Report

GabrielGeo Consulting

Basement Impact Assessment

190 Goldhurst Terrace, London, NW6 3HN

for

Mr Joshua King

Ref: 16550/R3

April 2018

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Basement Impact Assessment

Site: **190 Goldhurst Terrace, London, NW6 3HN**

Client: **Mr Joshua King**

Report Status: **FINAL**		
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Foreword

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

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1. INTRODUCTION

1.1 This Basement Impact Assessment has been prepared in support of a revised planning application to be submitted to the London Borough of Camden (LBC) for construction of a single-storey basement beneath No.190 Goldhurst Terrace, NW6 3HN. The assessment is in accordance with the requirements of the London Borough of Camden (LBC) Development Policy DP27 in relation to basement construction, and follows the requirements set out in LBC's guidance document CPG4 'Basements and Lightwells' (July 2015).

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

1.3 A preliminary site inspection (walk-over survey) of the house was undertaken on Thursday 10th March 2016. Photos from that visit are presented in Appendix A. Desk study data have been collected from various sources including geological data, environmental data and historic maps from Groundsure which are presented in Appendices E, F and G. Relevant information from the desk study and site inspections is presented in Sections 2–6, followed by the basement impact assessment in accordance with CPG4 Stages 1–4 in Sections 7–10 respectively.

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

- **CLAGUE ARCHITECTS:**
  Drg No. 22447A_11 Existing Plans and Elevations
  Drg No. 22447B_50 Proposed Plans and Elevations
- **OMEGA GEOMATICS (Surveyors):**
  Drg No. 15-0228 MBS_03 Measured Building Survey
  Drg No. 15-0228 ELEV_03 Existing Building Elevations
- **S.C.Green Ltd (Structural Engineers):**
  Drg No. 4013/A1/01F Phase 1
  Drg No. 4013/A1/02 J Phase 2 (with Typical Sections)
  Drg No. 4013/A1/03F Phase 3
  Drg No. 4013/A1/04B Phase 3 – Sections
  Drg No. 4013/A1/05G Phase 4
  Method Statement. Calculations (Job 4013).

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

1.5 Instructions to prepare this revised Basement Impact Assessment (BIA) were received via email on 6th March 2018.

2. THE PROPERTY, TOPOGRAPHIC SETTING AND PLANNING SEARCHES

2.1 No.190 Goldhurst Terrace is a three-storey, terraced house built of brickwork beneath a clay tiled roof, situated within the South Hampstead Conservation Area, in the London Borough of Camden. This part of Goldhurst Terrace extends from Priory Road to the west, up to Fairhazel Gardens to the east, beyond which Goldhurst Terrace extends north-eastwards up to where it joins Finchley Road (the A41). To the north, Aberdare Gardens is orientated broadly parallel to Goldhurst Terrace, and the two roads share a junction just to the west of the site. As shown in Figure 1, No.190 is situated on the north side of Goldhurst Terrace, between the adjoining No.192 to the west, and No.188 to the east (see also Photo 1 in Appendix A). To the north, the site is bounded by the rear gardens of No's 196 & 188 Goldhurst Terrace.

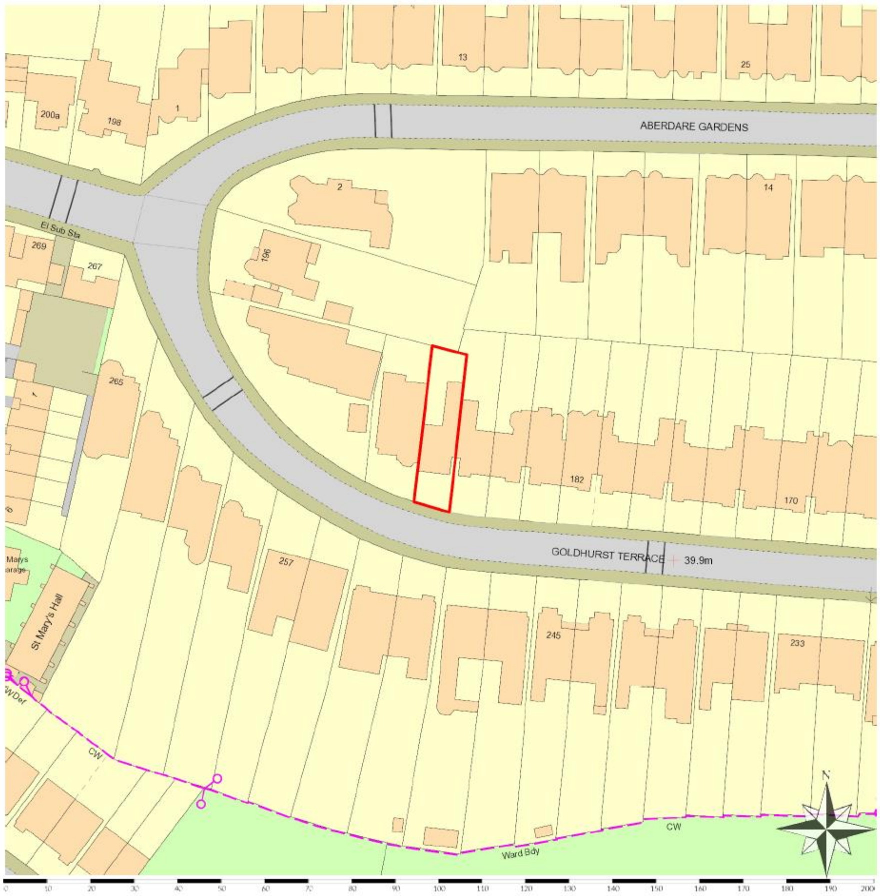

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Figure 1: Extract from 1:1,250 OS map (not to scale) with the site outlined in red.

2.2 Most of the houses in Goldhurst Terrace are of a similar design, although vary from large semi-detached properties, to smaller terraced houses. According to Elrington et al (1989) "*Building began from the east end with 20 houses by Charles Kellond in Goldhurst Terrace, the most southerly of the roads, in 1879 and another 50 there between 1880 and 1885; 101 houses, some flats, and a riding school were added between 1886 and 1900, mostly by T. K. Wells of Kentish Town.*". Like the majority of properties on Goldhurst Terrace, No.190 was subsequently divided into separate flats. Beneath the hallway there is a small cellar, accessed internally via a ladder, which consists of a storage room (formerly a coal store). It should be noted that a shallow depth of standing water was found within the southernmost part of this cellar, although the source of the water was not established.

2.3 Based upon the building's footprint in the 1915 historic Ordnance Survey (OS) map (see Appendix G), No.190, along with all of the terraced houses on the north side of Goldhurst Terrace, had a small original rear projection. Based on observations made during the recent site inspection, it would appear that the rear projection has also been rebuilt/heightened at some point, thought that is not shown on any of the appended OS maps, and an additional single-storey extension/conservatory been added behind this rear projection (see paragraph 2.13, and Photo 6).

2.4 No evidence was seen of major crack damage, though some cracking/displaced brickwork was evident around some of the windows (especially the brickwork lintels to the front bays). The front wall alongside the 190/188 party wall appeared to be in poor condition owing to poorly executed past alterations (and leaking pipes?).

2.5 Reference to the historic OS map dated 1871 in Appendix G shows the plot of No.190 Goldhurst Terrace within a field labelled No.399. The next available OS map, dated 1896, shows that construction of the Goldhurst Terrace and Aberdare Gardens carriageways had been completed prior to this date; however, only two houses had been built at this end of Goldhurst Terrace (No's 196 & 198), along with several other houses at the western end of Aberdare Gardens. This map also shows that prior to its publication, development had begun at the western ends of Greencroft Gardens and Canfield Gardens (further to the north). By the time the 1915 OS map was published, almost all of the properties on Goldhurst Terrace had been completed (including No.190), along with those on Aberdare Gardens and Greencroft Gardens further to the north.

2.6 Generally both the large- and small-scale historic OS maps show few significant changes in the area after this period of major development; however, to the south of No.190, significant redevelopment can be seen on the north side of the junction between Belsize Road and Abbey Road, between publication of the 1955 and 1968 OS maps. This redevelopment includes the demolition of No's 172-192 Belsize Road, and No's 100-122 Abbey Road, and the construction of two large blocks of flats labelled Snowman House and Casterbridge.

2.7 The WW2 bomb map for the Borough of Hampstead shows that a run of three bombs landed immediately to the north-east of No.190, affecting properties on both sides of Aberdare Gardens. This map also shows that three bombs landed on or very close to Goldhurst Terrace further to the east of the site, and that another landed near the junction between Priory Road and Goldhurst Terrace to the north-west of the site. There are no 'gaps' in bomb lines which might suggest that an unexploded bomb fell in No.190's garden, though this does not provide conclusive proof that the site is clear of unexploded ordnance.

2.8 The London County Council Bomb Damage Map for this area (London Topographical Society, 2005) shows that No.190 did not suffer any damage. To the north–east of the site however, No's 6 & 8 Aberdare Gardens are recorded as having suffered "General blast damage – not structural", and on the opposite (north) side of Aberdare Gardens, No's 17 & 19 are recorded as "*Seriously damaged, but repairable at cost*". This pattern of damage appears consistent with what is shown on the WW2 bomb map for the Borough of Hampstead.

2.9 Externally, at the front of the property, there is a large paved parking area (Photo 3) which slopes gently towards the Goldhurst Terrace carriageway, and is bounded by low brick walls, except at its access point with the Goldhurst Terrace footway, and along part of the 190/188 Goldhurst Terrace boundary, where there are metal railings, and a metal gate at the vehicular access. Between this paved parking area and the front bay is a small planting area, which also extends alongside the 190/192 boundary. Along the 190/188 boundary, to the front of the house, there is an established hedge growing in a raised planting area bounded by low brickwork walls.

2.10 Much of the rear garden to No.190 Goldhurst Terrace is laid to lawn, with patio areas both alongside the two-storey rear projection (with direct access from the rear bay) and behind the single-storey extension. Wooden panel fencing was present along the presumed boundaries to the rear garden, and a perimeter raised planting area (which includes several small trees and shrubs) was present along part of the garden's western boundary with No.192 Goldhurst Terrace, as well as the rear and eastern boundaries with No's 196 and 188 Goldhurst Terrace. The raised planting area was only absent alongside the patio area in front of the rear bay.

Topography:

2.11 The topographic setting of the western part of Goldhurst Terrace is the north-western side of a weakly developed SSW-facing valley, as illustrated by the contours in Figure 2. This slope rises to West End Lane, which follows a prominent southwards-falling topographic ridge to the north-west of this area (see Figure 2). This valley was created by a former tributary to the River Westbourne, one of the 'lost' rivers of London (see paragraph 5.1). Between the 40m contour, which passes just to the south of the site, and the 45m contour which passes further upslope to the north-west of the site, the overall slope angle has been calculated as around 1.0°; this overall slope angle reduces to less than 1° downslope of the 40m contour.

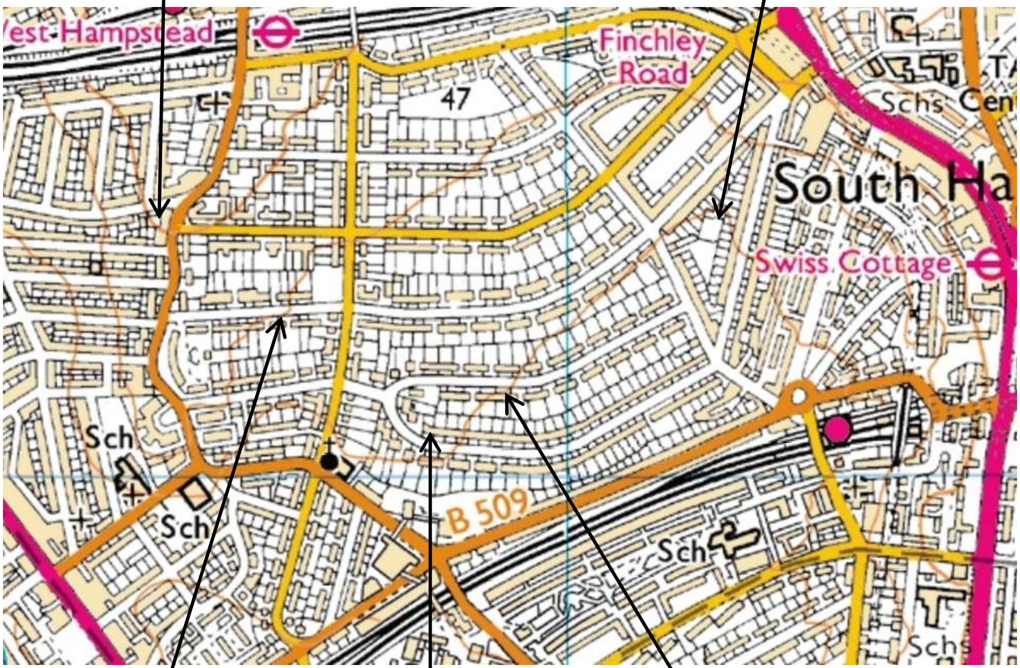

Figure 2: Enlarged extract from 1:25,000 Ordnance Survey map showing site location.

2.12 Based on observations made during the recent site inspection, and spot heights on the appended historic OS maps, the Goldhurst Terrace carriageway falls gently eastwards within the vicinity of No.190, towards a low point located between the curve in Goldhurst Terrace, and the junction of Goldhurst Terrace with Fairhazel Gardens; this 'low point' in the carriageway is about 300m from No.190. To the east of this junction, Goldhurst Terrace falls southwards (from Finchley Road) at gradients which ease from approximately 3.0° at its north-eastern end, to less than 1° near the junction with Fairhazel Gardens (calculated using contour intervals and spot heights). On the east side of this junction, the speed hump which has been constructed just upslope of the junction obstructs the full width of both carriageways, so restricts surface water run-off from ('upper') Goldhurst Terrace into (and down) Fairhazel Gardens. However, double gullies have been provided alongside the speed hump in order to increase the drainage capacity at this artificial restriction to run-off. A speed hump has also been constructed on the west side of the same junction, although no drainage gullies have been provided alongside the speed hump, because run-off within this part of Goldhurst Terrace will most likely flow westwards away from the junction, towards the low point.

