, may be required in excavations. No cross-slope excavations should be made in battered open cut. Basement design should ensure no (or minimal) increase in groundwater pressures.

3	Does the development neighbour land, including railway cuttings and the like, with a slope greater than 7°?	**Potential impact**: Slopes may be only marginally stable. Increases in groundwater levels/pressures could cause slope failure. **Action:** As Q1 above.
6	Will any tree/s be felled as part of the proposed development and/or are any works proposed within any tree root protection zones where trees are to be retained?	**Potential impact:** Heave from removal of trees; slope(s) become less stable; damage to trees. **Action:** Arboricultural assessment and review of potential impact on stability of buildings and/or slopes and/or the trees as relevant.
8	Is the site within 100m of a watercourse or potential spring line?	**Potential impact:** For watercourse(s) downslope of the proposed basement, and spring(s) upslope, as applies here, construction of the basement might block or divert the flow of groundwater, thereby increasing groundwater pressures and reducing the stability of slopes and/or retaining structures in the vicinity. **Action:** Review hydrogeology of the site and undertake a ground investigation.
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?	**Potential impact:** Inadequate provision of dewatering can lead to collapse of excavations. Inappropriate dewatering can cause removal of fines and/or unacceptable increases ineffective stress, both of which can cause structures to settle. **Action:** Ground investigation required in order to enable a proper assessment of the appropriate forms of groundwater control.
13	Will the proposed basement substantially increase the differential depth of foundations relative to neighbouring properties?	**Potential impact:** Loss of support to the ground beneath the foundations to neighbouring buildings if basement excavations are inadequately supported. This will apply to No.5a Templewood Avenue only, as there is sufficient separation between No's 5 and 3 Templewood Avenue. **Action:** Ensure adequate temporary and permanent support by use of best practice underpinning methods.
14	Is the site over or within the exclusion zone of any tunnels, eg railway lines.	**Potential impact:** Stress changes on any tunnel lining. **Action:** Undertake services search to check that there are no tunnels/services in the vicinity.

8.4 Surface flow and flooding scoping:

Issue (= Screening Question)		**Potential impact and actions**
3	Will the proposed basement development result in a change in the proportion of hard surfaced / paved external areas?	**Potential impact:** May increase flow rates to sewer, and thus increase the risk of flooding (locally or elsewhere). **Action:** Assess net change in hard surfaced/ paved areas and, if required, recommend appropriate types of SuDS for use as site-specific mitigation.
6	Is the site in an area known to be at risk from surface water flooding, such as South Hampstead, West Hampstead, Gospel Oak and King's Cross, or is it at risk from flooding, for example because the proposed basement is below the static water level of a nearby surface water feature?	**Potential impact**: Flooding of the basement. **Action:** Assess flood risk and potential threat to proposed scheme, and provide recommendations for flood resistance measures as appropriate.

8.5 Non-technical Summary – Stage 2:

The scoping exercise has reviewed the potential impacts for each of the items carried forward from Stage 1 screening, and has identified the following actions to be undertaken:

- A ground investigation is required (which has already been undertaken).
- An arboricultural assessment is required.
- Review of site's hydrogeology and groundwater control requirements.
- Assess the net change in area of hard surfacing and the potential for change in discharge to the ground.
- Review need to implement appropriate types of Sustainable Drainage System (SuDS) in order to offset (mitigate) any potential increase in discharge to mains sewer.
- Review ground investigation findings for potential flow from springs and recommend mitigation measures if appropriate.
- Assess potential for slope instability and provide appropriate recommendations.
- Ensure adequate temporary and permanent support by use of best practice working methods.
- Provide recommendations for groundwater control.
- Ensure adequate temporary and permanent support by use of best practice underpinning methods.
- Undertake a services search to ensure there are no deep tunnels/services.
- Investigate the existing surface water drainage system and provide appropriate flood resistance and mitigation measures.
- Review flood risk and include appropriate flood resistance and mitigation measures in the scheme's design.

All these actions are covered in Stage 4, or Stage 3 for the ground investigation.

9.0 STAGE 3 - GROUND INVESTIGATION

9.1 A site-specific ground investigation was undertaken by Chelmer Site Investigations (CSI) in December 2015, and consisted of two continuous flight auger boreholes (BH1 & BH2), both drilled to depths of 15.0m below ground level (bgl), and four hand dug trial pits (TP1 – TP4). The factual findings from the investigation are presented in Appendix C, including location plans, trial pit logs, borehole logs and laboratory test results.

9.2 Trial pits TP1 – TP4 were dug in order to investigate the foundations to No.5, and the soils beneath the footings, at their respective locations.

- TP1 was dug to a depth of 1.39m within the rear patio area, alongside the rear wall of the house. It revealed brickwork with three corbels at its base, resting on a 450mm thick concrete footing, founded at **1120mm** below ground level (bgl). The concrete footing was recorded as projecting 460mm from the face of the brick wall above, and the rear patio area was found to consist of 50mm thick York stone slabs over 50mm of concrete.
- TP2A was dug within the rear patio area, alongside the rear wall of the single-storey rear projection. This pit was terminated at 200mm bgl, due to mass concrete and suspected drainage, thus TP2B was dug on the opposite side of a soil & vent pipe (svp). This pit revealed brickwork with three corbels at its base, resting on a 300mm thick concrete footing, founded at **890mm bgl**. The concrete footing was recorded as projecting 210mm from the face of the brick wall above.
- TP3 was dug to a depth of 0.50m, immediately to the west of the single-storey rear projection, alongside the low brick retaining wall which separates part of the side access path and rear garden from the access road to the garages, located just to the north-west of the site. The pit revealed brickwork with a single corbel at its base, resting on a 150mm thick concrete footing, founded at **300mm** bgl. The concrete footing was recorded as projecting 100mm from the face of the brick wall above.
- TP4 was dug to a depth of 1.65m alongside the front wall, just to the west of the front entrance. This pit also revealed brickwork with three corbels at its base, resting on a 450mm thick concrete footing, founded at **1450mm bgl**. The concrete footing was recorded as projecting 360mm from the face of the brick wall above.

9.3 In all four trial pits, Made Ground was recorded from directly beneath the York stone slabs and concrete which formed the rear patio area, or from ground level in TP4, down to the founding depth of the foundations. In trial pits TP1, TP2B and TP3, the Made Ground was generally recorded as mid-brown, slightly sandy, silty to very silty **clay**, with brick fragments and pieces, whereas in TP2A and TP4, Made Ground consisting of "silty fine to medium **sand** with brick fragments" was recorded. Directly beneath the footings, all four trial pits recorded natural soils, described as "Firm, mid-brown (brown in TP1), slightly sandy silty CLAY". TP2A was terminated on concrete (see paragraph 9.2 above).

9.4 The site's geology as found by the boreholes may be summarised as:

- <u>Made Ground:</u> Found immediately beneath 0.20m of concrete in BH1, and beneath 0.2m of turf in BH2; the Made Ground was proved to depths of 1.2m and 0.5m below ground level (bgl) in BH1 and BH2 respectively. In BH1, 0.4m of "brick and concrete rubble" (hardcore) was recorded, over 0.6m of "brown, sandy silty **clay**, with brick fragments", whereas in BH2, the Made Ground was recorded as "firm, light brown, slightly sandy silty **clay**, with brick fragments".
- <u>Weathered Claygate Member</u>: Proved from the base of the overlying Made Ground to depths of 3.8m and 4.7m bgl in BH1 and BH2 respectively. In BH1, the Claygate Member was recorded as "Stiff,

brown/orange silty CLAY", whereas in BH2, it was recorded as "Firm (becoming stiff from 3.0m bgl), brown, slightly sandy silty CLAY".

- 'Un-weathered' Claygate Member/London Clay Formation?: Directly beneath the Weathered Claygate Member, BH1 and BH2 recorded "Stiff, grey, silty CLAY with crystals" and "Stiff (becoming very stiff from 9.0m bgl), grey, slightly sandy silty CLAY" to the base of both boreholes respectively.

9.5 Standard Penetration Tests (SPTs) were carried out in-situ in BH1. The resulting N values ranged from N = 17 at 1.00-1.45m bgl, up to N=25 at 14.00-14.45m bgl within the 'un-weathered' Claygate Member/London Clay, illustrating an overall gradual increase in strength with depth. In BH2, hand vane measurements of shear strength were taken in-situ within the Weathered Claygate Member and underlying un-weathered Claygate Member/London Clay. These readings also illustrated a gradual increase in strength with depth, with an averaged value of 67kPa recorded at 1.0m bgl, increasing to 93kPa at 7.0m bgl, then 89kPa at 8.0m bgl. Below 9.0m, all readings were >120kPa. These hand vane values do not allow for the clay's fabric such as fissures, so typically over-estimate the soil's strength and should NOT be used for design.

9.6 Roots of live and dead appearance were recorded to a depth of 0.3m bgl in TP3, and "Hair like and fibrous roots" were recorded to a depth of 0.8m bgl in BH1.

9.7 Both groundwater seepages and groundwater strikes were recorded during drilling, and the groundwater standing level in BH2 was recorded on completion, the results of which are summarised in Table 4 below. In order to monitor the groundwater levels, standpipes were installed to depths of 7.0m and 8.0m bgl in BH1 and BH2 respectively, and water level readings were taken on 12th and 27th January 2016 in BH1, and on 27th January 2016 only in BH2. During this short period of monitoring, the groundwater level in BH1 fell, from 1.53m to 2.79m bgl, whereas in BH2, a groundwater level of 2.05m bgl was recorded.

Table 4: Summary of groundwater records from ground investigation boreholes

Location	Seepage (m bgl)	Strike (m bgl)	Depth to water on completion (m bgl)
BH1	4.1	6.1	-
BH2	5.3	7.1	11.4

9.8 Laboratory Testing:

Laboratory tests were carried out by Chelmer Geotechnical Laboratories (CGL) and others on samples recovered from the two boreholes. The testing comprised classification tests, including moisture content and plasticity, strength and compressibility tests, and chemical testing to assess the potential for acid or sulphate attack on buried concrete. The results were presented in CSI's Factual Report (see Appendix C).

9.9 Plasticity tests were performed on two samples of the Weathered Claygate Member, recovered from BH1 at 3.0m and from BH2 at 3.5m, as well as four samples of the 'un-weathered' Claygate Member/London Clay, recovered from BH1 at 5.0m, 9.0m and 12.5m bgl, and from BH2 at 5.5m bgl. All six of these samples were found to be of High Plasticity, as classified by BS5930 (1999, 2010), and Medium volume change potential, as defined by the NHBC (NHBC Standards, 2013, Chapter 4.2, Building near Trees). It should be noted however that the sample recovered from BH2 at 5.5m bgl was found to be on the boundary between High Plasticity and Intermediate Plasticity.

