Report

Basement Impact Assessment

46 Agamemnon Road, London
NW6 1EN

Ref: GGC20807/R1.1 July 2020

Gabriel GeoConsulting Limited
Highfield House, Rolvenden Road, Benenden, Kent TN17 4EH
Company No. 6455714, registered in England and Wales. Registered office as above.
Tel: 01580 241044
e: info@gabrielgeo.co.uk www.gabrielgeo.co.uk

Basement Impact Assessment

Site: **46 Agamemnon Road, London NW6 1EN**

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

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 Gabriel GeoConsulting Ltd.

This report is specific to the proposed site use or development, as appropriate, and as described in the report; Gabriel GeoConsulting 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.

Contents Page

Foreword		i
1.	Introduction	1-2
2.	The Property, Topographic Setting and Planning Searches	3-6
3.	Proposed Basement	7
4.	Geological Setting	8-10
5.	Hydrological Setting (Surface Water)	11-15
6.	Hydrogeological Setting (Groundwater)	16-18
7.	Stage 1 – Screening	19-21
8.	Stage 2 – Scoping	22-23
9.	Stage 3 – Ground Investigation	24-28
10.	Stage 4 – Basement Impact Assessment	
	10.1 Conceptual Ground Model	29-30
	10.2 Subterranean (Groundwater) Flow – Permanent Works	31-32
	10.3 Subterranean (Groundwater) Flow – Temporary Works	33
	10.4 Slope and Ground Stability	34-37
	10.5 PDISP Heave/Settlement Assessment	38-42
	10.6 Damage Category Assessment	43-48
	10.7 Monitoring	49-50
	10.8 Surface Flow and Flooding	51-53
	10.9 Mitigation	54
11.	Non-technical Summary – Stage 4	55-56
References		57

Appendices

Appendix A Photographs
Appendix B Desk Study Data – BGS Boreholes
Appendix C Desk Study Data – Historic maps – Large Scales (1:1,056, 1:1,250 & 1:2,500)
Appendix D Desk Study Data – Historic maps – Small Scales (1:10,560 & 1:10,000)
Appendix E Desk Study Data – GroundSure's Enviro+Geo Insight report
Appendix F Ground Investigation Results – Oakland Site Investigation & Geolabs Ltd
Appendix G PDISP Heave/Settlement Analysis Figures G1-G6

1. INTRODUCTION

1.1 This Basement Impact Assessment (BIA) has been prepared in support of a planning application to be submitted to the London Borough of Camden (LBC) for the extension of the existing cellar in order to create a single-storey basement beneath the front part of No.46 Agamemnon Road, NW6 1EN. Further details of the proposed works are given in Section 3. This assessment is in accordance with the requirements of the London Borough of Camden (LBC) Local Plan 2017, Policy A5 in relation to basement construction, and follows the requirements set out in LBC's guidance document 'CPG Basements' (March 2018).

1.2 This assessment has been undertaken by Keith Gabriel, a Chartered Geologist with an MSc degree in Engineering Geology (who has specialised in slope stability and hydrogeology), and reviewed by 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 many assessments of basements in several London Boroughs.

1.3 **Desk Study**: A site inspection (walk-over survey) of the property was undertaken on 29th May 2020. Prior to that, photographs of relevant parts of the property had been provided by the client; a selection of those photos are presented in Appendix A. The authors are also familiar with the area around No.46, having compiled BIA reports for three other properties in the vicinity. Desk study data have been collected from various sources including borehole/well logs from the area around the site (Appendix B), flood reports and flood modelling specific to the borough, historic maps (Appendices C & D), and environmental & geological data in Groundsure's Insight report (Appendix E). Relevant information from the desk study and site inspection is presented in Sections 2–6.

1.4 **Ground Investigations:** Sitework for the ground investigation (borehole and trial pits) was undertaken on 20th March 2020, the findings from which are presented in Section 9 and Appendix F.

1.5 The Screening, Scoping and basement impact assessments in accordance with CPG Basements, Stages 1-4, are presented in Sections 7, 8 & 10 respectively.

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

DK Design (Architects):

- Drg No. Ro.01.101 — Existing ground floor, part basement and external elevations and sections
- Drg No. Ro.01.201 — Proposed ground floor, basement and external elevations and sections

David Joseph Consulting (Structural Engineers):

- BASEMENT IMPACT STATEMENT FOR THE PROPOSED BASEMENT EXTENSION, (Ref: 4388/BIS/003/AB, June 2020) which includes:
- Preliminary Method Statement
- Drg No's 4388_SK01 – SK03 Construction of underpinning for new basement (3 sheets).
- Drg No.4388_SK04 rev.A Proposed Preliminary Basement Floor Plan
- Drg No.4388_SK05 rev.A Proposed Preliminary Ground Floor Plan
- 'Preliminary loadings for ground analysis' annotated onto a copy of Drg ~SK04
- Preliminary retaining wall calculations.

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

1.7 Instructions to prepare this Basement Impact Assessment were confirmed by phone on 12th March 2020.

2. THE PROPERTY, TOPOGRAPHIC SETTING AND PLANNING SEARCHES

2.1 No.46 Agamemnon Road is a two-/three-storey terraced house (see cover photo and Photo 1 in Appendix A), in the Fortune Green area of the London Borough of Camden (LBC). A full-width single-storey rear/side extension was added in 2015, together with various changes to the original superstructure (consented planning application 2014/7175/P and Certificate of Lawfulness 2014/7176/P).

2.2 Agamemnon Road rises northwards from Hillfield Road towards Hampstead Cemetery. No.46 is situated on the west side of Agamemnon Road, near the northern end of the road. The adjoining houses are No.44 to the south and No.48 to the north, the front entrance to which is on the adjoining Gondar Gardens. To the rear, No.46's garden and all the adjoining properties back onto No.62 Gondar Gardens, as shown in Figure 1 below.

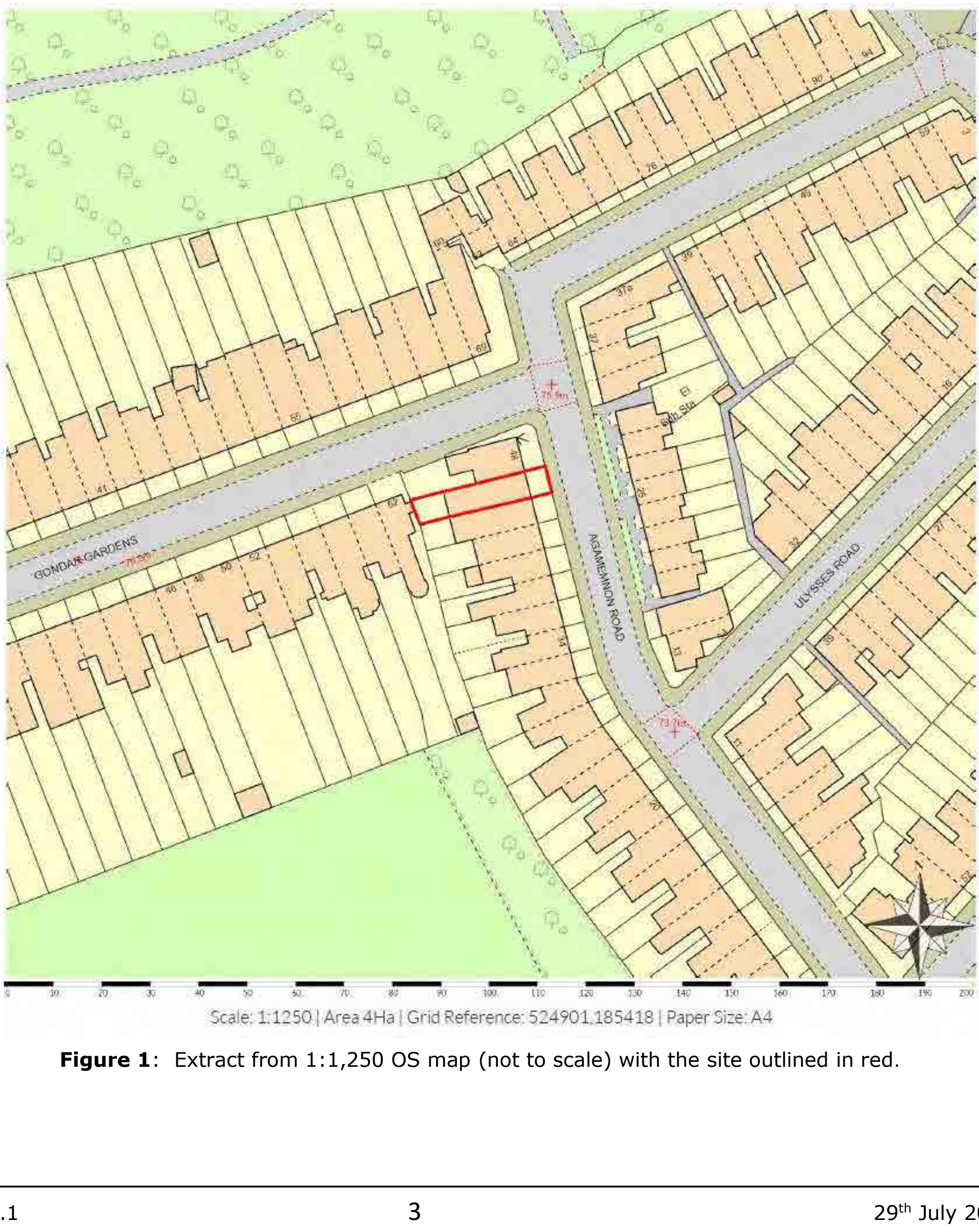

Figure 1: Extract from 1:1,250 OS map (not to scale) with the site outlined in red.

2.3 The first available historic Ordnance Survey (OS) map with coverage of this area, dated 1874 (1:10,560 scale, as presented in Appendix D) shows that the area remained undeveloped farmland, with a field boundary crossing the rear end of No.46’s future site. Mill Lane and Fortune Green Lane were present to the south and east respectively. By 1894/1896 Agamemnon Road and most of the adjoining roads were present and the area was close to being fully developed, including all the Victorian houses in a variety of architectural styles on the north-south oriented part of Agamemnon Road and a large covered reservoir to the south of No.46 (see 1:1,056 and 1:2,500 scale OS maps in Appendix C). Only the eastern-most part of Gondar Gardens had been developed.

2.4 Few changes have occurred to the properties in the immediate vicinity of No.46 since 1915, by when the area was fully developed. The most notable changes were the destruction of the original No’s 17-31 Agamemnon Road, directly opposite No.46, by bombing during WW2 (see paragraph 2.7 below) and their replacement with the current houses by 1953, as shown on the 1:1,250 scale map (those plots were vacant on the 1951 1:10,560 map). The 1:1,250 scale 1953 map also shows a revised footprint for the rear projection to No.88 (though that could have been built much earlier, as the 1:1:2,500 scale maps from 1915 and 1938 were not sufficiently detailed to shown that change).

2.5 Externally, No.46’s front garden is extensively paved except alongside the front wall where an ornamental hedge grows in a flower bed (Photos 2 & 3). The southern part of this hedge is in three pots in a recess, which screens the gas and electric meter boxes on the inner side of the front boundary wall. The front garden slopes towards the front hedge along the front boundary and, at the pedestrian entrance, there are two steps down to the public footway. All the paving in the front garden was renewed following the building works in 2015 and a short period with ornamental gravel surfacing. Prior to those works, the front garden was extensively paved with a small central flower bed and other flower beds alongside the boundary walls.

2.6 The proposed basement will not extend beneath the rear extension or the original rear projection (all the walls from which were removed at ground floor level during the 2015 works). Thus the rear garden and rear extension are not directly relevant to the proposed basement with the exception of the rear access arrangements, which comprise sliding patio doors with the external ground level flush with the internal finished floor level (FFL).

2.7 The London County Council Bomb Damage Map for this area (London Topographical Society, 2005) records extensive damage to the whole terrace on the east side of Agamemnon Road, to the north of Ulysses Road. Eight houses (No’s 17-31 were “*totally destroyed*” and the remainder (No’s 13, 15 & 33-37a) were recorded as “*seriously damaged but repairable at cost*”, as was No.32 on the west side of the road. The Hampstead bomb map shows that four high explosive bombs fell in that area. There are no gaps in the bomb lines which might indicate the presence of an unexploded bomb, though this must not be taken as conclusive proof of the absence of unexploded ordnance (UXO).

2.8 The photos of the front of No.46 and the adjoining No's 44 and 48 (Photos 1, 5 & 6) suggest that the brickwork is generally in good condition. The capital on the south side of the arch to No.44's front door is clearly lower than those on either side of No.46's arch, while the lintel to No.44's window above the front door also shows marked settlement on the south side (see mark-up on Photo 7a in Appendix A); both indicate settlement of No.44 relative to No.46. Minor crack damage, possibly passing through a previous repair, was also evident beneath the north end of the cill to the same window.

<u>Topographic Setting:</u>

2.9 No.46 Agamemnon Road is located on a south-southeast facing slope which forms the flank of a promontory that projects out from the broadly southwest facing slope that leads up to Hampstead Heath. The covered reservoir is also on the southern edge of this promontory, which is defined by the 80m AOD contour (metres above Ordnance Datum), while the northern WSW-ENE oriented part of Agamemnon Road occupies the ridge of the promontory. The slope on which No.46 stands leads down to the valley of the former western branch of the River Westbourne, one of the 'lost' rivers of London (see Figure 2 and paragraph 5.1).

Figure 2: Extract from 1:25,000 scale Ordnance Survey map showing site location.

