9.7 A standpipe was installed in BH1 to a depth of 4.0m bgl, and water level readings were taken on 13th February 2018, at which time the borehole was found to be dry and on 16th March when the water level was 3.68m bgl. These levels may not have equilibrated fully with water pressures in the clay, so may not have been representative of the groundwater levels/pressures in the surrounding ground.

Laboratory Testing:

9.8 Laboratory tests were carried out by Geolabs Ltd on samples recovered from the borehole and trial pits. The testing comprised classification tests, including moisture content and plasticity, and chemical testing to assess the potential for acid or sulphate attack on buried concrete. The results are presented in Appendix F.

9.9 Plasticity tests were performed on two samples of the Weathered London Clay, recovered from BH1 at depths of 1.35-1.40m and 3.70-3.80m bgl, and on a sample of the 'Head Deposits/Soliflucted London Clay' recovered from TP3 at a depth of 0.70m – 0.80m bgl. All three samples were found to be of Very High Plasticity, as classified by BS5930 (2015), and High Volume change potential, as defined by the NHBC (NHBC Standards, 2018, Chapter 4.2, Building near Trees). These results are displayed in Figure 9.

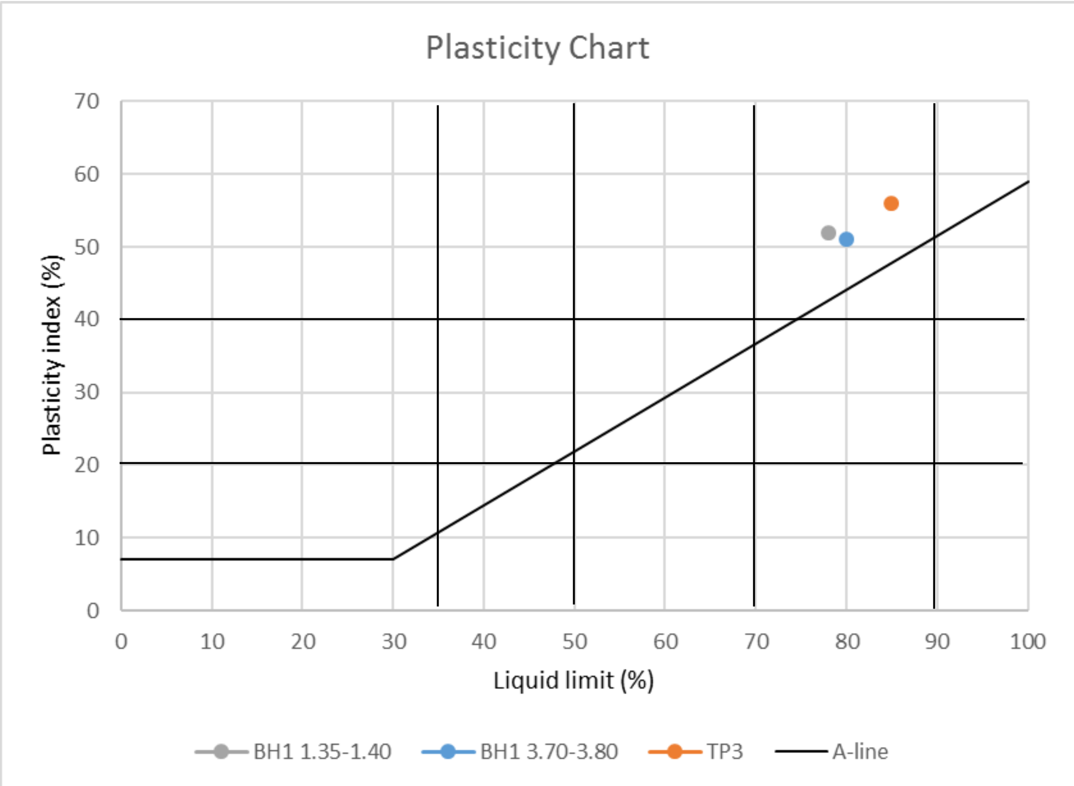

Figure 9: Plasticity chart for samples recovered from BH1

9.10 The water contents of fourteen samples recovered from BH1 between 1.00m and 6.00m bgl were found to vary between 28.7% and 34.0%. The plotted profile in Figure 10 shows an overall slight decrease in water content with depth in the

Weathered London Clay. The water content of a sample from TP3 at 0.70 - 0.80m was found to be 37.0%, within the Head/Soliflucted London Clay.