9.10 The moisture contents of the same six samples recovered from BH1 and BH2 were found to vary between 25% and 35%. The majority of the samples were all 6-15% above the plastic limit values from the same sample, which indicates that the samples were not desiccated. The moisture content of the sample recovered from BH1 at 5.0m bgl however, was just 1% above the plastic limit value from the same sample, thus would generally be considered indicative of some desiccation. Overall, the moisture contents in BH1 and BH2 show no overall trend with depth (see plotted profiles in CGL's reports).

9.11 Undrained strength tests (undrained triaxial compression) were undertaken on a total of five samples recovered from BH1 at 2.00-2.45m, 4.00-4.45m, 6.50-6.95m, 9.50-9.95m and 12.50-12.05m bgl. The resulting values of cohesion varied from 60kN/m² to 258kN/m², and have been plotted as a profile against depth in Figure 10.

9.12 One dimensional consolidation tests were also carried out on the two shallowest samples of Claygate recovered from BH1 at 2.00-2.45m and 4.00-4.45m bgl. For data, see the semi-log plots of voids ratio against pressure within CGL's report.

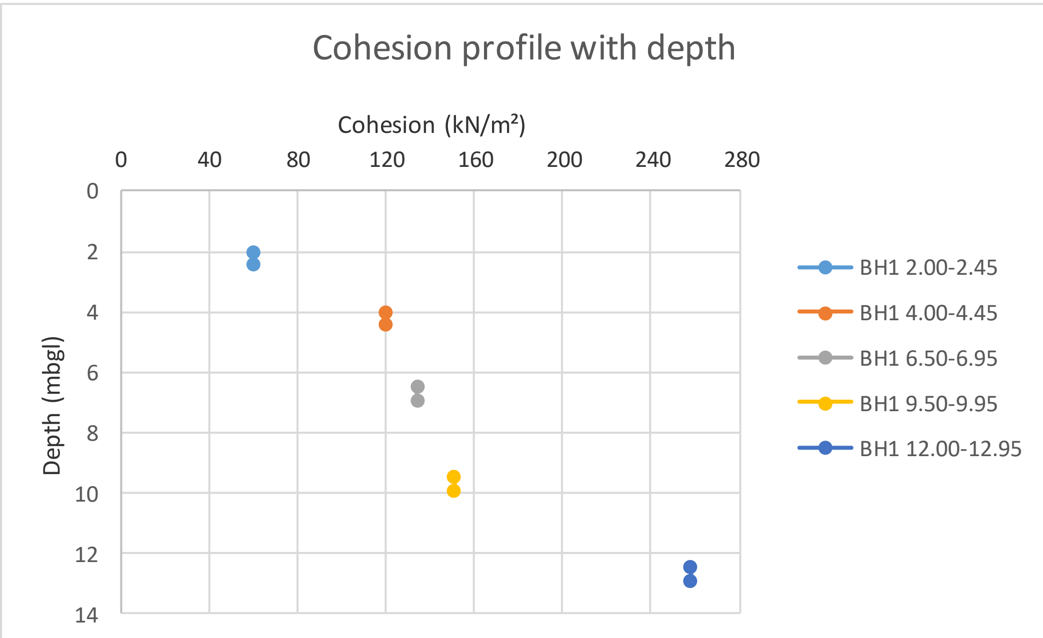


Figure 10: Cohesion values with depth.

9.13 The chemical tests were undertaken to assess the potential for acid or sulphate attack on buried concrete, and were carried out by Nicholas Colton Analytical on a total of five samples, broadly in accordance with BRE Special Digest 1 (2005). The samples were recovered from BH1 at 1.00m, 3.00m and 11.00m bgl, and from BH2 at 0.30m and 5.50m bgl, so included Made Ground, Weathered Claygate Member and 'un-weathered' Claygate Member/London Clay. The following ranges of results were recorded.

pH value:	7.4 – 8.3
Water-soluble sulphate:	79 – 630 mg/l
Total Sulphur:	0.02 – 0.54 mg/kg

Calculations following BRE Digest SD1 gave 'derived' values:

Total Potential Sulphate: 0.06 – 1.62%

Oxidisable sulphides: 0.02 – 1.48%.

These results indicated that the samples fell within the following Design Sulphate Classes, as defined by BRE Special Digest 1 (2005):

DS-1: Samples from the Made Ground

DS-2: Sample from the Weathered Claygate Member

DS-3 to DS-4 (based on TPS): Samples from the un-weathered Claygate Member/London Clay(?).

9.9 Non-technical Summary – Stage 3:

9.9.1 The site-specific ground investigation at No.5 Templewood Avenue recorded weathered CLAYS of the Claygate Member, over 'un-weathered' CLAYS which may form part of either the Claygate Member, or the underlying London Clay Formation (see discussion in Section 10.1.1). A variable thickness (0.3-1.45m) of Made Ground was also found overlying the natural strata across the site.

9.9.2 Groundwater seepages were recorded at 4.1m and 5.3m bgl, and groundwater strikes were recorded at 6.1m and 7.1m bgl, in BH1 and BH2 respectively. During the short monitoring period, groundwater levels of up to 1.53m bgl in BH1 and 2.05m bgl in BH2 were recorded; therefore the proposed basements will involve excavation below the relevant groundwater table.

9.9.3 The laboratory testing has shown that the clay specimens from the Weathered Claygate Member and underlying un-weathered Claygate Member/London Clay were all found to be of High plasticity, and 'Medium' volume change potential. Considerably higher Total Potential Sulphate concentrations (DS-3 to DS-4) were found in samples from the un-weathered Claygate Member/London Clay at depth, and high Oxidizable Sulphate concentrations (in the same two samples) is indicative of the presence of significant quantities of pyrite.

10.0 STAGE 4- BASEMENT IMPACT ASSESSMENT

10.1 Conceptual Ground Model

10.1.1 The desk study evidence together with the ground investigation findings suggest a conceptual ground model for the site characterised by:

- Made Ground: Made Ground was discovered within all of the exploratory holes, to a maximum depth of 1.45m below ground level (bgl). As usual, it varied in its composition, with descriptions of the Made Ground ranging from "slightly sandy silty **clay**" to "silty fine to medium **sand**". Brick fragments were found throughout the Made Ground; however other materials, as well as other soil types and greater thicknesses/depths, are also likely to be present on site, owing to the inherent variability of Made Ground.
  Perched groundwater may occur locally within this Made Ground, supported on horizons of lower permeability; such perched groundwater may only be present during the wetter winter and spring seasons.
- 'Head' Deposits?: Although Head deposits were not identified during the site investigation at No.5, the presence of Head deposits on site cannot be ruled out. These locally-derived re-worked soils typically consist of material that has been washed down from upslope, so would be expected to include a mixture of soil types from the Claygate Member, and potentially the Bagshot Formation.
- Claygate Member (part of the London Clay Formation): Predominantly silty and slightly sandy CLAYS of the Claygate Member were recorded directly beneath the Made Ground. No laminations of silt or fine sand of sufficient thickness to warrant separate identification on the borehole logs were recorded within these clays; however that may in part be a reflection of the CFA drilling method.
  Many of the nearby boreholes (see Table 2) recorded partings of silt and fine sand down to 84.92m AOD, approximately 15.5m below the ground level at BH1. As a result, there may be more frequent laminations or thin beds of silt and fine sand than the on-site ground investigation logs show, probably located at or close to the levels where the multiple groundwater seepages and strikes were recorded.
- 'Un-weathered' Claygate Member/London Clay Formation: Clays with a similar description to 'un-weathered' London Clay were described from 3.80mbgl to the base of BH1 (see paragraph 9.4). In BH2, the full 10.3m thickness of the same/equivalent stratum were described as "slightly sandy" and lacked "crystals", which is more typical of the Claygate, although the laboratory test results suggest that the materials in both boreholes were broadly the same.
  The Claygate Member was 16.0m thick in the Hampstead Heath borehole (see paragraph 4.3), therefore a reasonable thickness would be expected on-site, due to the mapped proximity of the overlying Bagshot Formation occurring just upslope of the site. As a result, it is unclear whether these clays belong to the lower part of the Claygate Member, or unit D (the uppermost unit) of the London Clay Formation.
  These clays are likely to be fissured and will undergo heave movements in response to unloading by the basement excavation. The "crystals" recorded in BH1 were probably selenite (a form of gypsum) which is aggressive to buried concrete as, potentially, is the indicated high pyrite content in those clays if they are disturbed sufficiently to permit oxidisation. The "stiff" and "very stiff" consistency descriptions were probably based in part on the vane test results in BH2, which are known to over-estimate the shear strength of fissured clays.

- Hydrogeology
    - Groundwater pressures may be close to fully hydrostatic (which means that the water pressure increases linearly with depth) or may be modified locally by seepage/flow pressures and/or under-drainage (via permeable layers which are drained further downslope). Groundwater flow through these clays is likely to be limited to minor seepage through any of the silt/sand horizons which are laterally extensive and/or sufficiently interconnected. Neither of the boreholes found any individual silt/sand layers of sufficient size to warrant separate identification, unlike many of the nearby boreholes (see above and Table 2). Groundwater strikes were recorded at 6.1 and 7.1m bgl in BH1 (rear garden) and BH2 (front parking area) respectively, and the highest groundwater standing level recorded during the short period of monitoring was 1.53m bgl in BH1 (see Table 3 in Section 6, and Table 4 in Section 9, for groundwater records on nearby sites and this site respectively).
    - The hydrogeology may be complicated further by the backfill in service trenches and granular pipe bedding (where present) forming preferential groundwater flow pathways within the strata they pass through.

10.1.2 The hydrogeological regime outlined above will be affected by long-term climatic variations as well as seasonal fluctuations, all of which must be taken into account when selecting a design water level for the permanent works. No multi-seasonal monitoring data are available, so a conservative approach will be needed, in accordance with current geotechnical design standards which require use of 'worst credible' groundwater levels/pressures. See paragraph 10.2.7 for the recommended provisional design groundwater levels.

10.1.3 No railway tunnels for the main operational lines are known to pass below the site. Other infrastructure including tunnels, sewers, cables or communications might be present within the zone of influence of the proposed basement, so an appropriate services search should be undertaken. If any such infrastructure is identified, then its potential influence on the proposed basement must be assessed. These searches will not identify any private services.