Planning Searches:

2.13 A search was made of planning applications on the Camden Council's website, in order to obtain details of any other basements which have been constructed or are planned in the vicinity of the property, the results of which are listed below. This search also found an application for the single-storey extension to the rear of No.190.

- **No.190 Goldhurst Terrace:** Application (8700456) involving the "*Enlargement of the existing single-storey rear addition as shown on drawings No.8711.01-04 inclusive*" was granted planning permission on 29th April 1987. Drawings of the proposed scheme on Camden's website showed that the two-storey layout already matched the current layout.
- **No.186 Goldhurst Terrace:** Application (2016/1112/P) involving the "*Erection of single storey part-replacement rear extension and lowering of internal lower ground floor level*" was granted planning consent on 22nd April 2016. Drawings of the proposed scheme on Camden's website showed that the "*internal lower ground floor*" comprised only the original cellar.
- **No.192 Goldhurst Terrace:** Application (2016/1504/P) involving "*Erection of a first floor rear extension over existing single storey rear extension*" was granted planning permission on 6th June 2016. Drawings of the proposed scheme were found on the website.
- **No.255 Goldhurst Terrace:** Application (2011/5554/P) involving "*Excavation of basement* (below rear part of house only) *and rear lightwell with balcony over at rear ground floor level and steps to garden, erection of extension at rear ground floor following removal of conservatory including raising of boundary wall and alterations to doors/windows at rear ground level all in connection with existing flat (Class C3)*" was granted planning permission on 22nd December 2011. Drawings of the proposed scheme were found on the website.
- **No.253 Goldhurst Terrace:** Application (2012/2911/P) involving "*Excavation at basement level for the provision of an enlarged extension [between basement and ground floor level throughout the footprint of existing building], installation of new balustrade and door to create raised roof terrace to the rear ground floor elevation and new obscured window to the side elevation in connection with the use as residential flats (Class C3)*" was granted planning permission on 27th July 2012. Drawings of the proposed scheme found on Camden's website show that the section in [ ] above is somewhat misleading, as the proposed basement extension was at the level of the existing basement and was only below the rear part of the building.
- **No.247 Goldhurst Terrace:** Application (2004/4834/P) involving "*Works of excavation in connection with the construction of an outdoor swimming pool and associated decking in rear garden*" was granted planning permission on 3rd June 2005. Drawings of the proposed scheme were found on the website.

2.14 Ground investigation records were also obtained from planning applications concerning No.27 Aberdare Gardens, No.65 Aberdare Gardens and No.17A Fairhazel Gardens (see Section 4).

3. PROPOSED BASEMENT

3.1 The proposed works at No.190 Goldhurst Terrace, for which planning permission will be sought, and as shown in Clague Architects' drawings (see paragraph 1.4), will comprise:

- Creation of a self-contained, single-storey basement beneath almost the full footprint of the house, including the rear projection, but excluding the single-storey rear extension/conservatory (see paragraph 2.13), with a proposed finished floor level (FFL) at 37.925m AOD, 2.775m below that of the ground floor (40.70m AOD).
- Open lightwells at basement level, alongside both the existing front and rear bays on the west side of the house. The proposed rear lightwell is shown on Clague's 'Proposed Plans and Elevations' (Drg No. 22447A_50) as projecting approximately 6.9m beyond the rear wall of the house, encompassing the entire existing rear patio area. The proposed front lightwell is shown as projecting approximately 2.65m from the front wall of the house, and extends across the full width of the front bay and almost the full width of the front entrance Hall to No.190, with a bridge over the lightwell providing access at ground floor level. Access to the basement will be externally, via a flight of stairs within the front lightwell, with a fixed ladder within the rear lightwell forming emergency access/egress (shown on CAD model by Clague).

3.2 The structural drawings by S.C.Green Ltd show that the scheme will comprise reinforced concrete (RC) underpinning beneath both of the party walls and the rear wall of the rear projection. The front and rear walls and bays, and the flank wall of the rear projection, will all be supported off new pads, strip footings or the new basement slab as shown on the drawings by SC Green Ltd. The main internal walls of the house will be carried by steel beams and columns resting on concrete pad footings. Two separate contiguous bored pile retaining walls are proposed for the lightwells at the front and rear of the house. Transitional underpins will be installed beneath the adjoining walls of the single-storey rear extension, and possibly also beneath some of the adjoining load-bearing walls of the neighbouring properties (see Drg No. 4013/A1/05G and paragraph 10.4.16 for further details).

3.3 S.C.Green's typical basement underpinning section in 'Phase 2' (Drg No. 4013/A1/02J) shows a thickness of 0.15m for the basement slab, which, when combined with 100mm of insulation, and 75mm of floor screed, gives a founding level (formation) of the proposed central basement slab of around 3.1m below the existing ground floor level (37.60m AOD). For the underpin bases, a thickness of 0.3m is given (also shown in S.C.Green's 'Phase 2'), therefore the founding level of the underpin bases will be around 3.25m below existing ground floor level (37.45m AOD). The pad footings will be either 0.6m thick (P1, P2 & F2) or 0.75m thick (F3 & F4), so would be founded at 37.15m and 37.0m AoD.

3.4 The depths of excavation required are expected to vary, from around 1.08m within the existing cellar, to around 3.05m for the underpinning within the footprint of the main part of the house and 3.29m for the underpins beneath the rear projection (see Table 1). The existing ground floor within No.190 Goldhurst Terrace is known to be suspended, however the exact depth of the void beneath the floor is unknown. Based on the ground level outside of the property, the ground level beneath the suspended floor has been assumed to be around 40.50m AOD (approximately 0.2m below the FFL of the existing ground floor). Details of the existing and proposed ground levels across the site are presented in Table 1 below.

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

Location	**Existing GL/FFL** (m AOD)	**Prop'd GL/FFL** (m AOD)	**Formation level – footings/U-pins** (m AOD)	**Excavation depths - U/pins** (m)	**Excavation depths – Slabs** (m)
Basement within footprint of main part of house (excl. existing cellar)	40.50 (beneath assumed suspended floor)	37.925	37.45	3.05	2.90
Basement/ existing cellar	38.70			1.25	1.10
Basement beneath 2-storey rear projection:	40.70			3.25	3.10
Pads F3, F4, F5 & central pier:	40.50	N/A	37.00	3.50	N/A
Rear Lightwell	40.46 to 40.54	37.925	Slab: 37.60	Pile length tbc	2.86 to 2.94
Front lightwell	40.45 to 40.60	37.925	Slab: 37.60	Pile length tbc	2.85 to 3.00

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 geology of the West Hampstead area.

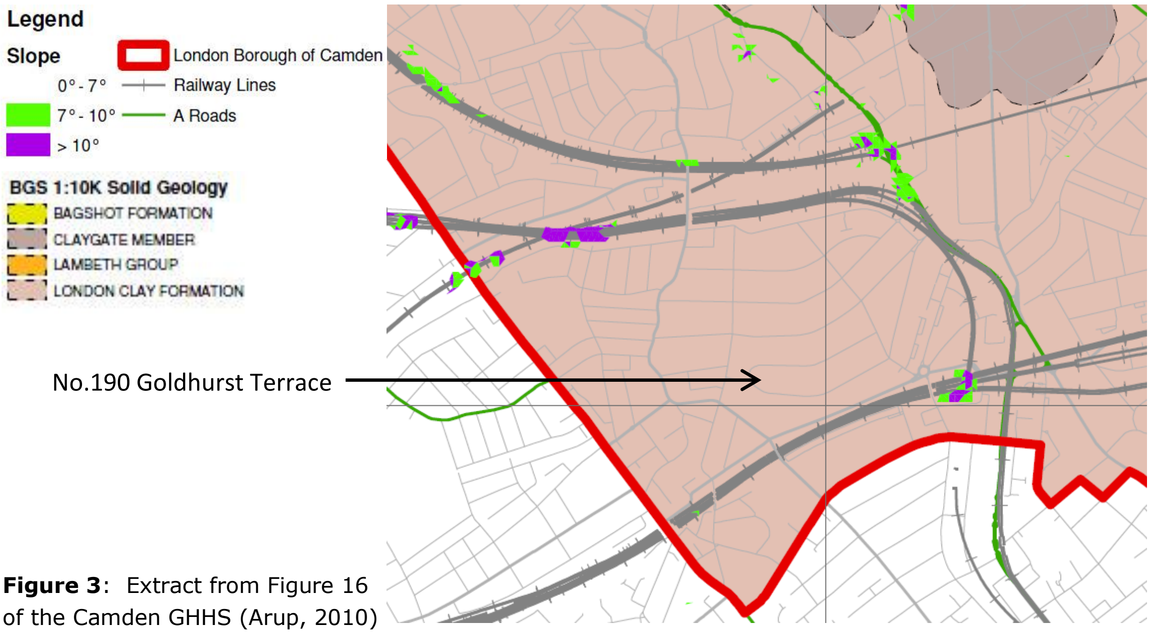

Figure 3: Extract from Figure 16 of the Camden GHHS (Arup, 2010)

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 1934 geological map (London IV.NE at 1:10,560 scale) records "London Clay in cuttings" to the south-east of the site, near Belsize Road, and "London Clay formerly dug" to the north-east of No.190, near the junction between Fairhazel Gardens and Greencroft Gardens. The map also appears to record "London Clay formerly dug, 6-10 FT deep", however part of the label is obscured. It should be noted that the lateral extent of these workings is not indicated.

4.4 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.

4.5 The results of the BGS natural ground subsidence hazard classifications are provided in the Groundsure GeoInsight report (Appendix E, Section 4); all except "shrink-swell clays" indicated "Negligible hazard" to "Very low hazard". The shrink-swell clay hazard is classified as "Moderate", which reflects the presence of the London Clay Formation.

4.6 The Groundsure GeoInsight report (Appendix E, Section 3) records the presence of a number of historic 'mining' features within 1000m of the site, the closest of which is an 'Air shaft' located 660m to the east. The tunnel at the same location is recorded in the historic underground workings section of the GeoInsight report (Appendix E, Section 2). Both the air shafts and tunnels appeared to be associated with London Underground's Metropolitan line. No historical surface ground working features were identified by Groundsure within 250m of the site. These databases are based on mapping evidence, so inevitably will provide an incomplete record of underground workings.

4.7 A search of the BGS borehole database was undertaken for information on previous ground investigations and any wells in the vicinity of the site. As shown on the location plan in Appendix B, few BGS boreholes were available within close proximity to the site. During the planning search however (see paragraph 2.13), a few boreholes were found closer to the site; thus the strata depths for a selection of these boreholes gleaned from the planning search are presented in Table 2 below. For full strata descriptions, reference should be made to the logs in Appendix B.

4.8 At No.65 Aberdare Gardens, to the south of Compayne Gardens, the two boreholes in the front garden recorded 1.8m of firm, moist, orange-brown, very silty CLAY with occasional gravel at 1.4-3.2m below ground level, beneath the Made Ground. These clays were attributed to a locally-derived Head deposit, whereas the boreholes drilled in the rear garden recorded only clays consistent with weathered London Clay Formation. Similar variability in the thickness of the Head deposits was also recorded at 17A Fairhazel Gardens (to the north-east of No.190, between Canfield Gardens and Aberdare Gardens), where logging of initial excavations for a basement recorded Head deposits to depths of 0.8-1.5m (Ashton Bennett Consultancy, 2012).

4.9 Groundwater readings from the boreholes at 65 Aberdare Gardens, both during drilling and from subsequent monitoring visits in 2013, are presented in Table 3.

Table 2: Summary of Strata in BGS Boreholes					
Strata (abbreviated descriptions)	**Depths (m) and levels (m AOD) to base of strata**				
	TQ28SE/ 445, 448 & 450		**TQ28SE/ 2062 & 2063**	**27Aberdare Gardens**	**No.65 Aberdare Gardens**
GL	Depth	**Level 40.78/ 41.46/35.81**	Depth	Depth	Depth
Made Ground and/or Topsoil	0.45/0.45/ 0.30	**40.33/41.01/ 35.51**	1.00	0.3-0.8	0.4-1.4
Firm, moist, orange-brown, veined grey very silty CLAY, with occasional gravel. (Head Deposit)	-	-	-	-	- / 3.2
Firm bec'g stiff, fiss'd, brown silty CLAY (Weathered London Clay Formation)	8.40/10.2/ 10.05	**32.38/31.26/ 25.76**	7.80/6.50 BH2: soft to c.3.0m	>6.0	> 6.0
Stiff, fissured, blue/blue-grey silty CLAY (London Clay Form'n)	>15.25	**<25.53/26.21/ 20.56**	>10.00		
Seepage/Strike	Not encountered while boring		dry	None - 3.3	See Table 3
Groundwater standing level	-	**-**	dry	1.09-0.82	See Table 3

Table 3: Groundwater records - 65 Aberdare Gardens				
	BH 1	**BH 2**	**BH 3**	**Original BH 1**
Date drilled	26/04/2013	26/04/2013	26/04/2013	22/11/2012
	(m bgl)	(m bgl)	(m bgl)	(m bgl)
Water seepage	2.3	None	3.8 (slight)	1.4
Water seepage	3.2	-	-	3.2
Water standing level on completion of boring	Moist at base	Dry	Moist at base	Moist at base
Base of standpipe	5.0	3.5	5.0	
Groundwater levels in standpipes				
Following inst'n on 26/04/2013	Dry	Dry	Dry	
27/04/2013	0.86	0.39	Dry	
29/04/2013	0.52	0.39	Dry	
30/04/2013	0.52	0.39	Dry	

5. HYDROLOGICAL SETTING (SURFACE WATER)

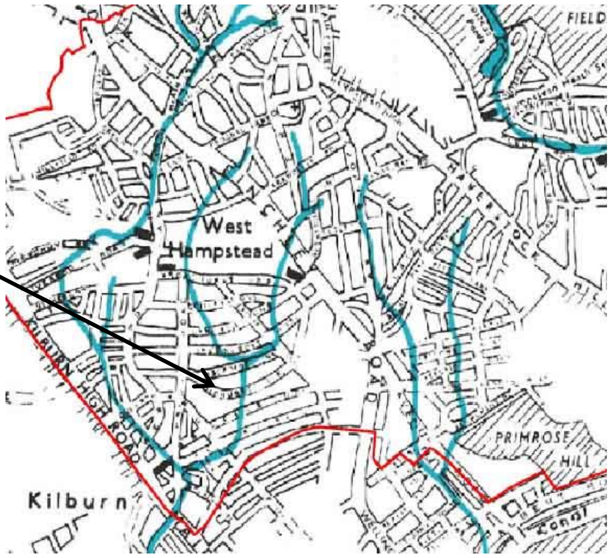

Figure 4: Extract from Figure 11 of the Camden GHHS (Arup, 2010) showing former watercourses, based on Barton (1992).

