Report

Basement Impact Assessment

24 Heath Drive,
London, NW3 7SB

for

Sarah Bard

Ref: GGC17597/R2.3 March 2018

Gabriel GeoConsulting Limited
Henwood Pavilion, Henwood, Ashford, TN24 8DH
Company No. 6455714, registered in England and Wales. Registered office: Highfield House, TN17 4EH
Tel: 01580 241044
e: info@gabrielgeo.co.uk www.gabrielgeo.co.uk

Project: **Basement Impact Assessment**

Site: **24 Heath Drive, London, NW3 7SB**

Client: **Sarah Bard**

Report Status: **FINAL**		
Role	**By**	**Signature**
Desk study, factual reporting, and screening & scoping by:	Roberta McAlister BSc MSc FGS	R McAlister
Impact Assessments by:	Roberta McAlister BSc MSc FGS and Keith Gabriel MSc DIC CGeol FGS UK Registered Ground Engineering Adviser	R McAlister K.R. Gabriel
Report checked and approved by: & Subterranean (groundwater) flow aspects approved by:	Keith Gabriel MSc DIC CGeol FGS UK Registered Ground Engineering Adviser	K.R. Gabriel
Slope/ground stability and surface flow & flooding aspects approved by:	Mike Summersgill MSc CEng MICE C.WEM FCIWEM	

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

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

Contents Page

Appendices

Appendix A Photographs

Appendix B Desk Study Data – BGS Boreholes

Appendix C Factual Report on Ground Investigation (Ref: 17595/R1)

Appendix D Desk Study Data – Geological Data (Groundsure Geo Insight)

Appendix E Desk Study Data – Environmental Data (Groundsure Enviro Insight)

Appendix F Desk Study Data – Historical OS Maps – Large Scale and Small Scale

Appendix G PDISP Heave/Settlement Analyses Figures

Appendix H HR Wallingford – Surface water storage requirements for proposed site – Site area 0.013ha

- Without mitigation
- With mitigation

1. INTRODUCTION

1.1 This Basement Impact Assessment has been prepared in support of a planning application to be submitted to the London Borough of Camden (LBC) for works including the extension of the existing partial footprint basement beneath 24 Heath Drive, NW3 7SB. Further details of the proposed scheme are provided in Section 3. The assessment is in accordance with the requirements of the London Borough of Camden (LBC) Development Policy DP27 in relation to basement construction, and follows the requirements set out in LBC's guidance document CPG4 'Basements and Lightwells' (July 2015).

1.2 Preparation of this assessment has been supervised/undertaken 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 property was undertaken on Friday 7th October 2016, prior to the site specific ground investigation which was then undertaken on 14th and 25th November 2016. Photos from the preliminary visit are presented in Appendix A. Desk study data have been collected from various sources, including geological data, environmental data and historical maps from Groundsure which are presented in Appendices D, E and F. 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 planning application have been considered:

- **Studio Kyson (Architects):**

  <u>Existing:</u>

Drg No.508-16/0500	Site Location & Block Plan
Drg No.508-16/999	Existing Basement Floor Plan
Drg No.508-16/1000	Existing Ground Floor Plan
Drg No.508-16/1100	Existing Front Elevation
Drg No.508-16/1101	Existing South-West (Side) Elevation
Drg No.508-16/1102	Existing Rear Elevation
Drg No.508-16/1103	Existing North-East (Side) Elevation
Drg No.508-16/1200	Existing Site Section
Drg No.508-16/1201	Existing Section A
Drg No.508-16/1202	Existing Section B

  <u>Proposed:</u>

Drg No.508-16/0501	Site Location & Block Plan
Drg No.508-16/1999/A	Proposed Basement Floor Plan
Drg No.508-16/2000/A	Proposed Ground Floor Plan

Drg No.508-16/3000	Proposed Front Elevation
Drg No.508-16/3001	Proposed South-West (Side) Elevation
Drg No.508-16/3002	Proposed Rear Elevation
Drg No.508-16/3003	Proposed North-East (Side) Elevation
Drg No.508-16/4000	Proposed Section A
Drg No.508-16/4001	Proposed Section B
Drg No.508-16/4002	Proposed Section C
Drg No.508-16/4003	Proposed Section D

Stripping Out:

Drg No.508-16/1499/A	Strip-Out Basement Floor Plan
Drg No.508-16/1500	Strip-Out Ground Floor Plan
Drg No.508-16/1600	Strip-Out Front Elevation
Drg No.508-16/1601	Strip-Out South-West Elevation
Drg No.508-16/1602	Strip-Out Rear Elevation
Drg No.508-16/1603	Strip-Out North-East Elevation
Drg No.508-16/1700	Strip-Out Section A
Drg No.508-16/1701/A	Strip-Out Section B

- **Form Structural Design Ltd:**

Drg No.162637/A(28)01/P3	Proposed Cross Sections A-A & B-B
Drg No.162637/A(28)02/P3	Proposed Cross Sections C-C & D-D
Drg No.162637/A(28)03/P3	Proposed Cross Sections E-E & F-F
Drg No.162637/L(17)01/P2	Proposed Plant Level Plan
Drg No.162637/L(17)02/P3	Proposed Basement Plan
Drg No.162637/L(23)01/P2	Proposed Lower Ground Floor Plan
Drg No.162637/L(23)02/P4	Proposed Ground Floor Plan
Drg No.162637/SK011-1/B	Indicative SLS Loading for Geo Analysis (Basement, Pool & Plant Level)
Drg No.162637/SK011-2/A	Indicative SLS Loading for Geo Analysis (Lower Ground Floor Level)

Construction Method Statement (DRAFT) dated 19th December 2017

- **Cowley White (Landscape Designers):**

Drg No.001-REV C	Plan: Landscape Design

Design Statement: Landscape
Materials and Specifications
Planting Schedule

- **Gleeds Building Surveying Ltd:**

Drg No.LNBS0490_FPB_Prov	[Existing] Basement Plan
Drg No.LNBS0490_FP00_Prov	[Existing] Ground Floor Plan
Drg No.LNBS0490_FP01_Prov	[Existing] First Floor Plan
Drg No.LNBS0490_FP02_Prov	[Existing] Second Floor Plan
Drg No.LNBS0490_FPR_Prov	[Existing] Roof Plan
Drg No.LNBS0490_T01	Topographical Survey

- **Eight Associates (Sustainability Consultants):**
  Tree Survey and Tree Constraints Plan (1948 24 Heath Drive Tree Survey Report 1610-31sc), containing:
  Writtle Forest Consultancy: 161002_Figure 001 Rev.1 Tree Constraints Plan.
  Arboricultural Implication Assessment (1948 24 Heath Drive AIA 1802-06rc)
  Arboricultural Method Statement (1948 24 Heath Drive AMS 1802-06rc).

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

1.5 Instructions to prepare this Basement Impact Assessment (BIA) were received via email on 23rd January 2018 from the Client's project manager, Sebastian Potiriadis from The Estate Office Shoreditch.

2. THE PROPERTY, TOPOGRAPHIC SETTING AND PLANNING SEARCHES

2.1 24 Heath Drive is a Grade II listed, detached, three-storey house with a cellar (see cover photo), located within the Redington and Frognal conservation area in the London Borough of Camden. Heath Drive is located between Redington Road to the north-east, Finchley Road (A41) to the south, and is transected by Kidderpore Avenue/Bracknell Gardens to the south-west of the property. No.24 is situated on the south-east side of Heath Drive, adjacent to the junction between Heath Drive and Ferncroft Avenue, between No.23 to the north-east and No.25 to the south-west. To the rear (south-east), No.24 is bounded by the rear gardens of No's 2 and 4 Oakhill Avenue (location as shown in Figure 1, property setting as cover photo and Photo 2 in Appendix A).

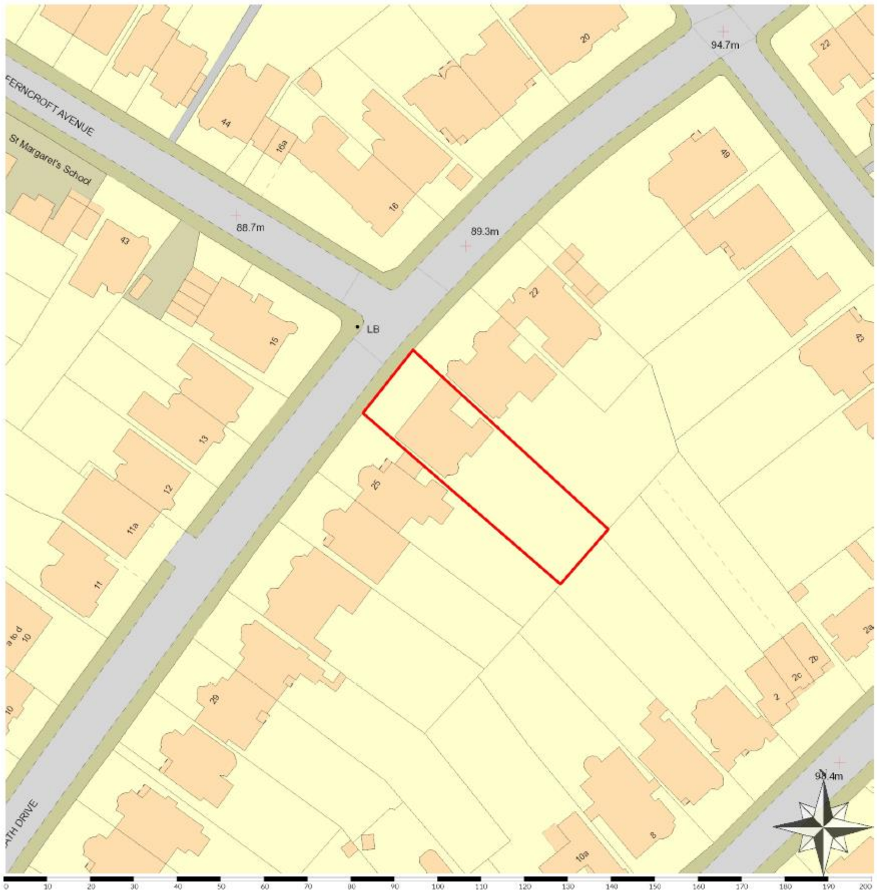

Figure 1: Extract from 1:1,250 OS map (not to scale) with the site outlined in red.
Ordnance Survey © Crown copyright 2016. All rights reserved. Licence number 100051531.

2.2 Externally, there is a paved driveway which extends across the full width of the front of the property. This area is bounded by flower beds/planting areas, located alongside the front wall of the house (either side of the front entrance), between the parking area and the Heath Drive footway (with two gaps at either end for access), and alongside both the 24/25 and 24/23 boundaries. The flower bed alongside the upslope 24/23 boundary is raised and supported by a low stone retaining wall, which was failing in the vicinity of the garage. A narrow side passage runs alongside the south-western side of the house, between No's 24 and 25, which connects the front drive to the rear garden. This side passage is surfaced mainly with asphalt, with crazy paving (in poor condition) in front of the side gate. The passageway is supported by a low retaining wall, on top of which sits a close-boarded wooden fence, which separates it from the adjoining side passage and single-storey garage to No.25 at lower level. On the north-eastern side of the house there is a two-storey side extension, which includes a garage. Crack damage was noted in the front and rear elevations of the garage (see Photo 10). To the rear of this extension is a small courtyard and another side passage, which also leads to the rear garden. This courtyard and side passage are surfaced with paving slabs, and, due to the difference in ground levels, there is a stone retaining wall/rockery at the boundary between No's 24 & 23, on top of which sits another close-boarded wooden fence.

2.3 To the rear of the house is large, split-level garden. The lower level is set two steps below the ground floor level of the house, and consists of a narrow, paved patio area. This patio is bounded by an irregular stone retaining wall along its south-eastern side, with retained heights of up to around 0.73m measured from Gleeds Building Surveying Ltd 'Topographic Survey' (Drg no. LNBS0490_T01). Three short flights of steps lead from this lower patio area, to a mid-level lawn, past large flower beds which contain various shrubs and semi-mature trees. Beyond this lawn area, the rearmost parts of the garden were largely overgrown, and included several large trees. There was also a stepped paved pathway which lead up towards an upper terrace alongside the site's rear boundary with No's 2 & 4 Oakhill Avenue. This boundary consists of a concrete crib retaining wall, on top of which sits a wooden fence (approximately 5.2-5.7m above the level of the patio). The upper parts of the garden, as well as the lawn and flower beds all fall gently towards the rear of the house (towards the north-west), which is broadly consistent with the contours in Figure 1 (see also paragraph 2.9 below).

2.4 Reference to the first available historical Ordnance Survey (OS) map dated 1870 (presented in Appendix F) shows that none of the immediately surrounding road network, including Heath Drive carriageway, nor any of the properties in this area (including No.24) had been built prior to that date, and that this part of Hampstead remained relatively un-developed. This map also shows that the site on which the property now lies appears to have formed farmland, and that tributaries of the River Westbourne (not labelled on these maps) ran directly in front (north-west) of the site, and just to the north-east (see paragraph 5.1). The Finchley Road carriageway (A41) and some of the properties which front onto it to the south of the site had already been built prior to 1870; Kidderpore Hall stood approximately 250m to the west of No.24's site, and the town centre of Hampstead, to the east of the site, was well developed by that time.

2.5 Development of the area began with the construction of some of the surrounding road network, including West Hampstead Road (now Heath Drive), Redington Road and Kidderpore Avenue, between publication of the 1874 and 1894 OS maps. The River Westbourne's tributaries can no longer be seen at surface on the 1894/1896 maps (see Section 5). Kidderpore Hall had become Westfield College and a new east wing had been built, along with a covered public water supply reservoir immediately upslope of the College, both of which are still present today. Development of this area then continued between publication of the 1896 and 1915 OS maps, with the construction of the Ferncroft Avenue carriageway, all of the properties on Heath Drive, including No.24, and most other properties within the surrounding area, including those on Redington Road and Ferncroft Avenue. Few significant changes to the site and surrounding area can then be observed between the 1968 and current OS maps. The only change evident to No.24 Heath Drive itself, since it was built, was the addition of the side extension/garage on the north-eastern side of the house between publication of the 1915 and 1953 OS maps. No.23's front bays were added during the same period, while a single-storey garage was added to No.25, adjoining the 24/25 boundary, between 1953 and 1966.

2.6 Upslope of No.24, No.2a Oakhill Avenue was constructed between 1953 and 1966. The OS maps also indicate that No.2 was demolished and replaced with a terrace of three houses between 1978 (revision date for the 1979 map) and 1991, however the online planning records show that permission was granted in 1970 indicate that works were in progress by 1971.

2.7 The bomb map for Hampstead indicates that no hits were recorded on properties in this part of Heath Drive, and that the closest recorded hit occurred on the north-eastern side of Redington Road, around 120m to the east of No.24. The London County Council Bomb Damage Map (LTS, 2006) for this area indicates that none of the houses on Heath Drive, including No.24, suffered any bomb damage, and that the hit on the north-eastern side of Redington Road caused only 'Blast damage, minor in nature' to No's 26 & 28 Redington Road.

Topographic Setting:

2.8 24 Heath Drive is situated towards the base of the south-east side of a valley, which itself falls towards the south-west. This valley feature, illustrated by the contours in Figure 2, has been carved out by the upper course of the River Westbourne, one of the 'lost' rivers of London (Barton & Myers, 2016). Since the Heath Drive carriageway is located broadly at the base of this valley, it falls towards the south-west. The front driveway/garden to No.24 is also located near the base of this valley feature, thus it also falls predominantly towards the south-west (although this varies locally), whereas the rear garden, which is located further up the south-east side of this valley, falls predominantly towards the north-west (see paragraph 2.3 above).