10.2 Subterranean (Groundwater) Flow – Permanent Works

10.2.1 The Made Ground, where seen in the exploratory holes, generally comprised sandy, silty clays, which are low permeability materials so will permit little or no flow of any perched groundwater. In two areas, alongside the front entrance (TP4) and as bedding to some of the concrete surfacing (TP2A), the Made Ground consisted of sand, which is likely to hold perched water at times and would permit much higher flow rates. While the house foundations and existing cellar/basement are likely to restrict seepage/flow of any perched groundwater from the rear to the front of the site (downslope), some flow is possible beneath the side access path and adjacent driveway to the rear garages (that flow, if any, is expected to follow broadly the fall of the slope towards the south-east). Thus, no significant flow is expected through the Made Ground, except possibly where it consists of sand, or where service trenches or granular pipe bedding facilitates channelled flow. On clay sites, groundwater in the backfill to footing trenches is typically static (until excavations are dug into/though the backfill).

10.2.2 The groundwater seepages and strikes recorded in BH1 at 4.1m and 6.1m bgl, and in BH2 at 5.3m and 7.1m bgl, indicate that mobile groundwater was present at multiple levels within the clays of the Claygate Member. This groundwater is probably confined to the beds/laminations of silty sand and sandy silt which are believed to be present within the Claygate Member, even though neither of the site-specific boreholes found any individual silt/sand layers of sufficient size to warrant separate identification; unlike many of the nearby boreholes (see 10.1.1 above and Table 2 in Section 4).

10.2.3 The proposed founding depths for the main part of the proposed basement and the pool area are around 4.5m and 7.3m below the existing ground floor level respectively. Thus, the basement is expected to be founded in the clays of the Claygate Member. The proposed basement will create an obstruction to any seepage/flow of groundwater in the silt/sand laminations/beds and may result in a local increase in water levels/pressures on the upslope side of the basement. Provided that the silt/sand horizons are sufficiently laterally extensive to reach the boundaries of the site, then any natural flow of groundwater in these horizons would be able to continue to flow around the new basement. This behaviour is acknowledged in the Camden GHHS, which noted that even extensive excavations for basements in the City of London have not caused any serious problems in 'damming' groundwater flow, with groundwater simply finding an alternative route (Arup, 2010, paragraph 205). Thus, given the apparent lack of deep basements beneath No.5a and the adjacent access driveway to the rear garages, it is considered unlikely that construction of this basement box would cause any unacceptable adverse impact. The car lift will be in the downstream 'shadow' of the main basement, so will not create any additional impact.

10.2.4 Elliott Wood's 'Lower Ground Plan' (Drg No.S-90 Rev P4) shows that a contiguous bored pile wall (BPW) will form the rear wall of the proposed basement, as well as small sections of the south-west and north-east side walls. This contiguous piled wall will provide a deeper restriction to groundwater flow but will permit flow between the piles below basement level, which will help to limit any build-up of groundwater pressure and levels on the upslope side of the BPW.

10.2.5 If the bored piles encounter a permeable sand deposit of limited lateral continuity (such as a channel deposit) which would be fully obstructed by the basement, then construction of a groundwater bypass would be required in order to allow continued flow of that groundwater past the basement. It is recommended that the rear part (upslope side) of the basement is constructed first, so that the extent of permeable horizons on that side of the basement can be assessed and a decision taken regarding the need for the bypass. The specific geometry of the collection and discharge elements of the bypass would depend on the location of the more permeable deposit, on both the upslope and downslope sides of the house, so cannot be designed/detailed until the works are in progress.

10.2.6 Standpipes were installed in both of CSI's site-specific boreholes, and a maximum groundwater standing level of 1.53m bgl was recorded in BH1 during the short period of monitoring. These standpipes should be maintained so that further readings can be taken during detailed design and immediately prior to the start of the works.

10.2.7 Current geotechnical design standards require use of a 'worst credible' approach to selection of groundwater pressures. Based on the above groundwater standing levels, use of provisional design groundwater levels are recommended at:

- 0.5m below the rear lawn,
- ground level around the rear of the house and alongside both flank walls,
- 0.5m bgl alongside the front (downslope) wall.

These levels are subject to a further review (upwards/shallower only) once new readings from the standpipes and the results from a trial excavation (if required, see Section 10.3) are available.

10.2.8 The basement structure must be designed to resist the buoyant uplift pressures which would be generated by groundwater at ground level. The variable depth of the proposed basement, along with the variable ground levels on site, mean that the uplift pressures will also vary, from approximately 28kPa to 68kPa within the main part of the basement, to around 58kPa to 73kPa within the proposed pool area (both un-factored). Use of tension piles are likely to be required to resist the hydraulic uplift forces, at least beneath the pool area within the basement.

10.2.9 The proposed basement will need to be fully waterproofed in order to provide adequate long-term control of moisture ingress from the groundwater. Detailed recommendations for the waterproofing system are beyond the scope of this report although it is noted that, as a minimum, it would be prudent for the system to be designed in compliance with the requirements of BS8102:2009.

10.2.10 The National House Building Council published new guidance on waterproofing of basements in November 2014 (now NHBC Standards, 2016, Chapter 5.4). Compliance would be compulsory if an NHBC warranty is required, otherwise it may provide a useful guide to best practice.

Cumulative Impact:

10.2.11 No cumulative impact on groundwater flow is anticipated from construction of the proposed basement, as set out in paragraph 10.2.3 above.

10.3 Subterranean (Groundwater) Flow – Temporary Works

10.3.1 As noted above, the seepages and strikes in both boreholes at depths of 4.1-7.1m showed that free groundwater is present at multiple levels within the depth of excavations which will be required for the basement. Such water is likely to occur in thin beds and laminations of silt/sand within the clays, and which were not recorded individually on the borehole logs.

10.3.2 The method of construction will influence the method of groundwater control required. For both the underpins beneath the house and for the excavations within the contiguous bored pile wall around the rear part of the basement (see 10.2.5 above), use of well-pointing techniques around the perimeter of the basement excavations is likely to be the most appropriate form of groundwater control, with suitably screened well-points to minimise the removal of fines, subject to advice from a dewatering specialist. Pumping from additional screened well-points or sumps in the centre of the excavation is also likely to be required to prevent hydraulic heave of the basement formation (the founding soils). Monitoring of groundwater pressures will be required to assess the need for, and efficacy of, such pumping.

10.3.3 Consideration should be given to undertaking a trial excavation during the detailed design phase, to the full depth of the proposed basement, in order to assess the likely extent of groundwater flow into the underpin excavations, and the adequacy of well-pointing to control that flow. The trial should be located centrally to the upslope side of the proposed pool area. If well-pointing is found to be inadequate, then use of a secant bored pile wall would be required, or some variation thereof, in order to fully exclude groundwater entries on the upslope side of the basement excavations.

10.3.4 A careful watch should be maintained to check that pumping from the well-points or sumps does not result in removal of fines from the adjoining ground; if any such removal of fines is noticed, then pumping should stop, the excavation may need to be partially backfilled, and the advice of a suitably experienced and competent ground engineer should be sought.

10.3.5 Where the formation level onto which the underpins and the basement slab will bear consists of clays, they must be protected from water, because they would soften rapidly if water gets onto these surfaces. Thus, the formation should be blinded with concrete immediately following excavation and inspection.

10.3.6 Irrigation systems in neighbouring gardens can also contribute significantly to water entries so, if such systems are present in the adjoining gardens, then the owners should be asked to avoid excessive use during the basement construction period.

10.4 Slope and Ground Stability

Slope Stability

10.4.1 Slope angles in excess of 7° have been recorded within the rear garden to this property, and on the access driveway to the garages. Construction of the upslope perimeter of the basement using a contiguous bored pile wall (BPW), as proposed by Elliott Wood, is therefore entirely appropriate owing to the possible presence of shear surfaces associated with soliflucted clays (if Head deposits are present). The bored piles must be designed accordingly (see 10.4.12).

10.4.2 Normal support requirements (see 10.4.5) will apply for the underpins beneath the remainder of the perimeter of this basement.

Retaining Wall Construction and associated Ground Movements - Underpinning

10.4.3 Underpinning techniques will be used for construction of those parts of the basement's perimeter walls located beneath walls in the house which will be retained, as shown on Elliott Wood's Drg No.2150493/S-90 P4. Similar reinforced concrete (RC) retaining wall panels will be required for the front and side lightwells, and for the internal perimeter of the swimming pool area (ie: where not formed by the BPWs). Underpinning methods involve excavation of the ground in short lengths in order to enable:

- the stresses in the ground to 'arch' onto the ground or completed underpinning on both sides of the excavation;
- the loads from the wall being underpinned to span across the unsupported section.

10.4.4 Some ground movement is inevitable when basements are constructed. When underpinning methods are used, the magnitude of the movements in the ground being supported by the new basement walls is dependent primarily on:

- the geology;
- the adequacy of temporary support to both the underpinning excavations and the partially complete underpins prior to installation of full permanent support;
- the quality of workmanship when constructing the permanent structure.

A high quality of workmanship and the use of high stiffness temporary support systems, installed in a timely manner in accordance with best practice methods, are therefore crucial to the satisfactory control of ground movements alongside basement excavations (see also 10.4.6 and 10.4.7 below).

10.4.5 The minimum temporary support requirements recommended for the proposed underpins and retaining walls at No.5, subject to inspection and review as described in 10.4.7 below, are:

- Once dewatered, closely spaced support must be installed as the basement excavations progress. If seepages are experienced from the silt/sand layers, then full face support would be required.
- Temporary support will be required to all the new underpins and RC retaining wall panels, and must be maintained until the full permanent support has been completed, including allowing time for the concrete to gain adequate strength.

10.4.6 Under UK standard practice, the contractor is responsible for designing and implementing the temporary works, so it is considered essential that the contractor employed for these works should have completed similar schemes successfully. For this reason, careful pre-selection of the contractors who will be invited to tender for these works is recommended. Full details of the temporary works should be provided in the contractor's method statements.

10.4.7 In accordance with normal health and safety good practice, the requirements for temporary support of any excavation must be assessed by a competent person at the start of every shift, and at each significant change in the geometry of the excavations as the work progresses. The possible presence of shear surfaces in the clays must be allowed for.

10.4.8 The construction sequence will be covered in the structural engineer's Construction Method Statement.

Retaining Wall Construction and associated Ground Movements - Bored Pile Wall

10.4.9 The proposed bored pile wall (BPW) around the rear part of the basement (see paragraph 3.3 and Elliott Wood's Drg No.2150493/S-90 P4) will be constructed of 350mm diameter piles, with an internal 200mm thick lining wall. A 'bottom-up' construction sequence will be unavoidable for the rear lightwell (which, after installation of the BPW, involves installation of temporary support as the basement is excavated down to the level of the basement slab) whereas a 'top-down' construction sequence could be considered for the swimming pool area. The latter would involve construction of the new ground floor slab at pile head level prior to excavation of the basement space beneath the new slab). Use of high stiffness temporary support in accordance with CIRIA Report C580 will be required in both cases, in order to minimise the deflections of the BPW as the basement is excavated. The temporary support must be maintained in place until all the piles and the whole of the basement slab has gained full working strength.