5.1 Barton's map of the 'lost' rivers of London (Figure 4) indicates that this part of Goldhurst Terrace is situated just to the south-west of the confluence between two branches of one of the former tributaries to the Westbourne. However, for the reasons set out in 5.2 below, the correct location of this confluence is now considered to lie further to the south, between Goldhurst Terrace and Belsize Road.

5.2 The 1871 1:2,500 and 1874 1:10,560 maps show a stream/ditch along the (irregular) northern boundaries of the properties on the north side of Belsize Road, to the south of Goldhurst Terrace, and apparently terminating about 80m due south of No.190. That was probably the south-eastern end of a culvert carrying the western of these two branches of the Westbourne from Priory Road past St Mary's Church (see 5.4 below). These maps also show a ditch to the east of the site, which by comparison with one of the more modern maps, can be shown to pass beneath No.104 Goldhurst Terrace and No's 71 & 60/58 Aberdare Gardens, on an alignment close to north-south. This ditch was probably the realigned Westbourne tributary to the east of No.190, part of which can be seen slightly further to the east of that ditch on the 1874 map. The 1934 geological map shows the same stream linked to the ditch to the east of No.190, and shows the western branch following the east side of Priory Road (and probably already in culvert). This layout of the tributary seems more plausible than that given by Barton.

5.3 By 1896 the OS map shows that all these streams/ditches to the south and east of the site had disappeared and that much of the surrounding area had been developed. Thus, the Westbourne had been fully culverted in this area by that date or had been diverted into the new sewer system.

5.4 A map from SLR which illustrates the hydrology of the area was found on Camden Council's website, related to a planning application for the construction of a basement beneath No.85 Greencroft Gardens (attached to letter dated 11th March 2014). This largely confirms what is shown on the 1871 OS map.

5.5 In order to find out more about the possible alignment of the suspected culverts, enquiries were made to Thames Water, then the Environment Agency, and finally to LBC's Asset Management and Highways teams. None of these organisations had any record of any culverts in the vicinity of Goldhurst Terrace. The consensus opinion was therefore that either the stream/ditch was diverted into the mains sewers beneath the road network, or it might still be in an old culvert which no organisation is now maintaining.

5.6 The natural surface water catchment upslope of No.190 was large, but has been substantially altered by development and now comprises:

i. Under all normal circumstances, the catchment is expected to be restricted to the site itself, as well as parts of the adjoining rear gardens to No's 192 & 196 Goldhurst Terrace, where they are only separated by a wooden fence. Minimal or no run-off is expected from the adjoining rear garden to No.188, which with the general fall of the slope appeared to be very slightly lower than No.190's garden. All other surface water run-off upslope of No.190 is expected to be intercepted by the road network and discharged into the mains sewers.

ii. The front parking area to No.190 is mostly bounded by low brick walls, except along the section of the 190/188 boundary in the recess between the front part of these two houses, where there are metal railings; as the forecourt parking area slopes gently away from that area, towards the carriageway, and No.188's forecourt is slightly lower than No.190's, the surface water catchment area for No.190's front parking will be limited to direct rainfall (under normal circumstances).

iii. In the rear garden, infiltration is feasible within the lawn and perimeter planting area, when the ground is not saturated or frozen. In the paved patio areas infiltration will be minimal or nil, and there is a (relatively recent) channel drain around both the rear extension and rear projection in order to collect surface water and discharge it to the mains drainage system. At the front of the property, minimal infiltration is expected within the forecourt parking area, because of the extensive paving and limited areas of soft landscaping alongside the front bay, and parts of the boundaries with the adjoining properties. Overall however, infiltration will be limited on-site, because of the presence of the London Clay Formation at shallow depths.

5.7 Figure 5 shows that Goldhurst Terrace was subject to surface water flooding in both the 1975 and 2002 flood events. The implications of those historical events are addressed in Section 10.6.

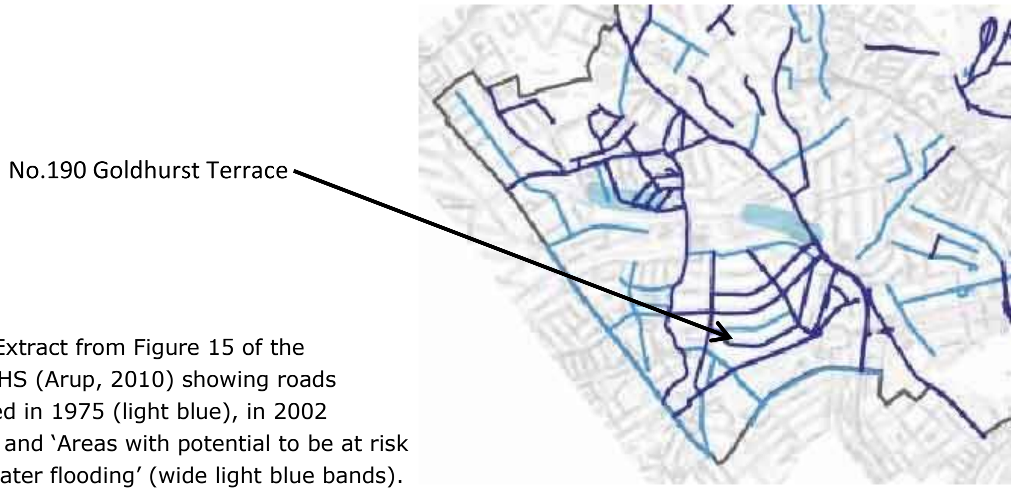

Figure 5: Extract from Figure 15 of the Camden GHHS (Arup, 2010) showing roads which flooded in 1975 (light blue), in 2002 (dark blue), and 'Areas with potential to be at risk of surface water flooding' (wide light blue bands).

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

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

- There are no surface water features within 250m of the site and no 'Detailed River Network' entries within 500m (App.F, Sections 6.10 & 6.11).
- There are no surface water abstraction licences within 2000m of the site (App.F, Section 6.4).
- There are no flood defences, no areas benefitting from flood defences, and no flood storage areas within 250m of the site (App.F, Sections 7.4, 7.5 & 7.6).

5.10 The Environment Agency's (EA) new map of 'Flood Risk from Surface Water' is available on the Government's 'Long Term Flood Risk Information' website, an extract from which is presented in Figure 6. This map identifies four levels of risk (high, medium, low and very low), and it appears to be based primarily on topographic levels, flood depths and flow paths. This modelling shows a 'Very Low' risk of flooding for the site of No.190 Goldhurst Terrace and the immediately surrounding area, which is the lowest, national background level of risk. The 'Low' to 'High' risk of flooding shown close to Belsize Road, to the south-west and south-east of No.190, closely follows the former alignment of the Westbourne tributaries. Towards the eastern end of this section of Goldhurst Terrace (west of Fairhazel Gardens), areas at a 'Low' risk of flooding from surface water are shown on the north side of Goldhurst Terrace, as well as within the Goldhurst Terrace carriageway, which coincides with the topographic 'low point' for the road. The areas at a 'Low' to 'High' risk of flooding from surface water shown to the east of the junction between Goldhurst Terrace and Fairhazel Gardens are associated with the reduction of gradient on the carriageways and the presence of traffic-calming ramps at the junction (for further detail, refer to 2.12 above).

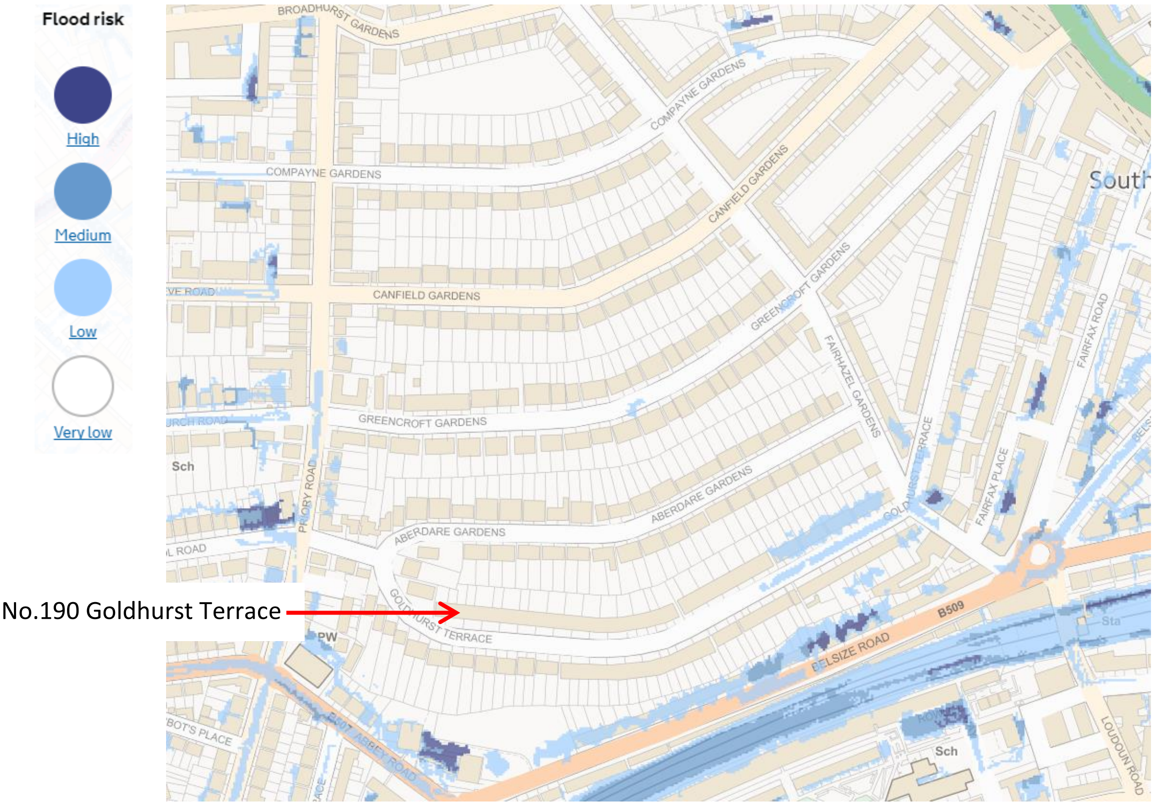

Figure 6: Extract from the Environment Agency's map of 'Flood risk from Surface Water'.
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5.11 More recently, surface water flood modeling has been undertaken by URS as part of a Strategic Flood Risk Assessment for the London Borough of Camden, which was published in July 2014; an extract from their model is presented in Figure 7. As per the Environment Agency modeling, this map identifies the same four levels of risk (high, medium, low and very low), and shows a 'Very Low' risk of flooding for the site of No.190, and the surrounding area. Areas of flooding, similar to those on the EA's model, were shown along the Westbourne tributaries. Although this map also shows the same area at a 'High' risk of flooding from surface water adjacent to the junction between Goldhurst Terrace and Fairhazel Gardens, it is not possible to say whether the same areas at a 'Low' risk of flooding from surface water, as shown by the EA's model, are present in/near to the topographic low point for this road because of the broad green marking/band along the road (which indicates that two of the properties on Goldhurst Terrace have been affected by historic surface water flooding).

5.12 Figure 7 also shows that Goldhurst Terrace falls within Critical Drainage Area Group3_010, while Figure 6 in the Camden SFRA also shows that the site lies within the 'Goldhurst' Local Flood Risk Zone (LFRZ).