2.9 The 90m contour line runs approximately north-east/south-west through the site, and the boundaries of the site lie entirely within the 85m and 95m contour lines. The contours on Figure 2 indicate that the base of the valley has an overall slope angle within the vicinity of No.24 of approximately 2.6° towards the south-west (calculated between the 80m and 95m contours), and the rear garden of No.24 has an overall slope angle of approximately 5.7° towards the north-west (calculated between the 90m and 95m contours), not taking into consideration any localised re-profiling of the slope. However, Gleeds Building Surveying Ltd 'Topographic Survey' (Drg no. LNBS0490_T01) allows site-specific slope angles to be measured. When localised levelling works are taken into account, slope angles in the rear garden range from approximately 5.5° to 19° towards the north-west. This is confirmed by slope modelling in Figure 16 of the Camden GHHS (Arup, 2010), an extract from which is presented in Figure 3 (see Section 4), which indicates that slope angles within the immediate vicinity of 24 Heath Drive range from < 7°, to 7-10°, to >10°.

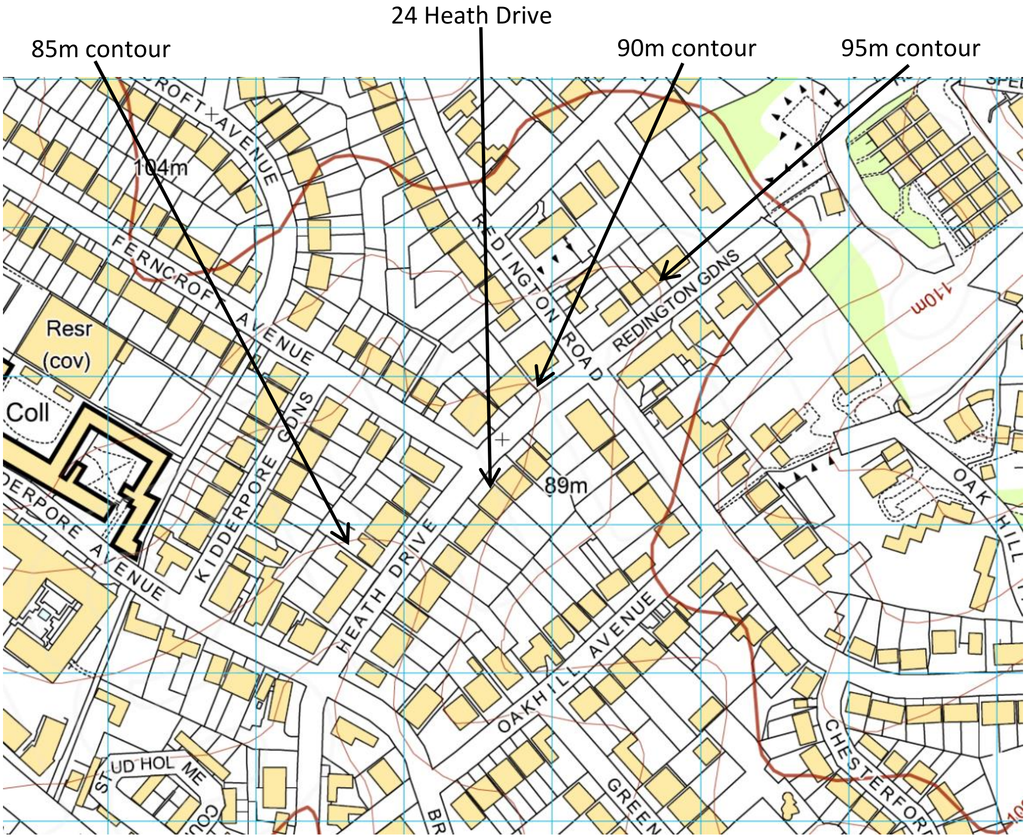

Figure 2: Enlarged extract from 1:5,000 Ordnance Survey map showing site location. Ordnance Survey © Crown copyright 2016. All rights reserved. Licence number 100051531.

<u>Planning Searches:</u>

2.10 A search was made of planning applications on Camden Council's website, in order to obtain details of any other basements which have been planned, constructed or extended in the vicinity of the property. The search included the properties adjacent to and opposite 24 Heath Drive, elsewhere in the vicinity on Heath Drive, as well as those to the rear, on Oakhill Avenue. No applications relating to the extension of the existing basement, or relating to significant other alterations/extensions were found for the property itself. There were however several applications found relating to significant alterations/extensions to some of the adjacent properties on Heath Drive, and relating to basement extensions/construction at some of the properties opposite and to the rear of No.24, the most relevant of which are listed below:

- **No.23 Heath Drive**: Application (PW9802906R1) for the "*erection of single storey side and rear extensions and alterations to existing garage*" was granted planning permission on 24th March 1999. Drawings of the proposed scheme were found on the website.
- **No.25 Heath Drive**: Application (2011/1468/P) for "*rebuilding and enlargement of side extension to existing dwelling (Class C3)*" was granted planning permission on 17th May 2011. Plans for the proposed scheme were found on the website, but the scheme did not appear to have been implemented. Another application (P9601296) involving "*the erection of a timber and glass conservatory on a brickwork wharf* [dwarf] *wall*" was granted conditional planning permission on 28th June 1996. Drawings of this proposed scheme were also found on the website.
- **No.14 Heath Drive**: Application (2009/5153/P) for "*change of use and conversion from three self-contained flats into two units (1x 1 bed at basement, 1x 6 bed maisonette on ground, first and second floor levels), and associated additions and alterations, including enlargement of basement, creation of a sunken garden, rear extension at ground floor level, including terrace above rear extension*" was granted planning permission on 21st January 2010. Plans and elevations of the proposed scheme were found on the website.
- **No.18 Heath Drive**: Application (2014/4824/P) involving "*alterations to side and rear elevations including the formation of new window openings, further excavation of basement level with excavation of lightwells at front and rear elevations*" was granted planning permission on 6th October 2014. Plans, elevations and a Basement Impact Assessment (BIA) for the proposed scheme were found on the website.
- **2 Oakhill Avenue (rear)**: Application (2013/6162/P) involving "*basement excavation and extensions to rear and side in connection with conversion of existing single family dwelling into 2 x 3 bedroom maisonettes (Class C3)*" was granted planning permission on 16th February 2016. Plans, elevations, a BIA and a Ground Movement Assessment (GMA) for the proposed scheme were found on the website.

It should be noted however that while the applications outlined above were granted planning permission, no information is available detailing whether or not construction subsequently went ahead.

3. PROPOSED BASEMENT

3.1 The proposed works at 24 Heath Drive for which planning permission will be sought, as shown in Kyson's drawings (see Section 1.4), will comprise:

- An extension to the existing basement, to include excavation for increased ceiling height, and lateral expansion beneath the full footprint of the main part of the existing house (i.e. excluding the existing garage extension, utility and rear bay window). This proposed basement will include a swimming pool (on its north-eastern side), which will extend beyond the rear wall of the house beneath the rear garden (with a walk-on roof light proposed), and will be set below the main basement level. A 'sunken pit' level will be created beneath the Lounge in the northern corner of the basement, with a suspended floor slab above at the level of the main basement, which will house a plant room and pool attenuation tank.
- Demolition of the existing two-storey extension and single-storey projection on the north-eastern side of the house, and the construction of a new part single-storey, part two-storey extension in their place, which will extend from front to rear of the house. This extension will comprise a new garage at the front, which will extend to the north-eastern boundary of the site. The remainder of the extension will extend between approximately 2.6m from the north-eastern flank wall of the house, leaving a narrow (1.05m to 1.24m wide) access path between the house and the site's north-eastern boundary, which will lead from the door in the back wall of the garage to the rear garden. The floor level in the rear part of this extension is shown on Kyson's drawings to step up by 0.51m from the garage.
- Demolition of the existing rear bay on the south side of the house, and the construction of a new single-storey rear extension, which will extend approximately 5.7m from the main rear wall of the house. The proposed extension will include a flat roof.
- Landscaping of the rear garden, as shown in Cowley White's landscaping design drawing (Drg No.001-REV C), which will include excavations into the rockery retaining wall and large flower beds which separate the existing rear patio area from the rear lawn, and the formation of four new terraced levels and associated retaining walls.
- Additional minor alterations including the continuation of the main stairwell down to basement level, alterations to fenestration and the construction of dormers at second floor level in the main roof. See Kyson's stripout and proposed scheme drawings as listed in Section 1.4 for further details.

3.2 Several young to mature trees will be removed from the site to facilitate the works described above, and several more will be planted (see Section 10.4).

3.3 The proposed section drawings by Kyson (Sections A, B & D; Drg No's 508-16/4000, 4001 & 4003) and Form SD (Sections A-A to F-F; Drg No's 162637/A(28)01-03) provide finished floor levels (FFL's) and structural slab levels (SSL's) for the main basement level, the pool level and the sunken plant pit, which are included in Table 1 below. The underpin and retaining wall bases will be 450mm thick throughout, and the basement slab will be 300mm thick with a 225mm thick anti-heave void former (Pecavoid RD Range) beneath. The proposed founding levels (formations) throughout the basement, pool level and sunken pit have been calculated using these dimensions, and are also included in Table 1.

3.4 Existing floor/ground levels were found to vary across the site where the proposed basement will be located, from 86.27m AOD where the existing lower ground floor is located, up to approximately 89.0m AOD in the rear garden above the retaining wall (see Gleeds Building Surveying's Basement Plan, Ground Floor Plan and Topographical Survey; Drg No's LNBS0490_FPB, FP00 & T01). These levels give rise to varying excavation depths down to the proposed formation levels, which are also summarised in Table 1 below.

Table 1: Summary of proposed formation levels and proposed excavation depths

Location	**Proposed Finished Floor Level (m AOD)**	**Proposed Structural Slab Level (m AOD)**	**Proposed Formation Level (m AOD)**	**Existing Level (m AOD)**	**Proposed Excavation Depth (m)**
Basement Level (Underpin & Retaining Wall Bases)	84.69	84.50	**84.05**	86.27-88.35	**2.22-4.3**
Basement Level (Slab)	84.69	84.50	**83.975**	87.5-88.35	**3.525-4.375**
Pool Level	83.39	83.06	**82.61**	86.27-89.0	**3.66-6.39**
'Sunken Pit' (Plant Room)	82.08	81.93	**81.48**	86.27-88.10	**4.79-6.62**

4. GEOLOGICAL SETTING

4.1 Mapping by the British Geological Survey (BGS) indicates that the site is located on the boundary between the Claygate Member of the London Clay Formation, which is shown within the central and rear parts of the site, and the underlying London Clay Formation proper, which underlies the rest of the site. Figure 3 (below) presents an extract from Figure 16 of the Camden GHHS (Camden Geological, Hydrogeological and Hydrological Study by Arup, November 2010) which illustrates both the geology of the area, and highlights areas with slope angles greater than 7°. The map in Section 1.3 of Groundsure's GeoInsight report (see Appendix D, page 12) suggests that the boundary is slightly lower, crossing No.24's front driveway. Although the boundary between these two strata is mapped as running approximately through the site, LBC's CPG4 guidance document advises that *"boundaries are indicative and should be considered to be accurate to 50m at best"*. This is particularly applicable to the Claygate-London Clay boundary because the uppermost Unit D of the London Clay proper contains silt/sand horizons similar to those in the Claygate Member, and because there are few exposures at surface where the boundary can be identified. It can be seen that the boundary as mapped approximately follows the contours of the valley in this area, as described in paragraph 2.8 and illustrated in Figure 2.

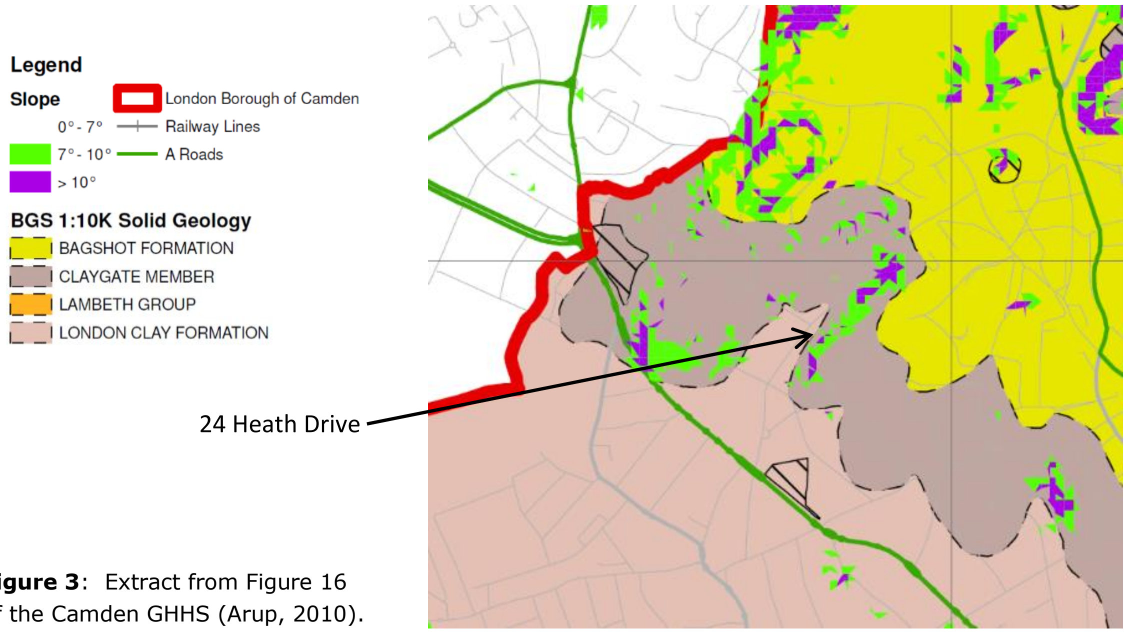

Figure 3: Extract from Figure 16 of the Camden GHHS (Arup, 2010).

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

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

4.4 The London Clay Formation is well documented (e.g. Ellison et al., 2004) as consisting of over-consolidated, firm to very stiff, grey to blueish grey, fissured, bioturbated, slightly calcareous, silty to very silty clay. It contains well-graded (ie: poorly sorted, with a range of particle sizes) beds of clayey silt to silty fine sand, pyrite, and variously sized carbonate concretions (claystones) which sometimes obstruct boreholes and piles. The London Clay Formation is known to have a weathered, oxidised zone at its top (usually between 3m and 6m thick where the London Clay is not overlain by other strata). This weathered zone and the transitional zone below are typically brown in colour, often becoming grey-brown or chocolate brown with depth, and contains selenite (a form of gypsum), which is aggressive to buried concrete. The clays of the London Clay Formation are typically of high or very high plasticity and high volume change potential. As a result, the clays undergo considerable volume changes in response to variations in natural moisture content (they shrink on drying and swell 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 trees whose roots abstract moisture from the clays. The clays will also swell when unloaded by excavations such as those required for the construction of basements.

4.5 The London Clay Formation is known to reach thicknesses of between 90m and 130m below parts of London, therefore exceeds the depth considered relevant to the proposed basement. As a result, the geology beneath the London Clay Formation is not considered further.

4.6 The results of the BGS classifications of six natural ground subsidence/stability hazards are presented in the Groundsure Geo Insight report (see Appendix D, Section 4); all indicate "Negligible" or "Very low" hazard ratings with the exception of 'Shrink – Swell Clay' for which a "Moderate" hazard rating is given, which reflects the outcrop of the London Clay Formation at/near to surface. In spite of the BGS hazard classification of "Very low" for landslides on site, Figure 17 of the Camden GHHS (Arup, 2010), which is based on Forster et al. (2003), indicates that the site lies within an area of significant landslide potential due to its location on the boundary between the Claygate Member and London Clay Formation.

4.7 The Groundsure Geo Insight report (Appendix D, Sections 2, 3 & 7) records:

- No historical surface ground working features within 250m of the site, and no current ground workings within 1000m of the site (see App. D, Sections 2.1 & 2.3).
- Various historical underground working features within 1000m of the site. These are described as 'Tunnel' or 'Tunnels', and are located 815-824m and 991-998m to the south-east of the site (see App. D, Section 2.2), so are irrelevant to the proposed basement.
- No historical or active mines or natural cavities within 1000m of the site (see App. D, Sections 3.1-3.10).
- No historical or active railways or tunnel features within 250m of the site (see App. D, Sections 7.1-7.4).