10.4.10 Consideration should be given to extending the BPW further forward along the right flank wall, instead of the deep underpins currently proposed, to include at least the remainder of the flank wall of the swimming pool area (which will be feasible because the existing two-storey projection on that side of the house will be taken down and replaced).

Geotechnical Design

10.4.11 Design of the basement retaining walls must take into consideration:

- Earth pressures from the surrounding ground (see also paragraph 10.4.12 below);
- Dead and live loads from the superstructure of the house;
- Loads acting on the basement slab, which will create surcharge loads on the internal underpins/retaining walls for the swimming pool area;
- Loads from the adjacent part of No.5a which will create a surcharge load on the perimeter basement wall;
- Surcharge from vehicles on the forecourt parking area and on the driveway to the garages;
- Surcharges, or increased earth pressure coefficients, to allow for the rise of the rear garden alongside the upslope wall to the basement;
- Normal surcharge allowances elsewhere;
- Swelling displacements/pressures from the underlying clays;
- A provisional design groundwater level at ground level to 0.5m below the adjoining ground level (see paragraph 10.2.7);
- Precautions to protect the concrete from sulphate attack (see paragraph 9.13).

10.4.12 The following geotechnical parameters should be used when calculating earth pressures:

Made Ground:	Unit weight, γ_b:	18.0 kN/m^3
(sandy clays)	Effective cohesion, c':	0 kPa
	Angle of internal friction, φ':	25°
Claygate Member:		
Firm Clay, possibly soliflucted (for BPW, to 3.0m bgl):		
	Unit weight, γ_b:	19.0 kN/m^3
	Effective cohesion, c':	0 kPa
	Angle of internal friction, φ':	15°
Stiff Clays:	Unit weight, γ_b:	20.0 kN/m^3
	Effective cohesion, c':	0 kPa
	Angle of internal friction, φ':	25° for sandy CLAYS
	Angle of internal friction, φ':	22° for silty CLAYS
Sands:	Unit weight, γ_b:	18.0 kN/m^3
	Effective cohesion, c':	0 kPa
	Angle of internal friction, φ':	32°

Coefficient of earth pressure at rest, k_0: 1.5 for bored pile walls, 1.0-1.5 for underpins (depending on stiffness of support), both after release of higher pressures in response to installation of piles/ underpins.

These parameters should be used in conjunction with appropriate partial factors dependent upon the design method selected.

10.4.13 The formation level clays onto which the underpins and the basement slab will bear must be protected from water to prevent softening and loss of strength, as described in 10.3.5 above.

10.4.14 The low boundary retaining wall which supports part of the driveway to the rear garages will need to be extended alongside the rear lightwell and to the rear of that lightwell (see Brod Wight Architect's 'Proposed South West Side Elevation', Drg No.1046-AP10). Sufficient movement joins must be provided to allow for any heave which develops following removal of the nearby large Sycamore tree.

10.4.15 Transition underpins should be installed beneath load-bearing walls which adjoin the retaining walls for the swimming pool area. These underpins should step up in accordance with the Building Regulations and general good practice.

Cumulative Impact:

10.4.16 No cumulative impact would be expected on ground stability aspects from construction of the proposed basement.

10.5 Settlement/Heave and Damage Category Assessment

Basement Geometry and Stresses:

10.5.1 Analyses of vertical ground movements (heave or settlement) have been undertaken using PDISP software, in order to assess the potential magnitudes of movements which may result from the changes of vertical stresses caused by excavation of the basement. These preliminary analyses have not modelled the horizontal forces on the retaining walls, so have simplified the stress regime.

10.5.2 Figure D1 (see Appendix D) provides an extract from Elliott Wood's 'Lower Ground Plan' (Drg No.2150493-S-90 Rev.P4), which shows the layout of the proposed basement beneath No.5 Templewood Avenue. Figure D2 illustrates the layout of the PDISP zones used to model the underpins, basement slab, and the bored pile wall, based on information provided by Elliott Wood, including their load takedown summary, hand annotated on a copy of the same 'Lower Ground Plan' drawing. The maximum overall dimensions of the proposed basement are approximately 22.3m wide by 27.5m long.

10.5.3 Table 5 presents the net changes in vertical pressure for four major stages of the stress history of the basement's construction, as detailed in paragraph 10.5.7 below. Based on the load takedown data provided by Elliott Wood, net pressure changes were calculated, including the gross unloading from the excavation, and the gross bearing pressures which will be applied by the underpins and contiguous bored pile wall. The contiguous bored pile wall was assumed to extend 6.0m below basement formation level, giving pile lengths of around 9.5m within the main upper part of the basement, and 12.3m within the proposed pool area. The BPW was then divided into a total of four sections (zones a-d), with the upper section (zones a) modelled from ground level to basement formation level, and three 2.0m long sections (zones b-d) modelled below basement formation level, with the loads on BPW acting at the base of each of these four sections. It should be noted that Zones 67, 68 & 69 (coloured green, pink and yellow respectively) are superimposed zones, which have been added in order to modify the excavation depths within the basement, due to the changes in existing ground levels on site. These superimposed zones were omitted from Stage 1 of the PDISP analysis, as bulk excavation of the basement to formation level will not occur until Stage 2 of the PDISP analysis (see paragraph 10.5.7 below).

10.5.4 The tension piles which have been recommended to resist hydraulic uplift have not been included in these analyses, as they will have only a limited influence on the heave resulting from stress relief.

Table 5: Coordinates and net bearing pressures for PDISP							
ZONE	Centroid		Dimensions		Net change in vertical pressure (kPa)		
#	Xc(m)	Yc(m)	X(m)	Y(m)	Stage 1	Stage 2	Stages 3 and 4
1a	11.04	0.13	3.25	0.75	26.80	26.80	26.80
2a	9.79	2.38	0.75	3.75	26.80	26.80	26.80
3a	9.04	4.63	2.25	0.75	26.80	26.80	26.80
4a	8.29	6.48	0.75	2.95	26.80	26.80	26.80
5a	6.71	7.58	2.40	0.75	55.57	55.57	55.57
6a	5.89	10.13	0.75	4.35	66.77	66.77	66.77
7a	5.69	12.68	1.15	0.75	66.77	66.77	66.77
8a	5.49	14.80	0.75	3.50	66.77	66.77	66.77
9a	5.69	16.93	1.15	0.75	66.77	66.77	66.77
10a	5.89	18.65	0.75	2.70	66.77	66.77	66.77
11	20.32	21.01	3.92	1.80	-46.53	-46.53	-46.53
12	23.58	21.01	2.60	1.80	22.25	22.25	22.25
13	23.41	18.31	2.25	3.60	-62.70	-62.70	-52.70
14	26.16	18.31	3.25	3.60	-6.83	-6.83	-6.83
15	27.86	15.31	4.55	2.40	-18.58	-18.58	-18.58
16	Polygonal zone				7.31	7.31	7.31
17	28.37	2.51	3.52	2.39	-18.42	-18.42	-18.42
18	24.26	2.91	4.70	2.19	-11.15	-11.15	-11.15
19	Polygonal zone				-33.77	-33.77	-33.77
20	17.29	1.20	9.25	2.00	-2.88	-2.88	-2.88
21	15.71	7.26	5.15	2.58	-92.26	-92.26	-92.26
22	20.28	8.27	4.00	3.60	13.14	13.14	13.14
23	24.86	5.75	3.50	3.50	21.50	21.50	31.50
24	25.36	10.65	2.50	2.50	80.50	80.50	90.50
25	20.28	10.55	4.00	0.96	-99.34	-99.34	-99.34
26	15.71	10.43	5.15	1.21	-115.90	-115.90	-105.90
27	21.77	12.78	1.02	3.50	-80.16	-80.16	-80.16
28a	8.54	19.63	4.55	0.75	36.08	36.08	36.08
29a	11.19	20.13	0.75	1.75	36.08	36.08	36.08
30a	14.96	20.63	6.80	0.75	36.08	36.08	36.08
31	10.21	18.61	4.90	0.55	-17.65	-17.65	-17.65
32	7.51	15.18	0.50	7.30	-44.70	-44.70	-44.70
33	14.26	11.28	14.00	0.50	18.30	18.30	18.30
34	21.01	12.78	0.50	2.50	-44.70	-44.70	-44.70
35	20.26	14.28	2.00	0.50	-44.70	-44.70	-44.70

Table 5: Coordinates and net bearing pressures for PDISP (continued)							
ZONE	Centroid		Dimensions		Net change in vertical pressure (kPa)		
#	Xc(m)	Yc(m)	X(m)	Y(m)	Stage 1	Stage 2	Stages 3 and 4
36	19.51	16.43	0.50	3.80	-44.70	-44.70	-44.70
37	18.61	18.58	2.31	0.50	22.30	22.30	22.30
38	17.20	19.01	0.50	1.36	22.30	22.30	22.30
39	15.06	19.44	3.79	0.50	22.30	22.30	22.30
40	12.91	19.01	0.50	1.36	22.30	22.30	22.30
41	11.65	3.60	2.97	2.80	0.00	-62.70	-52.70
42	15.71	4.09	5.15	3.77	0.00	-62.70	-52.70
43	15.71	9.19	5.15	1.27	0.00	-115.90	-105.90
44	10.90	6.48	4.47	2.95	0.00	-54.34	-44.34
45	10.90	8.75	4.47	1.60	-105.90	-105.90	-105.90
46	10.90	10.29	4.47	1.48	0.00	-115.90	-105.90
47	7.46	9.49	2.40	3.08	-115.90	-115.90	-105.90
48	6.06	14.80	0.40	3.50	0.00	-115.90	-105.90
49	6.76	14.93	1.00	7.80	0.00	-115.90	-105.90
50	23.20	10.65	1.83	2.50	0.00	-62.70	-62.70
51	28.39	8.90	3.56	10.40	0.00	-62.70	-52.70
52	24.45	8.45	4.33	1.90	0.00	-62.70	-52.70
53	23.93	15.31	3.30	2.40	0.00	-62.70	-52.70
54	24.45	13.01	4.34	2.21	0.00	-62.70	-52.70
55	21.33	17.34	1.90	5.55	-95.88	-95.88	-95.88
56	13.51	14.93	11.50	6.80	0.00	-115.90	-105.90
57	20.01	12.78	1.50	2.50	0.00	-115.90	-105.90
58	15.06	18.78	3.79	0.89	0.00	-115.90	-105.90
59	8.91	19.07	5.30	0.37	0.00	-115.90	-105.90
60	11.41	1.35	2.50	1.70	0.00	-62.70	-52.70
61	12.11	19.54	1.10	1.42	0.00	-115.90	-105.90
62	17.91	19.54	0.91	1.42	0.00	-115.90	-105.90
63	19.06	19.48	1.40	1.26	0.00	-115.90	-105.90
64	15.06	19.97	4.79	0.56	0.00	-115.90	-105.90
65	20.07	17.34	0.62	5.55	0.00	-115.90	-105.90
66	Polygonal zone				0.00	-62.70	-52.70
67	14.01	4.02	3.55	7.64	0.00	-22.80	-22.80
68	10.83	3.80	2.81	8.09	0.00	-13.30	-13.30
69	9.50	12.13	3.17	15.75	0.00	-28.50	-28.50