5.13 The implications from these flood models are discussed in Section 10.8.

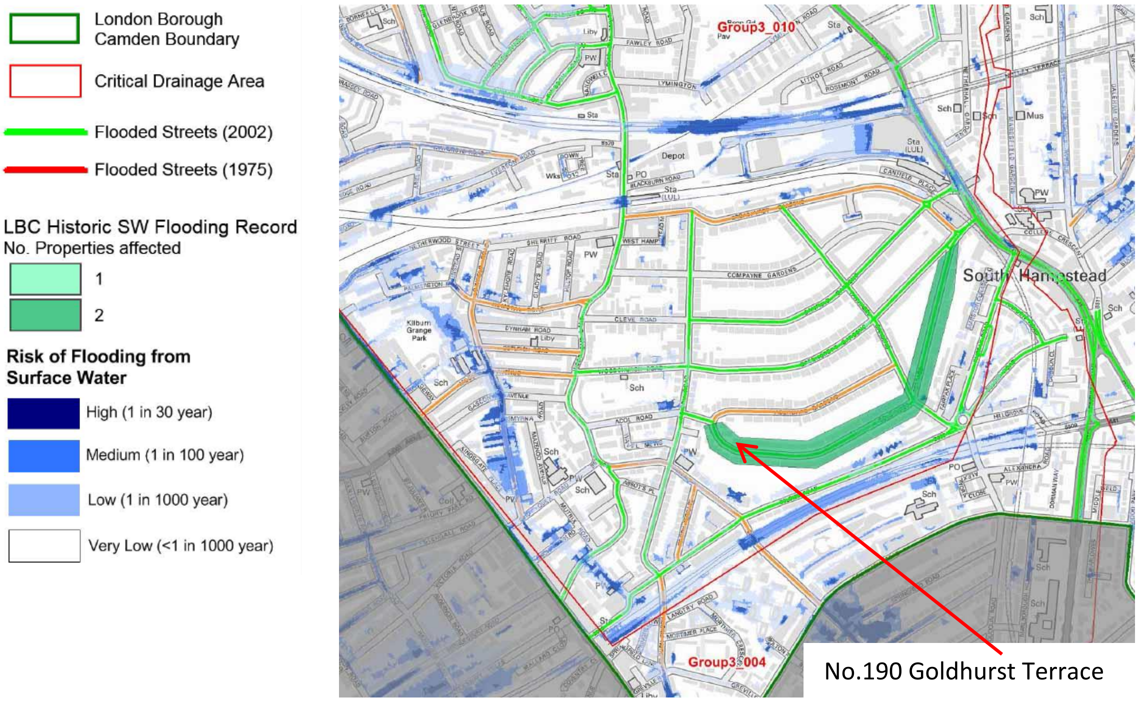

Figure 7: Extract from Figure 3v of the Camden Strategic Flood Risk Assessment (SFRA) (URS, July 2014) showing risk of flooding from surface water.

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5.14 A 'Sewer Flooding History Enquiry' report has been obtained from Thames Water Utilities Ltd (TWU). In response to the question 'Is the requested address or area at risk of flooding due to overloaded public sewers?' (TWU's wording) the response given was: "*The flooding records held by Thames Water indicate that there have been no incidents of flooding in the requested area as a result of surcharging public sewers*". A copy of the report is available on request.

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. Under the old groundwater vulnerability classification scheme, which now applies only to superficial soils, the area is unclassified.

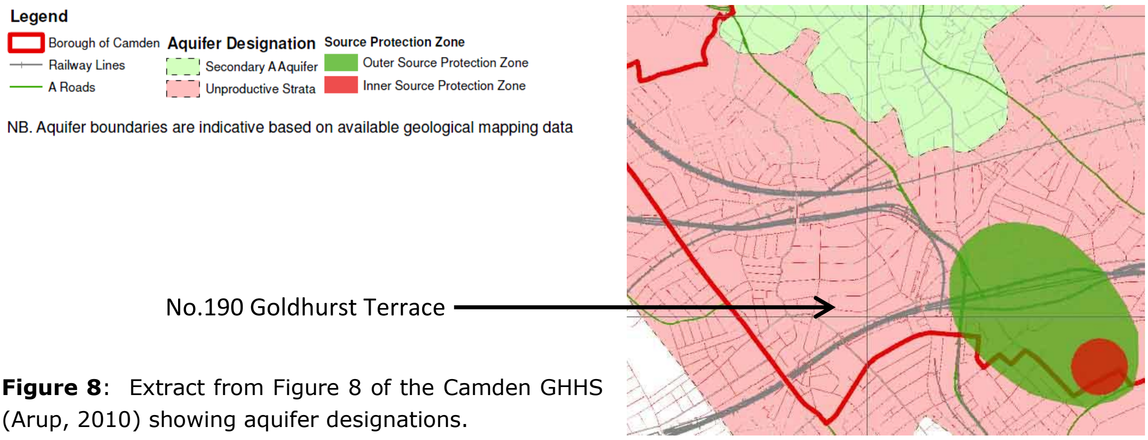

Figure 8: Extract from Figure 8 of the Camden GHHS (Arup, 2010) showing aquifer designations.

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

6.3 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.

6.4 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.

6.5 Details of what was found by the site-specific ground investigation in January 2016 are presented in Section 9.

6.6 The groundwater catchment areas upslope of No.190 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, as well as infiltration within No.190's own rear garden (and to a lesser extent the front planting area), 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 EnviroInsight report (Appendix F) include:

- The nearest Source Protection Zone (SPZ) is a Zone 2, 'Outer catchment' located approximately 730m to the east of the site (Figure 8 above, and App.F, Sections 6.6 & 6.7); this is irrelevant to the proposed basement.
- The nearest groundwater abstraction licence is 959m to the east of the site at the Swiss Cottage Open Space Borehole (TQ28SE/1769) (App.F, Section 6.3) with a maximum permitted abstraction of 28.8 m^3/day. This borehole is 159m deep with 6" steel casing grouted into the London Clay and abstracts water from the Chalk below -56mOD, so it will have no effect on the proposed basement.
- The closest abstraction licence for potable water is 1867m to the east of the site at Barrow Hill Pumping Station (App.F, Section 6.5), so will also be irrelevant to the proposed basement.
- The BGS has classified the area within 50m of the site as 'Not Prone' to groundwater flooding, based on the presence of London Clay to surface (App.F, Section 7.7).

7. STAGE 1 - SCREENING

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

7.2 Subterranean (groundwater) flow screening flowchart:

Question		Response, with justification of 'No' answers	Clauses where considered further
1a	Is the site located directly above an aquifer?	No – Site underlain by London Clay	4.1
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 any perched groundwater in the Made Ground/Head Deposits overlying the London Clay.	8.2, 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. The former Westbourne tributary (possibly culverted on line) passes about 80m to the south of the site.	5.1 to 5.5, & 5.9
3	Is the site within the catchment of the pond chains on Hampstead Heath?	No – Site is approx 2.2km SW of Hampstead No.1 Pond	
4	Will the proposed basement development result in a change in the proportion of hard surfaced/ paved areas?	Yes – Proposed front lightwell is likely to increase hard surfaced area.	Carried forward to Scoping: 2.8 & Section 3.
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; SuDS may be used, but volume of water discharged to ground will not increase.	
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 around 1.1km to NE (at London Clay-Claygate Member interface).	5.9 & 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 clays. 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 overall slope across the site is approximately 1.0°	2.10
2	Will the proposed re-profiling of landscaping at site change slopes at the property boundary to more than 7°?	No – No significant re-profiling is proposed.	
3	Does the development neighbour land, including railway cuttings and the like, with a slope greater than 7°?	No – Max. overall slope angle in the vicinity of the property is 1°; (and there are no railway cuttings in the vicinity of the site).	2.10
4	Is the site in a wider hillside setting in which the general slope is greater than 7°?	No – As Q3 above.	2.10
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, 8.3, Sections 9, 10.4 & 10.5
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 – Proposed basement does not extend beneath canopy of large (Ash?) tree within adjoining rear garden to No.196 Goldhurst Terrace.	(Photo 7)
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 the general area, though these houses appear to have suffered less than others in the area, with only minor cracking observed around some of the windows to No.190.	Carried forward to Scoping: 2.9, 8.3, & 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.	
9	Is the site within an area of previously worked ground?	No – See BGS map extract (Figure 3 herein).	4.1
10	Is the site within an aquifer? If so, will the proposed basement extend beneath the water table such that dewatering may be required during construction?	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 2.2km from Hampstead No.1 Pond.	
12	Is the site within 5m of a highway or a pedestrian right of way?	No – Proposed basement does not extend to within 5m of the Goldhurst Terrace footway in-front of the property.	Section 3

13	Will the proposed basement substantially increase the differential depth of foundations relative to neighbouring properties?	Yes – The adjoining No's 188 & 192 Goldhurst Terrace do not have modern basements (although they are both likely to have existing cellars similar to No.190).	Carried forward to Scoping: 8.3, Section 10.4
14	Is the site over or within the exclusion zone of any tunnels, eg railway lines.	No – Re railway tunnels. Unknown - re other tunnels.	Carried forward to Scoping: 8.3, 10.1.3

7.4 Surface flow and flooding screening flowchart:

Question		**Response, with justification of 'No' answers**	**Clauses where considered further**
1	Is the site within the catchment of the pond chains on Hampstead Heath?	No – Site is approx 2.2km from the nearest part of the catchment (Hampstead No.1 Pond).	
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 front and rear lightwells (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?	Yes – Proposed front lightwell is likely to increase hard surfaced area slightly.	Carried forward to Scoping: Section 3, 8.4 & Section 10.7
4	Will the proposed basement result in changes to the profile of the inflows (instantaneous and long-term) of surface water being received by the adjacent properties or downstream watercourses?	No – Run-off from front and rear gardens to adjacent properties is minimal. The possible historic natural watercourse downslope of the property is thought to have been culverted.	5.1 to 5.6
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 surface run-off from this property reaches a watercourse.	5.6
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 – Goldhurst Terrace was subject to surface water flooding in both the 1975 and 2002 flood events, though the construction in 1994 of the NW Storm Relief Sewer should have been beneficial.	5.7, 5.10-5.12 & Figures 5 - 7. Carried forward to Scoping: 8.4 & Section 10.7

7.5 Non-technical Summary – Stage 1:

The screening exercise in accordance with CPG4 has identified seven issues which need to be taken forward to Scoping (Stage 2); one is related to subterranean (groundwater) flow, four are related to Ground Stability and two are related to Flooding potential. The presence of groundwater in the clays 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 Subterranean (groundwater) flow scoping:

Issue (= Screening Question)		**Potential impact and actions**
4	Will the proposed basement development result in a change in the proportion of hard surfaced/ paved areas?	**Potential impact:** Increased hard surfacing would decrease infiltration of surface water into the ground. **Action:** Review appropriate types of SuDS for use as site-specific mitigation.

8.3 Slope/ground stability scoping:

Issue (= Screening Question)		**Potential impact and actions**
5	Is the London Clay the shallowest strata at the site?	**Potential impact:** Continued seasonal shrink/swell below shallow foundations and heave following unloading by the basement excavations. **Action:** Ground investigation required, then 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 No.190 and the adjoining No's 188 & 192 once the proposed basement has been constructed**.** **Action:** Review potential impact of seasonal water content changes in the clays, and any planned vegetation removal and future vegetation growth. Designer and contractor to take account of any weakening of the structure caused by past movements.
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 the adjoining No's 188 & 192 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**.** **Action:** Undertake services search to check that there are no tunnels/services in the vicinity.

8.4 Surface flow and flooding scoping:

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

8.5 Non-technical Summary – Stage 2:

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

- A ground investigation is required (which has already been undertaken).
- Appropriate types of Sustainable Drainage System (SuDS) should be reviewed for the increase in hard surfaced/ paved areas and rainwater discharge route.
- 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.
- Undertake a services search to ensure there are no deep tunnels/services.
- Owing to Goldhurst Terrace being recorded as having flooded in 1975 and 2002, the future flood risk should be assessed.

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

9. STAGE 3 – GROUND INVESTIGATION

9.1 A site-specific ground investigation was undertaken by Chelmer Site Investigations (CSI) on 28th January 2016, and comprised two continuous flight auger boreholes (BH1 & BH2) drilled to a depth of 8.0m below ground level (bgl) within the rear garden and front parking area to No.190 respectively, as well as two hand dug trial pits. The findings from the investigation are presented in CSI's Factual Report (see Appendix C), including a site plan, trial pit logs, borehole logs, and laboratory test results.

9.2 Trial pits TP1 & TP2 were dug in order to investigate the foundations to No.190, and the soils beneath the footings, at their respective locations.

- TP1 was dug to a depth of 465mm within the southern part of the existing cellar, alongside its eastern side wall. The pit revealed brickwork with three corbels which projected 140mm resting at a depth of 325mm on 140+mm of concrete. The depth of the underside of the concrete footing was not established, as a clay pipe was encountered, and the pit was abandoned. At this location, a 50mm thick, ground bearing concrete floor slab was recorded.
- TP2 was dug to a maximum depth of 1275mm, immediately to the west of the rear bay, alongside both the rear wall of the house (Section A), and the flank wall to No.192 (Section B). Section A revealed brickwork with three corbels at its base, resting on a 300mm thick concrete footing which projected 230mm from the face of the wall and was founded at 1.075m below ground level (bgl). Similarly, section B revealed brickwork with three corbels at its base, resting on a 225mm thick concrete footing which projected 250mm from the face of the wall and was founded at 0.775m bgl. At this location, 50mm thick paving slabs were recorded over 100m of lean mix concrete.

9.3 In both trial pits, Made Ground was recorded directly beneath the surfacing. In TP1 this Made Ground was described as "dark brown, gravelly, silty clayey **sand**, with numerous brick fragments", whereas, in TP2, Made Ground consisting of "dark brown, gravelly, sandy clayey **silt**, with brick fragments" was recorded. Beneath the Made Ground in TP1, "wet, orange, silty coarse SAND" was recorded alongside the concrete footing (below 325mm bgl), and in section A of TP2, "Stiff, mid brown silty CLAY, with partings of brown and orange silt and fine sand" was recorded directly beneath the concrete footing. The base of the Made Ground was not intercepted in Section B of TP2, however it should be noted that this side of the pit was only dug to 975mm bgl, 0.1m above the base of the Made Ground recorded in Section A.