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

4.8 A search of the BGS borehole database was undertaken for information on previous ground investigations and any wells in the vicinity of the site. Few BGS boreholes were identified close to the site, and at a relevant height above Ordnance Datum, however several boreholes were found within close proximity of the site during a wider search of planning applications on the London Borough of Camden's website; the strata depths in a selection of these boreholes are summarised in Table 2. Reference should be made to the logs in Appendix B for full strata descriptions. General points of note from these boreholes were:

- The 15.0m deep BH4 at 2 Oakhill Avenue recorded silty sandy CLAY down to 84.13m AOD, which is below the level of No.24 Heath Drive's site; all these clays were interpreted by GEA to belong to the Claygate Member to the base of the borehole, whereas the mapping would suggest that this borehole should have passed into the London Clay (unless an unidentified fault is present).
- Neither the logs of the boreholes drilled at 4 Templewood Avenue nor those drilled at 69 Redington Road recorded the ground level at the top of the boreholes relative to Ordnance Datum, thus only the depths below ground level are given. Levels estimated from the site plan for the 4 Templewood Avenue boreholes were 98.3m and 97.8m AOD respectively for BHs 1 & 2, which would place the Claygate-London Clay boundary at 86.3m and 84.8m AOD respectively.
- Considerable variation in strata depths can be seen between boreholes drilled on the same site, which may in part be sue to the variable topography in this area.
- The level of No.24's site ranges from 87.05m to 93.8m AOD.

Table 2: Summary of Strata in Nearby Boreholes						
Strata (abbreviated descriptions)	**Depths (m) and levels (m AOD) to base of strata**					
	2 Oakhill Avenue (BH1-4)		**10A Oakhill Avenue (WS1-3)**		**4 Templewood Avenue (BH1 & BH2)**	**69 Redington Road (BH1 & BH2)**
GL (mAOD)	Depth	**Level 99.27-96.60**	Depth	**Level 96.15-92.60**	Depth	Depth
Date drilled	25-28/03/13		02/05/13		14/07/10 & 07/12/10	11/06/12
Made Ground/ Topsoil	0.40-1.10	**98.63-96.20**	0.15-0.70	**95.85-92.45**	0.30	0.30-1.50
Soft to firm, orange-brown to grey, silty to very silty, sandy to very sandy CLAY, with occasional gravel (Head Deposits?)	-/1.00	**-/95.60**	-	**-**	1.75-1.80	-/0.9
Soft to stiff, generally brown to orange brown mottled grey, silty sandy CLAY, with pockets/partings of silt and fine sand ("Weathered" Claygate Member)	4.30-6.20	**93.07-92.00**	4.90-5.95	**90.20-87.70**	12.00-13.00	1.8 – >5.2
Firm to stiff, grey, silty sandy CLAY, with occ'nl pockets of silty fine sand (Claygate Member)	>7.00/ >15.00	**<92.27/ <84.13**	-	**-**	-	-
Stiff to very stiff, fissured, slightly sandy, silty CLAY ("Weathered" London Clay Fm)	-	**-**	-/ >5.00 - >7.00	**-/ <89.15- <87.60**	-	-/3.3
Stiff to very stiff, mid grey to grey blue, slightly sandy, silty to v. silty CLAY (London Clay Fm)	-	**-**	-	-	>20.00	-/>6.0
Groundwater Strikes	2.0/1.8/ 3.8/7.40	**94.6/95.1/ 95.5/91.73**	4.4/-/3.6	**91.75/- /89.00**	6.0,8.0,10.0++/8 .0,14.0,18.0	-
Groundwater Standing Levels	1.6/2.0/ NA/5.2	**95.0/94.9/ NA/93.93**	3.55/4.55/ 2.20	**92.60/9 0.2/ 90.4**	**5.9,7.0,4.5/6.0**	-

5. HYDROLOGICAL SETTING (SURFACE WATER)

5.1 Barton and Myers' map (2016) showing the 'lost' rivers of London indicates that the former course of one of the headwater branches of the River Westbourne once flowed in the base of the valley which is now occupied by Heath Drive, as illustrated in Figure 4 below. This stream flowed from north-east to south-west in this area. The 1870 historical OS map shows this stream flowing just to the north-west of the site, and shows two tributaries, one to the north-west merging with this stream downslope of No.24's site (so not as shown in Figure 6) and another a short distance to the north-east of the site. All three streams had disappeared by the survey for the 1894 OS map at 1:10,560 scale (see Sections 2.5 and 2.6), most likely indicating they had been culverted when development of the area began. Barton and Myers (2016) describe the River Westbourne as having been diverted into the Middle Level Interceptor Sewer when it was culverted, with storm flows having been diverted into the Ranelagh Sewer.

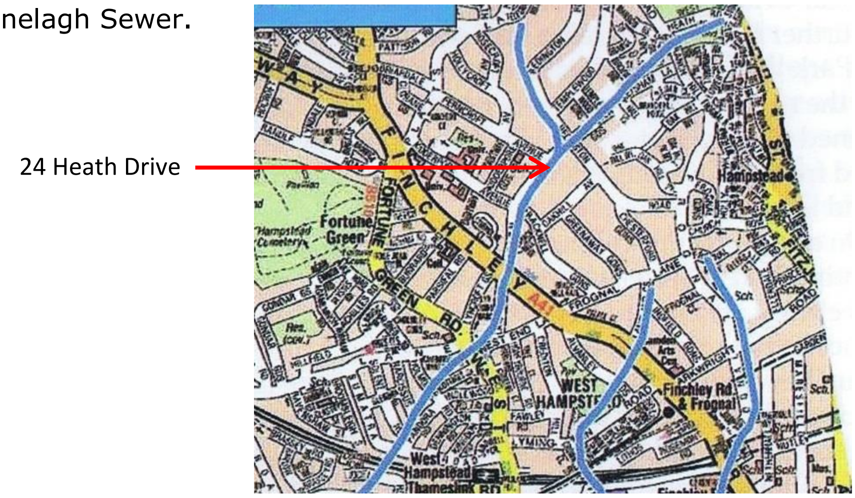

Figure 4: Extract from Map 21 of Barton & Myers' Lost Rivers of London (2016) – 'The course of the Westbourne through Hampstead to Maida Vale'.

5.2 Figure 12 of the Camden GHHS (Arup, 2010) shows that the closest surface water feature to the site is an extremely small pond feature located approximately 300m to the south-west of the site, with a larger pond/lake (Whitestone Pond) located approximately 750m to the north-east. Figure 14 of the GHHS shows that the site is not within any of the Hampstead Heath surface water catchments; the closest being the Golders Hill Chain Catchment approximately 400m to the north and the Hampstead Chain Catchment approximately 800m to the north-east of the site.

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

- There are no rivers (or more specifically "Detailed River Network entries") within 500m of the site, and no surface water features within 250m of the site (App. E, Sections 6.10 & 6.11 respectively).

- There are no surface water abstraction licences within 2000m of the site (App. E, Section 6.4).
- There are no flood storage areas, flood defences, or areas which benefit from flood defences within 250m of the site (App. E, Sections 7.4, 7.5 & 7.6).

5.4 Mapping by the Environment Agency (EA) on the Government's 'Flood Map for Planning' website indicates that the site lies within Flood Zone 1, which is defined as land having a low probability of river and sea flooding, with a less than 0.1% (1 in 1000) chance of such flooding occurring each year, **not** taking into account the presence of any flood defences. The closest Flood Zone 2/3 (0.1-1%/>1% chance of river flooding) is located approximately 3km to the north-west of the site. According to the EA's 'Long Term Flood Risk Information mapping, also available on the Government's website, the site has a 'Very Low' risk of flooding from rivers and the sea (with a less than 1 in 1000 [0.1%] chance), which **does** allow for the beneficial effects of any flood defences and the possibility that they may be over-topped or breached. This mapping also shows that the site does not fall within an area at risk of reservoir flooding.

5.5 The gentle fall of the footway away from the property, together with the south-westwards fall of Heath Drive are likely to prevent surface water on the carriageway from reaching the property under most conditions. The low retaining walls and hedges which separate the front parking/amenity area from the adjoining front parking/amenity areas to No.23 & 25, are unlikely to prevent surface water flow from or to these areas. Thus, the surface water catchment for the front parking/amenity area will include the area immediately upslope of the site, as well as direct rainfall. The front parking/amenity area is predominantly surfaced with paving slabs, so infiltration will be limited or nil in most of this area, although infiltration is likely to occur in the flower beds and soft landscaped areas (see Photo 8).

5.6 The access path on the north-eastern side of the house, to the rear of the garage, is separated from the adjoining site of No.23 Heath Drive by a stone retaining wall/rockery. These are unlikely to prevent surface water flow to No.24's side passage, thus the surface water catchment for this area will include the adjoining areas immediately upslope, as well as direct rainfall. This area is also predominantly surfaced with paving slaps, so infiltration will be limited or nil in most of this area, although some infiltration is likely to occur within the flower beds (see Photos 7 & 8). The narrow access path on the south-west side of the house is supported by a low retaining wall, on top of which sites a wooden fence, thus surface water run-off is likely to be able to flow from this area, to the adjoining side passage to No.25 located downslope. Its surface water catchment is likely to include the adjoining areas of No.24's own rear garden, as well as direct rainfall. Although this area is surfaced with asphalt, it was observed to be in poor condition, thus some infiltration may occur within this area.

5.7 No.24's rear garden is separated from the adjoining rear gardens to No's 23 & 25 by wooden fences, thus it is likely to receive some excess overland run-off from the adjoining upslope rear garden of No.23. The rear garden to No.24 predominantly falls towards the rear of the house, with only a very slight fall across the garden, towards the south-west. The adjoining rear garden to No.25, located downslope, has been terraced, so is broadly level, and if the rear garden to No.23 has also been terraced, then the amount of run-off which No.24's rear garden receives from the upslope rear garden of No.23 may be minimal. To the south-east, the rear garden is separated from the adjoining rear gardens of No's 2 & 4 Oakhill Avenue by a concrete crib retaining wall, on top of which sits a wooden fence. These are also unlikely to prevent surface water flow from these areas, however the amount of surface run-off which No.24's rear garden receives may be minimal, if these rear gardens have also been terraced and levelled (which is likely).

5.8 Both Figure 15 of the Camden GHHS (Arup, 2010) and Figure 3v of the Camden Strategic Flood Risk Assessment (SFRA) (URS, July 2014) show that Heath Drive did not flood during either the 1975 or 2002 surface water flood events, as illustrated in Figure 5 below. Ferncroft Avenue is shown as having flooded during the 1975 event. These figures record the whole length of affected roads as having flooded, though the floods generally affected only a short length of each affected road; in the case of Ferncroft Avenue, localised flooding probably occurred at its lowest points, which lie at the road's south-eastern and north-western ends, where it joins Heath Drive and Platt's Lane respectively. Since Platt's Lane is shown as having flooded during both the 1975 and 2002 flood events, it seems likely that the flooding on Ferncroft Avenue was at its north-western end.

Figure 5: Extract from Figure 3v Rev.1 of the Camden Strategic Flood Risk Assessment (SFRA) (URS, July 2014) showing risk of flooding from surface water.

5.9 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 below. 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. The EA's definitions of these risk categories are:

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

5.10 The EA's modelling indicated that the risk of flooding from surface water at 24 Heath Drive is 'Very Low' (see Figure 6 below), which is the national background level of risk. The adjacent properties on the south-east side of Heath Drive, along with those to the rear of the property on Oakhill Avenue are also within an area at a 'Very Low' risk of flooding from surface water. Directly in front of No.24, within the Heath Drive carriageway, an area at a 'Low' risk of flooding from a surface water flow route is shown extending the full length of the road, becoming 'Medium' risk further downslope. An area at a 'Low' risk of flooding from surface water can also be seen within the footprint of No.15 Heath Drive, as well as part of No.14.

Figure 6: Extract from the Environment Agency's map of 'Risk of Flooding from Surface Water'.
Ordnance Survey © Crown copyright 2018. All rights reserved. Licence No.100051531.
Also contains public sector information licensed under the Open Government Licence v3.0.

5.11 The SFRA by URS for the London Borough of Camden, was published in July 2014. Surface water flood modelling was carried out as part of this SFRA, the results of which are presented in Figures 3i-3v of that document. Figure 3v (an extract from which is presented in Figure 5) identifies the same four levels of risk of surface water flooding as the EA modelling (described as high, medium, low and very low), and the results correspond well with the EA's modelling; this map also indicates a 'Very Low' risk of surface water flooding for 24 Heath Drive, and a 'Low' risk for the Heath Drive carriageway and No.15 Heath Drive opposite.

5.12 Figure 6 of the SFRA (2014) shows that Heath Drive is located within the Group3_010 Critical Drainage Area, but is not located within any of the 'Local Flood Risk Zones' identified within this CDA. The Surface Water Management Plan (SWMP), published as part of the wider Drain London project in 2011, describes the Group3_010 West Hampstead CDA as having a "*pluvial/sewer capacity issue*".

5.13 Recorded sewer flooding incidents were summarised and mapped by postcode in Figures 5a and 5b of the SFRA (2014). No internal or external sewer flooding events were recorded within the 'NW3 7' sub-postcode (in which 24 Heath Drive lies).

5.14 The implications of the various flood models are discussed in Section 10.8.

6. HYDROGEOLOGICAL SETTING (GROUNDWATER)

6.1 The Claygate Member is classified by the Environment Agency as a 'Secondary A Aquifer', whereas the underlying London Clay Formation is classified as an 'Unproductive Stratum'. This hydrogeology is illustrated in Figure 7, which presents an extract from Figure 16 of the Camden GHHS (Arup, 2010). Under the old groundwater vulnerability classification scheme, which now applies only to superficial soils, the site is within an area classed as 'Minor Aquifer High' groundwater vulnerability.

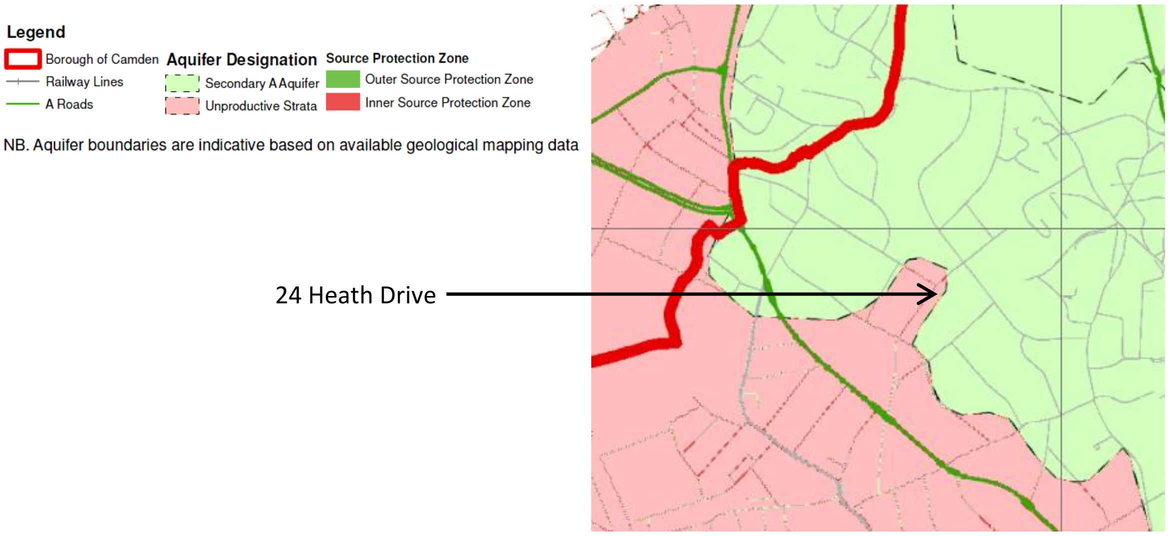

Figure 7: Extract from Figure 8 of the Camden GHHS (Arup, 2010) – 'Camden aquifer designation map'.

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

6.3 Perched groundwater would typically be expected in any overlying Made Ground, and possibly also in any Head Deposits which may be present, 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.