2.10 No.46 is located just above the 75m AOD contour, as shown on Figure 2 and the 1976 & 1993 1:10,000 scale maps, while the spot height on the carriageway at the junction of Agamemnon Road with Gondar Gardens gives a level of 75.9m AOD. The contours on Figure 2 indicate an overall slope between the 70m and 75m contour lines varying between approximately 3.9° and 8.8° (at the south-east corner of the covered reservoir); however the latter is probably inappropriate because no slope greater than 7° has been identified in this area in Figure 16 of the Camden geological, hydrogeological and hydrological study (Arup, 2010; see extract in Figure 3, in Section 4 below). Using the centreline spot heights along Agamemnon Road, given on the current 1:1,250 scale OS map (Figure 1) and the 1896 1:1,056 scale Town Plan, overall slope angles of around 1.7-3.6° towards the south-southeast were calculated as follows:

Gondar Gardens to Ulysses Road:	1.7°	(adjacent to No.46)
Ulysses Road to Achilles Road:	3.5°	
Achilles Road to Hillview Road:	2.9°	

Planning Searches:

2.11 Searches were made of planning applications on Camden council's website (on 8th and 14th April 2020) in order to obtain details of any other basements which have been constructed, or are planned, in the immediate vicinity of the property. Plans of the immediately adjoining/adjacent properties were also sought and were downloaded where available. These searches found:

- **Adjoining No.44 Agamemnon Road:** Application (2015/5623/P) involving "*Erection of a single storey side return infill extension and installation of rooflight on existing rear extension*" was granted planning permission on 20th November 2015. Existing cellar to remain unchanged. No evidence was found for any proposed basement. Superseded by:
- **No's 42 & 44 Agamemnon Road:** Application (2015/6355/P) involving "*Erection of a single storey infill extension at No. 42 and a wraparound single storey side and rear extension at No. 44 Agamemnon Road (following removal of single storey rear extension at No. 44)*" was granted planning permission on 26th January 2016. Drawings of both existing and proposed plans, elevations and sections were found on the website. Aerial photos show that these extensions have been built.
- **Adjoining No.48 Agamemnon Road:** A Certificate of Lawfulness was issued on 9th November 2004 for "*Erection of rear dormer window, rooflight and various elevational alterations*" (Application 2004/4103/P); layout plans were obtained. No evidence was found for any existing cellar or proposed basement.
- **No.42 Agamemnon Road:** No applications, other than the joint application with No.44 described above.
- **No.40 Agamemnon Road:** No applications.
- **No.62 Gondar Gardens:** No applications.

3. PROPOSED BASEMENT

3.1 The proposed works at No.46 Agamemnon Road for which planning permission will be sought, as shown in the scheme drawings by DK Design (see paragraph 1.6), will comprise:

- A single-storey basement beneath most of the footprint of the main part of the house (with the basement's rear wall set slightly forwards from the main rear wall of the house in order to avoid breaking out the deep strip footing which supports the ground floor level box frame). A small services cupboard under the stairs into the basement will extend a short distance beneath the rear projection, alongside the 44/46 party wall.
- A lightwell at the front of the property, symmetric on the front bay and extending out 1.20m from the bay (internal dimension) and 2.1m from the front wall (external dimension).
- Removal of the temporary pad foundation and a stub column which were installed during the 2015 works to support the loads from column C5. That column acts on a transverse steel beam at ground floor level which spans between the party walls. (This temporary footing was designed to minimise load on the existing party wall footings until such time as they are underpinned as part of the now proposed basement works).
- Revised bin store in the front garden.

3.2 The drawings by DK Design show that the finished floor level (FFL) in the basement will be **2.675m** below the ground floor's FFL and an allowance of **225mm** is specified for insulation, cavity drainage and floor structure. The structural drawings by David Joseph Consulting show that both the underpin bases and the central basement slab will be **300mm thick**, thus the founding level (formation) of the underpins and slab will be approximately **3.20m** below the ground floor's FFL, whereas the front lightwell will be founded 225mm higher at **2.975m** below the same floor level.

3.3 Excavation depths for the proposed basement are expected to range from **1.65m** below the existing cellar floor (= 3.20 – 1.275 – (2.675 – 2.400); see dimensions on DK Design's sections) to **2.82m** below the remainder of the ground floor, where there is a clear air void of approximately 160-170mm beneath the 200mm deep timber joists (and allowing 21mm for floor boards). For the front lightwell, the excavation depth will be approximately **2.88m** from the existing front garden level (= 2.675 + 0.30 – 0.09).

3.4 Based on a search of the LBC's planning applications (paragraph 2.11), the adjoining No.44 has an existing single-storey cellar similar to No.46's cellar (though the plan drawing attached to the 2015 application by Pelican Architecture & Design is clearly incorrect because the layout shown would extend under/through the party wall). It is also understood that the owner of No.48 has confirmed that there is no cellar or basement beneath that house.

4. GEOLOGICAL SETTING

4.1 Mapping by the British Geological Survey (BGS) indicates that the site is underlain by the London Clay Formation. 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 site geology of the Hampstead area.

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

4.2 In urban parts of London, the London Clay is 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 London Clay 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.

4.4 The results of the BGS natural ground subsidence hazard classifications are provided in the Groundsure Insight report (Appendix E, Section 17); all indicated 'Negligible' or 'Very Low' hazard ratings with the exception of 'Shrink swell clays' for which a 'Moderate' hazard rating was given, which reflects the outcrop of the London Clay Formation at surface.

4.5 The Groundsure GeoInsight report (Appendix E, Sections 18, 3 & 7) records:

- Historical surface ground working features, the closest of which were the excavations for the covered reservoir located 39m to the south of the site (see App.E, Section 18.3).
- No historical underground workings within 1000m of the site (see App.E, Section 18.3).
- No historical 'non-coal mining' features or 'mining cavities' within 1000m of the site (see App.E, Sections 18.6 & 18.7).
- No records of mining on site for five specific mineral deposits (see App.E, Sections 18.9 to 18.13).

It should be noted that these databases are based on mapping evidence so inevitably will provide an incomplete record of underground workings.

4.6 A search of the BGS borehole database was undertaken for information on previous ground investigations and any wells in the vicinity of the site, the locations of which are presented on the location plan in Appendix B. The strata depths in a selection of these boreholes are summarised in Table 1. For full strata descriptions, reference should be made to the logs in Appendix B. General points of note from these boreholes were:

- BGS Boreholes TQ28NE/119 (BH1-BH4) were all drilled by Soil Mechanics Ltd, as part of a ground investigation at Kidderpore Avenue, to the north-east of the site. The boreholes display similar information; thus in Table 1, only the minimum and maximum depths are recorded, giving the range of depths found across these four boreholes.
- BGS Boreholes TQ28NW/20 (BH1-BH4) were all drilled as part of a ground investigation to the north-west of the site, at Hampstead School. Again, these boreholes display similar information so in Table 1 only the minimum and maximum depths are recorded.
- BGS Borehole TQ28NW/32: This shallow (1.28m deep) pit was dug to assess the founding conditions for the West Hampstead Fire Station (and found 'Hard yellow CLAY' below 0.67m). Identical strata depths, strata descriptions and ground level are given for borehole TQ28NW/21, the location of which is shown further to the north-east (see Figure B1), so that record is believed to be a mis-plotted duplicate. The record for TQ28NW/21 does indicate that the hard yellow CLAY was considered by the BGS to be Weathered London Clay.

Table 1: Summary of Strata in BGS Boreholes							
Strata (abbreviated descriptions)	**Depths (m) and levels (m AOD) to base of strata in BGS Boreholes**						
	TQ28NE/119 (BH1-BH4)	**TQ28NE/32 (and ~/21)**		**TQ28SW/ 85**	**TQ28SW/ 74**		**TQ28NW/20 (BH1–BH4)**
GL (mAOD)	Depth	Depth	**Level 60.15**	Depth	Depth	**Level 50.93**	Depth
Surfacing/ Made Ground	0.15-0.53	0.67	**59.48**	0.23	0.30	**50.63**	0.15-0.30
Soft/firm becoming firm/stiff, grey and brown mottled, sandy clayey SILT (Claygate Member)	4.27-5.79	-	**-**	-	-	**-**	-
Firm-very stiff fissured brown silty CLAY with crystals (Weathered London Clay Fm)	-	>1.28		6.71	8.23	**42.70**	7.01-7.16
Firm-very stiff, fissured, grey/dark grey silty CLAY with crystals (London Clay Fm)	>10.67-15.39	-		>15.24	>45.72		>7.62-12.19
Seepage/Strike	-	-	**-**	-	**-**	**-**	-
Groundwater standing level	1.27-7.47	-	**-**	-	**-**	**-**	-

5. HYDROLOGICAL SETTING (SURFACE WATER)

Approximate location of No.46 Agamemnon Road

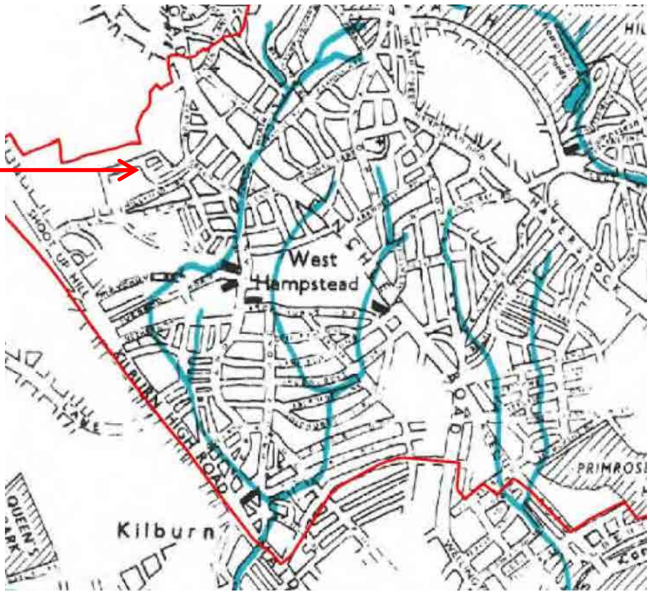

Figure 4: Extract from Figure 11 of the Camden GHHS (Arup, 2010) showing former watercourses, based on Barton (1992).

5.1 The site lies to the north-west of a tributary to the river Westbourne, one of the 'lost' rivers of London, as shown in Figure 4. Most of these 'lost' rivers now run in dedicated culverts or the sewer system. The location of this tributary is confirmed by the 1865/1874 historic OS map (small scale), which shows a stream passing beneath the Mill Lane-West End Lane junction then coming within some 480m to the south-east of the site. This tributary was most likely culverted when the area was developed.

5.2 The gentle fall of No.46's almost fully paved front garden towards the front boundary, including the flower bed for the hedge and the positively drained recess by the meter cabinets and the front gate, ensures that surface water drains away from the front of the house (Photos 2-5). Similarly, the two steps up at the pedestrian access gate and the cross-fall on the public footway towards the gutter (Photo 6), together with the steeper southwards fall of Agamemnon Road, are expected to ensure that surface water drains away from the property under normal conditions.

5.3 The front garden to No.46 is bounded by a wooden fence on the upslope side which is unlikely to prevent excess surface water run-off entering the site from No.48's garden, though, as that is largely soft landscaped with gravel and flower beds, run-off is expected to be minimal, if any. On the downslope side, a low rendered wall is present along the 44/46 boundary. Therefore the surface water catchment for the front garden is expected to be restricted to the site itself, plus any surplus overland run-off water seeping from the adjoining front garden to No.48.

5.4 As No.46's front garden is almost fully paved, infiltration will be limited to the flower bed for the hedge alongside the front boundary wall (though infiltration would become ineffective when the ground is saturated or frozen). That flower bed will be unaffected by the proposed basement works.

5.5 Figure 5 shows that Agamemnon Road, along with the adjoining Hillfield Road and Achilles Road, 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, those floods generally affected only a short length (usually the 'low point') of these roads.

No.46 Agamemnon Road

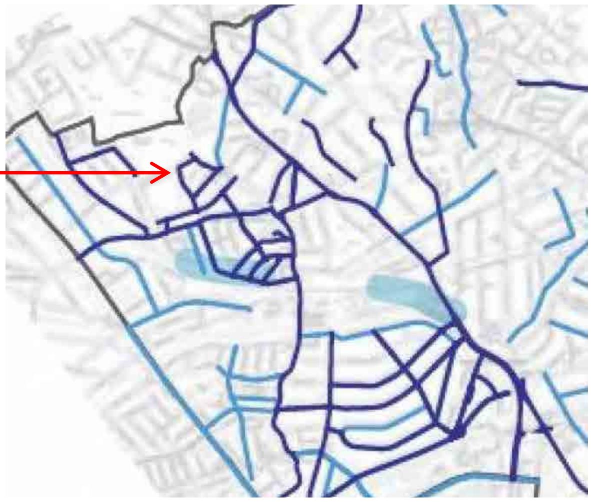

Figure 5: Extract from Figure 15 of the Camden GHHS (Arup, 2010) showing roads which flooded in 1975 (light blue), in 2002 (dark blue), and 'Areas with potential to be at risk of surface water flooding' (wide light blue bands).

5.6 The Environment Agency's classifications for the risk of flooding from rivers and sea at the Agamemnon Road (available on the GOV.UK website), has shown that the site is:

- Within Flood Risk Zone 1, so has less than 1 in 1000 annual probability of river or sea flooding (<0.1% in any given year), **not** taking into account the presence of any flood defences;
- Classified under the Environment Agency's Risk of Flooding from Rivers or Seas (RoFRaS) dataset, which **does** allow for the beneficial effects of any flood defences (though irrelevant in this part of Camden), with a 'Very Low' probability of flooding, which is once again defined as "*each year this area has a chance of flooding of less than 0.1%*" (<1 in 1,000).

These are all as expected, given the remote position of Agamemnon Road relative to the River Thames floodplain, and the former course of the nearest of the 'lost' rivers (see paragraph 5.1).

5.7 The Environment Agency's modelling also shows that this area does not fall within an area at risk of reservoir flooding; with the nearest potentially affected areas (the "maximum extent of flooding") being over 1.9km to the west of No.46, from Brent Reservoir, and 2.25km to the east from the Hampstead Pond Chain.

5.8 Some hydrological data for the site has been obtained from the Groundsure Enviro+Geo Insight report (see Appendix E), including:

- There are no Water Network records (of rivers, streams, lakes or canals, from 'OS MasterMap' data) within 250m of the site (App.E, Section 6.1);
- No surface water features were identified within 250m of the site (App.E, Section 6.2);
- Under the EU's Water Framework Directive, the site is in a Coastal catchment, draining to the Thames, and does not fall within a 'River Water Body' catchment (App.E, Sections 6.3 & 6.4);
- There are no surface water abstraction licences within 2000m of the site (for more than 20m^3 per day; App.E, Section 5.7).
- The Environment Agency have no records of historical flooding within 250m of the site since 1946 (App.E, Section 7.2);
- There are no flood defences, no areas benefiting from flood defences, and no flood storage areas within 250m of the site (App.E, Sections 7.3, 7.4 & 7.5).
- Flood modelling by Ambiental Risk Analytics gives a 'Negligible' risk of surface water flooding affecting No.46 in all four rainfall event return periods modelled (1 in 30 years to 1 in 1,000 years) (App.E, Section 8).