Table 5: Coordinates and net bearing pressures for PDISP (continued)							
ZONE	Centroid		Dimensions		Net change in vertical pressure (kPa)		
#	Xc(m)	Yc(m)	X(m)	Y(m)	Stage 1	Stage 2	Stages 3 and 4
2b-d	9.79	2.38	0.75	3.75	17.80	17.80	17.80
3b-d	9.04	4.63	2.25	0.75	17.80	17.80	17.80
4b-d	8.29	6.48	0.75	2.95	17.80	17.80	17.80
5b-d	6.71	7.58	2.40	0.75	46.57	46.57	46.57
6b-d	5.89	10.13	0.75	4.35	40.97	40.97	40.97
7b-d	5.69	12.68	1.15	0.75	40.97	40.97	40.97
8b-d	5.49	14.80	0.75	3.50	40.97	40.97	40.97
9b-d	5.69	16.93	1.15	0.75	40.97	40.97	40.97
10b-d	5.89	18.65	0.75	2.70	40.97	40.97	40.97
28b-d	8.54	19.63	4.55	0.75	10.28	10.28	10.28
29b-d	11.19	20.13	0.75	1.75	10.28	10.28	10.28
30b-d	14.96	20.63	6.80	0.75	10.28	10.28	10.28

Ground Conditions:

10.5.5 The ground profile was based on the site-specific ground investigation by CSI, as presented in Sections 9 and 10.1 above, together with the desk study information.

10.5.6 The short-term and long-term geotechnical properties of the soil strata used for the PDISP analyses are summarised in Table 6. They were based on the findings of site-specific investigation and data from previous projects.

Table 6: Soil parameters for PDISP analyses				
Strata	**Level** (m bgl)	**Undrained Shear Strength** Cu (kPa)	**Short term, undrained Young's Modulus,** Eu (MPa)	**Long term, drained Young's Modulus,** E' (MPa)
Stiff, sandy, silty CLAY (Claygate Fm/ London Clay Fm)	3.29 57.20	80 404	40 242	24 145
Where: For CLAY: Undrained Young's Modulus, Eu = 40 + 3.75 z where z = depth below the top of the stratum (3.29mbgl). Drained Young's Modulus, E' = 0.6 Eu				

PDISP Analyses:

10.5.7 Three dimensional analyses of vertical displacements have been undertaken using PDISP software and the basement geometry, loads/stresses and ground conditions outlined above in order to assess the potential magnitudes of ground movements (heave or settlement) which may result from the vertical stress changes caused by excavation of the basement. PDISP analyses have been carried out as follows:

- Stages 1 – Construction of underpins, retaining walls and BPW – Short-term condition
- Stage 2 - Bulk excavation to formation level – Short-term condition
- Stage 3 – Construction of basement slabs – Short-term (undrained) conditions
- Stage 4 – As Stage 3, except – Long-term (drained) conditions

10.5.8 The results of the analyses for Stages 1 to 4 are presented as contour plots on the appended Figures D3 to D6.

Heave Assessment and associated Recommendations:

10.5.9 Construction of the basement will cause immediate elastic heave and settlements in response to the stress changes identified in Table 5, followed by long-term plastic swelling as the unloaded over-consolidated clays take up groundwater. The rate of plastic swelling in over-consolidated clays will be determined largely by the permeability of the clay and the availability of water. As a result, the rate of swelling may be relatively rapid where water-bearing laminations of silt/sand are present in the sandy clays of the Claygate Member, whereas in the low permeability of the homogenous, unbedded, silty clays in the Claygate Member/London Clay Formation the swelling can take decades to reach full equilibrium. The structure of this basement will need to be designed so as to enable it to accommodate the swelling displacements/pressures developed underneath it.

10.5.10 The ranges of predicted short-term and long-term movements for each of the main parts of the proposed basement are presented in Table 7 below. These analyses indicated that the perimeter walls are likely to experience movements ranging from minor heave and settlement in Phase 1 to moderate heave movements following excavation of the basement, while the ground beneath the basement slabs is likely to experience larger total heave movements (to in excess of 20mm). In general, the predicted heave movements are greater around the proposed pool and Jacuzzi, owing to the greater depth of excavation in that area. It should be noted that, due to the relatively coarse contour intervals, the ranges of predicted movements recorded in Table 7 are only approximate.

Table 7: Summary of predicted displacements – Basement beneath House				
Location	**Stage 1** **(Figure D3)**	**Stage 2** **(Figure D4)**	**Stage 3** **(Figure D5)**	**Stage 4** **(Figure D6)**
Front wall	0 – 1mm Heave	2 - 4mm Heave	0 – 2.5mm Heave	2.5 – 5mm Heave
SW Flank wall	0 – 2mm Heave	2 - 7mm Heave	2 - 6mm Heave	2.5 - 10mm Heave
Bored pile wall	0 – 3mm Settlement	0 – 7.5mm Heave	0 – 7mm Heave	0 – 12mm Heave
NE flank wall house/annex	0 – 3mm Heave	2 - 7mm Heave	2 - 7mm Heave	2.5 - 12mm Heave
Upper level of basement slab	5mm Heave to 2mm Settlement No slab present	2.5-10mm Heave	0-10mm Heave	2.5 - 17.5mm Heave
Pool area	(5mm Heave to 2mm Settlement) No slab present	3 – 13mm Heave	3 – 13mm Heave	3 – 21mm Heave

10.5.11 All the short-term elastic displacements would have occurred in Stages 1-3, before the concrete of the basement slab has set/cured, so only the post-construction incremental heave (from Stages 3 to 4) is relevant to the slab design. The analyses indicated that the maximum predicted post-construction displacements beneath the slab are likely to range from around 5 - 10mm heave, within both the upper part of the basement slab, and the pool area. Differential movements across the slab are also unlikely to exceed the same range.

10.6 Damage Category Assessment

10.6.1 When underpinning or constructing RC retaining walls in underpin-like panels, it is inevitable that the ground will be un-supported or only partially supported for a short period during excavation of each pin, even when support is installed sequentially as the excavation progresses. This means that the behaviour of the ground will depend on the quality of workmanship and suitability of the methods used, so rigorous calculations of predicted ground movements are not practical and there are no published empirical data on such ground movements. However, provided that the temporary support follows best practice as outlined in Section 10.4 above, then extensive past experience has shown that the bulk horizontal movements of the ground alongside the basement caused by underpinning for a single-storey basement should not exceed 5mm horizontally. Similarly, in the absence of any published data on settlements caused by relaxation of the ground alongside underpinning works, these have been estimated using data from CIRIA Report C580 (Gaba et al, 2003) for embedded retaining walls.

10.6.2 The worst case scenarios, for damage category assessment purposes, will be:

- No.3: the rear wall of the main house (which has been assumed to continue on the same line at the junction with the rear extension), because the depth of excavation for the proposed basement will be greater there, while the rear wall of No.3's rear extension is relatively less vulnerable to damage owing to its restricted width;
- No.5a: the rear wall of the front part of the house, which is very close to the front wall of the existing projection on the north-east side of No.5, while there is understood to be a large covered courtyard behind that wall (the OS plan in Figure 1 is misleading, because it shows the glazed roof over the courtyard as though it were part of the masonry structure.

The car lift will be more remote from the front wall of No.3 than the main basement, and will provide a small beneficial increase in heave, so does not require further consideration.

10.6.3 In order to relate these typical ground movements to possible damage which adjoining properties might suffer, it is necessary to consider the strains and the angular distortion (as a deflection ratio) which they might generate using the method proposed by Burland (2001, in CIRIA Special Publication 200, which developed earlier work by himself and others).

<u>No.3's main Rear Wall:</u>

10.6.4 Ground movements associated with the construction of retaining walls in clay soils have been shown to extend to a distance up to 4 times the depth of the excavation. No.3's main rear wall is on the same line as No.5's main rear wall, so it is opposite a section of the basement which will be constructed using underpinning techniques, with a depth of excavation of 3.3m (internal) and about 4.1m below the external path level (101.1m AOD). Measuring off the 'Proposed Front Elevation' by Brod Wight Architects (Drg No.1046-AP07) gives a separation of 4.82m between the two houses. So the damage category calculations for the rear wall of No.3 are as follows:

Zone of influence from basement = 4.1 x 4 = **16.4m**
Distance to No.3 = 4.8m
Width (L) of No.3 affected = 16.4 – 4.8 = **11.6m**
Height (H) = 7.45m to eaves level
Hence L/H = **1.56**

Thus, for the anticipated 5mm maximum horizontal displacement, the strain beneath No.3 would, theoretically, be in the order of $\varepsilon_h = 3.05 \times 10^{-4}$ (0.031%).

10.6.5 The nil vertical displacement predicted by the PDISP analysis during Stage 1 for this part of No.5's south-west flank wall represents the worst case scenario, because the heave in subsequent stages would be beneficial. Thus, only the settlement caused by relaxation of the ground alongside the basement in response to excavation of the underpins is relevant to the damage category assessment. Using the low support stiffness profile on Figure 2.11b from CIRIA Report C580, with allowance for the 4.8m gap between the houses (= 1.2 x depth of excavation), gave a maximum deflection beneath the rear wall of No.3, Δ = 0.024% of excavated depth = 0.98mm. This represents a deflection ratio, Δ/L = 8.45 x 10^{-5} (0.008%).

10.6.6 Using the graphs for L/H = 1.5, these deformations represent a damage category of 'negligible' (Burland Category 0, ε_{lim} = <0.05%) as given in CIRIA SP200, Table 3.1, and illustrated in Figure 11 below.