6.4 The beds of silty sand and sandy silt within the Claygate Member would generally be expected to be water-bearing and where these are laterally continuous they can give rise to moderate water entries into excavations. There is potential for multiple perched water tables to be present within the Claygate Member, given that five separate water strikes were recorded in one of the nearby boreholes (see Table 2), and depending on the lateral continuity and interconnection of the higher permeability beds. The clay and silty clay beds would also be expected to be saturated, with water pressures controlled by the water levels/ pressures in adjacent silty and sandy beds, by tree root activity or by the influence of man-made changes such as utility trenches (which can act either land drains or as sources of water and high groundwater pressures). Boreholes drilled through low permeability layers can also homogenise groundwater pressures between permeable layers if they are not adequately sealed.

As with Made Ground and Head Deposits, variations in groundwater levels and pressures will occur seasonally and with other man-induced influences.

6.5 While the London Clay Formation is classified as unproductive, it can still be water-bearing. Any partings, laminations or thicker beds of silt or sand are likely to contain free groundwater and, where these are laterally continuous, they can also give rise to moderate water entries into excavations. In most cases however, there will be only very limited or no natural flow in these silt/sand horizons. 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.

6.6 The presence of interbedded sands, silts and clays of the Claygate Member often gives rise to various springs, located at and above the Claygate/London Clay boundary. While no springs are recorded on the historical Ordnance Survey maps in the vicinity of Heath Drive, the streams visible on the 1870 map were clearly fed by springs, and these have since (historically) been collected and channelled into drains/culverts before the area was developed.

6.7 The groundwater catchment areas upslope of No.24 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 upslope, in No.23's site, as well as in No.24's own garden, except where the trenches for drains and other services provide greater interconnection.
- Claygate Member and London Clay Formation: The catchment for the Claygate Member and underlying London Clay Formation will comprise recharge from the overlying soils in the vicinity of the site plus, a much wider area determined by the lateral extent of any interconnected silt/sand horizons.

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

- The nearest groundwater abstraction licence is located 1852m to the south-east of the site at the Swiss Cottage Open Space Borehole (TQ28SE1769) (see App.E, Section 6.3), so is irrelevant to the proposed basement.
- There are no abstraction licences for potable water within 2000m of the site (App.E, Section 6.5).
- There are no Source Protection Zones (SPZ) within 500m of the site (App.E, Section 6.6 & 6.7). The nearest is around 2km to the south-east of the site, so is irrelevant to the current issue.
- For an area within 50m of No.24 the BGS has classified the susceptibility to groundwater flooding as '**Limited Potential**', at a 'Low' confidence level (App.E, Sections 7.7 and 7.8). Such groundwater flooding is defined as "*the emergence of groundwater at the ground surface or the rising of groundwater into man-made ground under conditions where the normal range of groundwater levels is*

exceeded". This classification is described as relating to 'clearwater flooding' in this area, where flooding is "*associated with unconfined aquifers*" (App.E, Section 7.7). The implications of this classification are discussed in paragraph 10.2.10.

6.9 Figure 4e of the SFRA (2014) summarises and maps past groundwater flooding incidents as recorded by both the EA and LBC. An extract from this is presented below in Figure 8 below, which shows that no groundwater flooding incidents were reported by either source within the near vicinity of Heath Drive. The closest LBC historical groundwater flooding record was recorded on Lyncroft Gardens, over 360m to the south-west of No.24, and the closest EA groundwater flood incident was recorded on Church Row, about 600m to the ESE of the property. Figure 4e of the SFRA also maps areas with an increased susceptibility to elevated groundwater, however these are all within the south-eastern part of the borough, remote from Heath Drive, where River Terrace Deposits can be found overlying the London Clay Formation.

Figure 8: Extract from Figure 4e of the SFRA (2014) – 'Increased Susceptibility to Elevated Groundwater'.

6.10 Details of what was found by the site-specific ground investigation in November 2016 are presented in the Factual Report on Ground Investigation (in Appendix C and summarised in Section 9).

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 desk study, the site-specific ground investigation (see Appendix C), the Camden GHHS (Arup, 2010), historical maps and data obtained from Groundsure (see Appendices D, E & F), the site-specific documents from Kyson, Form SD, Cowley White and Eight Associates (see paragraph 1.4), 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?	Potentially, Yes – Mapping by the BGS indicates the central and rear parts of the site are underlain by the Claygate Member of the London Clay Formation, however it should be noted that this was not encountered during the site specific ground investigation.	Carried forward to Scoping: Paragraphs 6.1, 8.2 & 9.3, and Section 10.2
1b	Will the proposed basement extend beneath the water table surface?	Potentially, Yes – Due to the mapped presence of the Claygate Member beneath parts of the site, although this was not encountered during the site specific ground investigation. The basement will however extend below the phreatic surface of the groundwater in the London Clay. The design of the basement and all temporary works must allow for the presence of groundwater in the Made Ground, which was found to be predominantly clayey but included gravels, and the London Clay.	Carried forward to Scoping: Paragraphs 6.1, 8.2 & 9.3, and Sections 10.2 & 10.3
2	Is the site within 100m of a watercourse?	No – Although one of the former Westbourne tributaries which is thought to have been culverted in the 1800's may be located beneath the Heath Drive carriageway.	Paragraphs 5.1-5.3
3	Is the site within the catchment of the pond chains on Hampstead Heath?	No – As shown on Figure 14 of the Camden GHHS (Arup, 2010), the site is approximately 400m south of the Golders Hill Pond Chain catchment, which is the nearest of the pond chains.	Paragraph 5.2
4	Will the proposed basement development result in a change in the proportion of hard surfaced/ paved areas?	Yes – There will be an increase in hard surfaced/ paved areas from the proposed rear extension, the proposed swimming pool extending out beneath the rear garden, and the landscaping of the rear garden.	Carried forward to Scoping: Paragraphs 3.1 & 8.2, and Section 10.8
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.	Section 10.8

6	Is the lowest point of the proposed excavation (allowing for any drainage and foundation space under the basement floor) close to, or lower than, the mean water level in any local pond (not just the pond chains on Hampstead Heath) or spring line?	Yes – There are no surface water features within 250m of the site, but a spring line does occur in places at the Claygate Member – London Clay interface, which, based on BGS mapping in this area, is located on site.	Carried forward to Scoping: Paragraphs 5.3, 6.6 & 8.2, and Sections 10.2 & 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)	Yes - Gradients within the site vary from level to >10°.	Carried forward to scoping: Paragraphs 2.9 & 8.3, Figure 3, and Section 10.4
2	Will the proposed re-profiling of landscaping at site change slopes at the property boundary to more than 7°?	No – Re-profiling of the rear garden is proposed, to create sub-horizontal terraces separated by new retaining walls. In addition, the flank wall of the garage on the 23/24 boundary, which is a retaining wall, will be re-built.	Paragraphs 2.2 & 3.1
3	Does the development neighbour land, including railway cuttings and the like, with a slope greater than 7°?	Yes – At least parts of the adjoining rear gardens have also been identified as having slope angles of 7-10°, and localised areas have been identified as >10°.	Carried forward to scoping: Paragraphs 2.9 & 8.3
4	Is the site in a wider hillside setting in which the general slope is greater than 7°?	No – Overall slope angles in the vicinity of the site range between around 2.6° and 5.7°.	Paragraph 2.9 and Figure 2
5	Is the London Clay the shallowest strata at the site?	Yes – Based on the results of the site-specific ground investigation.	Carried forward to Scoping: Paragraphs 4.1 8.3 & 9.3, Sections 10.1 & 10.4
6	Will any tree/s be felled as part of the proposed development and/or are any works proposed within any tree root protection zones where trees are to be retained?	Yes – a majority of the trees n site are to be removed. Several new trees are also proposed.	Carried forward to Scoping: Paragraphs 3.2 & 8.3, and Section 10.4
7	Is there a history of seasonal shrink/swell subsidence in the local area, and/or evidence of such effects at the site?	Yes, possibly – Significant crack damage was observed within the two-storey side extension to No.24, although this could have been from other causes.	Carried forward to Scoping: Paragraphs 2.2 & 8.3, and Section 10.4
8	Is the site within 100m of a watercourse or potential spring line?	Yes - A potential spring line. The former Westbourne tributary may be located beneath the Heath Drive carriageway, but is thought to have been culverted in the 1800's.	Carried forward to Scoping: Paragraphs 5.1-5.3, 6.7 & 8.3, and Section 10.4
9	Is the site within an area of previously worked ground?	No – The closest area of worked ground is approximately 260m to the south of the site. See maps on pages 8 & 15 of the GeoInsight report (in App. D).	Paragraph 4.1 and Figure 3

10	Is the site within an aquifer? If so, will the proposed basement extend beneath the water table such that dewatering may be required during construction?	Potentially, yes and yes – See answers to Q's 1a & 1b of Subterranean (groundwater) flow screening flowchart.	Carried forward to Scoping: Paragraphs 6.1, 8.2 & 9.3, and Sections 10.2 & 10.3
11	Is the site within 50m of the Hampstead Heath ponds?	No – As shown on Figure 14 of the Camden GHHS (Arup, 2010).	Paragraph 5.2
12	Is the site within 5m of a highway or a pedestrian right of way?	No – The proposed basement will be around 9m from the Heath Drive footway at its closest point.	
13	Will the proposed basement substantially increase the differential depth of foundations relative to neighbouring properties?	Yes – In relation to No.23, the rear part of No.25 and its garage. No's 23 and 25 have existing cellars beneath the front left (north) corners of the main house.	Carried forward to Scoping: Paragraphs 3.1 & 8.3, and Section 10.4
14	Is the site over or within the exclusion zone of any tunnels, eg railway lines?	No – A services/infrastructure search has already been carried out.	Paragraph 10.1.3

7.4 Surface flow and flooding screening flowchart:

Question		**Response, with justification of 'No' answers**	**Clauses where considered further**
1	Is the site within the catchment of the pond chains on Hampstead Heath?	No – As shown on Figure 14 of the Camden GHHS (Arup, 2010).	Paragraph 5.2
2	As part of the proposed site drainage, will surface water flows (eg volume of rainfall and peak run-off) be materially changed from the existing route?	No – The surface water flow routes will not change significantly.	
3	Will the proposed basement development result in a change in the proportion of hard surfaced / paved external areas?	Yes – See answer to Q4 of Subterranean (groundwater) flow screening flowchart.	Carried forward to Scoping: Paragraphs 3.1 & 8.4, and Section 10.8
4	Will the proposed basement result in changes to the profile of the inflows (instantaneous and long-term) of surface water being received by the adjacent properties or downstream watercourses?	No – There will be no or insignificant change in run-off to adjacent properties. The historical natural watercourse downslope of the property (beneath the Heath Drive carriageway) has been culverted since the 1800's.	Paragraphs 5.1, 5.3 & 5.4
5	Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream watercourses?	No – The change in surfaces generating the run-off to neighbouring property will not have a detrimental effect on water quality; no run-off goes direct to a watercourse.	
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?	No – Heath Drive was not affected by either the 1975 or 2002 events (Camden GHHS, Arup, 2010), and surface water flood modelling by both the Environment Agency and in the Camden SFRA indicated a 'very low' flood risk (the lowest) for this property, and a 'very low' to 'low' risk for the surrounding area.	Paragraphs 5.9 & 5.11, and Figures 7 & 8

7.5 Non-technical Summary – Stage 1:

The screening exercise in accordance with CPG4 has identified thirteen issues which need to be taken forward to Scoping (Stage 2) in Section 8; four related to groundwater, eight related to ground stability and one related to flooding potential.

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
1a	Is the site located directly above an aquifer?	**Potential impact**: Local restriction of groundwater flows (perched groundwater or below groundwater table). **Action:** Ground investigation required, then review.
1b	Will the proposed basement extend beneath the water table surface?	The anticipated groundwater regime is described in Section 6, Hydrogeological Setting. **Potential impact**: Local restriction of groundwater flows (perched groundwater or below groundwater table). **Action:** Ground investigation required, then review. Use of groundwater bypass as mitigation if necessary.
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 potential impacts of proposed changes, including appropriate types of SuDS for use as site-specific mitigation when relevant.
6	Is the lowest point of the proposed excavation (allowing for any drainage and foundation space under the basement floor) close to, or lower than, the mean water level in any local pond (not just the pond chains on Hampstead Heath) or spring line?	**Potential impact:** Risk of inundation of the basement excavations and long-term seepage into basement if waterproofing inadequate. Temporary dewatering and/or the permanent works might cause a spring to run dry. **Action:** Review potential impacts in relation to the site's hydrogeology; recommend appropriate site-specific mitigation when relevant.

8.3 Slope/ground stability scoping:

Issue (= Screening Question)		Potential impact and actions
1	Does the existing site include slopes, natural or man-made, greater than 7°? (approximately 1 in 8)	**Potential impact**: Clay slopes may be only marginally stable owing to past solifluction, excavations or placement of fill material. Increases in groundwater levels/pressures could cause slope failure. **Action:** Additional support, both temporary and permanent, may be required in excavations. No cross-slope excavations should be made in battered open cut. Basement design should ensure no (or minimal) increase in groundwater pressures.
3	Does the development neighbour land, including railway cuttings and the like, with a slope greater than 7°?	**Potential impact**: Slopes may be only marginally stable. Increases in groundwater levels/pressures could cause slope failure. **Action:** As Q1 above.
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, followed by appropriate design**.**

6	Will any tree/s be felled as part of the proposed development and/or are any works proposed within any tree root protection zones where trees are to be retained?	**Potential impact:** Heave from removal of trees (within area of roof growth); slope(s) become less stable; damage to retained tree roots caused by the basement excavations; loss of stability of retained trees. Potential for trees to be protected by a Tree Protection Order, or be protected within the Conservation Area if over certain dimensions. **Action:** Arboricultural assessment and review of potential impact on stability of buildings and/or slopes and/or the trees as relevant. Revise the scheme if required to prevent unacceptable impacts.
7	Is there a history of seasonal shrink/swell subsidence in the local area, and/or evidence of such effects at the site?	**Potential impact:** Weakened structures from past movement would be more susceptible to damage during works. Future differential movement between the building above the proposed basement and the adjoining structures which remain on shallow footings. **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. Foundations for new extensions to be stepped in accordance with best practice and/or sufficiently deep to prevent differential movement.
8	Is the site within 100m of a watercourse or potential spring line?	**Potential impact:** For sub-surface spring(s) from the Claygate Member within or upslope of the site, as applies here, construction of the basement might block or divert the flow of groundwater in superficial soils derived from the spring line, thereby increasing groundwater pressures and reducing the stability of slopes and/or retaining structures in the vicinity. **Action:** Review hydrogeology of the site, undertake a ground investigation and include mitigation measures if required.
10	Is the site within an aquifer? If so, will the proposed basement extend beneath the water table such that dewatering may be required during construction?	**Potential impact:** Inadequate provision of dewatering can lead to collapse of excavations. Inappropriate dewatering can cause removal of fines and/or unacceptable increases in effective stress, both of which can cause structures to settle. **Action:** Ground investigation required in order to enable a proper assessment of the appropriate forms of groundwater control.
13	Will the proposed basement substantially increase the differential depth of foundations relative to neighbouring properties?	**Potential impact:** Loss of support to the ground beneath the foundations to neighbouring buildings if basement excavations are inadequately supported. **Action:** Ensure adequate temporary and permanent support by use of best practice underpinning methods.

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:** Reduced infiltration, which may increase flow rates to sewer, and thus increase the risk of flooding (locally or elsewhere). **Action:** Assess net change in hard surfaced/ paved areas and, if required, recommend appropriate types of SuDS for use as site-specific mitigation.

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 – see Stage 3 in Section 9).
- Review of site's hydrogeology and groundwater control requirements, including potential flow from springs.
- Provide recommendations for groundwater control, to include adequate mitigation to prevent inundation of basement excavation works, and adequate waterproofing of the basement to avoid long-term seepage.
- Assess the surface water storage requirements for the site in terms of implementing a Sustainable Drainage System (SuDS) in order to offset (mitigate) any potential increase in discharge to mains sewer.
- An arboricultural impact assessment is required (which has already been undertaken – see paragraph 1.4).
- A review of the implications of the proposed tree removal and planting, for the adjoining and adjacent structures which remain on shallow footings (i.e. the remainder of No.24 with no basement beneath, and the neighbouring No's 23 & 25).
- 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 working methods.
- Provide suitably deep/stepped foundations for the proposed extensions to avoid possible differential movements relative to the basement.
- Assess potential for slope instability and provide appropriate recommendations.
- Review flood risk and include appropriate flood resistance and mitigation measures in the scheme's design.