5.9 The Environment Agency (EA) published a new map of 'Flood Risk from Surface Water' in January 2014, and a more detailed version has since become available on the 'Check your Long Term Flood Risk' pages of the GOV.UK website, an extract from which is presented in Figure 6 below. This map identifies four levels of risk (high, medium, low and very low), and is based primarily on topographic levels (from LiDAR data), flood depths and flow paths. The EA's definitions of these risk categories are:

'Very low' risk:	Each year, these areas have a chance of flooding of less than 1 in 1000 (<0.1%).
'Low' risk:	Each year, these areas have a chance of flooding of between 1 in 1000 (0.1%) and 1 in 100 (1%)
'Medium' risk:	Each year, these areas have a chance of flooding of between 1 in 100 (1%) and 1 in 30 (3.3%).
'High' risk:	Each year, these areas have a chance of flooding of greater than 1 in 30 (>3.3%).

5.10 The EA's modelling presented in Figure 6 shows the No.46 and the adjoining properties, and the adjacent part of the Agamemnon Road carriageway, all have a 'Very Low' risk of flooding from surface water; this is the national background level of flood risk. Their modelling also predicts areas at a 'Low' and 'Medium' risk of flooding from surface water in the southern (downslope) part of the Agamemnon Road carriageway, which supports the likelihood of the 2002 flooding having affected only that lowest part of the road (see paragraph 5.5).

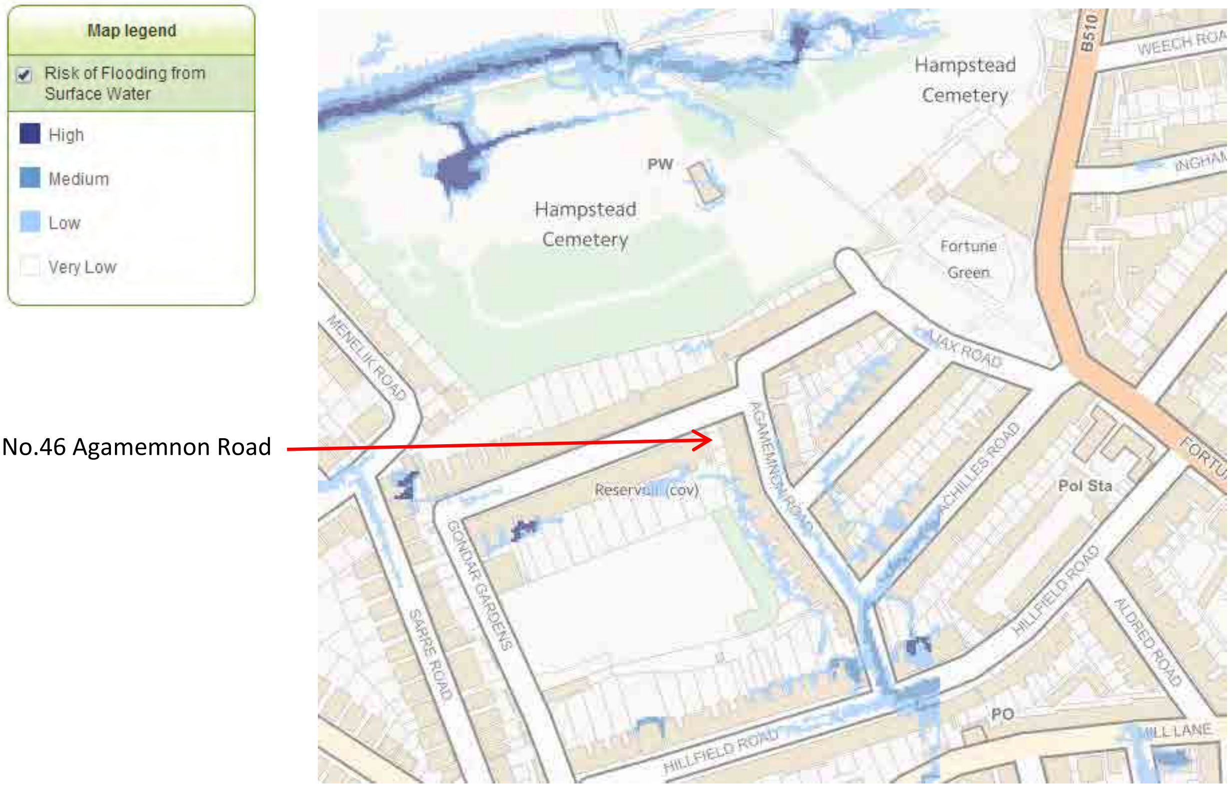

Figure 6: Extract from the Environment Agency's 'Flood Risk from Surface Water'.

5.10 Surface water flood modeling has also 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). This modelling is less clear than the EA's, though also shows that No.46, the adjoining properties and the adjacent part of the Agamemnon Road carriageway are classified as being at 'Very Low' risk of flooding from surface water.

5.11 Figure 7 also shows that Agamemnon Road falls just within the Group3_010 Critical Drainage Area, but it does not fall within any of the Local Flood Risk Zones (see SFRA Figure 6, Rev.2).

5.12 The implications from these flood models are discussed in Section 10.8. Dual gullies have been installed at the junction between Agamemnon Road and Hillfield Road, probably in response to past flooding (the 2002 event, perhaps).

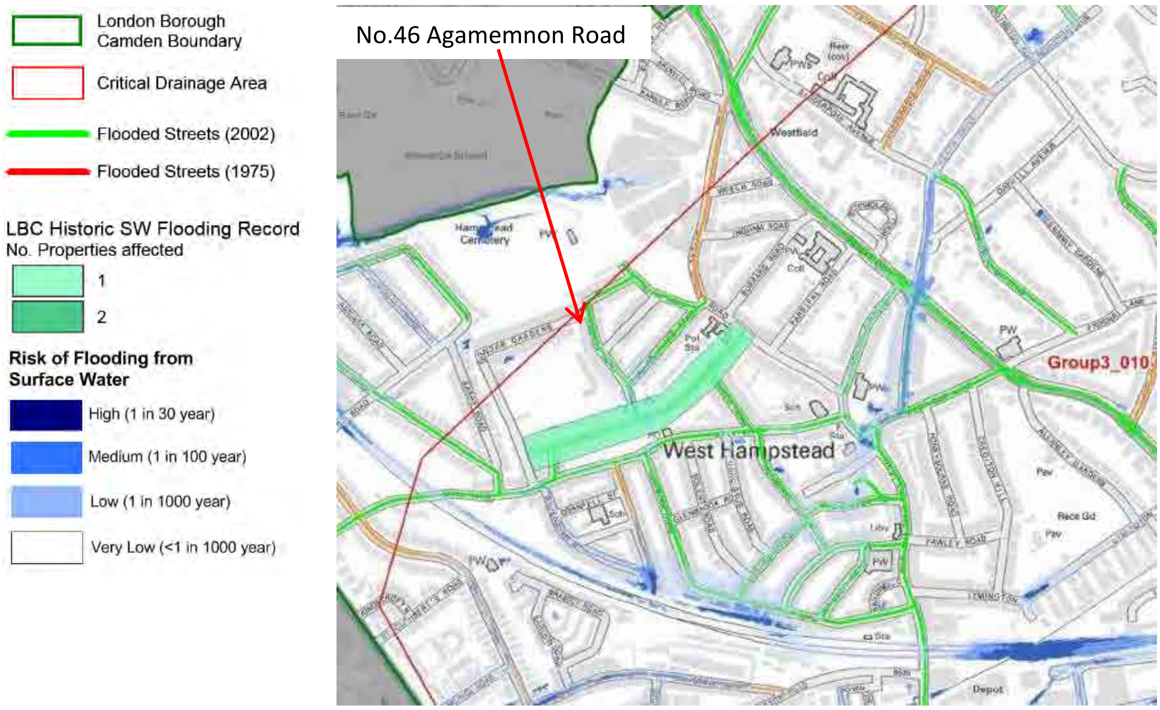

Figure 7: Extract from Figure 3v of the Camden Strategic Flood Risk Assessment (SFRA) (URS, July 2014) showing risk of flooding from surface water.

5.13 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), only one property within this postcode was recorded by Thames Water as having been affected by internal sewer flooding in the previous 10 years, and none were recorded as having been affected by external sewer flooding in the previous 10 years.

6. HYDROGEOLOGICAL SETTING (GROUNDWATER)

6.1 The London Clay Formation is classified by the Environment Agency as an 'Unproductive Stratum', as indicated by Figure 8. .

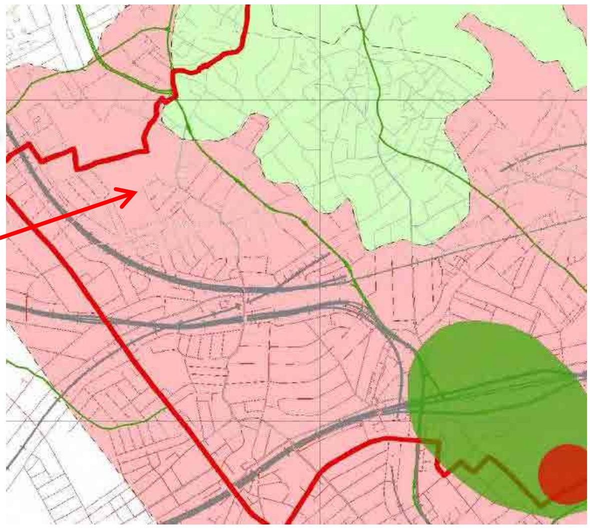

Figure 8: Extract from Figure 8 of the Camden GHHS (Arup, 2010) showing aquifer designations and SPZs. (Red = Zone I, Dark Green = Zone II).

6.2 New groundwater vulnerability mapping has been undertaken jointly by the BGS and Environment Agency; this mapping presents an assessment of the vulnerability of groundwater to a pollutant discharged at ground level based on the hydrological, geological, hydrogeological and soil properties within a one kilometre square grid. Groundwater vulnerability is described as High, Medium or Low based on the leachability and permeability of the soils concerned, with superficial and bedrock aquifers classified separately. Unproductive aquifers such as the London Clay Formation are also classified as having 'unproductive' vulnerability with a 'Low' infiltration Value within 50m of No.46 (see Groundsure's Enviro+Geo Insight report in Appendix E, Section 5.3).

6.3 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.

6.4 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 (which can act either as land drains or as sources of water and high groundwater pressures). 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.

6.5 Perched groundwater would typically be expected in any Made Ground, and possibly also in any Head deposits which overlie the London Clay, in at least the winter and early spring seasons. Variations in groundwater levels and pressures will occur in response to seasonal climatic changes and with other man-induced influences.

6.6 The groundwater catchment areas upslope of No.46 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 the immediately adjoining areas of Made Ground, except where the trenches for drains and other services provide greater interconnection.
- London Clay Formation: The catchment for the underlying London Clay will comprise recharge from the overlying soils in the vicinity of the site, plus potentially a wider area determined by the lateral extent of any interconnected silt/sand horizons.

6.7 Other hydrogeological data obtained from the Groundsure Enviro+Geo Insight report (Appendix E) include:

- There are no Source Protection Zones (SPZs) within 250m of the site (App.E, Sections 5.9 & 5.10, and Figure 8 above).
- There are no licensed groundwater or potable water abstractions (of greater than 20 cubic metres per day) within 2000m of the site (App.E, Sections 5.6 & 5.8).
- For No.46's site and an area within 50m of the site, Ambiental Risk Analytics has classified the susceptibility to groundwater flooding as '**Negligible'** for a 1 in 100 year return period (App.E, Section 9.1).

6.8 Groundwater flooding incidents were presented on Figure 4e of the Camden SFRA (see Figure 9 below). Around 24 incidents have been reported in the entire borough, the closest of which was around 60m to the west of the site, on the south side of Gondar Gardens. Given the London Clay geology of this area, it is more likely that the incident involved surface water flooding of a cellar which was misidentified as groundwater because it remained present for a prolonged period (the characteristic feature of groundwater flooding) owing to the low permeability of the underlying clays.

6.9 Details of what was found by the site-specific ground investigation in March 2020 are presented in Section 9.

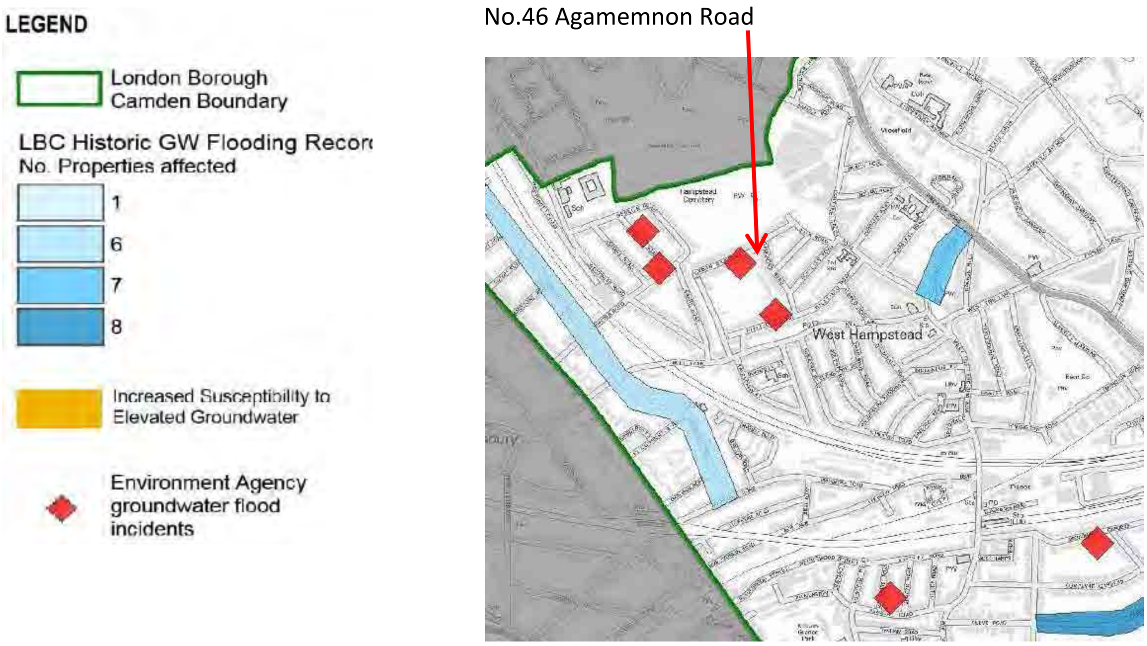

Figure 9: Increased Susceptibility to Elevated Groundwater – extract from Figure 4e of the SFRA.