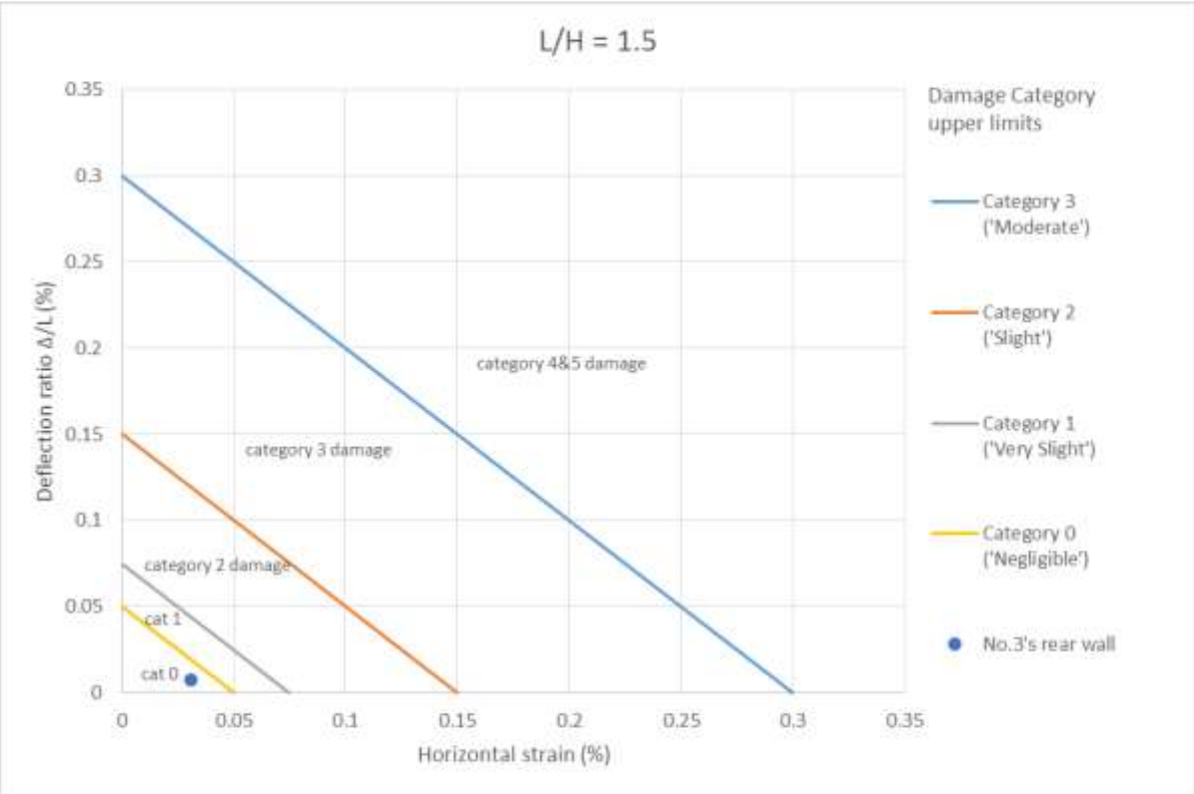


Figure 11: Damage category assessment for front wall of No.12, the critical location.

No.5a, Rear Wall to front part of house:

10.6.7 The separation between the proposed basement and the rear wall to the front part of No.5a is so small that it has been ignored. The depth of excavation for the underpins will be 3.45m below the external ground level (100.45m AOD). So the damage category calculations for this wall are as follows.

Zone of influence from basement = 3.45 x 4 = **13.8m**

Width (L) of No.5a = approx. **12.4m** (scaled from Figure 1, and assumes wall continues full width)

Height (H) = 104.54 – 100.45 + 1.0 = **5.1m** (single-storey, mainly parapet wall + footing depth)

Hence L/H = **2.43**

Thus, for the anticipated 5mm maximum horizontal displacement, the strain beneath No.5a would, theoretically, be in the order of $\varepsilon_h = 3.62 \times 10^{-4}$ (0.036%).

10.6.8 Once again a zero vertical displacement was predicted by the PDISP analysis in Stage 1 for this part of No.5's north-east flank wall and only the settlement caused by relaxation of the ground alongside the basement in response to excavation of the underpins is relevant to the damage category assessment. Using the low support stiffness profile on Figure 2.11b from CIRIA Report C580, gave a maximum deflection beneath this wall of, Δ = 0.054% of excavated depth = 1.86mm. This represents a deflection ratio, $\Delta/L = 1.5 \times 10^{-4}$ (0.015%).

10.6.9 Using the graphs for L/H = 2, these deformations represent a damage category of 'very slight' (Burland Category 1, ε_{lim} = 0.05-0.075%) as given in CIRIA SP200, Table 3.1, and illustrated in Figure 12 below.

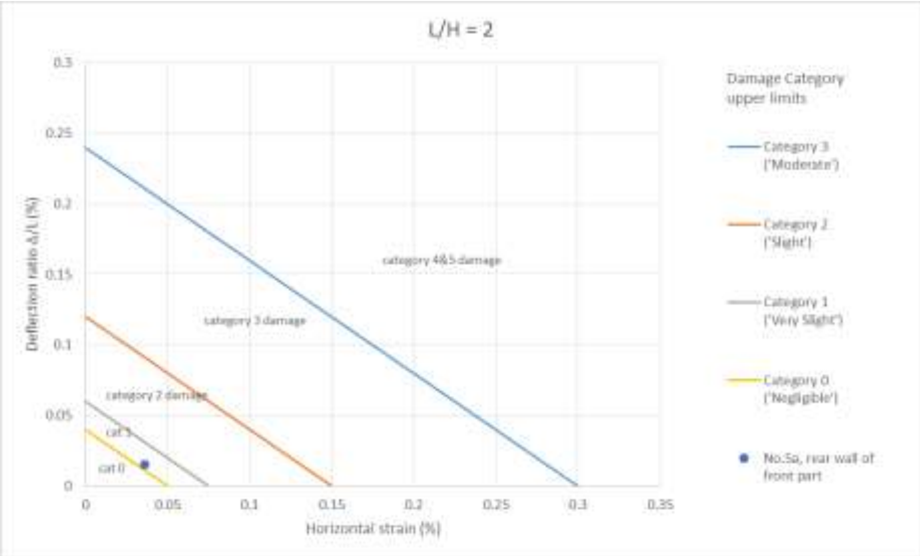


Figure 12: Damage category assessment for rear wall of front part of No.5A.

10.6.10 Use of best practice construction methods, as outlined in paragraphs 10.4.4 to 10.4.7 above, will be essential to ensure that the ground movements are kept in line with the above predictions.

10.7 Monitoring

10.7.1 Condition surveys should be undertaken of the neighbouring properties before the works commence, in order to provide a factual record of any pre-existing damage. Such surveys are usually carried out while negotiating the Party Wall Agreement and are beneficial to all parties concerned, although in this instance the Party Wall Act may not be triggered for No.3. The search of planning applications on LBC's planning website found an application for No.5a Templewood Avenue, relating to tree works (Application Ref: 2014/4018/T). A 'Technical Report' was found with this application on the website, which gave details of damage to No.5a which was attributed to a magnolia tree located alongside the south-west flank wall to No.5a. It is likely that this damage has already been repaired.

10.7.2 Precise movement monitoring should be undertaken weekly throughout the period during which the basement walls and slabs are constructed, with initial readings taken before excavation of the basement starts (preferably three initial sets, in order to assess any on-going movements from other causes). Readings may revert to fortnightly once all the perimeter walls and the basement slabs have been completed. This monitoring should be undertaken with a total station instrument and targets attached at a minimum of two levels at the following locations:

- at four equally spaced intervals along the NE flank wall of No.3, including the rear extension,
- at the west (rear left) corner of the rear extension to No.3,
- at the middle of the front wall to No.3,
- at the front and rear corners of the SW flank wall of the single-storey extension to No.5a,
- at the front and rear corners of the SW flank wall of No.5a's living room,
- at the middle and north-east end of the rear wall to the front part of No.5a,
- at the front left (SW) corner and middle of the front wall to No.5a,
- at the client's discretion, since outside any Party Wall Agreement, it would also be sensible to monitor all the load-bearing walls in No.5 which will be underpinned.

10.7.3 The accuracy of this system of monitoring is usually quoted as +/- 2mm. Thus, if recorded movements in either direction reach 5mm, then the frequency of readings should be increased as appropriate to the severity of the movement and consideration should be given to installing additional targets. If the recorded movements in either direction reach 8mm, then work should stop until new method statements have been prepared and approved by the appointed structural engineer.

10.7.4 If any structural cracks appear in the main loadbearing walls, then those cracks should be monitored using the Demec system (or similar) on the same frequency as the target monitoring.

10.8 Surface Flow and Flooding

10.8.1 The evidence presented in Section 5 has shown that:

- the site lies within the Environment Agency's Flood Zone 1, which means that it is considered to be at negligible risk of fluvial flooding;
- the site is not at risk of flooding from reservoirs, as mapped by Environment Agency;
- Templewood Avenue is recorded as having been affected by the surface water flooding event in 2002, but not in 1975, and given the local slope angle it is likely that the area affected was downslope of No.5;
- there are no surface water features within 250m of the site;
- the nearest river is the Tertiary grade upper waters of the Golders Hill pond chain, 442m to the north of the site, which is in a different catchment and so is irrelevant to the proposed basement;
- the latest flood modelling by both the Environment Agency and within the Camden SFRA gave a 'Very Low' risk of surface water flooding (the lowest category, which represents the national background level of risk) for No.5 and the adjacent properties (see Figure 6).

10.8.2 The site is in the catchment of the former river Westbourne, on an interfluve between the uppermost two tributaries to that river. Those tributaries were culverted, or diverted into a sewer, before the area was developed. Whether the culverts remain connected hydraulically to the perennial surrounding groundwater is unknown.

<u>Change in Paved Surfacing & Surface Water Run-off:</u>

10.8.3 The changes to surfacing shown on the drawings by Brod Wight Architects will include:

- An extension at the rear of the house which will cover much of the existing rear patio;
- A new rear patio, steps, rooflights and rear lightwell which will extend beyond the existing patio, into the existing soft landscaped area.
- Extension of the basement (swimming pool area) below the rear lawn, with 1.1m of soil over its roof.
- Lightwells alongside the front and flank walls which will be in areas that are already fully paved.
- Replacement of the existing raised planter in front of the two-storey projection on the north-east side of the house, with a new one of similar size, located slightly closer to the road in order to provide a screen in front of that lightwell.

10.8.4 Infiltration of surface water will be limited because of the high groundwater levels and the predominance of clays, despite the status of the Claygate Member as a secondary aquifer.