All these actions are covered in either Stage 3 (ground investigation) or Stage 4.

9. STAGE 3 – GROUND INVESTIGATION

9.1 A site-specific ground investigation has been undertaken by Gabriel GeoConsulting (GGC), with the drilling of boreholes and excavation of trial pits carried out by on 14th and 25th November 2016. The scope consisted of one windowless sampler borehole (BH1) drilled within the front parking/amenity area, two continuous flight auger (cfa) boreholes (BH2 & BH3) drilled within the rear patio area and rear lawn, and three hand dug trial pits (TP1, TP2 & TP3), which were completed in order to investigate the foundations to No.24. Logging of the trial pits, the recovered continuous 'core' samples from the windowless sampler, and the recovered disturbed samples from the cfa boreholes was undertaken on site by GGC (Keith Gabriel, Alexander Goodsell and Roberta McAlister).

9.2 The results of this ground investigation have been presented in the Factual Report on Ground Investigation (Ref: 17597/R1) which is reproduced in Appendix C, including:

- Figure GI-01 — Location Plan (includes boreholes and trial pits);
- Figures GI-02 to GI-06 — Trial Pit logs;
- Figure 17597.1 BH1 — Borehole log (BH1);
- Figure 17597.2 BH2 — Borehole log (BH2);
- Figure 17597.3 BH3 — Borehole log (BH3);
- Standard Penetration Test Results sheet; and
- Laboratory test results – Chelmer Geotechnical Laboratories, Project No. 7734.

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

The findings of the ground investigation may be summarised as follows:

- The hand dug trial pits revealed that the flank wall of the main house is supported on corbelled (stepped) brickwork bearing onto a 0.15m thick clinker concrete strip footing, at a depth of 0.89m bgl. The footings beneath the front wall of the single-storey projection were also revealed to be founded at 0.89m bgl, however consisted of corbelled brickwork bearing onto two clinker concrete footings, the upper with a thickness of 0.28m, and the lower with a thickness of 0.15m. The internal walls of the cellar were found to be supported on a 0.47m thick concrete footing, which varied slightly in geometry, and the footings beneath the front wall of the house were not located within the 1.50m depth excavated/probed (even though the cellar does not extend to that side of the house).
- Made Ground was encountered within all of the exploratory holes, with a maximum thickness of 2.25m, recorded within the front parking/amenity area in BH1. In the rear garden, at the locations of BHs 2 & 3, the thickness of the Made Ground was only 0.25-0.30m. The Made Ground generally consisted of **clays** with various included fragments of artificial material, although in TP3 beneath the cellar the upper parts consisted primarily of **gravel**.
- Beneath the Made Ground, all three of the boreholes recorded clays of the Weathered London Clay Formation, to a maximum depth of 6.70m bgl.

- Directly beneath the Weathered London Clay Formation, all three boreholes recorded 'un-weathered' London Clay to the base of the boreholes, at 10.0m bgl in BH1, and 6.0m bgl in BH2 & BH3.
- The highest groundwater levels recorded in the standpipes during the brief monitoring period was 0.51m bgl. Groundwater was also encountered in TP3, with groundwater standing at 0.36m below the cellar floor on completion.
- The chemical testing revealed very high Total Potential Sulphate (TPS) readings with the Weathered London Clay Formation, of up to 4.5% (DS-5).

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:

- Made Ground: Made Ground was discovered in all of the exploratory holes, and varied in both its thickness and its composition, with descriptions of the Made Ground ranging from dark brown, organic, sandy silty **clay** to mid brownish grey, wet, very sandy **gravel**. Brick fragments were found throughout much of the Made Ground, along with various other included artificial fragments (see Factual Report on Ground Investigation in Appendix C for further details); 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.
  The maximum recorded depth was to 2.25m below ground level (bgl), within the front parking area/driveway, although the base of the Made Ground was not intercepted in TP1, which was dug to a depth of 1.05m alongside the front wall of the house and then extended to 1.50m using a hand auger. Within the existing cellar, the base of the Made Ground was recorded at 0.51m below the cellar floor level.
  Perched groundwater was found locally within this Made Ground, supported on the underlying low permeability London Clay, as recorded in TP3 within the cellar; such perched groundwater may only be present during the wetter winter and spring seasons.
- Head (?): Although Head deposits were not described during the recent site investigation at No.24, the presence of Head deposits on site cannot be ruled out. These locally-derived re-worked soils typically consist of material that has been washed down from upslope, so would be expected to consist of clays or sandy clays derived from the London Clay Formation or the overlying Claygate Member.
- Weathered London Clay Formation: Soft to stiff, predominantly brown, CLAYS/silty CLAYS were found beneath the Made Ground in all of the site-specific boreholes (see Factual Report on Ground Investigation in Appendix C). The uppermost parts of the Weathered London Clay were predominantly mid brown to mottled grey and orangey brown in colour, which gradually changed with depth to a more uniform mid brownish-grey. These clays were sometimes fissured with depth, which reduces their shear strength, and were mottled/veined grey around the roots/rootlets.
  Occasional pockets/partings of sandy clay and sands were observed throughout much of the Weathered London Clay.
  These clays will undergo heave movements in response to unloading by the basement excavation. They were also observed to contain pockets of coarse grained selenite (a form of gypsum) which is aggressive to buried concrete,

and they often contain claystone nodules/horizons, the larger ones of which can obstruct boreholes and piles. The Standard Penetration Tests (SPTs) in BH1 recorded an increase in blowcount with depth (see Figure 2 in Appendix C), indicating that the strength of the London Clay increased with depth.

- 'Un-weathered' London Clay Formation: Stiff, generally mid-dark grey CLAYS of the London Clay Formation were encountered beneath the Weathered London Clay Formation, to the maximum depths excavated. The boundary between these two units was observed to be transitional (see Factual Report on Ground Investigation in Appendix C).
- Hydrogeology
  - Groundwater pressures are expected to be essentially hydrostatic in the silty CLAYS within the depth of current interest in the London Clay, except where modified by seepage (see below), tree root activity or human interference. Groundwater flow through these High to Very High plasticity clays is likely to be minimal, in practice being limited to seepage through any of the silt/sand partings which are sufficiently interconnected.
  - Partings/laminations of silt/sand were recorded in parts of the London Clay, and some thin horizons of sandy clay were observed in BH1, the levels of which were identified on the borehole log. Higher permeabilities may be present within these horizons compared with other parts of the London Clay.
  - No groundwater entries were recorded in BH1 during or on completion of drilling, and only a slight groundwater seepage was observed in BH2 at 3.70m bgl. A slight groundwater seepage was also observed in BH3, at 0.50m bgl, and a groundwater strike was observed at 2.20m bgl. Groundwater was standing at 2.40m bgl on completion of BH3. During the subsequent short period of monitoring, the highest groundwater standing level was 0.51m bgl, recorded in BH2, where the response zone was at 2.0-4.0m bgl.
  - The hydrogeology may be complicated further by the backfill in service trenches and granular pipe bedding (where present) forming preferential groundwater flow pathways within the strata they pass through.

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

10.1.3 No railway tunnels for the main operational lines are known to pass below the site. A full services search has been undertaken in order to check for adopted services at No.24 Heath Drive. The responses received have been compiled as a factual record and issued separately. These records do NOT include any private services.

10.2 Subterranean (Groundwater) Flow – Permanent Works

10.2.1 Perched groundwater was not observed within the Made Ground during the site-specific ground investigation, except for beneath the cellar floor within TP3, however perched groundwater can be expected in the Made Ground above the underlying natural clays during at least the wetter winter and spring seasons. With the exception of TP3, in which the upper parts of the Made Ground consisted of very sandy gravel, the Made Ground generally consisted of sandy/slightly sandy silty clays (where seen), thus any flow is likely to be limited to minor seepage. Groundwater in the backfill to footing trenches excavated within plastic clays is typically static (until excavations are dug into/though the backfill).

10.2.2 Partings/laminations of silt/sand were recorded in parts of the London Clay, and some thin horizons of sandy clay were observed in BH1, which may give rise to slightly higher permeabilities compared with the surrounding clays. However, the only groundwater entries recorded in the three boreholes were slight seepages in BHs 2 & 3 (at 3.70m and 0.50m bgl respectively), and a groundwater strike at 2.20m bgl in BH3 (approximately 87.45m AOD, so about 0.35-0.75m below the level of the rear patio). Groundwater was standing at 2.40m bgl on completion of BH3.

10.2.3 In general, the lack of groundwater entries from parts of the London Clay Formation 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 standpipe was installed in the borehole, and any water in the partings of silt/sand was sealed in by smearing of clays during the drilling process. The rise in groundwater level to 2.20m bgl recorded in BH1, and 0.51m bgl recorded in BH2 during the short period of monitoring is entirely consistent with this ground model; even higher groundwater levels must be allowed for in the basement's design.

Existing Basements

10.2.4 Plans of the adjacent houses on Heath Drive were obtained during the search of planning application on Camden Council's website. The plans for No.25 Heath Drive reveal that it also has an existing cellar, located beneath the front left (north) corner and central parts of the house only. Plans of No.23's basement were not found on Camden Council's website, however ground floor plans of the property were found, which appear broadly similar in size and layout to No.24. The ground floor plan of No.23 also includes two staircases, one of which is located in the same place as No.24's basement staircase, thus it is assumed that No.23 also has an original cellar, which may be broadly similar in size and footprint to No.24's, so is probably located on the north-east side of the house, away from the 23/24 boundary. The findings from TP1 at No.24 showed that the footings were abnormally deep on the downslope side of the house, at front at least, possibly because they were excavated through the Made Ground to bear onto natural ground. Since most of the surrounding properties on Heath Drive are all of a similar style, and were constructed at broadly the same time, it is considered likely that most of the surrounding properties on Heath Drive have original cellars.

<u>Other Proposed Basements:</u>

10.2.5 Planning consent has been granted for the extension of the existing basements beneath No's 14 & 18 Heath Drive, located on the opposite side of Heath Drive, just to the west and north of No.24 respectively. Neither of these properties are located sufficiently close to No.24 to create a cumulative impact on groundwater seepage/flows. Planning consent has also been granted for the construction of a basement beneath No.2 Oakhill Avenue, located upslope of No.24 to the south-east, however this property is located sufficiently far upslope of No.24 that it should not have any impact on groundwater seepage/flows around No.24.

<u>Proposed Basement at No.24:</u>

10.2.6 The existing foundations and cellar to No.24 will already obstruct any flows of perched groundwater at shallow depth, including any enhanced seepage from the potential springline which may exist just upslope of the site. Details of the proposed basement are provided in Section 3. The proposed founding depth (formation level) of the basement is around 4.3m below the internal ground floor level, increasing to 5.73m below the ground floor level for the proposed swimming pool. As a result, the basement's formation level will be below the level of the groundwater entry recorded at 2.20m bgl in BH3 (about 0.35-0.75m below the level of the rear patio), which, if from a laterally persistent horizon, would already be blocked by the existing footings and cellar to No.24. The formation level of the swimming pool will also be below the level of the seepage recorded in BH2, as well as potentially more permeable horizons of sandy clay recorded in BH1. However, the construction of this proposed basement at No.24 is not expected to create any unacceptable cumulative obstruction or adverse impact on groundwater seepage/flows, because the seepage/flow in any water-bearing permeable horizons intersected by the basement, is likely to be able to continue around the basement, between it and the adjacent cellars to No's 23 & 25 Heath Drive. Thus the proposed basement is considered acceptable in relation to groundwater flow.

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

10.2.8 The highest groundwater level reading from the standpipes during the limited monitoring period was 0.51m bgl, recorded in BH2. Current geotechnical design standards require use of a 'worst credible' approach to selection of groundwater pressures. On sites such as this where high plasticity clays are present close to surface, the groundwater table (or phreatic surface) may rise to surface, or into the overlying Made Ground where a significant thickness is present, 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, use of design groundwater levels at the adjacent ground level is recommended for the whole basement, so the design groundwater level will increase slightly from the front right corner to the rear left corner of the basement, as the ground levels around the house and basement rise from 87.55m to 88.34m AOD respectively.

10.2.9 The basement structure must be designed to resist the buoyant uplift pressures which would be generated by groundwater at the design level. The slight variation in design water level depth means that the uplift pressures will also vary slightly across the basement from up to around 35kPa to 43kPa beneath the main part of the basement, up to 52-57kPa beneath the proposed swimming pool, and up to 63-69kPa beneath the 'sunken pit' (all un-factored).

10.2.10 The proposed basement will need to be fully waterproofed in order to provide adequate long-term control of moisture ingress from the groundwater, especially given the high groundwater levels/pressures which have already been recorded by the limited monitoring exercise. The BGS classification of the site's susceptibility to groundwater flooding as 'Limited Potential', is therefore considered to be unusually lenient. 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, with particular attention to ensuring that all service connections through the basement walls are watertight.

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

<u>Cumulative Impact:</u>

10.2.12 No cumulative impact on groundwater flow is anticipated from construction of the proposed basement, as set out in paragraphs 10.2.5 & 10.2.6 above.

10.3 Subterranean (Groundwater) Flow – Temporary Works

10.3.1 Local groundwater entries/seepages must be expected into the excavations for the basement. On current evidence, they should be manageable by a combination of pumping from suitably screened sumps and use of well-pointing techniques, with suitably screened well-points, provided that the inflows are not being fed by defective drains or water supply pipes. The dewatering system should be configured with sufficiently close spacings in order to prevent seepages into the excavations causing erosion of fines. 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. A careful watch should be maintained to check that fine soils are not removed with the groundwater; if any such erosion/removal of fines is noticed, then pumping should cease and the advice of a suitably experienced and competent ground engineer should be sought. Temporary backfilling of the part of the excavation concerned might also be necessary.

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

10.4 Slope and Ground Stability

10.4.1 The overall slope angle in the rear garden and upslope of No.24 was calculated from the contours to be less than 6° however, the garden has been terraced so the overall slope angle from the rear wall of the house to rear site boundary is up to 9.1° and slope angles of up to 19° are present locally.

10.4.2 Natural slopes in the clays/silty clays of the London Clay Formation which are steeper than 7° are typically metastable (owing to past solifluction and cryoturbation processes under periglacial climatic conditions during the last Ice Age). No evidence of slope movements was seen in No.24's rear garden and no positive evidence was found of solifluction slip surfaces in the boreholes, although only disturbed samples were recovered from BHs 2 & 3 (because the very restricted access to the rear garden meant that continuous flight auger drilling methods had to be used). However, the Very High plasticities, fissuring, high groundwater levels and minimal amount of sand in most of these clays indicates that they would have been susceptible to solifluction processes. Accordingly, for the rear wall of the basement it is recommended that the design of both the permanent retaining structure and the temporary support for the underpin excavations should be based on 'effective residual' shear strength parameters for the Weathered London Clay (see 10.4.13 below), and the first underpin pit to be excavated in the rear wall of the basement should be inspected by an engineering geologist who is experienced in logging soliflucted clay textures.

10.4.3 The proposed rear extension will require excavation into the rockery retaining wall and the flower bed above that. A new retaining wall will therefore be required to support the ground around the path/patio alongside the new rear extension. Once again, that retaining structure should be designed using 'effective residual' shear strength parameters for the Weathered London Clay, the wall should be constructed in summer or autumn seasons in short panels not exceeding 2.0m length (parallel with face of the wall), and the first excavation for this retaining wall should be inspected by an engineering geologist who is experienced in logging soliflucted clay textures.

10.4.4 Normal support requirements (see 10.4.12) will apply for the underpins beneath the remainder of the perimeter of this basement.

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

10.4.5 The majority of the basement will be constructed using reinforced concrete (RC) underpinning techniques beneath the original building, together with a secant bored pile wall for the section of the swimming pool which extends to the rear of the existing house, all as shown on FormSD's drawings (as listed in paragraph 1.4).

10.4.6 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 10.4.7 to 10.4.9 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.

10.4.7 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 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.