7. STAGE 1 - SCREENING

7.1 The screening has been undertaken in accordance with the three screening flowcharts presented in LBC's CPG Basements (2018) 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 C, D & E) 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?	No – Site underlain by London Clay	4.1 & Figure 3
1b	Will the proposed basement extend beneath the water table surface?	No, not beneath the water table in an aquifer, though it will extend below the phreatic surface of the groundwater in the London Clay.	9.6, 9.7, and Sections 10.2 & 10.3
2	Is the site within 100m of a watercourse?	No – There are no surface water features within 250m of site.	5.1 & 5.8
3	Is the site within the catchment of the pond chains on Hampstead Heath?	No – Site is approx 1.1km to the south–west of the nearest pond chain catchment (Golders Hill Pond Chain).	
4	Will the proposed basement development result in a change in the proportion of hard surfaced/ paved areas?	No – the proposed front lightwell is in an area which is already fully paved.	2.5 & Photo 2 in Appendix A
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 – Soakaways would be inappropriate in London Clay.	
6	Is the lowest point of the proposed excavation (allowing for any drainage and foundation space under the basement floor) close to, or lower than, the mean water level in any local pond (not just the pond chains on Hampstead Heath) or spring line?	No – There are no surface water features within 250m of the site. Nearest springs are likely to be around 400m to the north-east (at the London Clay-Claygate Member interface).	5.8 & Figure 3

While the answer to question Q1b above was no, the design of the basement must allow for the presence of groundwater in the Made Ground, which was found to be predominantly clayey, and the London Clay. The Environment Agency's nearby record of groundwater flooding (see Figure 9) reinforces this requirement. The temporary works during construction must also allow for the presence of groundwater. These matters are considered in Sections 10.1 to 10.3.

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)	No – The site is broadly level and Figure 16 in the Camden GHHS shows no slopes greater than 7° in the vicinity of this property	2.10 and Figure 3
2	Will the proposed re-profiling of landscaping at site change slopes at the property boundary to more than 7°?	No – No re-profiling is proposed.	
3	Does the development neighbour land, including railway cuttings and the like, with a slope greater than 7°?	No – Figure 16 in the Camden GHHS shows no land greater than 7° in the vicinity of this property.	2.10 & Figure 3
4	Is the site in a wider hillside setting in which the general slope is greater than 7°?	No – Steepest slope angle on Agamemnon Rd is 3.6° and Figure 16 in the Camden GHHS shows no slopes greater than 7° in the vicinity of this property (so contour intervals are considered to be wrong).	2.10 & Figure 3
5	Is the London Clay the shallowest strata at the site?	Yes, it is the shallowest strata mapped by the BGS (though it may be overlain by Head Deposits).	Carried forward to Scoping: 4.1, 8.2, Section 9
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?	No – There is no vegetation in the footprint of the proposed front lightwell and only young trees in the vicinity.	Photo 2
7	Is there a history of seasonal shrink/swell subsidence in the local area, and/or evidence of such effects at the site?	Yes, in No.44.	Carried forward to Scoping: 8.2, Section 10.4
8	Is the site within 100m of a watercourse or potential spring line?	No – see Q2 & Q6 in subterranean flow screening above. No springs in the vicinity.	
9	Is the site within an area of previously worked ground?	No – See BGS map extract (Figure 3 herein) and map on page 84 of the Enviro+ GeoInsight report (in App.E).	4.1 & Figure 3
10	Is the site within an aquifer? If so, will the proposed basement extend beneath the water table such that dewatering may be required during construction?	No – London Clay Formation is classified as an 'Unproductive Stratum'.	6.1
11	Is the site within 50m of the Hampstead Heath ponds?	No – Site is approx 1.1km southwest of the nearest pond chain (Golders Hill).	
12	Is the site within 5m of a highway or a pedestrian right of way?	Yes.	Carried forward to Scoping: 8.2, Section 10.4
13	Will the proposed basement substantially increase the differential depth of foundations relative to neighbouring properties?	Yes	Carried forward to Scoping: 8.2, 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: 8.2, 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 – Site is approx 1.1km southwest of the nearest pond chain (Golders Hill).	
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 – Flow routes at surface should be unchanged. Only change to surface water flow route will be the lightwell (from where the surface water will have to be pumped into the drainage system)	
3	Will the proposed basement development result in a change in the proportion of hard surfaced / paved external areas?	No – the proposed front lightwell is in an area which is already fully paved.	2.5 & Photo 2 in Appendix A
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 – The lightwell will not affect surface water run-off to adjacent properties. There are no watercourses within 250m.	5.2, 5.3 & 5.8
5	Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream watercourses?	No – There should be no significant change in surfaces generating run-off. None of the run-off from this property goes directly to a surface watercourse.	5.3 & 5.8
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 – Agamemnon Road did flood in 2002 (though probably only a small part at the low southern end of the road).	5.5 & Figure 5. Carried forward to Scoping: 8.3 & Section 10.8

7.5 <u>Non-technical Summary – Stage 1:</u>

The screening exercise in accordance with CPG4 has identified six issues which need to be taken forward to Scoping (Stage 2); five are related to ground stability and one is related to flooding potential. In addition, the presence of groundwater in the Made Ground and the London Clay must also be allowed for in the design of the basement and the associated temporary works; these matters are considered in Sections 10.2 and 10.3.

8. 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 Slope/ground stability scoping:

Issue (= Screening Question)		**Potential impact and actions**
5	Is the London Clay the shallowest strata at the site?	**Potential impact:** Heave in response to the unloading caused by the basement excavations, and as Q7 below. **Action:** Ground investigation required, followed by appropriate design**.**
7	Is there a history of seasonal shrink/swell subsidence in the local area, and/or evidence of such effects at the site?	**Potential impact:** Weakened structures from past movement would be more susceptible to damage during works. Future differential movement between the building above the basement and the adjoining structures**.** **Action:** Review potential impact of future vegetation growth. Designer and contractor to take account of any weakening of the structure caused by past movements.
12	Is the site within 5m of a highway or a pedestrian right of way?	**Potential impact:** Construction of basement causes loss of support to footway/highway and damage to the services beneath them**.** **Action:** Ensure adequate temporary and permanent support by use of best practice underpinning methods**.**
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 No's 44 & 48 Agamemnon Road, if basement excavations are inadequately supported. Possible long term differential movement. **Action:** Ensure adequate temporary and permanent support by use of best practice underpinning methods. Consider the need for transition underpinning.
14	Is the site over or within the exclusion zone of any tunnels, eg railway lines.	**Potential impact:** Stress changes on any tunnel lining. Piles or boreholes penetrating the tunnel (though no piles in this scheme)**.** **Action:** Undertake services search to check that there are no tunnels / deep services in the vicinity.

8.3 Surface flow and flooding scoping:

Issue (= Screening Question)		**Potential impact and actions**
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:** Review flood risk and provide flood resistance measures as appropriate**.**

8.4 <u>Non-technical Summary – Stage 2:</u>

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).
- Designer and contractor to take account of the weakening of the structure caused by past movements.
- Ensure adequate temporary and permanent support by use of best practice underpinning methods.
- Consider the need for transition underpinning to mitigate differential foundation depths, subject to Party Wall Act protocols.
- Undertake a services search to check whether there are any deep services/ tunnels which might be affected by the basement.
- 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. STAGE 3 – GROUND INVESTIGATION

9.1 The ground investigation sitework was carried out by Oakland Site Investigation (Oakland SI) on 20th March 2020, and consisted of one 'windowless' sampled borehole (BH1) drilled to a depth of 8.0m below ground level (bgl) in the front garden, and two hand dug trial pits (TP1 and TP2) in the cellar. The factual findings from the investigation are presented in Appendix F, including a site plan, borehole log and trial pit logs; reference should be made to the logs for full details of the strata descriptions and footing geometries. Laboratory testing on the samples recovered from the borehole and trial pits was undertaken by Geolabs; their test reports are also presented in Appendix F.

9.2 Trial pits TP1 and TP2 were dug in order to investigate the foundations to the party wall and front wall of the house, and the soils beneath the footings at their respective locations.

TP1:	Location:	44/46 party wall, in cellar
	Ground level:	approx. 1.55m below ground floor level
	Footing depth:	0.50m bgl; projection = 0.13m
	Materials:	Brickwork with 2 corbels (0.30m thick 'footing')
	Surfacing:	0.08m concrete slab
	Geology under footing:	"MADE GROUND. Soft to firm, brown sandy CLAY with abundant fine to medium fragments of brick and concrete".
TP2:	Location:	Corner of front wall and internal wall, in cellar.
	Ground level:	0.18m below ground floor level
	Footing depth:	Front wall: 0.18m bgl; projection = 0.13m Internal wall: not found (greater than 0.18m but low headroom restricted excavation.
	Materials:	Brickwork with 2 corbels (thin bricks in upper course)
	Surfacing:	0.08m concrete slab
	Geology under footing:	As TP1.

9.3 The geological sequence as found by borehole BH1 may be summarised as follows:

- <u>Made Ground:</u> Stone paving slabs, over successively: screed(?) and fragments of concrete and brick (0.11m), over coarse yellow sand with brick fragments (0.05m), over "*Soft to firm brown, very sandy, gravelly CLAY with frequent roots and rootlets. … Gravel is … fragments of brick and concrete*" (0.20m).
- <u>Possible Head Deposits</u>: The presence of fine to medium gravel in the "*Stiff orangish brown slightly sandy … CLAY…"* at 2.70-3.05m bgl indicates that the slightly sandy clays from 0.40m to 3.05m bgl could be a Head deposit, though it is more likely that these clays are in-situ Weathered London Clay as described below.

- Weathered London Clay Formation: Recorded from the base of the Made Ground (0.40m bgl in BH1) to a depth of 7.3m bgl. The Weathered London Clay was recorded as "*orangish brown slightly sandy CLAY"*. Its consistency was given as "*firm to stiff*" to 2.70m bgl, then "*stiff*" until 5.00m bgl below which it became "*very stiff*". "Occasional blue mottling" was present down to 5.0m bgl and occasional selenite (gypsum) crystals were recorded below 3.05m depth.
- 'Un-weathered' London Clay Formation: "Very *stiff, bluish grey,* (slightly) *sandy CLAY*" was recorded from 7.30m to the base of the borehole at 8.0m.

9.4 Standard Penetration Tests (SPTs) were carried out in BH1 at one metre intervals. The resulting 'N' values (blows to drive the 300mm test length, after 150mm of 'seating' driving) are recorded at the relevant depths on the borehole log (in Appendix F), and have also been plotted as a profile against depth in Figure 10 below. The SPT values show an overall trend of increasing shear strength with depth, as is typically found in this stratum.

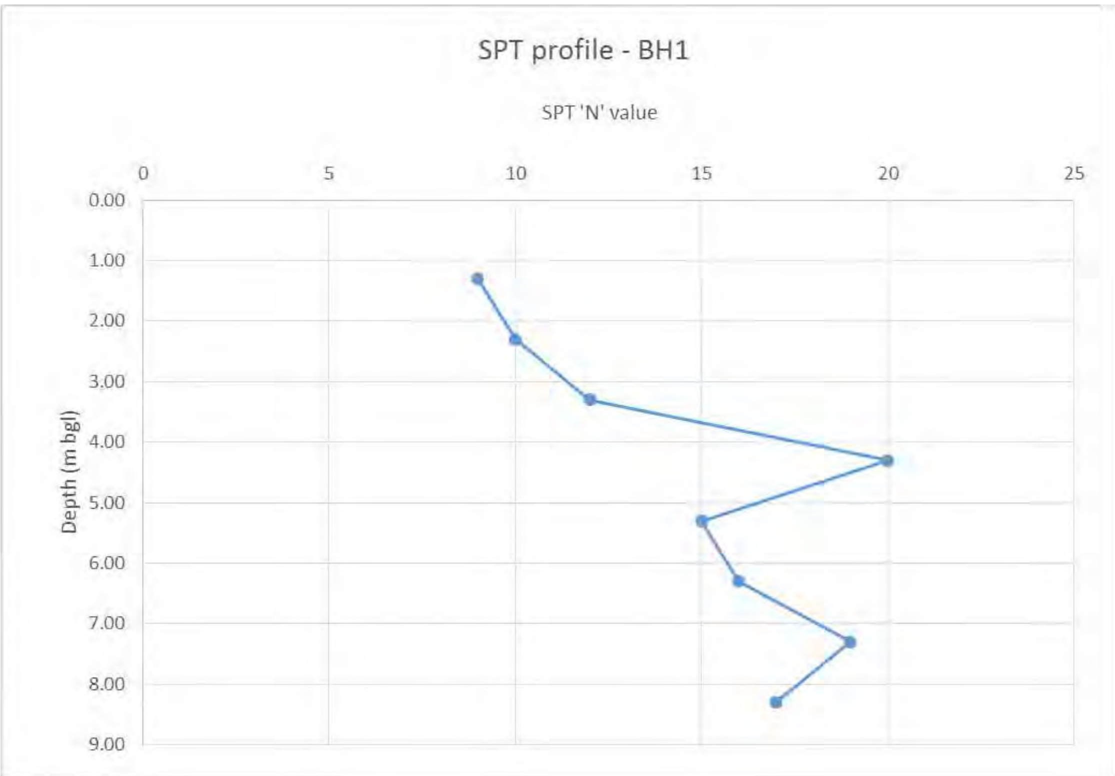

Figure 10: SPT 'N' values with depth

9.5 Roots were recorded only in the lower part of the Made Ground in BH1, to a maximum depth of 0.40m bgl. No roots were recorded in TPs 1 & 2.

9.6 No groundwater entries were recorded in any of the exploratory holes (BH1 and TPs 1 & 2).

9.7 A standpipe was installed to 4.00m bgl in BH1, comprising 1.0m of plain pipe at top, then 3.0m of slotted pipe. Bentonite seals were installed close to surface and at 4.0-4.5m bgl, beneath which the borehole was backfilled with arisings. Water level readings were recorded on two occasions; the results during this short monitoring period are presented in Table 2 below.

Table 2: Water levels from Groundwater Monitoring

Date	Depth to Water (m below ground level)	Approx. depth below ground floor FFL (m)
29-05-2020	3.08	3.2
08-06-2020	2.57	2.7

Laboratory Testing:

9.8 Geotechnical laboratory tests on samples recovered from borehole BH1 and trial pit TP1 were carried out by Geolabs Ltd. The testing comprised classification tests (including water content and plasticity) and chemical testing to assess the potential for acid or sulphate attack on buried concrete. The test reports from Geolabs (Project No. GEO/30889) are presented in Appendix F.

9.9 Plasticity tests were performed on three samples of Weathered London Clay, and one sample of 'unweathered' London Clay. The samples of Weathered London Clay were both found to be of Very High Plasticity (one on the borderline with High plasticity), as classified by BS5930 (2015), while the 'unweathered' London Clay from 8.0m bgl was found to be of High Plasticity; these results are displayed in Figure 11. All four samples were found to be of High Volume Change Potential, as defined by the NHBC (NHBC Standards, 2020, Chapter 4.2, Building near Trees).