10.8.5 The net change in the area of paved surfacing is anticipated to be a modest increase. A channel drain was present along most to the rear wall of the house, and various surface water gullies were evident, so it is anticipated that most of the water from the existing paved surfaces already discharges to the combined sewer. This facility should be replaced with a similar new channel drain alongside the rear wall of the rear extension. The potential increase in discharge of surface water to the mains drainage system may be smaller than the increase in paved surfacing would suggest, because it appeared that some of the surface water run-off from the rear lawn flows onto the patio and thence into the drainage system. The net potential increase in surface water run-off could easily be mitigated by the inclusion of appropriate Sustainable Drainage Systems (SuDS) in the scheme, such as:

- Intervention storage;
- Rainwater harvesting;
- Directing some roof water to rain gardens (which would provide only a small proportion of the mitigation required, owing to the limited permeability of such soils);

- Use of permeable paving (which, once again, would provide only a small proportion of the mitigation required).

Quantitative analysis will be required to confirm the net increase in paved surfacing and to enable design of the mitigation works.

10.8.6 Soakaways should not be used because of the high groundwater levels, and because concentrated local increases in groundwater levels would be detrimental to the stability of the slopes concerned. For the same reason, the soils used over the top of the garden entertainment room should match the existing clays as closely as possible, and (under-)drainage from them should be permitted along the full length of the perimeter of that part of the basement.

Surface Water (Pluvial) Flooding:

10.8.7 In view of the 'Very Low' risk of surface water flooding predicted by both the Environment Agency and the Camden SFRA, only basic flood resistance measures will be required to protect the basement from local surface water flooding, including:

1. Provision of upstands around the proposed lightwells:
   a. for the lightwells alongside the front and flank walls, where the ground surface falls away the lightwell, these upstands may be limited to a nominal height (provided that any surface water which runs off from No.5a's garden, through the narrow gap between No's 5 & 5a, is directed away from the lightwell and is able to run-off down slope);
   b. for the rear lightwell, the upstand must be sufficient to protect the lightwell from all run-off from the rear garden, including at the top of the steps, so that the run-off is directed down the side access path or onto the rear patio;
2. Provision of a raised threshold or ramped paving leading up to the entrance door into the 1-bed flat (at rear left corner of the ground floor);
3. Installation of suitably raised thresholds at the external entrances to the basement from the lightwells;
4. Protection of the car lift in accordance with the manufacturer's instructions.

Sewer Flooding:

10.8.8 The Camden SFRA indicates that Thames Water had no records of flooding from public sewers affecting this postcode area (see 5.12). However, no drainage system can be guaranteed to have adequate capacity for all storm eventualities, and all drainage systems only work at full capacity when they are properly maintained, including emptying gullies and regular checks of the sewers themselves for condition and blockages. Maintenance of the adopted sewers is the responsibility of Thames Water, so is outside the Applicant's control and largely outside of the Council's influence. Thus, the probability of future sewer flooding affecting No.5 is considered to be very low, provided that the sewer system is well maintained and appropriate flood resistance measures are implemented, as set out below.

10.8.9 Drainage systems are designed to operate under 'surcharge' at times of peak rainfall, which means that the level of effluent in the sewers may rise to ground level. When this happens, the effluent can back-up into un-protected properties with basements or lower ground floors. During major rainfall events it is possible for some sewers to overflow at ground level, though this is rare.

10.8.10 Camden's CPG 4 requires all basements to be "*protected from sewer flooding by the installation of a positive pumped device*" (paragraph 5.11). Non-return valves and pumped loop systems must therefore be fitted on the drains serving the basement, the lightwells and the rear patio, in order to ensure that water from the mains sewer system cannot enter the basement when/if the adjacent sewer is operating under surcharge.

All drains which discharge via the same outfall as the basement must be protected, including those carrying foul water and roof/surface water. The loop systems are generally required to rise above ground level in order to provide complete protection (although there is quite a significant height difference to the road/manholes outside). A battery powered reserve pump should be fitted to ensure that the system remains functional during power cuts.

10.8.11 The pumped loops must rise high enough to create sufficient pressure head to open the non-return valves when the mains sewer flow is surcharged to ground level, otherwise the basement would once again be vulnerable to flooding while the surcharged flow continues. If it is not possible to achieve a sufficient rise of the loop then temporary interception storage would be required, to hold temporarily the predicted maximum volume of water from all relevant sources which discharge via the valve-protected outfalls (including surface water from the various roofs and the lightwells, and foul water), for the duration of the predicted surcharged flows in the sewer. If decking is used in the rear lightwell, then the area beneath the decking could be used for some of the interception storage, deepened as necessary to provide extra capacity, though it must be protected from backup of foul sewage. This temporary interception storage would require formal design to ensure satisfactory performance.

Cumulative Impact:

10.8.12 No cumulative impact would be expected on surface water flooding in the vicinity from construction of the proposed basement.

10.9 Mitigation

10.9.1 The following mitigation measures have been recommended in Sections 10.2-10.8:

- a groundwater bypass would be required if the bored piles for the rear wall encounter a permeable sand deposit of limited lateral extent which would be fully blocked by construction (10.2.5);
- tension piles are likely to be required to resist the hydraulic uplift forces, at least beneath the swimming pool area (10.2.8);
- transition pins beneath all the load-bearing walls which adjoin walls in the swimming pool area (10.4.15);
- a condition survey should be undertaken of the adjacent parts of No.5a (which is known to have suffered previous subsidence damage) before the works start; a condition survey of the adjacent parts of No.3 would also be prudent, even though the Party Wall Act may not apply (10.7.1, 10.7.2);
- use of appropriate SuDS system(s) for management of surface water, but not soakaways (10.8.5 & 10.8.6);
- provision of upstands around the lightwells, a raised threshold or ramped paving to protect the entrance to the 1-bed flat, and raised thresholds to protect all external entrances to the basement (10.8.7);
- provision of non-return valves and above ground loop systems on all outfall drains which connect to the mains drainage system, and possibly also temporary interception storage (10.8.10 & 10.8.11).

11.0 NON-TECHNICAL SUMMARY- STAGE 4

11.1 This summary considers only the primary findings of this assessment; the whole report should be read to obtain a full understanding of the matters considered.

11.2 A services search should be undertaken (10.1.3).

11.3 The proposed basement will be constructed using a combination of bored piles, underpinning, and RC retaining walls constructed in panels of limited width. Construction of this basement is unlikely to cause any unacceptable adverse impact, because groundwater flow would be able to continue around it where permeable materials are present. The car lift will not create any additional restriction on groundwater flow (10.2.1 to 10.2.4).

11.4 If the bored piles in the rear wall encounter a permeable sand deposit which would be fully blocked by the basement, then a groundwater bypass may be required (10.2.5).

11.5 Shallow groundwater levels were recorded in the standpipes to within 1.53m of ground level, so the standpipes must be maintained to allow further groundwater monitoring (10.2.6). Provisional design groundwater levels at ground level to 0.5m below the relevant external ground level are proposed, subject to review of the monitoring readings. This means that the basement must be able to resist maximum buoyant uplift pressures (un-factored) which could vary from 28kPa to about 73kPa, for which tension piles will be required (10.2.7, 10.2.8). The basement will need to be fully waterproofed (10.2.9).

11.6 Groundwater control will be required while excavating the basement, probably using screened well-points. A trial excavation would be useful to check the adequacy of well-pointing and the suitability of the proposed contiguous piled wall and underpin combination (10.3.1 to 10.3.3). The clays onto which the basement slab will bear must be blinded with concrete immediately following excavation and inspection (10.3.5).

11.7 Given the slope angles in the rear garden and on the driveway to the garages, it is possible that solifluction shear surfaces may be present, so the bored pile retaining walls on the upslope side of the excavation must be designed accordingly (10.4.1).

11.8 For those parts of the basement which will be constructed using underpinning techniques and RC retaining walls, use of best practice methods and high stiffness temporary support systems, installed in a timely manner, will be crucial to the satisfactory control of ground movements around the basement (10.4.2 to 10.4.8).

11.9 A 'bottom-up' construction sequence will be unavoidable for the rear lightwell whereas a 'top-down' construction sequence could be considered for the swimming pool area. It is also recommended that the BPW should be extended further forward along the right flank wall, to include at least the remainder of the flank wall of the swimming pool area (10.4.9, 10.4.10).

11.10 Various other guidance is provided in relation to the geotechnical design and construction of the basement's perimeter walls (10.4.11 to 10.4.14).

11.11 Good practice requires stepping up between footings at different depths, so transition underpins should be provided for the load-bearing walls which adjoin the swimming pool area (10.4.15).

11.12 The basement walls and slabs must be designed to accommodate the displacements and swelling pressures generated by consolidation and heave of the underlying clays. A preliminary heave/settlement assessment has been undertaken using PDISP software, which predicted between 3mm of settlement (short-term only) and 12mm of heave beneath the perimeter walls, and up to 21mm of heave below the basement slabs, including the internal RC retaining walls to the swimming pool area. However, only the preliminary predicted 5-10mm of post-construction incremental heave is likely to be relevant to the design of the basement slabs (Section 10.5).

11.13 Damage category assessments indicated that, provided best practice construction methods are employed, the worst case predicted deformations in adjacent properties are likely to fall within Burland Category 1, termed 'very slight', for No.5a and Burland Category 0, termed 'negligible', for No.3. The car lift will be more remote from the front wall of No.3 than the main basement, and is expected to be beneficial, so does not require further consideration. (Section 10.6).

11.14 Condition surveys of the neighbouring properties should be commissioned, and a programme of monitoring the adjoining structures should be established, before the works start (Section 10.7).

11.15 Both the Camden SFRA and the Environment Agency's recent modelling of risk of flooding from surface water predicted a Very Low flood risk within No.5's site and the adjoining properties (10.8.1). Only basic flood resistance measures will be required in view of the 'Very Low' risk of surface water flooding; specific guidance is given (10.8.7).

11.16 The basement and new rear patio will increase the hard-surfaced area. A quantitative analysis of the net change will be required in order to assess the potential change in surface water run-off, and to enable design of one or more of the identified suitable SuDS systems (10.8.3 to 10.8.6).

11.17 The SFRA indicated that Thames Water had no records of flooding from public sewers affecting this postcode, so the probability of future sewer flooding affecting No.5 is considered to be very low, provided that the sewer system is well maintained and appropriate flood resistance measures are implemented (10.8.8).

11.18 Non-return valves and pumped loop systems must be fitted to the drains serving the basement, lightwells and rear patio. Temporary interception storage may also be required, with sufficient capacity for the predicted maximum volume of discharges (from all sources) via the 'protected' outfall pipe(s), for the duration of the predicted surcharged flows in the sewer; formal design would be required (10.8.10, 10.8.11).

11.19 Construction of the proposed basement is not expected to create any unacceptable cumulative impact in relation to groundwater flows (10.2.11), ground stability (10.4.16) or surface water (10.8.12).

11.20 The mitigation measures recommended in various parts of Sections 10.2 to 10.8 have been summarised in Section 10.9.