9.4 The site's geology, as found by the two boreholes drilled in the gardens by CSI, may be summarised as:

- <u>Made Ground:</u> Found immediately beneath 0.20m of turf and topsoil in BH1, and beneath 0.1m of paving slabs and concrete in BH2; the Made Ground was proved to depths of 0.9m and 1.2m below ground level (bgl) in BH1 and BH2 respectively. In BH1, this Made Ground was recorded as "dark brown, gravelly silty **clay**, with brick fragments and pieces"; similar Made Ground was recorded in BH2, except the clays there were sandy and contained "numerous brick and concrete fragments".

- Weathered London Clay Formation: Recorded from the base of the Made Ground to depths of 5.5m and 6.0m bgl in BH1 and BH2 respectively. The upper part of the Weathered London Clay Formation was described in both BHs 1 & 2 as "Stiff (becoming very stiff from 3.0m bgl in BH1), brown, grey veined, silty CLAY, with partings of brown and orange silt and fine sand, and occasional gravel (fine gravel in BH1)". The presence of the gravel suggests that the clays above 3.0m may be a locally-derived Head deposit (see paragraph 4.2). Below 3.5m in BH1, and below 5.0m in BH2, the lower part of the Weathered London Clay was described as "Very stiff, mid to dark brown CLAY, with partings of brown and orange silt and fine sand and crystals".
- London Clay Formation: Proved from the base of the Weathered London Clay Formation to the base of both boreholes at 8.0m bgl; the London Clay was recorded as "Very stiff, mid-grey, silty CLAY, with crystals" in both BHs 1 & 2.

9.5 Hand vane measurements of shear strength were taken in-situ in the boreholes in the weathered and 'un-weathered' London Clay. In BH1, at 1.0m and 2.0m bgl, these tests gave averaged values of 81kPa and 102kPA respectively. In both BH1 and BH2, at 3.0m and below, all readings taken were >140kPa. These values do not allow for the clay's fabric such as fissures, so typically over-estimate the soil's strength and should NOT be used for design.

9.6 No roots were observed in any of the exploratory holes.

9.7 A groundwater strike was recorded in BH1 (rear garden) at 0.8m bgl, and on completion, the borehole was described as open (ie: stable), with a groundwater standing level at 0.9m bgl. Conversely, no groundwater entries were recorded in BH2, and the borehole was described as 'dry' and open on completion.

9.8 Standpipes were installed to depths of 6.0 and 8.0m bgl in BH1 and BH2 respectively, and water level readings were taken on 3rd and 10th March 2016. During this short period of monitoring, the water level in both boreholes rose slightly, from 0.67m to 0.56m bgl in BH1 (located at the rear of the property), and from 5.91m to 5.43m bgl in BH2 (located at the front of the property). These levels may not have equilibrated fully with water pressures in the clay, so may not have been representative of the groundwater levels/pressures in the surrounding ground.

Laboratory Testing:

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

9.10 Plasticity tests were performed on a total of seven samples of Weathered London Clay, recovered from BH1 at depths of 1.0m, 2.5m and 4.0m bgl, BH2 at 1.5m, 3.5m and 5.5m bgl, and TP2A at 1.1m bgl. Plasticity tests were also performed on two samples of 'un-weathered' London Clay, recovered from BH1 and BH2 at 8.0m bgl. All nine of the samples tested were found to be of Very High Plasticity as classified by BS5930 (1999, 2010), and High volume change potential, as defined by the NHBC (NHBC Standards, 2013, Chapter 4.2, Building near Trees). It should be noted that three of the samples recovered from BH1 at 1.0m, 2.5m and 4.0m bgl were found to be at the boundary between High and Very High Plasticity, with Liquid Limits of 70%.

9.11 The moisture contents of the same nine samples recovered from BH1, BH2 and TP2 were found to vary between 30% and 38%; the variation with depth has been plotted as profiles against depth in CGL's report. All of the moisture contents were 4-12% above the plastic limit values from the same sample, so would not generally be considered indicative of significant desiccation.

9.12 The chemical tests were undertaken on a total of seven samples, recovered from BH1 at 0.5m, 2.5m and 8.0m bgl, from BH2 at 1.0m, 1.5m and 5.5m bgl, and from TP2A at 0.25m bgl thus included samples from the Made Ground, Weathered London Clay and (un-weathered London Clay). These tests were carried out in order to assess the potential for acid or sulphate attack on buried concrete, broadly in accordance with BRE Special Digest 1 (2005). The following results were recorded.

pH value:	7.5 – 8.3
Water-soluble sulphate:	90 – 3000 mg/l
Total Sulphur:	<0.02 – 0.81 %

Calculations following BRE Digest SD1 gave 'derived' values:

Total Potential Sulphate (TPS):	0.06 – 2.43%
Oxidizable sulphides:	0 – 2.03%.

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

DS-1 to DS-2: Samples from the Made Ground
DS-1 to DS-3 (based on TPS): Samples from the Weathered London Clay
DS-5 (based on TPS) and high pyrite content: Sample from the London Clay.

Non-technical Summary – Stage 3:

9.13 The site-specific ground investigation at No.190 Goldhurst Terrace recorded CLAYS of the London Clay Formation, as mapped by the British Geological Survey (BGS), although the weathered clays above a depth of approximately 3.0m may have been moved downslope during the Ice Age (a Head deposit). A variable thickness of Made Ground was also found overlying the natural strata across the site.

9.14 A groundwater strike was recorded at 0.8m bgl in BH1; during the short monitoring period after installation, groundwater levels of up to 0.56m bgl were measured, which probably represented perched groundwater in the Made Ground. Conversely, no groundwater entries were recorded in BH2 during drilling, and the highest groundwater level recorded during the short period of monitoring was 5.43m bgl. These recorded levels may not have equilibrated fully with water pressures in the surrounding clay, so may not be representative of the groundwater pressures (phreatic surface levels) which existed at that time.

9.15 Exceptionally high sulphate and Total Potential Sulphate concentrations were found in one sample from the London Clay at depth, and the same sample's high Oxidizable Sulphate concentration is indicative of the presence of significant quantities of pyrite. These must be taken into account when selecting the piling method and grade of concrete (see paragraph 10.4.14).

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 by the following sequence:

- Made Ground: Discovered within all of the exploratory holes and was proved to a maximum depth of 1.2m below ground level (bgl) externally, and a maximum thickness of 0.325m internally, below the concrete floor slab in the cellar. Descriptions of the Made Ground varied from "gravelly, clayey, silty **sand**", to "gravelly silty **clay**", with included fragments of brick, and concrete; however other materials, as well as other soil types and greater thicknesses/depths, are also likely to be present on site, owing to the inherent variability of Made Ground.
  Perched groundwater should be expected in the Made Ground during at least the winter and spring seasons as was observed in BH1 in the rear garden.
- Head Deposits: Typically firm, silty clays with variable amounts of rounded flint gravel where overlying the London Clay; probable Head deposits were recorded to depths of 3.0m in both boreholes. To the east of No.190 Goldhurst Terrace, the thickness of these Head Deposits appears to be unusually variable, with the recorded range being from 0.2m to 0.9m at 17a Fairhazel Gardens (where the consistency was only soft-to-firm), to 3.2m at No.65 Aberdare Gardens (by interpretation of the strata descriptions).
- Weathered in-situ London Clay: Stiff to very stiff, mid to dark brown, silty CLAYS were found directly beneath the probable Head deposits in both of the boreholes and beneath the Made Ground and concrete footing in TP2 (see paragraph 9.4 for a more detailed description). These clays are likely to be fissured and will undergo heave movements in response to unloading by the basement excavation. The "crystals" recorded were probably selenite (a form of gypsum) which is aggressive to buried concrete as, potentially, is the indicated high pyrite content. The "stiff" and "very stiff" consistency descriptions were probably based in part on the vane test results, which are known to over-estimate the shear strength of fissured clays.
- London Clay Formation ('un-weathered'): Very stiff, mid-grey, silty CLAY of the London Clay Formation was encountered below depths of 5.5m/6.0m and continued to the maximum depth drilled (see paragraph 9.4). The logs of other boreholes in the area show that the top of the 'unweathered' London Clay could extend deeper, with depths ranging from 6.5m to 10+m bgl recorded in the area.
- Hydrogeology
  - 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 limited to minor seepage through any of the silt/sand partings which are sufficiently interconnected. In both BH1

& BH2, "partings of brown and orange silt and fine sand" were recorded throughout the Weathered London Clay; however neither of the boreholes found any individual silt/sand layers of sufficient size to warrant separate identification. A groundwater strike was recorded at 0.8m bgl in BH1 (rear garden), and a groundwater standing level of 0.56m bgl was recorded during the short period of monitoring.

- 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.
- Water was found on the front end of the cellar floor, although its origin was not apparent (so it could have been a plumbing defect).

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 10.2.6 for the recommended provisional design groundwater levels.

10.1.3 No railway tunnels are known to pass below or close to the site. The NW Storm Relief Sewer is understood (from Thames Water's drawings) to run beneath part of Goldhurst Terrace to the east of No.190. 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 Groundwater has been recorded at 0.56m bgl in the rear garden (in BH1) on the upslope side of the house, whereas to the front of the house the groundwater beneath the forecourt parking area was recorded at 5.43-5.91m bgl. The latter may not have been representative of the true groundwater levels/pressures in that area (owing to the generally low permeability of these clays), though similar marked differences in groundwater levels between front and rear gardens have been recorded in properties on both Aberdare Gardens and Canfield Gardens. This suggests that the groundwater in the rear garden is perched at sufficiently shallow depth to be blocked by the existing foundations to these houses (though there was no groundwater entry into TP2). The water standing at the front end of the cellar floor was completely clear, even after being disturbed, so might have come from a leaking pipe rather than being groundwater, though evidence of past damp was visible in the lower parts of the cellar walls (which is typical of such cellars in London Clay).

10.2.2 The sand around the drain pipe found beneath the cellar floor (in TP1) is probably artificial bedding for the pipe rather than a natural, in-situ sand. It was noted to be wet, which is not surprising given the high groundwater level in at least parts of the rear garden (in BH1, but not in TP2). Where exposed, the foundations to the rear wall of the main part of the house (TP2, Section A) were bearing onto in-situ natural clays, so the foundations probably block all flow through the Made Ground from front to rear of the site. The Made Ground was a clayey silt in TP2, and clays in BHs 1 & 2; these are relatively low permeability materials, though more permeable than the underlying clays and locally sufficiently permeable to give rise to the groundwater strike in BH1. The suspected Head deposits, which extended to 3.0mbgl, may also be more permeable than the clays that they are derived from. Thus significant flow in the Made Ground is only likely to be feasible where service trenches or granular pipe bedding facilitates channelled flow. Groundwater can also collect in the backfill to footing trenches, but is typically static (until excavations are dug into/though the backfill).

10.2.3 The lack of any record of silt/sand horizons, or groundwater entries, in the weathered London Clay in both boreholes (other than the thin partings) indicates that these silty clays are likely to be towards the lower end of the permeability scale, so will permit little or no flow of groundwater. However, the lack of groundwater entries into boreholes from the London Clay during drilling does not necessarily mean that groundwater was absent, rather the low permeability of the clays merely means that the flow rate was too slow for groundwater entries to occur before the instrumentation was installed in the borehole, and any water in silt/sand partings was potentially sealed in by smearing of clays during the drilling process.

10.2.4 The basement will be founded at approximately 3.0-3.3m below ground level. It will therefore extend down through most or all of the Head deposits and onto/into the underlying "stiff", silty clays. This in-situ London Clay contains thin partings of silt/sand; flow through these partings, if any, would only occur where the partings are sufficiently interconnected, which is generally rare, and even then is likely to involve very low flow rates and volumes. The proposed basement will not increase the width of the existing obstruction to flow (if any) created by the existing foundations and the cellar so, given the anticipated negligible flow in the London Clay, the proposed basement is considered acceptable in relation to groundwater flow.

10.2.5 No cumulative impact is anticipated from the construction of the proposed basement, owing to the lack of deep basements on either side of the proposed basement.

10.2.6 In the unlikely event that the basement excavations encounter a local deposit of more permeable soils containing mobile groundwater which has remained undetected within the London Clay (or the suspected/potential 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 any 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. This groundwater bypass would only be required where a permeable soil unit extends across the full width/length of the basement, such that the basement would fully or substantially block flow through that unit. It will comprise a collector zone on the upstream side of the basement, a connecting drain pipe set in concrete beneath the basement slab, and a discharge zone in the same permeable soil unit on the downstream side of the basement. The collector and discharge zones must be installed at appropriate levels against the rear face of the underpins concerned (with the base of the discharge zone at the same level as or below the base of the collector) and would typically comprise either perforated pipes wrapped in non-woven geotextile or a drainage geocomposite connected to a suitable collector pipe. The sequence of underpin construction should be planned such that the upslope underpins are completed before the downslope underpins. Temporary over-pumping might be required until the discharge zone has been completed.

10.2.7 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.

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

10.2.9 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 may rise to ground level, at least 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, and in view of the high groundwater levels already recorded, use is recommended of design groundwater levels equal to ground level around the perimeter of the basement, in both short-term and long-term situations (in accordance with the Eurocode 7, BS EN 1997-1). It would be prudent for one additional set of groundwater level readings to be taken from the standpipes immediately prior to the start of excavation works on this site in order to enable the contractor to finalise their methodology for groundwater control.

10.2.10 The basement structure must be designed to resist the buoyant uplift pressures which would be generated by groundwater at ground level. For the founding depths currently proposed, the uplift pressures would be up to 33kPa (factored).

10.3 Subterranean (Groundwater) Flow – Temporary Works

10.3.1 Local groundwater entries into the excavations for the basement should be expected, especially from the Made Ground on the upslope side of the basement, though, on current evidence, they 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 any groundwater removed by sump pumping.