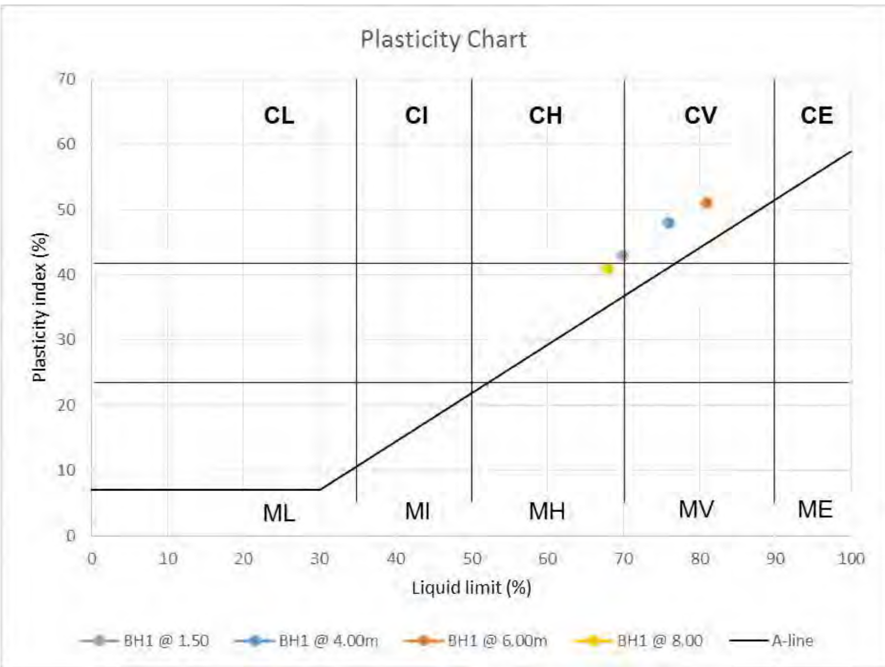

Figure 11: Plasticity chart for samples recovered from BH1.

9.10 The Water Contents of twelve samples recovered from BH1 were found to vary from 26.2% to 32.6%, showing an overall trend of decreasing Water Content with depth, as plotted in Figure 12 below. The Water Contents are unusually uniform in the Weathered London Clay, which probably reflects the front garden having been extensively paved for many years. The only exception is the slightly higher reading from the 1.0m sample, which is also entirely normal as the Water Content in the top 1.5m or so fluctuates in response to seasonal climatic changes.

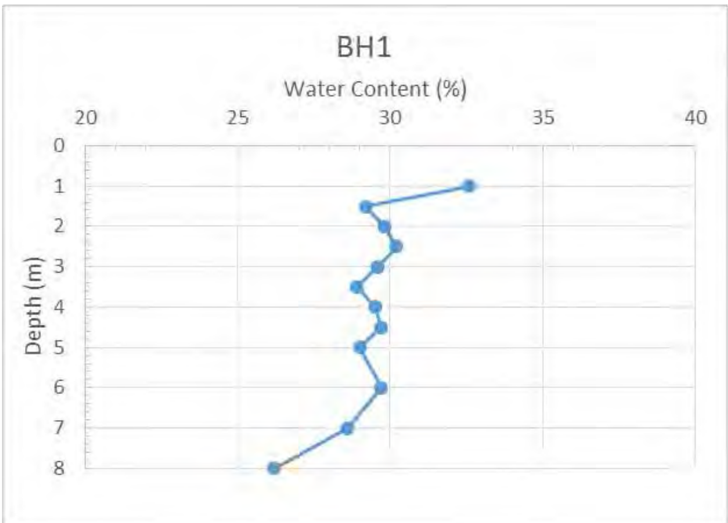

Figure 12: Profile of Water Content with depth in BH1.

9.11 The Water Contents of the four samples that were also tested for plasticity were found to range progressively from 2.2% above the respective sample's Plastic Limit (1.00m) to 0.8% below (at 8.00m). These comparisons suggest that there was no significant desiccation of the Weathered London Clay samples.

9.12 The chemical tests were undertaken on a total of five samples in order to assess the potential for acid or sulphate attack on buried concrete. The samples tested included two samples of Made Ground (from TP1 at 0.50m bgl and BH1 at 0.30m bgl) while the remainder were from the Weathered London Clay. The following ranges of results were recorded:

pH value:	7.8 - 9.6	
Water-soluble sulphate:	<10 - 1400mg/L	
Total acid-soluble sulphate:	0.016 – 1.1%	(London Clay only)
Total sulphur:	0.013 - 0.29%	(London Clay only)

Calculations following BRE Digest SD1 gave 'derived' values for the London Clay:

Total Potential Sulphates (TPS):	0.039 - 0.216%
Oxidizable sulphides:	0 - 0.056%

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

Made Ground:	DS-1 to DS-2
Weathered London Clay:	DS-1 to DS-3

<u>Non-technical Summary – Stage 3:</u>

9.13 The site-specific ground investigation at No.46 Agamemnon Road confirmed the presence of London Clay at shallow depth below the front garden. This is consistent with mapping by the British Geological Society (BGS). Made Ground was found immediately underneath the foundations which were exposed in the cellar.

9.14 No groundwater entries were recorded in the borehole (BH1) or in the two trial pits, though this does not mean that groundwater is absent. A standpipe was installed in BH1 to enable recording of groundwater levels/pressures. The highest water level recorded during the short monitoring period was 2.57m below ground level.

10. 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 as follows. For further details of the geology found by the ground investigation, and the results of in-situ and laboratory testing, reference should be made to Section 9 and Appendix F.

- Made Ground: Made Ground was present in all the exploratory holes. Only a limited thickness was found beneath the front garden (0.40m) whereas the maximum thickness was not proven beneath the cellar floor (where the trial pits reached at least 0.55m below floor level). The Made Ground was generally described as "soft to firm, variably sandy CLAY with fragments of brick and concrete"; the latter were abundant beneath the cellar floor where the lower part could be London Clay which has been disturbed in-situ. In the front garden, the clayey Made Ground was overlain by a thick bedding layer of granular materials (brick and concrete fragments, and sand) beneath the stone paving. 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.
- Head Deposits: Head deposits were not positively identified in the exploratory holes though the presence of fine to medium gravel in the horizon at 2.70-3.05m bgl indicates that those clays, and the overlying clays, might be a locally-derived Head deposit (see paragraph 9.3). Head deposits have been recorded elsewhere in the vicinity, where they comprised clays similar to the underlying Weathered London Clay (from which they were derived).
- Weathered London Clay: Firm to stiff, becoming stiff with depth, orangish brown slightly sandy CLAYS were found beneath the Made Ground in the front garden (see paragraph 9.3), and are expected to underlie the whole site.
  These clays contained occasional blue mottling down to 5.00m bgl, below which they were recorded as very stiff, although the in-situ Standard Penetration Tests indicated lower strengths. These lower 'mass' strengths are attributed to the presence of fissuring within these clays, which makes such clays less stable in excavations than would otherwise be expected. Selenite (a form of gypsum, which is aggressive to buried concrete) was recorded between 3.05m and 5.00m depths, and may be more widespread. These clays also often contain claystone nodules/horizons which can obstruct boreholes and piles.
  The logs of other boreholes in the area, indicate that the base of the Weathered London Clay can be found at depths ranging from 6.7m to 8.2m bgl.

- London Clay Formation ('un-weathered'): Apparently un-weathered, very stiff, bluish grey, (slightly) sandy CLAY of the London Clay Formation was encountered below 7.30m (see paragraph 9.3), and is likely to extend to a depth of more than 65m based on the finding from BGS borehole TQ28SW/74. These clays are expected to be fissured, and will also undergo heave movements in response to net unloading by basement excavations.

- Hydrogeology
  - Perched groundwater should be expected in the Made Ground during at least the winter and spring seasons, though it may be present only locally.
  - Groundwater pressures are expected to be essentially hydrostatic within the depth of current interest in the London Clay. Groundwater flow through these clays is likely to be minimal, in practice being limited to seepage through any of the silt/sand partings which are sufficiently interconnected. One sandy "pocket" was recorded at 5.55m bgl, and other fine partings of silt and fine sand are commonly found throughout the London Clay.
  - 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 10.2.8 for the recommended design groundwater level.

10.1.3 No railway tunnels are known to pass below or close to the site, though this must be verified. Other infrastructure (including tunnels) for 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 comprises variably sandy CLAYS with artificial debris, together with granular bedding for the paving in the front garden. The clays are likely to be of low permeability so seepage of any groundwater perched above the in-situ Weathered London Clay (of even lower permeability) is expected to be minimal. No perched water was found in the Made Ground during the site-specific ground investigation, though perched water may develop locally at times, at least during wetter winter and spring seasons.

10.2.2 The common lack of groundwater entries into boreholes while drilling through the London Clay is caused by the low permeability of the clays and the temporary sealing of any slightly more permeable layers by the drilling process, rather than by an absence of water.

10.2.3 The highest groundwater standing level (phreatic surface) below the front garden during the brief monitoring period was 2.57m below ground level (bgl) and higher water levels must be expected in winter months.

Existing Basements:

10.2.4 With the exception of No.48, all the similarly-styled properties along the terrace in which No.46 is situated (No's 22 to 46) are expected to have original single-storey cellars similar to No.46's. The owner of No.48 has confirmed that it does not have a cellar.

Other Proposed Basements:

10.2.5 No applications were found on Camden's planning website for modern basements beneath the adjoining and adjacent properties (No's 42, 44 & 48 Agamemnon Road and No.62 Gondar Gardens).

Proposed Basement at No.46:

10.2.6 Details of the proposed works are given in Section 3. The proposed basement is expected be founded at approximately **3.20m** below the ground floor's FFL (the formation level; see paragraph 3.2). Based on the strata levels in BH1, the basement's formation level will be in the Weathered London Clay. The basement will therefore obstruct any flows of perched groundwater but it is not expected to create any significant impact, locally or cumulatively, because the existing footings already block any downslope seepage/flow in the Made Ground, the existing cellars already cause intermittent 'obstructions', and because of the naturally very low permeability of the London Clay. The service trenches beneath the carriageway and footway are also likely to provide flow paths with higher permeabilities than most of the surrounding natural ground. Thus, the proposed basement is considered acceptable in relation to groundwater flow.

10.2.7 In the unlikely event that the basement excavations encounter, and would completely obstruct, a local deposit of more permeable soils containing mobile groundwater which has remained undetected within the London Clay (or Head deposits), of sufficient thickness and extent to permit significant flow, then it is possible that an engineered groundwater bypass might be required. This bypass would have to be detailed once the geometry of the permeable soil unit is known. Water-bearing claystone horizons in the London Clay can also permit significant seepage/flow and might require similar treatment if encountered.

10.2.8 Current geotechnical design standards require use of a 'worst credible' approach to selection of groundwater pressures. On sites such as this where high plasticity clays are present close to surface, the groundwater table (or phreatic surface) may rise to surface, or at least into the overlying Made Ground in the wettest winters, unless mitigation measures such as land drainage can be installed. No acceptable disposal location exists for such water (because there is no accessible watercourse nearby, and Thames Water will not allow long-term disposal of groundwater to the mains drainage system). As a result, use of a design groundwater level at ground level is recommended for the whole basement, for both short-term and long-term situations (in accordance with Eurocode 7, BS EN 1997-1).

10.2.9 The basement structure should be designed to resist buoyant uplift pressure that would be generated by the 'worst credible' groundwater levels. For the design groundwater level suggested above, buoyant uplift pressures of up to 31kPa (un-factored) would have to be accommodated.

10.2.10 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.

10.2.11 The National House Building Council published new guidance on waterproofing of basements in November 2014 (now NHBC Standards, 2020, Chapter 5.4). Compliance would be compulsory if an NHBC warranty is required, otherwise it may provide a useful guide to best practice.

10.3 Subterranean (Groundwater) Flow – Temporary Works

10.3.1 Local groundwater entries/seepages may occur into the excavations for the basement though, on current evidence, they are likely to be minor and should be manageable by sump pumping, provided that they are not being fed by defective drains or water supply pipes. It would be prudent to ensure the external isolation stopcock is both accessible and operational before the start of the works. An appropriate discharge location must be identified for the groundwater removed by sump pumping.

10.3.2 All groundwater control measures should be supervised by an appropriately competent person. A careful watch should be maintained to check that fine soils are not removed with the groundwater; if any such erosion/removal of fines is noticed, then pumping should cease and the advice of a suitably experienced and competent ground engineer should be sought.

10.3.3 The unloaded clays at/beneath formation level will readily absorb any available water which would lead to softening and loss of strength. It will therefore be important to ensure that the clays at formation level (onto which the underpins and the basement slab will bear) are protected from all sources of water, with suitable channelling to sumps for any groundwater seeping into the excavations. The formation clays should be inspected and then blinded with concrete immediately after completion of final excavation to grade. Any unacceptably soft/weak areas must be excavated and replaced with concrete.

10.4 Slope and Ground Stability

10.4.1 With slope angles of <2° in the immediate vicinity of this property and up to 3.9° in the surrounding area (see paragraph 2.10), the proposed basement excavation raises no concerns in relation to slope stability.

Basement Retaining Wall Construction – Underpinning:

10.4.2 Use of reinforced concrete (RC) underpinning is anticipated for construction of the proposed basement, as shown on the drawings by David Joseph Consulting (DJC; see paragraph 1.6), subject to agreement under Party Wall Act protocols. For the front lightwell, where the basement extends out beyond the footprint of the existing building, the perimeter walls will be constructed as 'L' shaped, cantilevered, RC retaining walls in panels not exceeding 1.0m in width, on the same 'hit and miss' basis as the underpinning. The rear wall of the basement will require RC underpinning of the 0.9m wide mass concrete strip footing which was installed in 2015 to support the box frame at ground floor level, together with an RC lining wall above the 1.10/0.80m RC toe. The estimated founding (formation) levels and excavation depths of the underpins, retaining walls and basement slab are explained in paragraphs 3.2 and 3.3.

10.4.3 Underpinning methods involve excavation of the ground in short lengths (not exceeding 1.0m is recommended) in order to enable stresses in the ground to 'arch' onto the ground or completed underpinning on both sides of the excavation. Loads from the structure above will similarly arch across the excavation, provided the structure is in good condition.

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 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 use of best practice methods of temporary support are therefore crucial to the satisfactory control of ground movements alongside basement excavations (see 10.4.5 to 10.4.7 below). Any cracks and past repairs in load-bearing walls which have weakened their structural integrity should be fully repaired in accordance with recommendations from the appointed Structural Engineer before any underpinning is carried out. The structural significance of the distortion noted in the external walls should also be reviewed by a suitably competent Structural Engineer.