10.4.9 For the proposed basement beneath No.24:

- It should be assumed that full support will be required to any Made Ground, any natural granular soils and all soft, firm or firm-to-stiff clays exposed in the excavations.
- Closely spaced temporary support may be adequate in the stiff or very stiff clays of the London Clay Formation, depending on the degree of fissuring; if there is any doubt regarding the presence of fissures then full face support should be used.
- Temporary support must also be installed to support all the new underpins and RC retaining wall panels and must be maintained until the full permanent support has been completed, including allowing time for the concrete to gain adequate strength.

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

10.4.10 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.

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

Basement Retaining Wall Construction – Bored Pile Walls:

10.4.12 Use of secant bored pile retaining walls is proposed by FormSD, around the three sides of the swimming pool which projects to the rear of the house. This bored pile wall will be finished with reinforced concrete capping beams before the interior is excavated. The minimum spacing between the pile locations and the existing house walls should also be checked with specialist contractors, in order to confirm the feasibility of achieving the locations currently proposed.

10.4.13 A piling platform must be designed and constructed so as to provide a stable working platform for the piling rig – see paragraph 10.4.28 for further details.

10.4.14 FormSD's Construction Method Statement proposes a 'bottom-up' construction sequence, so adequate high stiffness temporary support must be installed at appropriate levels in order to minimise lateral movement of the piles before the permanent base slab and roof slab have been constructed. These slabs must be designed to act as permanent high-stiffness props.

10.4.15 The quality of workmanship has a significant impact on the magnitude of ground movements adjacent to a bored pile wall, 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:

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

- Earth pressures from the surrounding ground (see paragraph 10.4.17 below);
- Dead and live loads from the superstructure;
- Surcharge loads from the higher ground levels and retaining walls to the rear of the basement (depending on new landscaping layout);
- Vehicle loads on the front driveway and in the garage, and normal surcharge allowances elsewhere;
- The available bearing capacity (see paragraph 10.4.18);
- Swelling displacements/pressures from the underlying clays;
- Design groundwater level at the adjacent ground level (see paragraph 10.2.8);
- Precautions to protect the concrete from sulphate attack.

10.4.17 The following geotechnical parameters should be used when calculating earth pressures acting on the basement's retaining walls:

Made Ground:	Unit weight, γ_b:	18.0 kN/m^3
	Effective cohesion, c':	0 kPa
	Angle of internal friction, φ':	25° for clays
	Angle of internal friction, φ':	30° for sands/gravels
London Clay Formation:		
Weathered, firm CLAYS, possibly soliflucted:		
	Unit weight, γ_b:	19.0 kN/m^3
	Effective cohesion, c':	0 kPa
	Effective residual angle of internal friction, φ':	15°
'Unweathered', stiff CLAYS:		
	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:		1.0, after the likely existing

higher stresses have been released by the excavations.

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

10.4.18 The available bearing capacity (Presumed Bearing Value) at the anticipated founding level of the proposed basement would be 125 kPa, based on traditional design methods (which allow up to 25mm of settlement), the in-situ hand vane tests in BH2 and the penetrometer values on the samples recovered from BH1 with allowance for the presences of fissures. However, it would be preferable if the applied bearing pressures could be kept below 125kPa in order to minimise settlements.

10.4.19 The formation level clays onto which the underpins and the basement slab will bear must be protected from water to prevent softening and loss of strength, as described in 10.3.3 above.

Trees:

10.4.20 The net loads which will be applied to the proposed side and rear extensions, where no basement is proposed beneath, provisionally range from 40-55kN/m run dead and 2-15kN/m run live (see Form SD's 'Indicative SLS Loading' for lower ground floor level; Drg No.162637/SK011-2/A). The recorded consistencies of the clays and the in-situ tests in the boreholes indicate that adequate bearing capacity is available to support trench fill footings for these proposed side and rear extensions. Abnormally deep root growth was recorded in BH1, but no desiccation was evident in the samples which were tested below proposed basement level. However, desiccation of the clays should be considered in the vicinity of the trees, with the potential for further desiccation and shrinkage from trees which are retained, and for heave if the trees are removed or when they die naturally.

10.4.21 A minimum footing depth of 1.0m below the adjacent external ground level will be required for the footings for the proposed side and rear extensions, as the foundations will bear onto clays of high volume change potential (see paragraph 6.2 of the Factual Report on the Ground Investigation; ref: 17597/R1, included in Appendix C). However, the existing trees present on site, the existing trees to be removed on the site, and the proposed new trees to be planted within the site, will require the depths these footings to be increased. Table 3 below outlines the minimum footing depths recommended for the proposed side and rear extensions, to ensure adequate protection of the building from tree-related ground movements. Those depths have been calculated following guidance in NHBC's 2018 Standards (Chapter 4.2), and are based on distances measured from Writtle Forest Consultancy's Figure No.161002-001 Rev.1 and Cowley White's Drg No.Drg No.001-REV C, and tree species information taken from Eight Associates' Tree Survey and Tree Constraints Plan (1948; 1610-31sc) and Cowley White's Planting Schedule.

Table 3: Maximum recommended footing depths for proposed extensions			
Location	Min. Footing Depth (m)	Critical Tree No.	Retained or Removed
Side Extension (Garage, Utility/Dressing Room & Breakfast Room/Master Bathroom)	>2.5	T7 & T8	Existing, to be removed
Rear Extension (Dining Room)	2.4 (Potentially >2.5; see paragraph 10.4.22)	T12 T13	Existing, to be removed

10.4.22 It should be noted that Eight Associates' T13 (rhododendron) is located within the footprint of the rear extension, but as the exact species of rhododendron has not been provided, its mature height and water demand remain unknown. Depending on its water demand, a foundation of a minimum depth of either 1.8m (low water demand), 2.4m (moderate water demand) or >2.5m (high water demand) would be necessary. The required footing depth given in Table 3 for the rear extension is already at least 2.4m, but it may be necessary to increase this depth to greater than 2.5m. Of the two Cherry trees close to the rear extension (T12, 10m and T14, 14m) T12 is the more significant because it is immediately alongside the footprint of the proposed extension.

10.4.23 It is unlikely that the use of trenchfill foundations to the depths identified in Table 3 would be economic; particularly given the likely onerous practical requirements of installing anti-heave precautions, cleaning the base of the excavations and shoring the excavations should manual cleaning be necessary. As a piling rig will be on site for the bored pile wall, use of a system of pile foundations and groundbeams would therefore be recommended for both the side and rear extensions. The loads carried on piles should be taken down into the London Clay Formation, below the zone of influence of tree roots. Given the site's setting at the foot of a slope, a small number of larger diameter piles would be preferable to an increased number of smaller piles. Suitable pile types for these ground conditions and this site would include:

- Bored, cast in-situ piles
- Continuous flight auger piles (hollow stem CFA).

10.4.24 Preliminary estimates of pile load capacities have been made for isolated bored piles of a diameter of 450mm (as proposed by Form SD for the south-eastern end of the swimming pool at basement level) installed in BH1-3 type ground conditions. These estimates are presented in Table 4 below as a preliminary guide, for project planning purposes only, to the possible load capacities for various pile lengths. They are based on a strength profile indicated by the Standard Penetration Tests using the correlation proposed by Stroud (1974).

Table 4: Preliminary pile capacity analyses for isolated bearing piles				
Pile Diameter (mm)	Pile Length Below Maximum Assumed Depth of Tree Root Activity (m)	Pile Toe Level (m bgl)	Ultimate Load Capacity (kN)	Working Load Capacity (Factors as EN1997-1, UK NA) (kN)
450	6.0	9.5	364	151
	8.0	11.5	505	212
	10.0	13.5	665	282

10.4.25 The load capacities given in Table 4 are derived from a combination of shaft adhesion in the clays (alpha = 0.45 – 0.50) and end bearing on the pile base. Lower shaft adhesion values are likely to apply if driven, rather than bored cast in-situ, piles are used. It is recommended that a quantified risk assessment for unexploded ordnance (UXO) should be carried out if piles are to be driven.

10.4.26 It is standard practice in the UK for the design of bearing piles to be undertaken by the piling contractor, using their specialist knowledge of their particular piling systems. As a result, these preliminary analyses will be superseded by the final design analyses.

10.4.27 If pile groups are required in order to provide sufficient load capacity below more heavily loaded parts of the buildings, then, in addition to the design of the individual piles, the load capacity of each pile group will need to be checked against failure as a block.

10.4.28 A piling platform will be required in order to provide a stable base and to enable safe operation of the piling rig(s). Piling platforms typically comprise suitable granular fill material such as crushed rock, crushed concrete or other good quality hardcore, placed over a geogrid or geotextile separator membrane (which reduces the thickness of granular fill required). Formal design of the piling platform, taking into account both static and dynamic loads, is now required by all leading piling companies. Construction of the platform must comply with the design, including maximum gradients and the lateral extent of the platform beyond the required working positions for the rig at each pile location and along transit routes.

10.4.29 Provided that the piles are designed and constructed in accordance with best practice then settlements at pile head level are unlikely to exceed 10mm. Reduced settlement tolerance could be specified, if required in order to minimise differential movements relative to the basement.

10.4.30 All groundbeams used on site with pile foundations should be protected from future vertical heave of the ground beneath the new building using a suitable compressible material placed in accordance with guidance in NHBC Standards Chapter 4.2 (2018) and the manufacturer's recommendations, but would not need protection from lateral heave if all the adjacent soils within the footprint of the building are soft or firm (as defined in BS5930:2015) at the time of construction. Piles within the zone of influence of tree roots as defined in NHBC Standards Chapter 4.2 (2018) should either be sleeved to a depth of 3.5m bgl (assumed depth of tree root activity) to isolate them from the heave, or reinforced sufficiently to enable them to accommodate the tensile forces developed on the pile both directly from the soil and indirectly via the groundbeams.

10.4.31 While not required for Party Wall Act purposes, it may be beneficial to carry out similar foundation assessments for the neighbouring properties in relation to the proposed removal of several of the trees on site, with reference to NHBC Standards Chapter 4.2 (2018). However, the removal of any trees is only likely to have a significant effect on the neighbouring properties if the trees which are removed are older than the property itself.

Transition Underpins:

10.4.32 Normal good practice in foundation construction requires progressive stepping up between foundations of different depths beneath a single structure. However, this would not be necessary should a system of pile foundations and groundbeams be used on site for the side and rear extensions.

Cumulative Impact:

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

10.5 PDISP Heave/Settlement Assessment

Basement Geometry and Stresses:

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

10.5.2 Figures G1a and G1b in Appendix G illustrate the layout of the proposed basement, along with the layout of the PDISP zones used to model the piles, underpins, retaining walls, columns, basement slab, beams and sumps, based on Form Structural Design's proposed basement and plant level plans (Drg No's 162637/L(17)01/P2 & P3). It should be noted that the 'superimposed' peach-coloured zones (Zones 34 & 35) have been used to model the increased excavation for the sumps, the 'superimposed' sky blue zones (Zones 34-41) have been used to model the loads from the beams over the swimming pool, and the 'superimposed' green zones (Zones 42-50) have been used to either reduce or increase the excavation depths where the existing site levels vary. The purple zone (Zone 1), which has been used to model the piles, has been split by depth into four zones (1a, 1b, 1c & 1d), in order to model more realistic settlement patterns; Zone 1a stretches from the top of the capping beam to the base of the adjacent basement excavations (4.46m), and Zones 1b-d are all 2m deep. The maximum overall dimensions of the basement are approximately 14.9m wide by 22.9m long.

10.5.3 Table 5 presents the net changes in vertical pressure for all 50 of the PDISP zones during five major stages of the stress history of the basement's construction, as detailed in paragraph 10.5.6 below. The gross bearing pressures which will be applied by the underpins and retaining walls were calculated from Form Structural Design's load takedown (dated 23rd January 2018), an extract from which is presented in Figure G2, along with further information provided by email on 26th and 29th January 2018. The widths of Zones 12 and 13 were increased by 0.5m in the Stage 1 analysis in order to reduce the bearing pressures, as no underpin bases were proposed during this stage.

Table 5: Changes in vertical stress for PDISP Zones				
ZONE	**Net change in vertical pressure (kPa)**			
#	**Stage 1**	**Stage 2**	**Stage 3**	**Stages 4 and 5**
1a	26.76	26.76	26.76	26.76
1b	10.00	10.00	10.00	10.00
1c	10.00	10.00	10.00	10.00
1d	10.00	10.00	10.00	10.00
2	-55.57	-55.57	-55.57	-46.12

Table 5 Continued: Changes in vertical stress for PDISP Zones				
ZONE	**Net change in vertical pressure (kPa)**			
#	**Stage 1**	**Stage 2**	**Stage 3**	**Stages 4 and 5**
3	-55.57	-55.57	-55.57	-46.12
4	-56.17	-56.17	-56.17	-46.72
5	-56.17	-56.17	-56.17	-46.72
6	-47.23	-47.23	-47.23	-39.46
7	-48.44	-48.44	-48.44	-40.82
8	-24.16	-24.16	-24.16	-16.85
9	-50.93	-50.93	-50.93	-43.16
10	-6.64	-6.64	-6.64	12.02
11	-28.49	-28.49	-28.49	-20.93
12	119.74	119.74	119.74	0.00
13	85.23	85.23	85.23	0.00
14	-31.82	-31.82	-31.82	0.00
15	-28.45	-28.45	-28.45	-28.45
16	3.73	3.73	3.73	3.73
17	47.85	47.85	47.85	47.85
18	89.95	89.95	89.95	89.95
19	3.50	3.50	3.50	3.50
20	108.47	108.47	108.47	108.47
21	54.52	54.52	54.52	54.52
22	0.00	-66.97	-56.97	-56.97
23	0.00	-83.12	-73.12	-73.12
24	-14.94	-14.94	-14.94	-46.83
25	-29.60	-29.60	-29.60	-29.60
26	-14.54	-14.54	-14.54	-14.54
27	0.00	0.00	0.00	-29.88
28	0.00	0.00	0.00	-36.33
29	0.00	0.00	0.00	103.38
30	0.00	0.00	0.00	13.42
31	0.00	0.00	0.00	-40.27
32	0.00	0.00	0.00	-57.19
33	0.00	0.00	0.00	-97.53
34	0.00	0.00	0.00	-10.45
35	0.00	-36.58	-36.58	-36.58

Table 5 Continued: Changes in vertical stress for PDISP Zones				
ZONE	**Net change in vertical pressure (kPa)**			
#	**Stage 1**	**Stage 2**	**Stage 3**	**Stages 4 and 5**
36	4.35	4.35	4.35	4.35
37	4.30	4.30	4.30	4.30
38	4.35	4.35	4.35	4.35
39	4.30	4.30	4.30	4.30
40	4.35	4.35	4.35	4.35
41	4.30	4.30	4.30	4.30
42	-39.52	-39.52	-39.52	-39.52
43	-16.15	-16.15	-16.15	-16.15
44	23.37	23.37	23.37	0.00
45	-34.77	-34.77	-34.77	0.00
46	-11.40	-11.40	-11.40	-11.40
47	0.00	0.00	0.00	23.37
48	0.00	0.00	0.00	23.37
49	0.00	0.00	0.00	-34.77
50	0.00	0.00	0.00	-11.40

Ground Conditions:

10.5.4 The ground profile was based on the site-specific ground investigation by GGC, as presented in GGC's Factual Report on the Ground Investigation (Ref: 17597/R1), which is reproduced in Appendix C and summarised in Sections 9 and 10.1 above, together with 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 6, which were based on this investigation and data from other projects.