10.3.2 All groundwater control measures should be supervised by an appropriately competent person. The contractor's site staff should constantly monitor the excavations for changes in groundwater conditions and the Site Manager or other responsible person should be notified immediately on the discovery of any such change. If significant continuing flow is noted from any local feature within the London Clay then the advice of a hydrogeologist should be sought, who will assess whether an engineered groundwater bypass will be required (see 10.2.6). A careful watch should be maintained to check that fine soils are not removed with groundwater; if any such erosion/removal of fines is noticed, then pumping should cease, the excavation concerned may need to be partially backfilled temporarily, and the advice of a suitably experienced and competent ground engineer or dewatering specialist should be sought. If such removal of fines is encountered then it is likely to be from a localised deposit of more permeable, granular soils, so use of suitably screened well-points or a screened abstraction well located upslope of the excavation might be considered appropriate by the dewatering specialist.

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, so these clays must be protected from all sources of water, as described more fully in paragraph 10.4.8 below.

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

10.4 Slope and Ground Stability

10.4.1 With slope angles of approximately 1.0° upslope of this property, the proposed basement excavation raises no concerns in relation to slope stability.

10.4.2 The structural drawings by SC Green Ltd show that the basement's retaining walls will be constructed using reinforced concrete (RC) underpinning techniques beneath the building, together with contiguous bored pile retaining walls around the front and rear lightwells where the perimeter extends beyond the footprint of the existing buildings.

<u>Basement Retaining Wall Construction - Underpinning:</u>

10.4.3 Underpinning methods involve excavation of the ground in short lengths (not exceeding 1.0m is recommended) in order to enable the 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 that the structure is in good condition. Paragraphs 3.2 & 3.3 and Table 1 in Section 3 present the estimated founding (formation) levels and depths of excavation which are likely to be required for the underpins and basement/lightwell slabs.

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

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

A high quality of workmanship and 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 10.4.8 below). Any cracks 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.

10.4.5 All temporary support should be in accordance with ASUC guidelines and should use high stiffness continuous propping of the excavated face, installed in accordance with best practice prior to casting, in order to minimise the ground movements. The clays seen at shallow depth in the investigation at No.190 were all described as "stiff", however, the possibility remains that weaker Head deposits might be present (see paragraph 4.8) so the condition of the clay immediately beneath the footings on both sides of each underpin excavation should be checked by a competent person when excavation beneath the footing starts; if vane testing indicates that the shear strength of the clay is less than 60kPa then temporary vertical propping should be installed close to the side of the excavation concerned, and progressively extended as the excavation progresses, in order to ensure that the load transfer from above the underpin excavation does not over-stress the adjacent weak soils. A sacrificial prop should also be left within the stem of the pin when the concrete is cast.

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. London Clay is usually 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 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.

10.4.9 The construction sequence is described in the structural engineer's Construction Method Statement by SC Green Ltd.

<u>Basement Retaining Wall Construction – Bored Pile Walls:</u>

10.4.10 Use of contiguous bored pile retaining walls is proposed by SC Green Ltd for the front and rear lightwells. These bored pile walls will be finished with reinforced concrete capping beams which will also overlap the brick facing to the piles. Adequate temporary support must be installed in order to minimise lateral movement of the piles before the permanent basement slab has been constructed (see also paragraph 10.4.12). The minimum spacing between the pile locations and the existing house walls (to No.192 as well as No.190) should also be checked with specialist contractors, in order to confirm the feasibility of achieving the locations currently proposed.

10.4.11 A piling platform must be designed and constructed so as to provide a stable working platform for the piling rig taking into account both static and dynamic loads.

10.4.12 Minimisation of ground movements will be particularly important where these piles will support the ground alongside/beneath No.192's single-storey rear extension. A 'bottom-up' construction approach is unavoidable because there will be no ground floor slab over this lightwell (and will also apply to the front lightwell). Providing an adequately stiff temporary support system will not be easy because this project does not involve a full bored piled wall 'box'. Substantial raking props onto special foundation blocks are likely to be the preferred solution to limit wall deflections alongside No.192's rear extension, although beams spanning the full width of the proposed basement may also be considered. The slabs in both lightwells and basement should be designed to act as permanent high-stiffness props, so the 150mm thickness currently proposed will probably need to be increased.

10.4.13 The quality of workmanship has a significant impact on the magnitude of ground movements so a specialist piling contractor should be selected, in part on the basis of their ability to provide a high quality of workmanship in accordance with industry good practice.

Design Considerations:

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

- Earth pressures from the surrounding ground (see paragraph 10.4.15 below);
- Dead and live loads from the superstructure, including loads from the adjoining No's 188 & 192 which are carried on the party wall;
- Loads from all adjoining/adjacent walls in No's 188 & 192 which are founded within the relevant active earth pressure zone;
- Vehicle loadings on the forecourt and normal surcharge allowances elsewhere;
- Swelling displacements/pressures from the underlying clays;
- A design groundwater level at ground level, as described more fully in paragraph 10.2.9;
- Precautions to protect the concrete from sulphate attack; the sulphate levels are unusually high in the 'unweathered' London Clay (Class DS-5) and the Oxidizable Sulphides indicated the presence of significant pyrite content, so the piling method used must not allow these soils to become oxidised.

10.4.15 The following geotechnical parameters are applicable to the strata in this area and should be used when calculating earth pressures acting on the basement's retaining walls:

Made Ground (clays):	Unit weight, γ_b:	17.0 kN/m^3
	Effective cohesion, c':	0 kPa
	Angle of internal friction, φ':	25°
London Clay Fm/Head:	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: where undisturbed, typically 1.0 up to 2.5-3.0 (varies with depth); the extent to which this stress is released depends on the stiffness of the temporary and permanent support, but might typically reduce to around 1.0.

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

10.4.16 Normal good practice in foundation construction requires progressive stepping up between foundations of different depths beneath a single structure. Transitional underpins should therefore be installed beneath No.190's single-storey rear extension/conservatory (as shown in both plan and section on SC Green's Drg No. 4013/A1/05G) and, subject to agreement under the Party Wall Act negotiations, should be considered for some of the adjoining load-bearing walls in No's 188 & 192 where moisture levels in the clays beneath the footings may vary in the future. If there is an existing cellar beneath No.188's front entrance porch and Hall (similar to No.190's), then that will already provide adequate transition. Variations in moisture levels beneath the footings would be possible where the footings are less than 1.2m deep and/or if fine tree roots are present in the clays beneath these footings. Thus the need for, and the depth and extent of, any such transitional underpins will need to be assessed during the works (as well as being subject to approval under Party Wall Act protocols). The locations of the potentially affected adjoining walls are also indicated on Drg No. 4013/A1/05G.

10.5 PDISP Heave/Settlement Assessment

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

10.5.2 Figure D1 in Appendix D illustrates the layout of the proposed basement, as well as the layout of PDISP zones used to model the underpins, bored pile walls (BPWs) and basement slab, based on information received from S.C.Green, including their 'Phase 4' (Drg No.4013/A1/05G), an extract from which is presented in Figure D2. The maximum overall dimensions of the proposed basement are approximately 8.53m wide by 20.97m long.

10.5.3 Table 4 presents the net changes in vertical pressure for four major stages of the stress history of the basement's construction, as detailed in paragraph 10.5.6 below. The load takedown data for the proposed new building were obtained from S.C.Green's 'Structural Calculations', and have been summarised in a table presented in Figure D2; net pressure changes were then calculated, including the gross unloading from the excavation, the gross bearing pressures which will be applied by the underpins, and the self-weight of the piles. The piles which form the contiguous bored pile walls (Zones 1 & 8) were assumed to be 9.1m long; below the formation level of the basement slab the piles were divided into three 2.0m long sections (sub-zones b-d), with the loads on the piles acting at the base of each section. Superimposed Zones 23-27 (coloured green) have been used to reduce the excavation depths within the footprint of the existing cellar beneath No.190, whereas Zones 30-32 (also coloured green) have been used to model the bearing pressures from columns/piers which bear onto underpin bases, so overlap the blue zones concerned.

Table 4: Net changes in vertical pressure for PDISP Zones

ZONE	Net change in vertical pressure (kPa)		
#	Stage 1	Stage 2	Stages 3 and 4
1a	18.60	18.60	18.60
2	-10.59	-10.59	-10.59
3	-21.08	-21.08	-21.08
4	-7.17	-7.17	-7.17
5	-14.10	-14.10	-14.10
6	-21.57	-21.57	-21.57
7	-12.62	-12.62	-12.62
8a	18.30	18.30	18.30
9	-4.34	-4.34	-4.34

Table 4 Cont'd: Net changes in vertical pressure for PDISP Zones			
ZONE	**Net change in vertical pressure (kPa)**		
#	**Stage 1**	**Stage 2**	**Stages 3 and 4**
10	6.24	6.24	6.24
11	-5.74	-5.74	-5.74
12	33.07	33.07	33.07
13	24.50	24.50	24.50
14	48.50	48.50	48.50
15	24.50	24.50	24.50
16	0.00	-55.10	-51.35
17	0.00	-55.10	-51.35
18	0.00	-55.10	-51.35
19	0.00	-55.10	-51.35
20	0.00	-55.10	-51.35
21	0.00	-58.90	-55.15
22	0.00	-56.05	-52.30
23	34.20	34.20	34.20
24	0.00	34.20	34.20
25	34.20	34.20	34.20
26	0.00	34.20	34.20
27	34.20	34.20	34.20
28	-2.13	-2.13	-2.13
29	-36.70	-36.70	-36.70
30	38.49	38.49	38.49
31	80.23	80.23	80.23
32	56.11	56.11	56.11
1 b-d	12.00	12.00	12.00
8 b-d	12.00	12.00	12.00

Ground Conditions:

10.5.4 The ground profile was based on the site-specific ground investigation by Chelmer Site Investigations, as presented in Sections 9 and 10.1 above, and the desk study information.

10.5.5 The short-term and long-term geotechnical properties of the soil strata used for the PDISP analyses are presented in Table 5, based on this investigation and data from other projects.

Table 5: Soil parameters for PDISP analyses

Strata	**Level** (m ASD)	**Undrained Shear Strength,** Cu (kPa)	**Short-term, undrained Young's Modulus,** Eu (MPa)	**Long-term, drained Young's Modulus,** E' (MPa)
London Clay	37.60 20.40	80 209	40 104.5	24 62.7

Where:

For CLAY:

Undrained Young's Modulus, Cu = 80 + 7.5z kPa

where z = depth below the top of the stratum (37.6m ASD)

Undrained Young's Modulus, Eu = 500 * Cu

Drained Young's Modulus, E' = 0.6 * Eu

PDISP Analyses:

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

- Stage 1 – Construction of underpins and contiguous BPWs – Short-term condition
- Stage 2 – Bulk excavation of central area to formation level – Short-term condition
- Stage 3 – Casting of basement slab – Short-term (undrained) condition
- Stage 4 – As Stage 3, except – Long-term (drained) condition

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

Heave/Settlement Assessment:

10.5.8 Construction of the underpins and excavation of the basement will cause immediate elastic heave/settlements in response to the stress changes, followed by long term plastic swelling/settlement as the underlying clays take up groundwater or consolidation occurs. The rate of plastic swelling/consolidation will be determined by the availability of water and the low permeability of the London Clay, so can take many decades to reach full equilibrium. The basement slab will need to be designed so as to enable it to accommodate the swelling displacements/pressures developed underneath it.

10.5.9 The ranges of predicted short-term and long-term movements for each of the main parts of the proposed basement are presented in Table 6 below. These analyses indicated that the perimeter walls are likely to experience negligible to very minor heave/settlement movements in Stage 1, followed by minor heave movements in the later stages, whereas the basement slab is likely to experience slightly greater heave movements. Movements beneath the contiguous BPWs were less than the RC underpins, since only the self-weight of these walls has been modelled.

Table 6: Summary of predicted displacements

Location	Stage 1 (Figure D3)	Stage 2 (Figure D4)	Stage 3 (Figure D5)	Stage 4 (Figure D6)
Front lightwell (BPW):	0.5mm settlement	0 – 1mm heave	0 – 1mm heave	0 – 1.5mm heave
Party wall between No's 190/192 Goldhurst Terrace:	0 - 0.5mm settlement	0.5 – 2.5mm heave	0.5 – 2mm heave	0.5 – 4mm heave
Rear lightwell (BPW):	0 – 0.5mm settlement	0.5 - 2mm heave	0 – 1.5mm heave	0.5 – 2.5mm heave
Rear wall of rear extension:	0 – 0.5mm heave	0.5 – 2.5mm heave	0.5 – 2.5mm heave	1 – 4mm heave
Party wall between No's 190/188 Goldhurst Terrace:	0.5mm settlement to 0.5mm heave	0.5 – 2.5mm heave	0.5 – 2.5mm heave	1 – 4mm heave
East flank and front walls to No.190	0 – 1mm settlement	0 – 1.5mm heave	0 – 1.5mm heave	0 – 2.5mm heave
Internal column/foundation loads	0.5 – 1.5mm settlement	0.5 – 2.5mm heave	0.5 – 2.5mm heave	1 – 4mm heave
Basement slab	(0 - 1mm Settlement) No slab present	(0.5 – 3.5mm Heave) No slab present	0.5 – 3mm Heave	0.5 – 5.5mm Heave

10.5.10 All the short-term elastic displacements would have occurred in Stages 1-3, before the concrete of the basement slab has set/cured, so, in theory, only the post-construction incremental heave/settlements (from Stages 3 to 4) might be relevant to the slab design, subject to the detailed construction sequence employed. The analyses indicated that the theoretical maximum predicted post-construction displacements beneath the slab would be around 2.5-3mm heave.