10.4.5 The minimum temporary support requirements recommended for the excavations for the proposed underpins and RC retaining walls, subject to inspection and review as described in 10.4.6 and 10.4.7 below, are:

- Full face support must be installed as the excavations progress for all excavations through the Made Ground and in any firm clay which is present at the top of the London Clay/Head deposits.
- Closely-spaced temporary support may be adequate in the stiff or very stiff clays of the weathered London Clay Formation, depending on the degree of fissuring.
- Temporary support must also be installed to support all the new underpins, and must be maintained until the full permanent support has been completed, including allowing time for the concrete to gain adequate strength.

All temporary support should use high stiffness systems, installed in accordance with best practice, in order to minimise the ground movements.

10.4.6 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 London Clay has been inferred to be fissured; such fissures can cause seemingly strong, stable excavations to collapse with little or no warning. Thus, in addition to normal monitoring of the stability of the excavations, a suitably competent person should check whether such fissuring is present and, if encountered, should assess what support is appropriate.

10.4.7 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.

10.4.8 The unloaded clays at/beneath formation level will readily absorb any available water which would lead to softening and loss of strength. It will therefore be important to ensure that the clays at formation level are protected from all sources of water, with suitable channelling to sumps for any water seeping into the excavations. The formation clays should be inspected then blinded with concrete immediately after completion of excavation to grade and inspection. Any unacceptably soft/weak areas must be excavated and replaced with concrete.

10.4.9 The provisional construction sequence should be provided by the appointed Structural Engineer or a specialist basement contractor. That can only be preliminary because the appointed contractor will be responsible for the temporary works and preparation of the final Construction Phase plan.

Geotechnical Design - Retaining Walls:

10.4.10 Design of the retaining walls for the basement must include all normal design scenarios (sliding, over-turning and bearing failure), and must take into consideration:

- Earth pressures from the surrounding ground (see paragraph 10.4.11 below);
- Dead and live loads from the superstructure, including loads from the adjoining No's 44 & 48 which are carried on the party walls;
- Loads from all adjoining/adjacent walls in No's 44 & 48 which are founded within the relevant active earth pressure zone;
- Vehicle loads on the public footway at the front of the site (approximately 2.4m from the front lightwell), and normal surcharge allowances elsewhere;
- Swelling displacements/pressures from the underlying clays;
- Design groundwater levels and hydraulic uplift forces on the basement structure, as described more fully in paragraphs 10.2.8 and 10.2.9;
- Precautions to protect the concrete from sulphate attack.

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

Made Ground (clays):	Unit weight, γ_b:	18.0 kN/m^3
	Effective cohesion, c':	0 kPa
	Angle of internal friction, φ':	25°
Head Deposits (clays):	As London Clay below.	
London Clay Fm:	Unit weight, γ_b:	20.0 kN/m^3
	Effective cohesion, c':	0 kPa
	Angle of internal friction, φ':	22°

Coefficient of earth pressure at rest, k_0: up to 2.5-3.0 where undisturbed (varies with depth); the extent to which this stress is released when the underpins are excavated depends on the stiffness of the temporary and permanent support, but might typically reduce to around $k_0 = 1.0$.

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

Geotechnical Design - Bearing Capacity:

10.4.12 The founding level (formation) of the underpins and slab will be approximately **3.20m** below the ground floor's FFL (paragraph 3.2) and the ground level at the position of BH1 was approximately 0.1m below the ground floor FFL. Thus, the Standard Penetration Test (SPT) 'N' values at and below 3.0m in BH1 are applicable (see paragraph 9.4 and Figure 10) and have been converted to undrained cohesion (Cu) values using the relationship identified by Stroud (1974). The SPT 'N' values reflect the presence of fissures in these clays. From a Cu value of **60kPa** just below the proposed founding depth for the basement the shear strengths increased with depth to **76kPa** at 8.00-8.45m.

10.4.13 Based on the derived undrained cohesion values given above, the minimum allowable bearing pressures for the underpins and retaining structures would be **100kPa** for up to 25mm settlement (long-term) based on a bearing capacity factor, N_C = 5.2 (after Skempton, 1951, for a strip footing with no adjacent surcharge) and a factor of safety, F = 3. This allows for the temporary situation when the central area within the underpins is excavated prior to casting the central basement slab. A lower factor of safety could be justified for temporary works, though settlements could then be greater.

Transitional footings:

10.4.14 Normal good practice in foundation construction requires progressive stepping up between foundations of different depths beneath a single structure. Subject to agreement under the Party Wall Act negotiations, transitional underpins should be considered for all adjoining load-bearing walls in No's 44 & 48 where the differential founding depth exceeds 1.0m. The cellar beneath No.44 will provide a transition, though the implications of the past differential settlement of the south side of that cellar (as evident in the front wall of No.44 – see Photo 7a in Appendix A) should be considered by the appointed Structural Engineer.

Trees:

10.4.15 The only trees in the vicinity of the proposed basement are a young pavement tree (Cherry?) outside No.46 and small conifers in the garden of No.48. Guidance from the NHBC (NHBC Standards, 2020, Chapter 4.2) indicates that the basement will be remote from its likely current zone of influence and, even once fully grown, the pavement tree would not impact the basement. If the conifers in No.48's garden are allowed to grow unrestrained they would have a detrimental impact on No.48 long before they affect the proposed basement (which would protect No.46 from root action by those trees unless they grow much larger). It is therefore reasonable to assume that the owner of No.48 will take appropriate precautions to protect their property before the conifers present a hazard to the proposed basement.

10.5 PDISP Heave/Settlement Assessment

Basement Geometry and Stresses:

10.5.1 Analyses of the 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 in vertical stresses caused by excavation of the basement extension and front vaults.

10.5.2 Figure G1 in Appendix G illustrates the layout of the proposed works at basement level based on DJC's 'Proposed Preliminary Basement Floor Plan' (drawing No.4388_SK04 rev.A), along with the layout of the PDISP zones used to model the proposed underpins, lightwell and basement slab (see Section 3 and paragraph 10.4.2). The load takedown data for the proposed building have also been provided by DJC, as presented in Figure G2. The horizontal forces acting on the retaining walls have not been modelled, so the stress regime has been simplified.

10.5.3 The overall dimensions of the basement excavations are approximately 5.95m wide by 10.5m long (including the 0.9m wide strip footing beneath the rear wall) extending locally to 11.75m for the under-stair plant room. For the purposes of these analyses, the founding depth of the underpins and basement slab was taken as **3.20m** below ground floor level, and **2.975m** below the same floor level for the front lightwell (see Section 3).

10.5.4 Table 3 presents the **net** bearing pressures which will result from the basement works for all the primary PDISP zones during the four major stress states associated with these works (see 10.5.7 below for details), and the **gross** loading/unloading values for the superimposed zones. All applied pressures were calculated from DJC's load takedown. For the front bay, temporary sacrificial underpins are proposed for the two shoulder sections (Zones 1 & 2) with a beam spanning between them to support the central section and allow access to the works via the lightwell excavation. Zones 1a & 2a comprised 0.45m wide underpins bearing onto 0.85m square pads, so excluded the flank wall of the lightwell and were calculated only in order to check the bearing pressure in that configuration. The PDISP analyses modelled Zones 1b & 2b which included the adjacent sections of the lightwell's flank walls.

Table 3: Bearing pressure changes for PDISP Zones				
ZONE	**TYPE**	**Change in vertical pressure (kPa)**		
#		**Stage 1**	**Stage 2 (Full excv'n)**	**Stages 3 and 4**
1a	Temporary underpin	92.70	N/A	N/A
2a	Temporary underpin	92.70	N/A	N/A
1b	RC U/pin & temp u/pin or bay	83.60	83.60	83.60
2b	RC U/pin & temp u/pin or bay	83.60	83.60	83.60
3	Front lightwell	-17.16	-17.16	-17.16
4	Front lightwell	-17.16	-17.16	-17.16
5	Front lightwell	-36.15	-36.15	-36.15
6	RC Underpin	71.37	71.37	71.37
7	RC Underpin	22.90	22.90	22.90
8	RC Underpin	43.04	43.04	43.04
9	RC Underpin	-3.90	-3.90	-3.90
10	RC Underpin	64.31	64.31	64.31
11	Mass concrete underpin on RC base + RC lining wall	23.42	23.42	23.42
12	RC Underpin	88.22	88.22	88.22
13	Basement slab	N/A	-53.49	-45.99
14	Existing pad footing	N/A	-98.77	-98.77
15	Basement slab	N/A	-53.49	-45.99
16	Basement slab	N/A	-53.49	-45.99
17	Basement slab	N/A	-53.49	-45.99
S1	Superimposed zone	8.67	8.67	8.67
S2	Superimposed zone	N/A	47.33	47.33
S3	Superimposed zone	N/A	73.33	73.33
S4	Superimposed zone	8.67	8.67	8.67
S5	Superimposed zone	22.00	22.00	22.00
S6	Superimposed zone	38.60	38.60	38.60
S7	Superimposed zone	N/A	N/A	101.86
X1	Reduced excavation depth	22.14	22.14	22.14
X2	Reduced excavation depth	N/A	22.14	22.14
X3	Reduced excavation depth	N/A	22.14	22.14
X4	Increased excavation depth	-22.04	-22.04	-22.04
Key:				
(green)	: Live loads omitted, as this will result in maximum differential settlement.			
(yellow)	: Live loads included, even though heave predicted, because increased heave would be beneficial to the Damage Category Assessment.			

<u>Ground Conditions:</u>

10.5.5 The ground profile was based on the site-specific ground investigation by Oakland SI Ltd, as presented in Sections 9 and 10.1 above, and the desk study information in Section 4.

10.5.6 The short-term and long-term geotechnical properties of the soil strata used for the PDISP analyses are presented in Table 4, based on this investigation and data from other projects.

Table 4: Soil parameters for PDISP analyses				
Strata	**Level** (m below ground floor)	**Undrained Cohesion,** Cu (kPa)	**Short term, undrained Young's Modulus,** Eu (MPa)	**Long term, drained Young's Modulus,** E' (MPa)
Head Deposits (?)	-2.1 -3.2	45 54	22.5 27	13.5 16
Weathered London Clay & London Clay	-3.2 -15.0	60 119	30 59.5	18 35

Where: For Head Deposits:
Undrained Shear Strength, Cu at top and base of stratum is based on the SPT profile (see Figure 10).

For Weathered London Clay and London CLAY:
Undrained Shear Strength, Cu at top of stratum is based on the SPT profile and firm to stiff becoming stiff descriptors.
Undrained Shear Strength within stratum assumed to increase at: 5.z kPa
where z = depth below the top of the stratum.
Undrained Young's Modulus, Eu = 500 * Cu
Drained Young's Modulus, E' = 0.6 * Eu

PDISP Analyses:

10.5.7 Three dimensional analyses of vertical ground movements in response to construction of the proposed basement extension have been undertaken using PDISP software and the basement and ground floor geometries, loads/stresses and ground conditions outlined above. PDISP analyses have been carried out as follows:

- Stage 1: Installation of sacrificial temporary underpins beneath shoulders of the bay window, followed by excavation of the front lightwell then installation of all perimeter underpins working via the front lightwell – Short-term (undrained) condition
- Stage 2: Excavation of central area of basement – Short-term condition
- Stage 3: Installation of central basement slab, thereby completing the structure of the basement – Short-term condition
- Stage 4: As Stage 3, except – Long-term (drained) condition

10.5.8 Stages 1 – 4 were analysed at formation level (3.2m below ground floor level) though the ground profile was taken from 2.1m below the ground floor in order to be able to model the unloading from removal of the temporary pad footing which currently supports column C5.

10.5.9 The results of the analyses for Stages 1-4 are presented as contour plots on the appended Figures G3 to G6 respectively.

Heave/Settlement Assessment:

10.5.10 The proposed works will cause immediate elastic displacements (settlement/heave) in response to the stress changes, followed by long-term plastic deformations (swelling/consolidation) as the pore water pressures in the over-consolidated clays, which underlie the site, adjust to the stress changes. The rate of plastic swelling/ consolidation will be determined by the availability of water and the permeability of the soils concerned; the low permeability of the London Clay typically results in these adjustments taking many decades to reach full equilibrium. The underpins and basement slab will need to be designed so as to enable them to accommodate the swelling displacements/pressures developed beneath them and the resultant distortions.

10.5.11 The ranges of predicted short-term and long-term movements for each of the main parts of the proposed basement are presented in Table 5 below. The predicted displacements have been rounded to the nearest 0.5mm. These analyses predicted the largest settlements beneath the 44-46 party wall, particularly beneath the small rear projection, with settlements of up to 7mm being predicted, and differential displacements up to 10mm. Elsewhere, the displacements predicted for the underpins ranged from 4mm settlement to 2.5mm heave, while for the front lightwell the range was slightly smaller.

10.5.12 The range of displacements quoted in Table 5 cover approximately the full range of predicted deflections, however the stiffness of the underpins is likely to reduce the range of displacements actually experienced.

10.5.13 All the short-term elastic displacements would have occurred before the central basement slab is cast, so only the post-construction incremental heave/settlements are relevant to the design of the slab. These preliminary analyses indicated that the maximum predicted post-construction differential displacements are likely to be approximately 5mm, while total differential displacements across the basement slab might reach about 10mm.

Table 5: Summary of predicted displacements				
Location	**Stage 1 (Figure G3)**	**Stage 2 (Figure G4)**	**Stage 3 (Figure G5)**	**Stage 4 (Figure G6)**
Front lightwell & Front bay (Zones 1b, 2b & 3-5)	2.5mm Settlement to 0.5mm Heave	1.5mm Settlement to 1.5mm Heave	1.5mm Settlement to 1.5mm Heave	3mm Settlement to 2mm Heave
Front wall (Zones 6 & pt 9)	0 - 3mm Settlement	0 - 2mm Settlement	0 - 2mm Settlement	0 - 4mm Settlement
No.44/46 party wall (left side) including under-stair cupboard (Zones 6, 7, 8 &12)	1 – 4.5mm Settlement	4mm Settlement to 2mm Heave	4mm Settlement to 1.5mm Heave	7mm Settlement to 2.5mm Heave
No.46/48 party wall (Zones 9 & 10)	0 – 2.5mm Settlement	2.5mm Settlement to 2mm Heave	2.5mm Settlement to 1.5mm Heave	4mm Settlement to 2.5mm Heave
Rear wall/box frame (Zone 11)	0.5 - 3mm Settlement	0 - 3mm Settlement	0 - 3mm Settlement	0 - 4mm Settlement
Central basement slab	N/A	1mm Settlement to 3.5mm Heave (No slab present)	1mm Settlement to 3mm Heave	1 – 5mm Heave

10.6 Damage Category Assessment

10.6.1 When underpinning, it is inevitable that the ground will be un-supported or only partially supported for a short period during the excavation of each pin, even when support is installed sequentially as the excavation progresses. This means the behaviour of the ground will depend on the quality of the workmanship and suitability of the methods used, so rigorous calculations of predicted ground movements are not practical. However, provided that the temporary support follows best practice as outlined in Section 10.4, then extensive past experience has shown that the bulk movements of the ground alongside a single-storey basement (typical depth 3.5m) should not exceed 5mm horizontally.