Table 6: Soil parameters for PDISP analyses				
Strata	**Level** (m bgl)	**Undrained Shear Strength,** Cu (kPa)	**Short-term, undrained Young's Modulus,** Eu (MPa)	**Long-term, drained Young's Modulus,** E' (MPa)
London Clay	84.5 51.5	38 285.5	19 142.5	11.4 85.5

Where:

Undrained Shear Strength, Cu at top of stratum is based on the SPT profile
Undrained Shear Strength, Cu at base of stratum assumed as Cu = 38 + 7.5z
where z = depth below the top of the stratum
Undrained Young's Modulus, Eu = 500 * Cu
Drained Young's Modulus, E' = 0.6 * Eu

PDISP Analyses:

10.5.6 Three dimensional analyses of vertical displacements have been undertaken using PDISP software for 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 and construction of the basement. PDISP analyses have been carried out as follows:

- Stage 1 – Installation of bored piles and pile cap, and excavation for and construction of underpins/retaining walls and internal columns to respective formation levels for main basement and pool – Short-term (undrained) condition
- Stage 2 – Excavation of basement slab areas to formation level – Short-term (undrained) condition
- Stage 3 – Construction of basement slabs – Short-term (undrained) condition
- Stage 4 – Excavation for and construction of underpins to sunken plant level, and installation of RC pool box, pool attenuation tank, suspended slab over plant room, and capping slab to rear – Short-term (undrained) condition
- Stage 5 – As Stage 4, except – Long-term (drained) condition.

10.5.7 The results of the analyses for the Stages 1, 2, 3, 4 and 5 are presented as contour plots on the appended Figures G3 to G7 respectively at multiple levels:

- Basement and pool levels in Stages 1-3; and
- Basement and sunken plant levels in Stages 4-5.

For all five stages, the basement level contour plots give a worst case scenario, indicating that settlements reduce with depth, which is as expected.

Heave/Settlement Assessment:

10.5.8 Construction of the basement will cause immediate elastic settlement/heave in response to the stress changes, followed by long-term plastic swelling/consolidation as the underlying clays take up groundwater or as the excess pore water pressures from the applied loads dissipate. The rate of plastic swelling in the in-situ clays will be determined largely by the availability of water and as a result, given the low permeability of the clays in the London Clay Formation, can take 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 PDISP analyses indicated that movements reaching 8mm settlement and 11mm heave are likely to develop beneath the basement slab and walls. The ranges of predicted short-term and long-term movements for each of the main sections at basement level are presented in Table 7 below.

10.5.10 All the short-term elastic displacements would have occurred before the basement slab is cast, so only the post-construction incremental heave/settlements (Stages 3 to 5) are relevant to the design of the central basement slab within the underpin bases. The analyses indicated that the maximum predicted post-construction displacements beneath the central slabs are likely to be about 4mm total, with differential displacements across the slab of up to 14mm.

Table 7: Summary of predicted displacements					
Location	**Stage 1 (Fig. D3)**	**Stage 2 (Fig. D4)**	**Stage 3 (Fig. D5)**	**Stage 4 (Fig. D6)**	**Stage 5 (Fig. D7)**
Front wall of basement level	2.0 – 3.0mm Settlement	1.5 – 2.0mm Settlement	1.5 – 2.0mm Settlement	0 – 2.0mm Settlement	0.5 – 3.0mm Settlement
Front wall of sunken plant level	2.5 – 4.0mm Settlement	2.0 – 4.0mm Settlement	2.0 – 4.0mm Settlement	1.0 – 2.5mm Settlement	1.5 – 4.0mm Settlement
NE side wall of sunken plant level	0 – 6.0mm Settlement	0 – 6.0mm Settlement	0 – 6.0mm Settlement	2.0mm Settlement to 1.5mm Heave	3.5mm Settlement to 1.5mm Heave
NE side wall of pool level	0.5mm Settlement to 3.0mm Heave	0.5mm Settlement to 3.0mm Heave	0.5mm Settlement to 3.0mm Heave	0.5mm Settlement to 2.5mm Heave	0.5 – 4.0mm Heave
Rear wall of pool level	1.0 – 4.5mm Heave	2.0 – 5.0mm Heave	2.0 – 5.0mm Heave	1.5 – 4.0mm Heave	2.0 – 6.5mm Heave
Pool beneath rear garden	0.5mm Settlement to 4.5mm Heave	0.5mm Settlement to 5.0mm Heave	0.5mm Settlement to 5.0mm Heave	0.5mm Settlement to 4.5mm Heave	0.5mm Settlement to 6.5mm Heave
Rear wall of basement level	2.5mm Settlement to 1.0mm Heave	1.5mm Settlement to 2.0mm Heave	2.0mm Settlement to 2.0mm Heave	2.0mm Settlement to 1.5mm Heave	3.0mm Settlement to 2.5mm Heave
SW side wall of basement level	0 – 4.5mm Settlement	3.5mm Settlement to 1.0mm Heave	3.5mm Settlement to 0.5mm Heave	3.5mm Settlement to 1.0mm Heave	6.0mm Settlement to 1.0mm Heave
Internal wall between basement and pool levels	1.0 – 3.5mm Heave	1.5 – 4.5mm Heave	1.5 – 4.5mm Heave	1.0 – 4.5mm Heave	2.5 – 7.0mm Heave
Staircase between basement and sunken plant level	2.5mm Settlement to 0.5mm Heave	2.0mm Settlement to 2.5mm Heave	2.0mm Settlement to 2.0mm Heave	1.0 – 6.0mm Heave	1.0 – 10.0mm Heave
Central basement slabs	4.5mm Settlement to 2.0mm Heave (No slab present)	2.0mm Settlement to 7.0mm Heave (No slab present)	2.0mm Settlement to 6.5mm Heave	2.0mm Settlement to 6.5mm Heave	3.0mm Settlement to 10.5mm Heave

10.6 Damage Category Assessment

10.6.1 When underpinning it is inevitable that the ground will be un-supported or only partially supported for a short period during 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 calculations of predicted ground movements can never be rigorous. However, provided that the temporary support follows best practice as outlined in Section 10.4 above, then extensive past experience has shown that the bulk horizontal movements of the ground alongside the basement caused by underpinning for a single-storey basement should not exceed approximately 5mm.

10.6.2 In order to relate these typical ground movements to possible damage which adjacent 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).

10.6.3 The neighbouring properties of No's 23 and 25 Heath Drive do not adjoin No.24. The superstructure of No.23 is broadly similar to No.24, but No.25 differs slightly from these two properties in that it is semi-detached, and appears to have four storeys. Both No's 23 and 25 have adjoining single-storey garages on their north-eastern sides. No.25 has a similar cellar to No.24, also located beneath the northern corner of the main part of the house. No.25 is known to have a cellar, but its exact location and dimensions are unknown; based on the location of the stairs on the ground floor plan taken from Camden Council's planning website (see paragraph 2.10), it is likely to be located on the south-western side of the house, close to No.24. The proposed basement will be located within approximately 0.80m of No.25 at its closest point, based on drawings taken from Camden Council's planning website (see paragraph 2.10). No.23 will be located approximately 3.70m from the proposed basement at its closest point, based on Form SD's Proposed Basement Plan (Drg No.162637/L(17)02/P3). The structural walls of No.23 which are perpendicular to the No.23/24 boundary are not located in close proximity to the basement of No.24, and the front western corner of No.23 is likely to have an existing cellar, so the differential foundation depths will be decreased. The worst case scenario will therefore occur at No.25, due to both its proximity to the proposed basement to No.24, and the results of the PDISP analyses, which indicated that maximum settlement will occur alongside No.25. However, the exact location of the worst case scenario for potential damage to No.25 was uncertain; the candidates were:

1. At the front wall of No.25's single-storey garage; or
2. At the main rear wall of No.25, although it is unclear whether or not this wall continues internally across to the main flank wall of the house where the 'store' is located.

The PDISP analyses indicated that the settlements will radiate further from the proposed footprint of No.24's basement into No.25's footprint in Stage 1, which therefore represents the worst case stage.

10.6.4 Separate damage category assessments have been undertaken for both of the locations identified above, which 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; and
- ground movements alongside the proposed underpins 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>Front wall of No.25's garage:</u>

10.6.5 The relevant geometries, based on information in Table 1 in Section 3, from the ground investigation at No.24 (see Section 9), and the relevant drawings from No.25 (see paragraph 2.10), are summarised below:

Depth of foundations = 0.9m (assumed) below internal ground floor level of approx. 87.72m AOD

Level of No.24's proposed basement at closest point = 84.05m AOD

Depth of excavation beneath ground level at No.25 = 87.72 – 84.05 = **3.67m**

Width of zone of affected soils = 3.67 x 4 = **14.68m**

Width of No.25's garage (L) = **3.70m** (closest point located **0.80m** from No.24's basement; see Figure 9 below)

Height (H) = 2.60 + 0.90 = **3.50m** (wall height + foundation)

Hence L/H = **1.06**.

10.6.6 Thus, for the anticipated (theoretical) horizontal displacement of 5mm (the typical value for a single-storey basement), the strain beneath the front wall of No.25's garage would be in the order of $\varepsilon_h = 3.41 \times 10^{-4}$ (0.034%).

10.6.7 The maximum settlement predicted by the PDISP analysis adjacent to the front wall of No.25's garage was just under 2mm in Stage 1 (see Figure G3 in Appendix G). This must be combined with the settlement caused by relaxation of the ground alongside the basement in response to excavation of the underpins, which can be estimated using the settlement profile for the worst case (low stiffness) scenario presented in Figure 2.11(b) of CIRIA Report C580. The settlement profiles are then summed to find the maximum deflection, Δ. Figure 9 presents these settlement profiles for the front wall of No.25's garage. The maximum Δ = 0.78mm, which represents a deflection ratio, $\Delta/L = 2.11 \times 10^{-4}$ (0.021%).

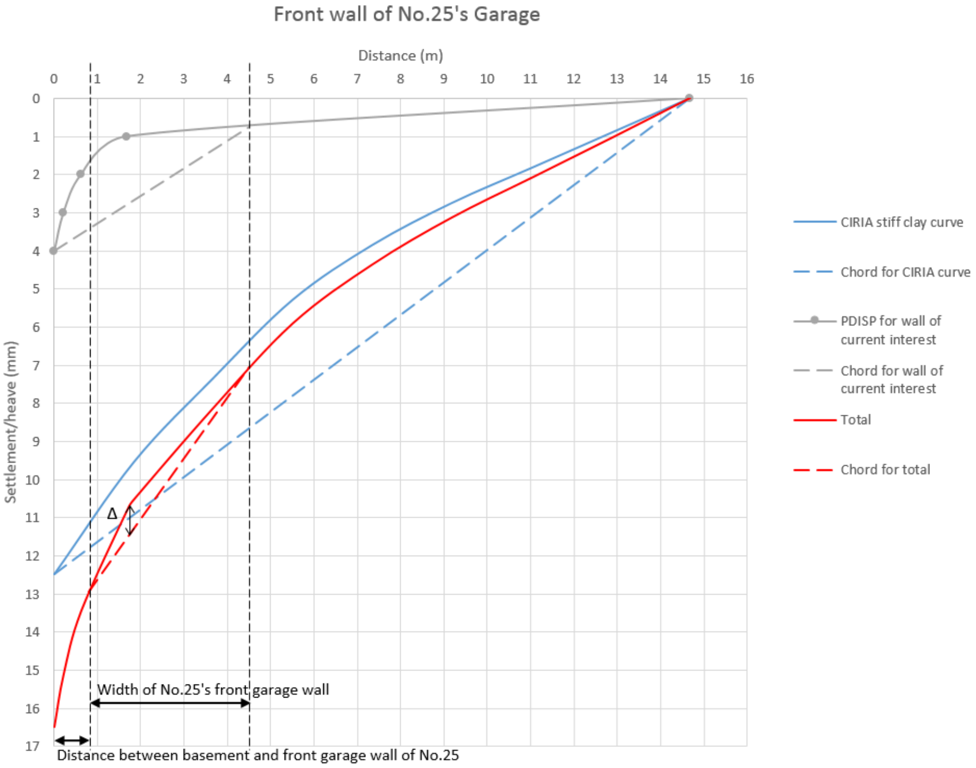

Figure 9: Displacement profile for front wall of No.25's garage.

10.6.8 Using the graphs for L/H = 1.5 (a conservative approach), these deformations represent a damage category of just into 'very slight' (Burland Category 1, ε_{lim} = 0.05-0.075%), as given in CIRIA SP200, Table 3.1, and illustrated in Figure 10 below.

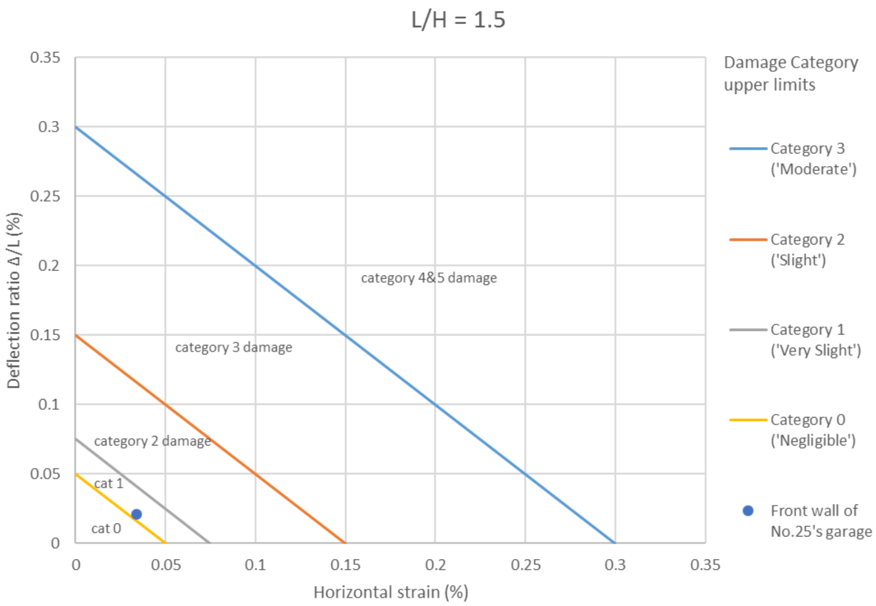

Figure 10: Damage category assessment for front wall of No.25's garage.

Main rear/internal wall of No.25:

10.6.9 The main rear wall of No.25 adjoins the main rear wall of No.26, so the wall of interest extends across the widths of both these properties. The relevant geometries, using the same methodology as above, are summarised below:

Depth of foundations = 1.1m (assumed, by correlation with No.24) below external ground level of approx. 87.4m AOD

Lowest level of No.93's proposed basement = 84.05m AOD

Depth of excavation beneath ground level at No.25 = 87.4 – 84.05 = **3.35m**.

Width of zone of affected soils = 3.35 x 4 = **13.40m**

Width of No.25 & No.26's rear/internal wall = **24.20m** (closest point located **1.55m** from No.24's basement; see Figure 11 below)

Width of potentially affected part of wall (L) = 13.40 – 1.55 = **11.85m**

Height (H) = 6.10 + 1.10 = **7.20m** (wall height + foundation).

Hence L/H = **1.65**.

10.6.10 Thus, for the anticipated (theoretical) horizontal displacement of 5mm (the typical value for a single storey basement), the strain beneath No.95 would be in the order of $\varepsilon_h = 3.73 \times 10^{-4}$ (0.037%).

10.6.11 The maximum settlement predicted by the PDISP analysis adjacent to No.25's rear/internal wall was less than 1mm in Stage 1 (see Figure G3 in Appendix G). As previously, this must be combined with the settlement caused by relaxation of the ground alongside the basement in response to excavation of the underpins. The resultant combined settlement profiles are presented in Figure 11. The maximum Δ = 1.57mm, which represents a deflection ratio, $\Delta/L = 1.19 \times 10^{-4}$ (0.012%).