10.6 Damage Category Assessment

Underpins:

10.6.1 When underpinning it is inevitable that the ground will be un-supported or only partially supported for a short period during excavation of each pin, even when support is installed sequentially as the excavation progresses. This means that the behaviour of the ground will depend on the quality of workmanship and suitability of the methods used, so rigorous calculations of predicted ground movements are not practical. However, provided that the temporary support follows best practice, then extensive past experience has shown that the bulk movements of the ground alongside underpins for a single-storey basement 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 the angular distortion (as a deflection ratio) which they might generate using the method proposed by Burland (2001, in CIRIA Special Publication 200, which developed earlier work by himself and others). Separate assessment is required for each location which involves a critical combination of depth of excavation and foundation depth/ vulnerability beneath the adjoining/adjacent buildings.

10.6.3 No evidence has been found on the London Borough of Camden's planning website for modern basements below the adjoining No's 192 & 188 Goldhurst Terrace, although these properties are likely to have existing cellars, similar in footprint to No.190's. The uniform founding level for the proposed basement means that 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 C580 (Gaba et al, 2003), concerns movements at ground surface (and presents data for embedded retaining walls, but, as no equivalent data exist for underpins, this data is the best available so must be interpreted very cautiously). These movements will reduce with depth, so CIRIA C580 states that the ground above foundation level should be ignored when undertaking building damage assessments.

10.6.4 The worst case scenarios as predicted by the PDISP analysis will occur at the main front and rear walls to No's 192 & 188, where the analyses indicated that the heave/settlements will be nil in Stage 1 (because the greater heave movements in the later stages would be beneficial). The existing cellars to No's 192 & 188 are likely to be located beneath the front part of these properties, with No.188's almost certainly a mirror-image of, and adjoining, the cellar in No.190. Their greater foundation depths will be beneficial, thus the worst case scenarios are expected to occur at the main rear walls of No's 192 & 188. A single damage category assessment has been undertaken for these worst case scenarios; the assessment considered:

- 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;
- ground movements alongside the proposed underpins and retaining walls caused by relaxation of the ground in response to the excavations.

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

<u>Main rear walls to No.192 & No.188:</u>

10.6.5 Since the geometry of the rear walls to No's 192 & 188 are very similar, and the PDISP analysis predicted the same degree of movement alongside theses walls (nil in Stage 1, see paragraph 10.6.4 above), the damage category assessments for both of these walls have been considered together.

10.6.6 The relevant geometries are:

Depth of foundations = 0.775m (as recorded in TP2, alongside No.192)

Depth of excavation below the foundations = 3.25 – 0.775 = 2.48m

Width (L) of zone of affected soils = 2.48 x 4 = **9.9m**, so will extend the full width of the adjoining No.192, and just beyond the 190/188 party wall. (Width of No.192 = approximately 10.0m, measured from OS map in absence of any plans for No.192 with scale bars or dimensions).

Height of rear walls, to eaves (H) = 7.87 + 0.775 = **8.65m**.

Hence, L/H = **1.14**.

10.6.7 Thus, for an anticipated 5mm maximum horizontal displacement, the strain beneath No's 192 and 188 would be in the order of $\varepsilon_h = 3.8 \times 10^{-4}$ (0.038%).

10.6.8 The maximum settlement predicted by the PDISP analysis alongside the rear walls to No's 192 & 188, in Stage 1 was 0mm (see Figure D3 in Appendix D), thus this can be ignored. The typical settlement caused by relaxation of the ground alongside the basement in response to excavation of the underpins, was therefore estimated using only the convex settlement profile for a worst case (low stiffness) support system presented in Figure 2.11(b) in CIRIA Report C580. This settlement profile gives a maximum deflection, Δ = 0.07% of the excavation depth, occurring at a distance greater than the depth of excavation from the excavation. The maximum Δ = 2.28mm, which represents a deflection ratio, $\Delta/L = 1.75 \times 10^{-4}$ (0.018%).

10.6.9 Using the graphs for L/H = 1.0, these deformations represent a damage category of 'very slight' (Burland Category 1, ε_{lim} =0.05-0.075%), on the boundary with Burland Category 0, 'negligible', as given in CIRIA SP200, Table 3.1, and illustrated in Figure 9 below.

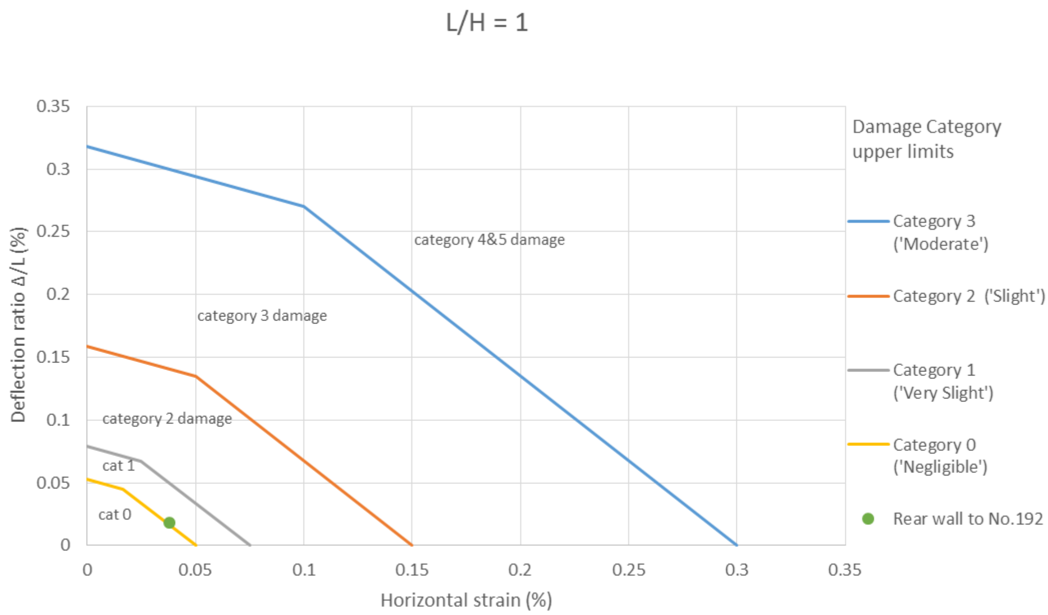

Figure 9: Damage category assessment for rear walls to No's 192 & 188.

10.6.10 Use of best practice construction methods, including timely installation of appropriate high-stiffness temporary support to both the excavations and the newly cast underpins, will be essential to ensure that the ground movements are kept in line with the above predictions.

Bored Pile Walls:

10.6.11 Some ground movement is inevitable when basements are constructed, even when using bored pile walls. Ground movements alongside the piles have been assessed using relationships developed from empirical case history data published in CIRIA's report C580 (Gaba et al, 2003). That report noted that "*ground movements cannot be predicted accurately, but it is possible to estimate them based on … an empirical approach* …" as presented in the following paragraphs. The movements in ground supported by a bored pile wall are highly dependent on the stiffness of the support system as a whole. The proposed 'bottom-up' construction method would be classified as 'Moderate support stiffness', provided that an appropriate construction sequence is followed with high stiffness props installed at high level.

10.6.12 CIRIA Report C580 presents charts which relate estimated ground surface movements alongside bored pile retaining walls in stiff clays to pile installation (as Figure 2.8 therein) and excavation in front of the wall (as Figure 2.11).

10.6.13 For 'Moderate support stiffness' walls designed and constructed in accordance with best practice, the estimated ground surface movements resulting from installing a contiguous bored pile wall (to an estimated depth of 8.0m below ground level) and then excavating to depths up to 2.9m would be as given in Table 7 (interpolated between high and low stiffness curves). The pile depth has been estimated because, under standard UK practice, the design analyses for bearing piles are undertaken by the piling contractor.

Table 7: Potential approximate movements of ground surface immediately alongside the bored pile wall		
Moderate support stiffness – 2.9m depth of excavation / 8.0m deep wall		
Ground surface movements due to:	**Horizontal movement**	**Vertical movement**
Bored pile wall installation:	0.04% of wall depth = 3.2mm	0.04% of wall depth = 3.2mm
Excavation in front of wall:	0.25% of excavation depth = 7.25mm	0.20% of excavation depth = 5.80mm
Totals:	**10.5mm**	**9.0mm**

10.6.14 For present purposes, however, we are interested in the displacements immediately beneath the foundations to No.192's single-storey rear projection, so the above figures must be modified to allow for the offset of approximately 1.0m between the piled wall and the footing concerned, and an assumed footing depth of 0.75m.

Table 8: Potential approximate ground movements below foundations to flank wall to No.192's rear extension (at 1.0m from bored pile wall)		
Moderate support stiffness – 2.15m depth of excavation / 8.0m deep wall		
Ground surface movements due to:	**Horizontal movement**	**Vertical movement**
Bored pile wall installation:	0.035% of wall depth = 2.80mm	0.037% of wall depth = 2.96mm
Excavation in front of wall:	0.24% of excavation depth = 5.16mm	0.16% of excavation depth = 3.44mm
Totals:	**8.0mm**	**6.4 mm**

10.6.15 Following the same methodology as used for the underpins (in paragraphs 10.6.6 to 10.6.10) the strain beneath No.192's single-storey rear projection would be in the order of $\varepsilon_h = 8.1 \times 10^{-4}$ (0.081%).

10.6.16 The lower bound line for vertical movements in response to installation of a bored pile wall in stiff clays is linear, so will generate no deflection. The Low support stiffness graph was used to estimate the deflection likely to occur in response to excavation of the lightwell/basement alongside the bored pile wall (because CIRIA

Report C580 does not provide a curve for Moderate support stiffness systems); this graph is likely to over-estimate the deflection. The width of No.192's rear extensions is approximately 5.5m; when the separation between the bored pile wall and the extension is also taken into account, the maximum deflection Δ = 0.75mm, which represents a deflection ratio, $\Delta/L = 1.36 \times 10^{-4}$ (0.014%).

10.6.17 Using the graphs for L/H = 1.5, these deformations represent a damage category of 'slight' (Burland Category 2, ε_{lim} =0.075-0.15%), as given in CIRIA SP200, Table 3.1, and illustrated in Figure 10 below.

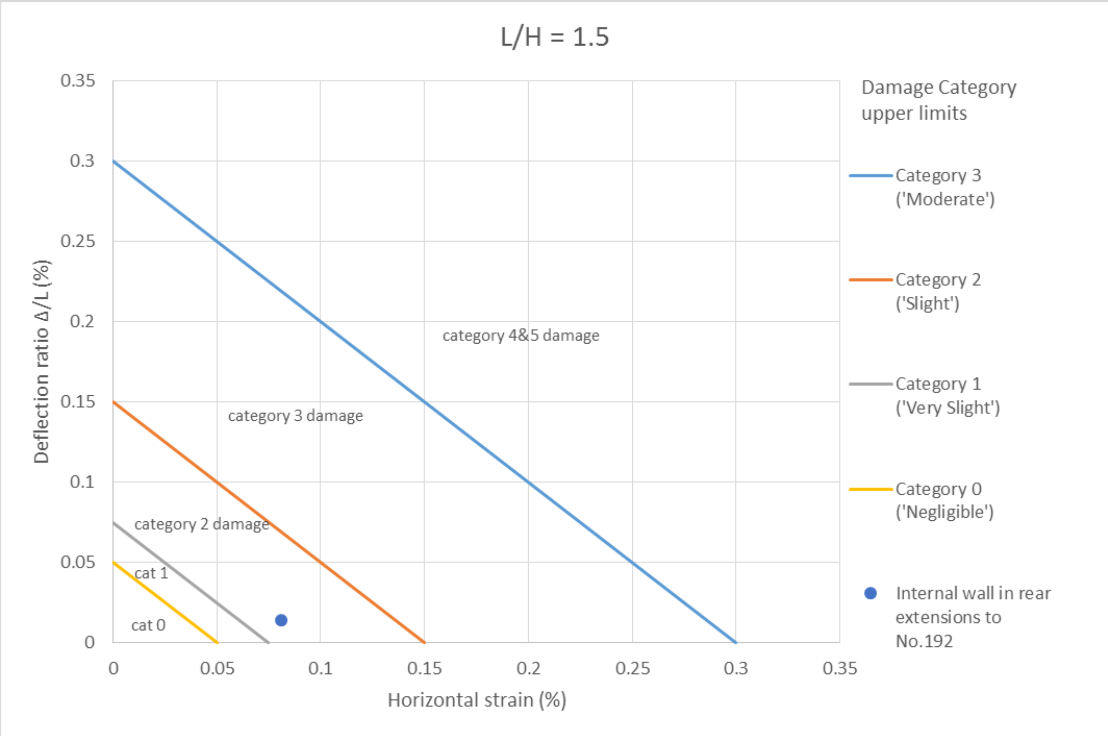

Figure 10: Damage category assessment for internal wall in No.192's rear extensions.

10.6.18 The CIRIA data were obtained from much larger retaining walls, with a minimum excavation depth of 8.0m, and the short length of this bored pile wall also means that the movements associated with its construction are likely to be smaller than those predicted by the CIRIA data. An observational approach is therefore recommended, as a form of mitigation, to assure all concerned that the displacements will remain within Category 1. Monitoring locations on the bored pile wall's capping beam and No.192's rear extension are recommended in Section 10.7; a horizontal strain not exceeding 0.06% will be required in order to keep any damage within Category 1, which means that the horizontal movement in the ground below the foundations should not exceed 6mm. Thus, an initial trigger level of 4mm movement in the flank wall of No.192 should be adopted, rather than 5mm as recommended more generally in paragraph 10.7.3.