10.6.2 In order to relate these typical ground movements to possible damage which adjoining properties might suffer, it is necessary to consider the strains and 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).

10.6.3 The potentially critical locations will be determined by the displacements predicted by the PDISP analyses and the geometries of the adjoining buildings. For these damage category assessments, we are interested in the ground movements at the foundation level of the neighbouring buildings, whereas the empirical data for ground movements alongside excavations presented in CIRIA Report C760 (Gaba et al, 2017) concerns movements at ground surface (and presents data for embedded retaining walls, but, as no equivalent data exist for underpins, this data is deemed the best available, though it must be interpreted very cautiously).

10.6.4 The adjoining No.44 has a very similar layout to No.46, with similar wrap-around rear extension. Plans attached to planning application 2015-6355-P indicate that (in both No's 44 & 42) the original transverse dividing walls between the two main ground floor reception rooms have largely been removed, so it would not be appropriate to apply Burland's methodology for assessing damage to the remaining sections of wall. The sections provided with the same application indicate that the main rear walls of both houses were also to be removed at ground floor level, whereas the plans show them remaining in place. As the maximum settlements predicted by the PDISP analyses were alongside those rear walls, it has been assumed that they are still in place at ground floor level and, as a result, they represent the most critical location along the 44/46 party wall for potential damage caused by ground movements in response to excavation of the basement.

10.6.5 No.48 is a larger, end-of-terrace house with a different internal layout compared to No.46. Its main west wall is approximately 3.0m to the west of No.46's main rear wall. The two north-south oriented internal walls in No.48 adjoin the party wall slightly to the rear of No.46's former internal wall (opposite the rear end of PDISP Zone S3) and close to the rear corner of the basement (opposite Zones 10 and S5). The PDISP analyses predicted the largest settlements along the 46/48 party wall at the rear corner of the basement so that location is considered to be potentially the most critical for present purposes. In the ground alongside the basement, a very slightly larger settlement (1.00mm instead of 0.96mm) was predicted by the front right corner of Zone S3; however there is no internal wall in No.48 adjoining that location, so no damage category assessment is warranted there.

10.6.6 Separate damage category assessments have been undertaken for both of the locations identified above. These assessments considered:

- ground movements alongside the proposed underpins caused by relaxation of the ground in response to the excavations, using empirical data from monitoring of large retaining walls during construction, as presented in CIRIA Report C760 (see 10.6.3 above);
- ground movements arising from the vertical stress changes, as assessed by the PDISP analyses (see Section 10.5), including an allowance for the stiffness of the foundations. Only the post-construction displacements (between Stages 3 & 4 of the PDISP analyses) have been considered, because the CIRIA data includes all movements during construction.

Ground movements associated with the construction of retaining walls in clay soils have been shown to extend to a distance up to 3.5 to 4 times the depth of the excavation.

<u>Main rear wall of No's 42 & 44 Agamemnon Road:</u>

10.6.7 The relevant geometries, based on information in Section 3, the ground investigation at No.46 (see Section 9 and Appendix F), and the relevant drawings for No's 42 & 44 (see 2.11) are:

Depth of excavation alongside rear wall = **3.2m** (from ground floor in No.44, which was raised as part of the approved works, though floor construction detail is not known).

Width, horizontal movement = 3.2 x 4 = **12.8m**, so will extend just into No.40's site. Horizontal movements are typically linear, so generate no deflection.

Width (L), settlement = 3.2 x 3.5 = **11.2m**, so will extend across the full width of No.44 and most of No.42's width.

Depth of foundations to party wall = **approx. 0.5m** (assumed)

Height (H) = 5.9 (to eves) + 0.5 = **6.4m**

Hence L/H = **1.75**

10.6.8 The typical horizontal displacement value of 5mm for a single-storey basement, combined with the geometry recorded above, indicates that the horizontal strain beneath No's 42 & 44 is likely to be in the order of $\varepsilon_h = 3.91 \times 10^{-4}$ (0.039%).

10.6.9 The settlement caused by relaxation of the ground alongside the basement, in response to excavation of the retaining wall, can be estimated using the settlement profile for the worst case (low stiffness) scenario presented in Figure 6.15b of CIRIA Report C760. This CIRIA data should be combined with the long-term movements predicted by the PDISP analysis, between Stage 3 (short-term) and Stage 4 (long-term); the settlement profiles are then summed to find the maximum deflection, Δ. Figure 13 presents these settlement profiles for the main rear wall of No's 42 & 44. The maximum Δ = 2.1mm, which represents a deflection ratio, Δ/L = 1.88 x 10^{-4} (0.019%).

Figure 13: Displacement profile for main rear wall of No's 42 & 44.

10.6.10 Using the graphs for L/H = 2.0, which is slightly conservative, 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 14 below.

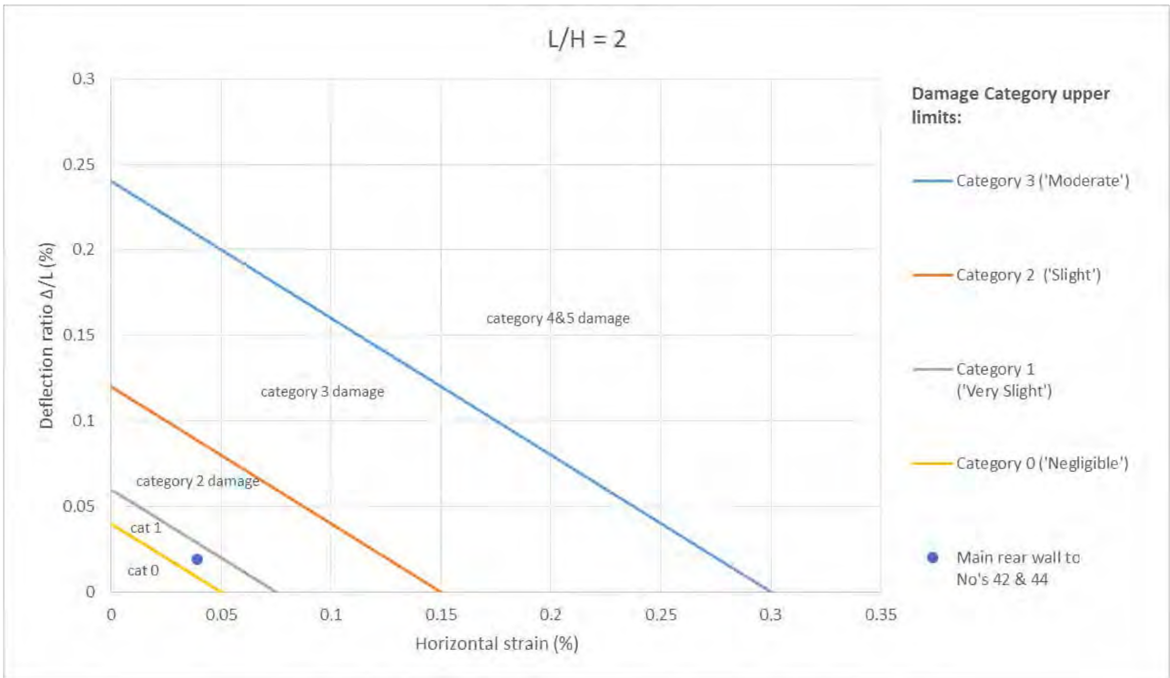

Figure 14: Damage category assessment for main rear wall of No's 42 & 44.

<u>Internal Wall in No.48 Agamemnon Road:</u>

10.6.11 This assessment has considered the westernmost of the two north-south oriented internal walls in No.48 which adjoins the party wall close to the rear corner of the basement, opposite PDISP Zones 10 and S5 (see paragraph 10.6.5). The relevant geometries for this 7.35m long wall, are:

Depth of excavation = **2.82m** (see paragraph 3.3), assuming that the underfloor void in No.48 is the same as that in No.46.

Width, horizontal movement = 2.82 x 4 = **11.28m**, so will extend well beyond the front wall of No.48 (which faces onto Gondar Gardens).

Width (L), settlement = 2.82 x 3.5 = **9.87m**, so also extends beyond the front wall of No.48.

Depth of foundations to party wall = **approx. 0.5m** (assumed)

Height (H) = 6.3 (to eves) + 0.5 = **6.8m**

Hence L/H = **1.45**

10.6.12 Following the same procedure as before, the anticipated strain beneath this internal wall would therefore be in the order of ε_h = 4.43x 10^{-4} (0.044%).

10.6.13 Assessment of the maximum deflection also followed the same procedure as before, using the low stiffness settlement profile in Figure 6.15b of CIRIA Report C760 combined with the long-term movements predicted by the PDISP analyses between Stage 3 and Stage 4. In this case however, allowance was also made for the affected wall being shorter than the zone of influence of the ground movements. The maximum deflection, Δ = 0.43mm, as shown in Figure 15, which represents a deflection ratio, $\Delta/L = 4.36 \times 10^{-5}$ (0.004%).

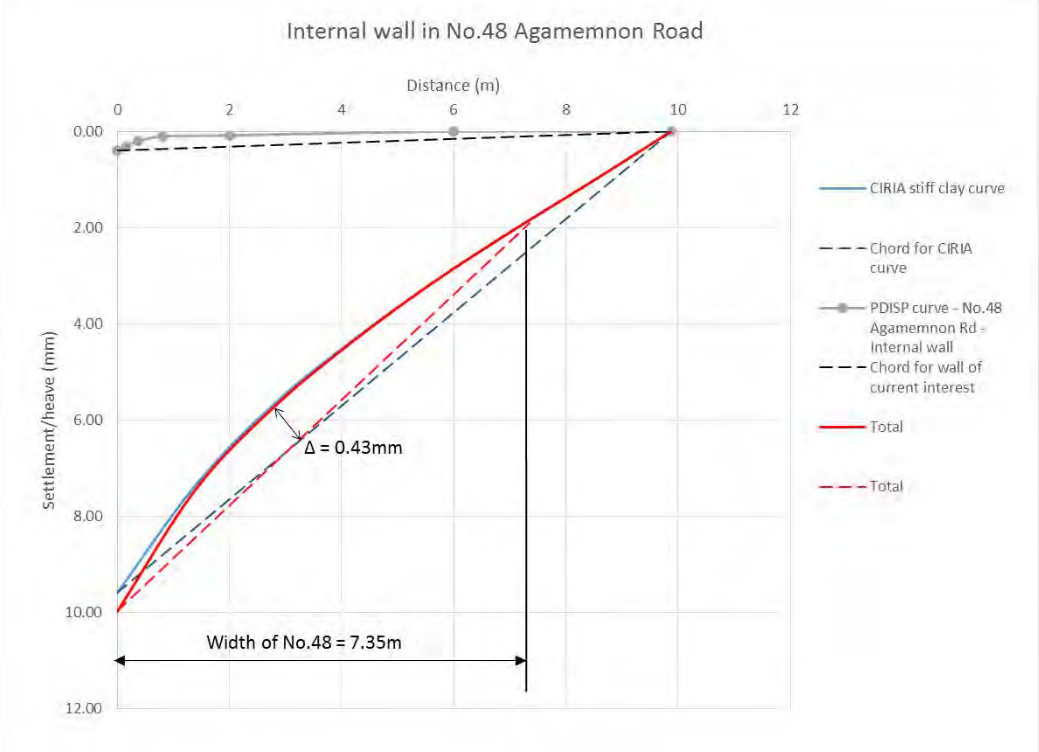

Figure 15: Internal wall in No.48 Agamemnon Road.

10.6.14 Using the graphs for L/H = 1.5, these deformations represent a damage category of 'negligible' (Burland Category 0, ε_{lim} = <0.050%), just on the boundary Category 1 'very slight' (ε_{lim} = 0.050-0.075%), as given in CIRIA SP200, Table 3.1, and illustrated in Figure 16 below.

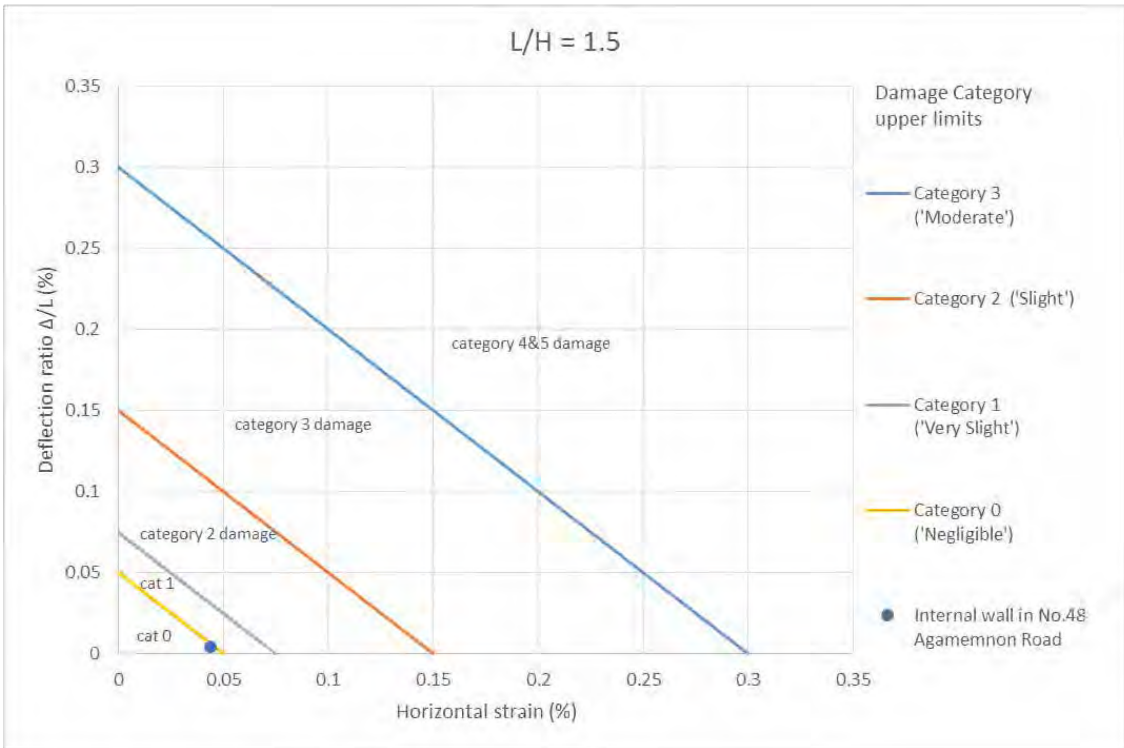

Figure 16: Damage category assessment for internal wall in No.48.