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Project:

5 Templewood Avenue, London, NW3 7UY

16525

No.3 Templewood Avenue

Photo 1: Front elevation (street scene). No.5 Templewood Avenue is a large, three storey detached house situated on the north side of Templewood Avenue.

Photo 2: The footway in front of No.5 falls gently towards the Templewood Avenue carriageway, away from the property. **In** the vicinity of No.5, the Templewood Avenue carriageway falls very gently to the south-west, however further to the north-east, it falls more steeply.

Title:	**Photographs - Sheet 1**					Sheet	**A1**
Date:	13 February 2016	Checked:	AG	Approved:	KRG	Scale :	NTS

Photo 3: Externally there is a large parking/amenity area at the front of the house which is predominantly surfaced with asphalt, with a distinct fall towards the Templewood Avenue carriageway, away from the front of the house.

Photo 4: Overall, the rear garden falls relatively steeply towards the rear of the house, however it has been terraced, to form two broadly level areas.

Project:

5 Templewood Avenue, London, NW3 7UY

16525

Photo 5: To the rear of the house, there is a large patio area which adjoins the rear wall of the house, within which are raised herbaceous borders. Upslope of this patio is an area which has been mostly laid to lawn.

Photo 6: Immediately to the rear of the single storey extension, the low retaining wall which separates part of the rear patio from the herbaceous border was seen to be failing. It was both leaning towards the rear of the house and had slid forwards on one of the mortar beds.

Title:	**Photographs - Sheet 3**					Sheet	**A3**
Date:	13 February 2016	Checked:	AG	Approved:	KRG	Scale :	NTS

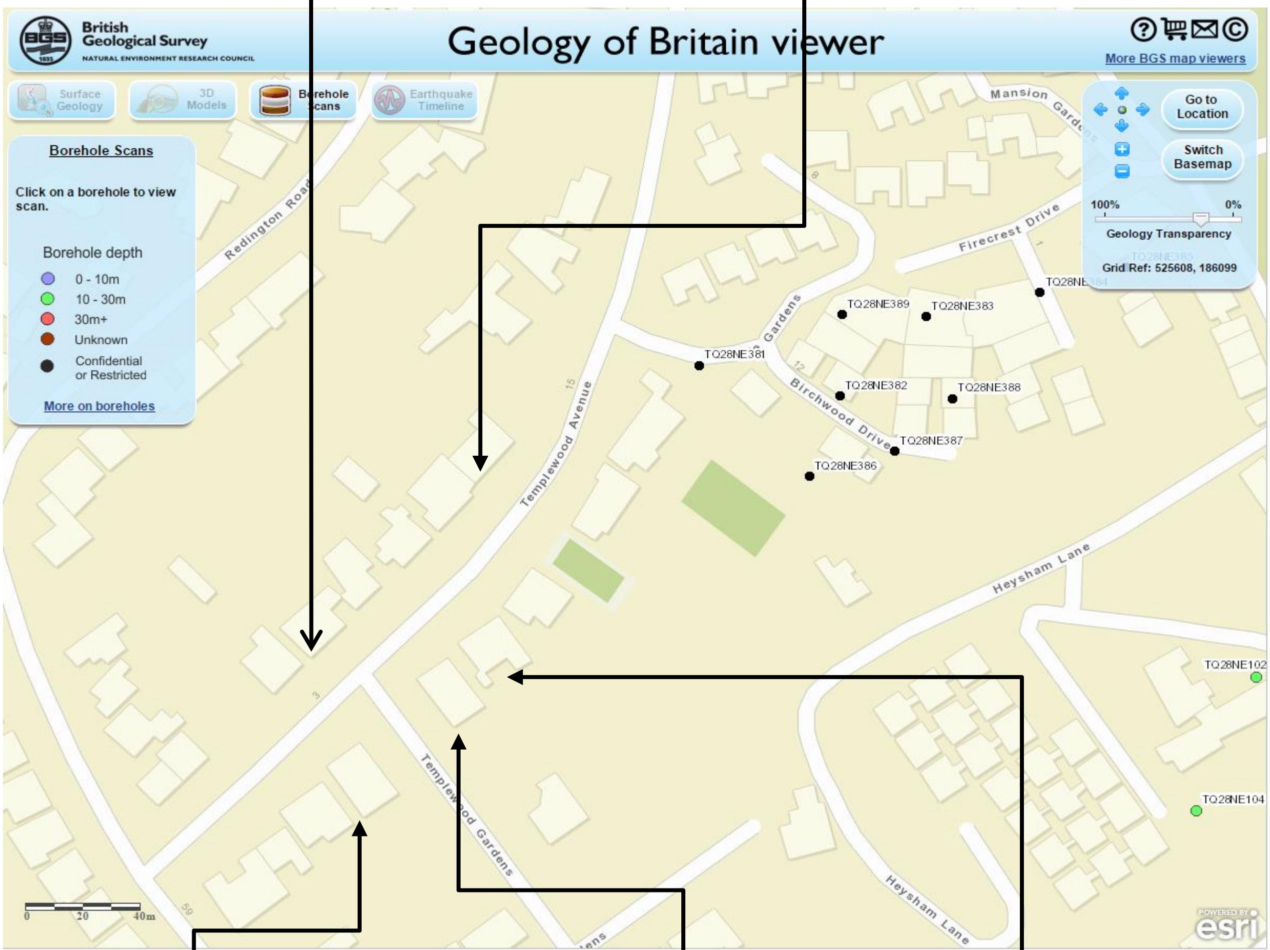
Project:

5 Templewood Avenue, London, NW3 7UY

16525

5 Templewood Avenue

No.11 Templewood Avenue (BH1 - BH3)

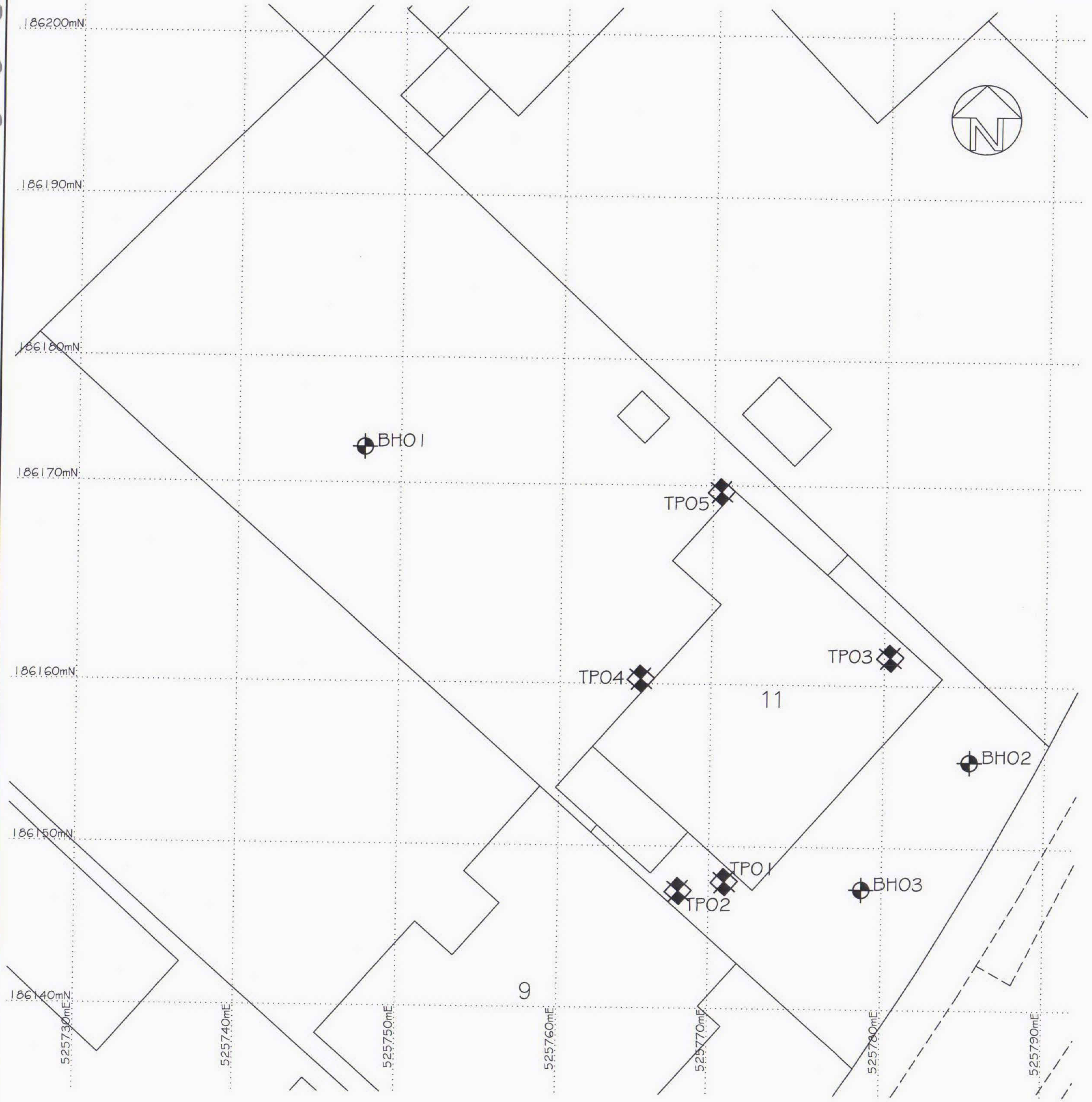


No.4 Templewood Avenue (BH1 & BH2)

No.6 Templewood Avenue (BH1 & BH2)

No.8 Templewood Avenue (BH1)

Title:	**Location Plan of Desk Study Boreholes**					Sheet	**B1**
Date:	April 2016	Checked:	AG	Approved:	KRG	Scale :	NTS



NOTES

1. This drawing should not be scaled.

COORDINATES & LEVELS

BH	EASTING (m)	NORTHING (m)	LEVEL (mOD)
BH01	525747.8	186174.5	106.24
BH02	525785.4	186155.3	105.01
BH03	525778.7	186147.4	104.52

TP	EASTING (m)	NORTHING (m)	LEVEL (mOD)
TP01	525770.2	186148.7	102.93
TP02	525767.0	186146.8	102.95
TP03	525780.3	186162.6	103.02
TP04	525765.4	186159.9	104.98
TP05	525770.2	186171.5	105.26

No	Revision	Drawn	Checked	Passed	Date

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Client:	Mr & Mrs Antony Fobel
Project:	11 Templewood Avenue London NW3
Title:	Exploratory Hole Location Plan
Dwg. No:	11/2397/01
Status:	Issue
Scale:	NTS

Drawn	Checked	Passed	Date
IP	DS	MD	August 2011