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 Agreements 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 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 basement slab have been completed. This monitoring should be undertaken with a total station instrument and targets attached at a minimum of two levels at the following locations:

- internally, at three equally spaced locations on the 188/190 and 190/192 party walls;
- externally, on the front and rear walls of No.188, on the centreline of the 188/190 party wall;
- externally, on both corners of No.188's front entrance bay and at the south-east corner of the front wall;
- externally, on No.192's front bay window and at the south-west corner of the front wall;
- externally, at the north end (north-east corner) of No.192's flank wall which is party with No.190, and at three equally spaced points on the capping beam to the bored pile wall on the 190/192 boundary (this capping beam must be constructed before the ground in front of these piles is excavated);
- externally, at two locations on the flank wall of No.192's rear single-storey extension;
- externally, at the north-west corner of No.192's main rear wall;
- externally, at the front and rear ends of the flank wall to No.188's rear projection, and on the rear wall on the centreline of the 186/188 party wall;
- at the client's discretion, since outside the Party Wall Agreements, it would also be sensible to monitor all other load-bearing walls in No.190.

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

10.7.4 Visual inspections should be undertaken daily throughout the period of basement construction in order to check for signs of new cracking in the structural walls being underpinned, and in adjoining/adjacent load-bearing walls, including the front and rear elevations of No's 188 & 192. Particular attention should be paid to the more vulnerable walls, and a record of each inspection should be maintained on site. If any new structural cracks appear in those walls, then those cracks should be monitored using the Demec system (or similar) on the same frequency as the target monitoring.

10.8 Surface Flow and Flooding

Flooding from Rivers, Sea & Reservoirs:

10.8.1 The evidence presented in Section 5, paragraphs 5.8 & 5.9, 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);
- the area is not at risk of flooding from reservoirs;
- there are no flood defences, no areas benefitting from flood defences and no flood storage areas within 250m of the site.

Surface Water (Pluvial) Flooding:

10.8.2 There are no surface water features within 250m of the site (see paragraph 5.9).

10.8.3 The site is known to lie about 80m to the north of one of the former tributaries to the 'lost' river Westbourne (as described in Section 5 above). These tributaries have been culverted or diverted into the sewer system a century ago, so they are no longer able to receive surface water run-off. Whether the culverts remain connected hydraulically to the perennial surrounding groundwater is unknown, as discussed in more detail in paragraphs 5.1 to 5.5.

10.8.4 The *'Floods in Camden'* report (LBC Floods Scrutiny Panel, 2003) and LBC's CPG4 guidance document record that Goldhurst Terrace flooded in both the 1975 and the 2002 local pluvial flood events. The Camden Strategic Flood Risk Assessment (the SFRA, by URS, 2014) identified a Goldhurst Local Flood Risk Zone, which includes Goldhurst Terrace, because of these events (see Figure 7). Construction of the NW Storm Relief Sewer in 1994 will have helped to prevent flooding in some of the surrounding roads since then, although it too became overloaded in 2002 because it was only designed for a 1 in 10 year storm.

10.8.5 The latest flood models by both the Environment Agency, and by URS for the Camden SFRA, gave a 'Very Low' risk of surface water flooding, the lowest category which represents the national 'background' level of risk, for No.190's site, and for all other properties in the vicinity (see Figures 6 & 7). Flood mitigation measures to protect the basement from local surface water flooding, may be restricted to:

- Providing upstands to the retaining walls around the lightwells in order to prevent surface water from the adjoining areas from draining into the lightwells. Where the retaining walls are constructed as bored pile walls either the capping beam should form the upstand or dwarf walls should be built off the capping beam. If the mains drainage is running under surcharge (see paragraphs 10.8.8 to 10.8.12 below) then surface water run-off could get trapped in the rear garden, so the top of the upstand around the rear lightwell should be at the same level as the ground floor inside the house.
- Installing raised thresholds to the external doors in the lightwells.
- Installing temporary interception storage for surface water which might become trapped in the rear gardens (see 10.8.11 below).

Changes to Hard Surfacing & Surface Water Run-off:

10.8.6 The location for the rear lightwell is currently occupied by a patio formed of paving slabs bedded on concrete, and surface water is collected by a gully and channel drain, so is presumed to discharge to the mains drainage (because soakaways would not work in London Clay). Thus, the rear lightwell will not alter the area of hard surfacing or the volume of water currently being discharged to mains drainage.

10.8.7 Part of the location for the front lightwell is occupied by flower beds, so construction of this lightwell will increase the area of hard surfacing by a small amount. The potential increase in discharge to the mains drainage system must be attenuated by the inclusion of one or more appropriate Sustainable Drainage Systems (SuDS) in the scheme. The options for SuDS are limited in this circumstance, but could include:

- An equal increase in soft landscaping elsewhere within the site;
- Intervention storage or discharge control on roofwater/downpipes;
- Rainwater harvesting;
- Use of a grey water system.

The latter three SuDS schemes would require formal design, including accurate quantification of the design run-off volumes. Soakaways are not considered suitable for discharge into the London Clay, despite the classification in the Camden SFRA (URS, 2014, Figure 4c) of this area as "Opportunities for bespoke infiltration SuDS" (the least favourable category). This seems unrealistically optimistic, given the low permeability of the near-surface London Clay.

Sewer Flooding:

10.8.8 Thames Water has no records of flooding from public sewers affecting No.190 (see 5.14). 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. Given the lack of any recorded history of sewer flooding affecting this property, the probability of future sewer flooding affecting No.190 is considered to be very low, provided that the sewer system is well maintained and appropriate flood resistance measures are implemented, as set out below.

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

10.8.10 Non-return valves and pumped above ground loop systems must therefore be fitted on the drains serving the basement and the lightwells, in order to ensure that water from the mains sewer system cannot enter the basement when the adjacent sewer might be operating under surcharge. All drains which discharge via the same outfall as the basement must be protected, including those carrying foul water and roof water. A battery-powered reserve pump should be fitted to ensure that the system remains functional during power cuts.

10.8.11 If non-return valves are used without an above-ground loop, then no effluent would at times be able to enter the mains sewer system when the flow in that sewer is sufficient to close the valves. The basement could then be vulnerable to flooding via the gullies in the lightwells and/or other low entry points on the drainage system within the basement. Sufficient temporary interception storage would therefore be required to hold temporarily the predicted maximum volume of water from all relevant sources which discharge via the valve-protected outfall (surface water from roof and lightwells and drains serving the terrace behind the single-storey extension, and foul water) for the duration of the predicted surcharged flows in the sewer. If decking is used in the lightwells, then the area beneath the decking could be used for interception storage, deepened as necessary to provide adequate capacity, though it must be protected from backup of foul sewage. This temporary interception storage would require formal design to ensure satisfactory performance.

10.8.12 If a non-return valve is fitted with a pumped above-ground loop, then the loop must rise high enough above ground level to create sufficient pressure head to open the valve when the 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 above ground level, then temporary interception storage should be provided as recommended above.

10.9 Mitigation

10.9.1 The following mitigation measures should be implemented:

- In the unlikely event that the basement excavations encounter a local deposit of more permeable soils, of sufficient thickness to permit significant flow, then an engineered groundwater bypass should be provided (see paragraph 10.2.6).
- Cracks in load-bearing walls which have weakened their structural integrity should be fully repaired, in accordance with recommendations from the appointed structural engineers, before any underpinning is carried out (10.4.4).
- Transitional underpins should be installed beneath No.190's single-storey rear extension/conservatory (10.4.16).
- Subject to Party Wall Agreement negotiations, transitional underpinning blocks should be included beneath the adjoining walls to No.188 & 192, except where existing cellars would provide sufficient transition (10.4.16).
- An observational approach is recommended to monitor the actual displacements of the flank wall to No.192's rear extension alongside the bored pile wall, in order to allow corrective action to be taken should greater than anticipated displacement be recorded before Category 1 is exceeded (10.6.18).
- Provision of upstands at the top of the retaining walls around the lightwells and installation of raised thresholds to the external doors in the lightwells (10.8.5).
- Use of one or more appropriate types of SuDS: Intervention storage, rainwater harvesting/control, and/or use of a grey water system (10.8.7).
- Non-return valves and/or pumped above ground loop systems should be fitted to the drains serving the basement and lightwells in order to ensure that water from the sewer system cannot enter the basement when the mains sewer is operating under surcharge (see paragraphs 10.8.10 to 10.8.12).

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 in order to check for any tunnelled/deep utilities (10.1.3).

11.3 The marked difference in groundwater levels between the front and rear gardens indicates that the existing footings already block downslope flow of perched groundwater. The proposed basement is considered acceptable in relation to the likely negligible groundwater flow in the natural strata, while sub-surface flows in any Made Ground may only occur where facilitated by service trenches or granular pipe bedding (as seen beneath the cellar). No cumulative impact will be caused to groundwater flow because there are no adjoining/adjacent basements (10.2.1 to 10.2.5).

11.4 In the unlikely event that the basement excavations encounter a local deposit of more permeable soils of sufficient thickness to permit significant flow, then an engineered groundwater bypass would be required (10.2.6).

11.5 The basement will need to be fully waterproofed (10.2.7, 10.2.8). The design groundwater level should be taken at external ground levels. This means that the basement must be able to resist buoyant uplift pressures (un-factored) up to 33kPa (10.2.9, 10.2.10).

11.6 Water entries into the basement excavations are expected, but 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 and 10.4.8).

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

11.8 The basement is expected to be constructed using a combination of underpinning techniques and bored pile walls. A high quality of workmanship and best practice methods of construction and temporary support will be crucial to the satisfactory control of ground movements. Requirements for temporary support are summarised (10.4.2 to 10.4.8).

11.9 The bored pile walls will not be part of a complete piled ‘box’ so substantial raking props onto special foundation blocks are likely to be required, particularly for the rear lightwell excavation. The slabs in the lightwells and basement should be designed to act as permanent high-stiffness props (10.4.10 to 10.4.13).

11.10 Various other guidance is provided in relation to the geotechnical design of the basement’s perimeter walls (10.4.14, 10.4.15).

11.11 Transitional underpins should be installed beneath No.190’s single-storey rear extension, and, subject to agreement under the PWA negotiations, should be considered for some of the load-bearing walls in No's 188 & 192 which adjoin No.190 (10.4.16).

11.12 PDISP heave/settlement analyses indicated that ground movements beneath the perimeter walls of the basement and the internal columns/piers are likely to range from 2mm settlement (in the short term) to 4mm heave in the long term, while up to 6mm heave beneath the basement slab was predicted (Section 10.5).

11.13 Damage category assessments for the perimeter underpins indicated that, provided best practice construction methods are employed, the worst case predicted deformation (in the main rear walls to the adjoining properties on both sides of No.190) is likely to fall on the boundary Burland Categories 1 and 0, termed 'very slight' and 'negligible' respectively (10.6.1 to 10.6.10).

11.14 For the contiguous bored pile walls, providing adequately stiff temporary support will be crucial to minimisation of ground movements alongside the piles. With a moderate support stiffness system for the proposed cantilevered 'bottom-up' construction sequence, the predicted ground movements at the assumed level of the footings to No.192's rear extension, based on empirical relationships in CIRIA Report C580, would be 8.0mm horizontal displacement and 6.4mm settlement, which places the predicted damage in Burland Category 2. This is likely to be an over-estimate because of the short length of this bored pile wall and the much shallower depth of excavation than applied to the data in CIRIA C580, so Category 1 damage is considered more likely (if any). An observational approach is therefore recommended to monitor the actual displacements and allow corrective action to be taken should greater than anticipated displacement be recorded before Category 1 is exceeded (10.6.11 to 10.6.18).

11.15 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).

11.16 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).

11.17 Goldhurst Terrace is close to a former tributary to the Westbourne, is within both a Critical Drainage Area and the Goldhurst LFRZ, and was recorded as having flooded during both the 1975 and 2002 events; however, the latest flood modelling by the Environment Agency and Camden SFRA gave a 'Very Low' risk of flooding by surface water to No.190 and the surrounding area/highways. This is the lowest, national background level of risk. Appropriate flood mitigation measures are recommended (10.8.2 to 10.8.5).

11.18 The basement's front lightwell will increase the area of hard surfacing slightly, so either an equal increase in soft landscaping should be created elsewhere within the site, or one or more appropriate, formally designed SuDS system should be implemented in order to control surface water discharge or store the water for re-use (10.8.6, 10.8.7).

11.19 Thames Water has no records of flooding from public sewers affecting No.190, so the probability of future sewer flooding affecting No.190 is considered to be very low, provided that the sewer system is well maintained and appropriate flood resistance measures are implemented (10.8.8).

11.20 Non-return valves and pumped above ground loop systems should be fitted to the drains serving the basement, lightwell and lower terrace. Temporary interception storage may also be required for the predicted maximum volume of discharges (from all sources) via the protected outfall pipe, for the duration of any predicted surcharged flows in the sewer; formal design would be required (10.8.10 to 10.8.12).

11.21 The mitigation measures recommended in various parts of Sections 10.2 to 10.8 have been 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.

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

BS 8002 (1994) Code of Practice for Earth retaining structures. British Standards Institution.

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) 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.

Elrington CR et al (1989) 'Hampstead: Frognal and the Central Demesne', *A History of the County of Middlesex: Volume 9: Hampstead, Paddington*. pp. 33-42

Gaba AR, et al (2003) Embedded retaining walls – guidance for economic design. CIRIA Report C580.

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 (2016) NHBC Standards, Chapter 4.2, Building Near Trees.

NHBC (2016) NHBC Standards, Chapter 5.4, Waterproofing of basements and other below ground structures.

URS (2009) Camden Infrastructure Study: Utilities and Physical Infrastructure Needs Assessment.

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