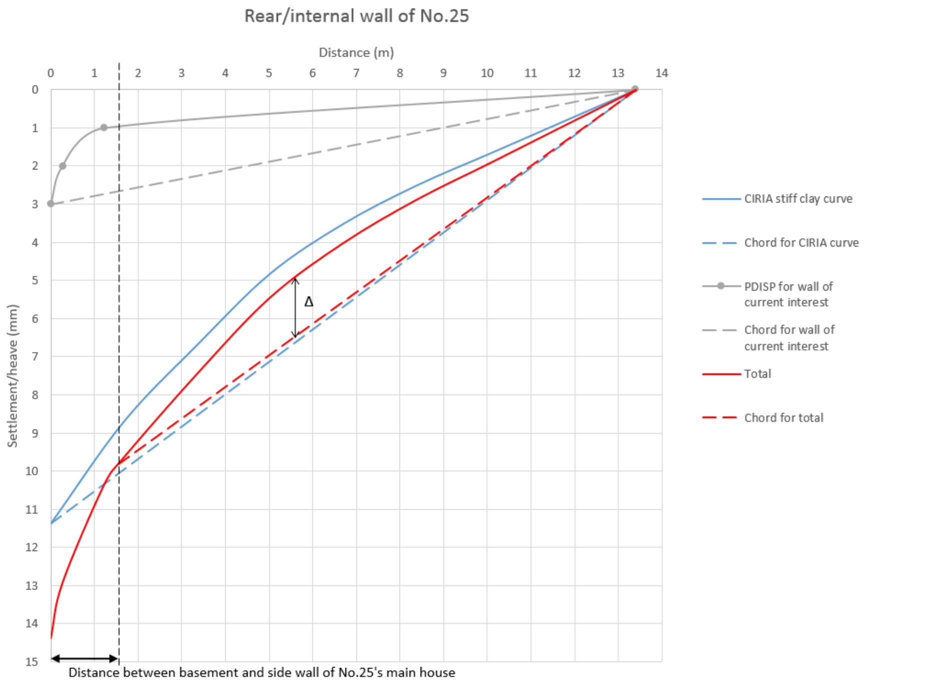

Figure 11: Displacement profile for rear/internal wall of No.25.

10.6.12 Using the graphs for L/H = 2.0 (a conservative approach), these deformations represent a damage category of just into 'very slight' (Burland Category 1, ε_{lim} = 0.05-0.075%), as given in CIRIA SP200, Table 3.1, and illustrated in Figure 12 below.

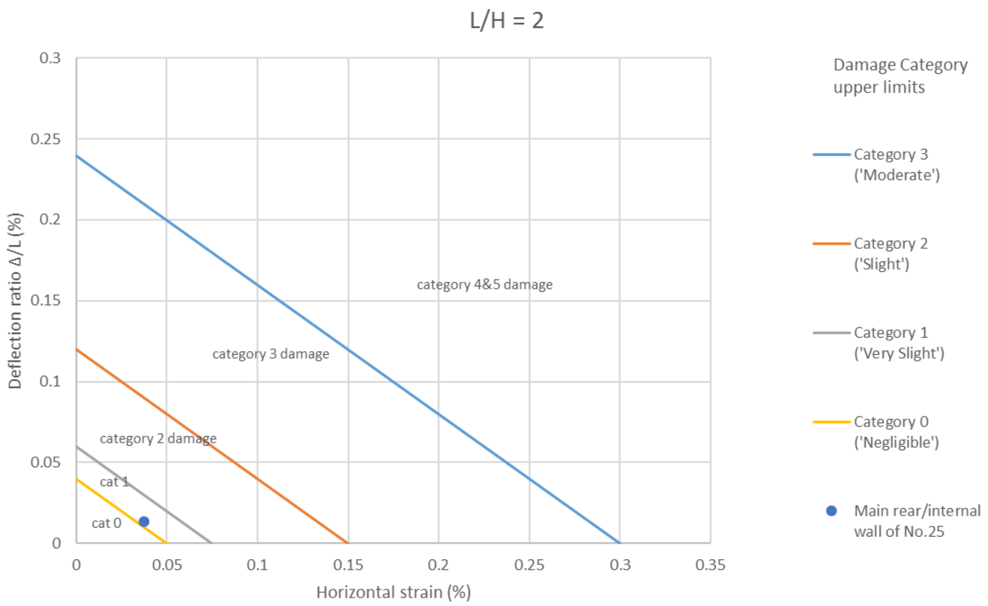

Figure 12: Damage category assessment for rear/internal wall of No.25.

10.6.13 Use of best practice construction methods, as outlined in Section 10.4, will be essential in order to ensure that the ground movements are kept in line with the above predictions.

10.7 Monitoring

10.7.1 Condition surveys should be undertaken of the neighbouring properties before the works commence, in order to provide a factual record of any pre-existing damage. Such surveys are usually carried out while negotiating the Party Wall Agreement 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 (preferably three initial sets, in order to assess any on-going movements from other causes). Readings may revert to fortnightly once all the perimeter walls and the basement slab have been completed. This monitoring should be undertaken with a total station instrument and targets attached at two (or more) levels at the following locations:

- externally on No.23 (on brickwork, not tile hanging) at:
    - the front, middle and rear of the flank wall;
    - close to the middle of the front wall (eg: on left side of right bay window);
    - close to the middle of the rear wall;
- externally, on both front corners and rear left corner to No.25's garage;
- externally, on front and rear corners to No.25's side extension;
- externally on the front, middle and rear of No.25's main flank wall;
- externally, on the front wall to No.25 close to the 25/26 party wall;
- externally, on the rear wall to No.25 close to its junction with the flank wall of the rear projection (access permitting);
- at the client's discretion, since outside any Party Wall Agreement, it would also be sensible to monitor all the load-bearing walls in No.24 which will be underpinned.

10.7.3 The wall movements detected by the monitoring exercise may be caused by rotation, flexing without cracking (especially for walls built using lime mortar) or lateral movements transverse to the plane of the wall. Movements such as these which occur without cracking would all fall within Burland's Category 0, so a twin-track approach to the monitoring will be required, combining both the target monitoring as proposed above and visual observations. Daily inspections of the subject property and the external walls of the adjoining and immediately adjacent buildings should be made and recorded by a member of the contractor's staff. If any new structural cracks appear in the main loadbearing walls, then the appointed structural engineer should be informed and those cracks should be monitored using the Demec system (or similar) on the same frequency as the target monitoring. Additional targets might also need to be installed, at the engineer's discretion, depending on the location of the cracks. It will be important to ensure that any pre-existing cracks in affected load-bearing walls which have weakened their structural integrity should be fully repaired in accordance with recommendations from the appointed structural engineer before any underpinning is carried out (as recommended in paragraph 10.4.6).

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

10.8 Surface Flow and Flooding

Flooding from Rivers, Sea & Reservoirs:

10.8.1 The evidence presented in Section 5 has shown that:

- the site lies within the Environment Agency's Flood Zone 1 which means that it is considered to be at negligible risk of fluvial flooding (from rivers or sea);
- the area is not at risk of flooding from reservoirs, as mapped by Environment Agency;
- 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 The evidence presented in Section 5 has shown that:

- there are no surface water features within 250m of the site (see paragraph 5.3);
- there are no "Detailed River Network entries" within 500m of the site;
- the latest flood modelling by both the Environment Agency and within the Camden SFRA gave a 'Very Low' risk of surface water flooding (the lowest category, which represents the national background level of risk) for No.24 and the neighbouring properties on both Heath Drive and Oakhill Avenue (see Figures 5 & 6).

10.8.3 One of the headwater branches of the 'lost' river Westbourne once flowed in the base of the valley which is now occupied by Heath Drive (paragraph 5.1 and Figure 4). The Westbourne's tributaries were culverted, or diverted into the sewer system, before the area was developed so they are no longer able to receive surface water run-off. A 1168/1194mm by 762mm sewer (probably Victorian brickwork) is present beneath Heath Drive, though it remains possible that there is also an earlier culvert at shallower depth. Whether that culvert, if present, remains connected hydraulically to the perennial surrounding groundwater is unknown.

10.8.4 The *'Floods in Camden'* report (LBC Floods Scrutiny Panel, 2003) and LBC's CPG4 guidance document record that Heath Drive did not flood in either the 1975 or 2002 local pluvial flood events.

10.8.5 The 'Very Low' risk of surface water flooding predicted by the latest flood modelling for No.24's site by both the Environment Agency and the Camden SFRA (see 10.7.2 above) is compatible with the lack of flooding in 1975 and 2002. Surface water flood resistance/mitigation measures for No.24 may therefore be restricted to:

- Ensuring that surface water run-off from the rear garden is not trapped at the rear of the house, by directing run-off from the upper parts of the rear garden towards the ramp which leads around the south-west flank wall into the front garden where possible, by installing a channel drain between the formal lawn and upper side of the patio steps, and installing further drainage within the lower patio.

- Providing raised thresholds at all external doors.
- Removing low air bricks/vents in the rear and flank walls and replacing them with solid bricks.

Changes to Hard Surfacing & Surface Water Run-off; SuDS Assessment:

10.8.6 Surface water gullies were evident in the front driveway/parking area and in the courtyard between the garage and the utility, so it is anticipated that some of the water from those areas already discharges to the combined sewer. No gullies were seen in the patio area to the rear of the house (and none are shown on the topographic survey).

10.8.7 Most of the proposed basement will be located beneath the current footprint of the house, except for the south-eastern corner of the basement, where the proposed swimming pool extends out beneath the rear garden, into the area of the existing rockery retaining wall and flower beds. Most of the proposed single/two-storey extensions will be built in areas which are already developed or fully paved, except for the rear extension, which will extend beyond the existing patio, into the area of the rockery retaining wall and flower beds. These will both cause a moderate increase in paved surface area. The proposed landscaping of the site will cause a further increase in paved surface area, with the creation of a two-tiered patio directly to the rear of the house, a further patio area towards the rear end of the rear garden, and the partial replacement of the flower bed in the front garden with paving. The total proposed increase in impermeable surface area totals approximately 150m², measured from Cowley White's landscaping design drawing (Drg No.001-REV C).

10.8.8 A site-specific assessment of surface water run-off has therefore been undertaken in general accordance with the 'CPG3 – Sustainability' (July 2015) policy document from the London Borough of Camden (LBC). The aim of this assessment is to provide adequate interception/attenuation storage within the proposed development to mitigate the increased surface water-run off caused by the increase in impermeable surface area, by incorporating one or more SuDS into the scheme in accordance with LBC's CPG3 (2015). The assessment has been carried out using the 'surface water storage volume estimation' tool which has been developed by HR Wallingford Ltd and is available on www.uksuds.com (see Appendix H).

10.8.9 The geology on site was recorded during the site-specific ground investigation to comprise Made Ground over London Clay (see Sections 9 & 10.1). As such, the site is classified as within SOIL type 4, as defined by HR Wallingford. A 'climate change allowance factor' of 1.4 has been applied in these analyses, which increases the design rainfall by a factor of 1.4, alongside an 'urban creep allowance factor' of 1.1.

10.8.10 The volume of run-off storage required to mitigate the proposed increase in impermeable surface area for the proposed development was calculated using the 'surface water storage volume estimation' tool with the volume control approach of 'flow control to a max of 2l/s/ha' (see Appendix H), without incorporating any mitigation measures. The total site area used in this calculation is equal to the proposed increase in impermeable surface area of 0.015ha (150m²). The calculated volumes of run-off storage required are summarised in Table 8 below.

Table 8: Summary of required storage volumes for the proposed development		
	Proposed (no mitigation)	**Proposed (with mitigation)**
Storage Volume (m^3)	Interception: 1 (to one significant figure [1 s.f.]) Attenuation: 0 (to 1 s.f.) Long term: 0 (to 1 s.f.) Total: 1 (to 1 s.f.) Additionally, Treatment: 2 (to 1 s.f.)	Interception: 0 (to 1 s.f.) Attenuation: 0 (to 1 s.f.) Long term: 0 (to 1 s.f.) Total: 0 (to 1s.f.) Additionally, Treatment: 0 (to 1 s.f.)

10.8.11 To mitigate the increase in impermeable surface area and subsequent increase in surface water-run off for the proposed development, the replacement of paving and asphalt with resin gravel (manufactured by Breedon) is proposed across most of the front driveway and along the pathway along the south-west flank wall of the house, to create permeable paving (see Cowley White's landscaping design drawing; Drg No.001-REV C). The total decrease in paved surface area from the inclusion of this permeable paving totals approximately 115m², measured from Cowley White's landscaping design drawing (Drg No.001-REV C).

10.8.12 The residual volume of run-off storage required on site, when the permeable paving mitigation measures are included in the scheme, was calculated using the 'surface water storage volume estimation' tool, with the same volume control approach of 'flow control to a max of 2l/s/ha' (see Appendix H). The total site area used in this calculation is again equal to the proposed increase in impermeable surface area of 0.015ha, and the mitigation includes an allowance in the tool for an impervious area of 0.0115ha (115m²) drained by infiltration. The calculated residual volumes of run-off storage required are summarised in Table 8 above; these were found to be 0m³. It is therefore indicated that the 115m² of permeable paving (resin gravel replacing impermeable paving and asphalt) proposed on site will entirely mitigate the 150m² proposed increase in paved surface area. Inherent in these calculations and the proposed mitigation is the fact that the entire forecourt area and the pathway along the south-west flank wall of the house currently drains to the mains system, either directly via the gullies or by run-off onto the highway and into the highway drains, so replacing most of this area with permeable surfacing is a net benefit.

10.8.13 It should be noted that for these permeable surfaces to mitigate the increase in impermeable surface area, they must be formed from **resin-bound** rather than resin-bonded gravel (i.e. they must be both porous and permeable). A permeable sub-base must be included beneath the gravel, of sufficient porosity and thickness to allow for storage of surface water run-off and its gradual release. Infiltration into the London Clay is likely to be limited (and not recommended on site anyway due to the high groundwater levels at times, and because concentrated local increases in groundwater levels would be detrimental to the stability of the slopes concerned), so the sub-base must be drained (gradually) into the combined sewer system via a suitable flow control device. However, limited infiltration on site may be possible, so where possible, the resin-bound gravel incorporated into the site should at least partially drain into adjacent soft-landscaped areas using a cross-fall, as recommended by Breedon in their literature, and partially drain into the sub-base and then into the sewer system. The sub-base should be separated from the underlying soils by a non-woven geotextile separator such as Terram 1000 (or suitable approved alternative), laid and lapped in accordance with the manufacturer's instructions, in order prevent mixing of the soils and the sub-base).

<u>Sewer Flooding:</u>

10.8.14 The Camden SFRA indicates that Thames Water had no records of flooding from public sewers affecting this postcode area (see 5.13). 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 also outside of the Council's influence. Thus, the probability of future sewer flooding affecting No.24 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.

10.8.15 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.

10.8.16 Camden's CPG 4 requires all basements to be "*protected from sewer flooding by the installation of a positive pumped device*" (paragraph 5.11). Non-return valves and pumped loop systems must therefore be fitted on the drains serving the basement, in order to ensure that water from the mains sewer system cannot enter the basement when/if the adjacent sewer is operating under surcharge. All drains which discharge via the same outfall(s) as the basement must be protected, including those carrying foul water and roof/surface water. The loop systems are generally required to rise above ground level in order to provide complete protection. A battery powered reserve pump should be fitted to ensure that the system remains functional during power cuts.

10.8.17 The pumped loops must rise high enough to create sufficient pressure head to open the non-return valves when the mains sewer flow is surcharged to ground level, otherwise the basement would once again be vulnerable to flooding while the surcharged flow continues. If it is not possible to achieve a sufficient rise of the loop then temporary interception storage would be required, to hold temporarily the predicted maximum volume of water from all relevant sources which discharge via the valve-protected outfalls (including surface water from the various roofs and the rear/side patio, and foul water), for the duration of the predicted surcharged flows in the sewer. This temporary interception storage would require formal design to ensure satisfactory performance.

Cumulative Impact:

10.8.18 No cumulative impact on surface water flooding would be expected in the vicinity of No.24 from construction of the proposed basement.

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 (10.2.7).
- 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 (10.4.6).
- Foundations for the new extensions which will adjoin the proposed basement should, be formed from a system of piles and groundbeams (see paragraphs 10.4.16-10.4.26).
- Surface water flood resistance/mitigation measures, as listed in paragraph 10.8.5.
- A Sustainable Drainage System (SuDS) on site comprising permeable paving (porous, permeable resin-bound gravel with a sub-base layer) should be included to replace existing impermeable paving and asphalt (across most of the front driveway and along the pathway along the south-west flank wall of the house, as per Cowley White's landscaping design drawing; Drg No.001-REV C) (see paragraphs 10.8.6-10.8.13).
- Non-return valves and/or pumped above ground loop systems should be fitted to the drains serving the basement and lightwell, in order to ensure that water from the sewer system cannot enter the basement when the mains sewer is operating under surcharge (see paragraphs 10.7.12 & 10.7.13).