10.6.15 Use of best practice construction methods, as outlined in Section 10.4, will be essential in order 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 Award and are beneficial to all parties concerned.

10.7.2 Precise movement monitoring should be undertaken weekly throughout the period during which the basement walls/raft, underpins and slab are constructed, with initial readings taken before excavation of the basement starts. Readings may revert to fortnightly once all the perimeter walls and the base slab have been completed, and may terminate three months after the new basement slab has reached working strength, the formwork has been struck and all temporary support has been removed, provided that there are no progressive on-going movements. This monitoring should be undertaken with a total station instrument and targets attached at a minimum of two levels at the following locations:

- internally, at three equally spaced locations on both the 44/46 and 46/48 party walls, above the front, middle and rear of the proposed basement;
- externally, on the front wall of No.46, on the centrelines of the 44/46 and 46/48 party walls;
- at the client's discretion, since outside the Party Wall Agreements, it would be sensible to monitor all other load-bearing walls and columns in No.46 which might be affected by the proposed excavations.

10.7.3 The wall movements detected by the monitoring exercise may be caused by rotation, flexing without cracking (especially for walls built using lime mortar, as may apply for a property of this age), or lateral movements transverse to the plane of the wall. Movements such as these which occur without cracking would all fall within Burland's Category 0, so a twin-track approach to the monitoring will be required, combining both the target monitoring as proposed above and visual observations. Daily inspections of the subject property and external walls of the adjoining buildings should be made and recorded by a member of the contractor's staff. If any new structural cracks appear in the main load-bearing walls, then the appointed Structural Engineer should be informed and those cracks should be monitored using the Demec system (or similar) on the same frequency as the target monitoring. Additional targets might also be installed, at the engineer's discretion, depending on the location of the cracks. It will be important to ensure that any pre-existing cracks in the affected load-bearing walls which have weakened their structural integrity should be fully repaired in accordance with recommendations from the appointed Structural Engineer before any underpinning is carried out (as recommended in paragraph 10.4.4).

10.7.4 While monitoring readings from this system are typically presented to the nearest 0.1mm, the accuracy (repeatability) is usually quoted as +/-2mm or +/-1.5mm. Thus, if recorded movements in either direction reach 5mm (amber trigger level), 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 recorded movements in either direction reach 8mm (red trigger level), then work should stop until new methods statements have been prepared and approved by the appointed structural engineer. Local temporary backfilling of the excavation adjacent to the movement of concern may be required.

10.8 Surface Flow and Flooding

<u>Flooding from Rivers, Sea & Reservoirs:</u>

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 (from rivers or sea), and is classified as having a Very Low risk of fluvial/tidal flooding under the Environment Agency's Risk of Flooding from Rivers or Seas (RoFRaS) dataset (paragraph 5.6);
- the site is not at risk of flooding from reservoirs, as mapped by the Environment Agency (paragraph 5.7);
- there are no flood defences, no areas benefitting from flood defences and no flood storage areas within 250m of the site.

<u>Surface Water (Pluvial) Flooding:</u>

10.8.2 There are no natural surface water features within 250m of the site (paragraph 5.8).

10.8.3 The *'Floods in Camden'* report (LBC Floods Scrutiny Panel, 2003) and Arup's 2010 guidance document (Camden GHHS) record that Agamemnon Road was flooded in the 2002 local pluvial flood event, but not in 1975, although the extent of the road affected was probably limited to the low point where it joins Hillfield Road (see Figure 5 above).

10.8.4 The Camden Strategic Flood Risk Assessment (SFRA, by URS, 2014) shows that Agamemnon Road is just within Critical Drainage Area 'Group3_010' (see Figure 7). However, Agamemnon Road was not in any of the Local Flood Risk Zones which were identified in the Camden SFRA, 2014 (see Figure 7). CDAs include both source areas and flood-prone areas; the evidence presented above and below indicates that No.46 is in a source area for flooding elsewhere and is **not** in a flood-prone area.

10.8.5 The current risk of surface water (pluvial) flooding within the sites of No's 40-48 (even numbers) is indicated to be 'Very Low' by the Environment Agency's latest modelling (see Figure 6 herein) and 'Negligible' according to modelling by Ambiental Risk Analytics (see paragraph 5.8). These are the lowest categories which represent the national 'background' level of risk. Surface water flood risk on the adjacent part of Agamemnon Road is also shown as 'Very Low' (increasing downslope to 'Medium' risk though that is irrelevant for the proposed basement).

10.8.6 Maintenance of existing flood resistance measures and implementation of the following new measures would enable the current 'Very Low' risk rating to be retained:

- Provision of an upstand to the retaining wall which will form the front lightwell in order to prevent surface water draining into the lightwell. The height of this upstand could be nominal (say 50mm) on the east and south sides of the lightwell where the ground slopes away from the lightwell, but should be at least 100mm high on the north side of the lightwell.

- Provision of positive drainage from the front lightwell; as the ground level in the lightwell will be 750mm below the window sill level that space could be used for temporary interception storage of surface water (see also 10.8.7 and 10.8.11 below).
- Surface water flooding to the basement could also occur, in an extreme rainfall event, from the rear garden via the sliding patio doors. Unless those doors can be made watertight, measures will be required to ensure that surface water cannot collect in the rear garden during even an extreme rainfall event. Those measures could comprise additional temporary interception storage of surface water.

Change to Hard Surfacing & Surface Water Run-off:

10.8.7 The front garden to No.46 is described in paragraph 2.5, and shown in Photos 2, 3 & 5 in Appendix A. The proposed front lightwell will be in an area which is currently fully paved, so there will be no increase in paved surface area. Currently, surface water run-off from the area of the proposed lightwell drains either out of the front access gate onto the public footway (and thence to highway main drainage) or to mains drainage via the gully in the recess alongside the gas and electricity boxes in the front wall, or to the flower bed for the remaining part of the front hedge. Thus, construction of the proposed lightwell could create a tiny increase in the volume of water discharged from the property's drainage system to the adopted sewer. If mitigation for this tiny potential increase in discharge is required, then the front lightwell could be used for temporary interception storage and consideration could be given to controlling the rate of discharge from that storage; both represent simple types of Sustainable Drainage Systems (SuDS).

Sewer Flooding:

10.8.8 The Camden SFRA noted that Thames Water's DG5 Flood Register had only one record of flooding from public sewers affecting this post code area ('NW6 1', 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. The probability of future sewer flooding affecting No.46 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 and lower ground floors. During major rainfall events, it is possible for some sewers to overflow at ground level, although this is rare.

10.8.10 Camden's CPG Basements requires all basements to be "*protected from sewer flooding by the installation of a positive pumped device*" (paragraph 6.16 in CPG, 2018). Non-return valves and pumped loop systems must therefore be fitted on the drains serving the basement and the lightwell, in order to ensure that water from the mains sewer system cannot enter the basement when 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, roof water, and surface water from the lightwell and rear garden (as relevant). 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 outfall(s) (including surface water from the various roofs, rear garden and lightwell, and foul water), for the duration of the predicted surcharged flows in the sewer. The front lightwell could be used for interception storage, deepened as necessary to provide adequate capacity, though it must be protected from backup of foul sewage, for which separate storage might be required. This temporary interception storage would require formal design to ensure satisfactory performance.

10.9 Mitigation

10.9.1 The following mitigation measures should be implemented:

- Cracks and past repairs which have weakened the structural integrity of load-bearing walls in the vicinity of the works should be fully repaired, in accordance with recommendations from the appointed Structural Engineers, before any underpinning is carried out (10.4.4).
- Subject to Party Wall Agreement negotiations, transitional underpins should be considered beneath the adjoining load-bearing walls to No's 44 & 48, where the differential founding depth exceeds 1.0m (10.4.15).
- Provision of an upstand at the top of the retaining wall around the front lightwell (10.8.6).
- If mitigation is required for the potential tiny increase in surface water discharge to the mains drainage system from the property, then temporary intervention storage of surface water in the front lightwell would provide a simple type of SuDS (10.8.7).
- Non-return valves and pumped above-ground loop systems should be fitted to the drains serving the basement, lightwell and rear garden in order to ensure that water from the sewer system cannot enter the basement when the mains sewer is operating under surcharge (10.8.10 & 10.8.11).

11. 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 is considered acceptable in relation to the likely limited or nil flow of groundwater through the essentially clayey Made Ground, the possible Head deposits and the weathered London Clay. There are no basements close enough to create any cumulative effect (10.2.1 to 10.2.6). In the unlikely event that the excavations encounter a local deposit of more permeable soils which has remained undetected, then it is possible that an engineered groundwater bypass might be required (10.2.7).

11.4 A provisional design groundwater level equal to ground level is recommended, which means that the basement must be able to resist buoyant uplift pressures (un-factored) which vary across the basement up to 31kPa (10.2.8, 10.2.9). The basement will need to be fully waterproofed (10.2.10, 10.2.11).

11.5 Water entries into the basement excavations are likely to be manageable by sump pumping (10.3.1). The clays onto which the underpins and the basement slab will bear must be blinded with concrete immediately following excavation and inspection (10.3.3).

11.6 There are no concerns regarding slope stability (10.4.1).

11.7 The basement will be constructed using underpinning techniques and RC retaining walls in panels of limited width. 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).

11.8 Various other guidance is provided in relation to the geotechnical design of the basement's perimeter walls (10.4.10, 10.4.11).

11.9 A net bearing pressure of 100kPa may be used for the underpins and RC retaining walls (10.4.12, 10.4.13).

11.10 Good practice requires stepping up between the footings at different depths beneath a single structure, so consideration should be given, during the Party Wall Act negotiations, to the inclusion of transition underpins beneath all load bearing walls in No's 44 & 48 that adjoin No.46, where the difference in founding level will exceed 1.0m. No.44's cellar should act as a transition, though the cause of the differential settlement across that cellar (evidenced externally at the front) should be investigated during detailed design and Party Wall Act negotiations (10.4.14).

11.11 The basement slab must be designed to accommodate swelling displacements/ pressures generated by heave of the underlying clays. A preliminary heave/settlement assessment has been undertaken (using PDISP software) which predicted between 7mm of settlement and 2.5mm of heave beneath the underpins and RC retaining walls, and up to 5mm of heave below the basement slab. However, only the preliminary predicted 5mm of post-construction incremental displacement is relevant to the design of the basement slab (Section 10.5).

11.12 Damage category assessments indicated that, provided best practice construction methods are employed, the worst case predicted deformation affecting No.44 (& No.42) is likely to fall within Burland Category 1, termed 'very slight', while only 'negligible', Burland Category 0, potential damage was predicted for No.48 (Section 10.6).

11.13 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).

11.14 The Environment Agency's maps show that the site is at negligible risk of flooding from rivers or the sea, and at no risk of flooding from reservoirs (10.8.1).

11.15 Agamemnon Road is recorded as having flooded during the 2002 event, but not in 1975; that flooding was almost certainly remote from No.46, in the downslope southern part of the road (10.8.3). The Camden SFRA shows that Agamemnon Road is just within Critical Drainage Area 'Group3_010'; however, it was not in any of the Local Flood Risk Zones and other evidence presented herein indicates that No.46 is **not** in a flood-prone area (10.8.4).

11.16 The Environment Agency's recent modelling of risk of flooding from surface water predict a Very Low flood risk in the sites of No's 40-48, while modelling by Ambiental Risk Analytics gave a 'negligible' risk of surface water flooding (10.8.5). Recommendations are given for mitigation measures to increase the property's resistance to surface water flooding (10.8.6).

11.17 The basement will not result in any increase in paved surface area, though part of the area for the front lightwell currently drains to a flower bed. If mitigation is required of the tiny potential increase in surface water draining to the sewer system, then SuDS options have been identified (10.8.7).

11.18 Thames Water had have only a single record of flooding from public sewers affecting postcode area 'NW6 1', so the probability of future sewer flooding affecting No.46 is considered to be very low, provided that the sewer system is well maintained and appropriate flood resistance measures are implemented (10.8.8).

11.19 Non-return valves and pumped above-ground loop systems should be fitted to the drains serving the basement and lightwell. 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 (10.8.9 to 10.8.11).

11.20 Mitigation measures which have been recommended in Sections 10.2-10.8, are summarised in Section 10.9.

References

Arup (November 2010) Camden geological, hydrogeological and hydrological study – Guidance for subterranean development. Issue 01. London.

Barton N (1992) The Lost Rivers of London. Historical Publications Ltd, London.

Barton N & Myers S (2016) The Lost Rivers of London. 3rd Edition. Historical Publications Ltd, London.

BS 1377-2 (1990) Methods of test for Soils for civil engineering purposes – Part 2: Classification Tests. British Standards Institution, London.

BS 5930 (2015) Code of practice for ground investigations. British Standards Institution, London.

BS 8002 (2015) Code of Practice for earth retaining structures. British Standards Institution, London.

BS 8102 (2009) Code of practice for protection of below ground structures against water from the ground. British Standards Institution, London.

BS EN 1997-1 (2004) +A1 (2013) Eurocode 7: Geotechnical Design – Part 1: General rules. British Standards Institution.

Ellison RA et al (2004) Geology of London. Special Memoir for 1:50,000 Geological sheets 256 (North London), 257 (Romford), 270 (South London) and 271 (Dartford) (England and Wales). British Geological Survey, Keyworth.

Gaba, A.R., Simpson, B., Powrie, W. and Beadman, D.R. (2003) CIRIA Report C580: 'Embedded retaining walls – guidance for economic design'. CIRIA, London.

Gaba, A.R., et al (2017) CIRIA Report C760: 'Guidance on embedded retaining wall design'. CIRIA, London.

London Borough of Camden (2003) Floods in Camden, Report of the Floods Security Panel.

London Topographical Society (2005) The London County Council Bomb Damage Maps 1939-1945. LTS Publication No.164.

NHBC (2020) NHBC Standards, Chapter 4.2, Building Near Trees.

NHBC (2020) NHBC Standards, Chapter 5.4, Waterproofing of basements and other below ground structures.

URS (2014) London Borough of Camden SFRA – Strategic Flood Risk Assessment. Final Report.