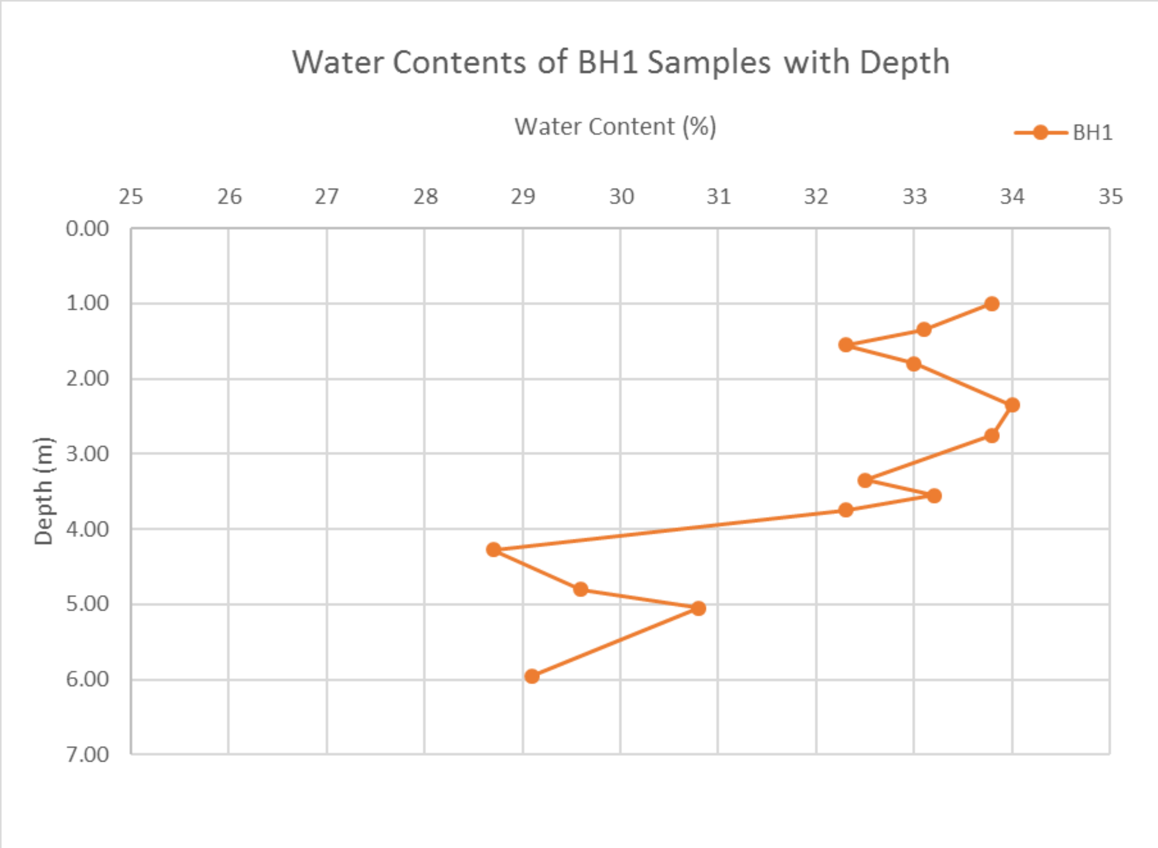

Figure 10: Profile of Water Content in BH1 with depth.

9.11 The chemical tests were undertaken on a total of three samples in order to assess the potential for acid or sulphate attack on buried concrete, in accordance with BRE Special Digest 1 (2005). The samples were recovered from BH1 at 0.50-0.60m, 2.30-2.40m and 3.50-3.60m bgl, so included samples from the Made Ground and the Weathered London Clay. The following ranges of results were recorded:

pH value:	7.5 – 8.0
Water-soluble sulphate:	80 – 2000 mg/l
Total Sulphur:	0.08 – 0.41 %

Calculations following BRE Digest SD1 gave 'derived' values:

Total Potential Sulphate (TPS):	0.24 – 1.23%
Oxidizable sulphides:	0 – 0.15%.

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

DS-2: Sample from the Made Ground

DS-2 to DS-4 (based on TPS): Samples from the Weathered London Clay

Non-technical Summary – Stage 3:

9.12 The site-specific ground investigation at No.71 Goldhurst Terrace recorded CLAYS of the (weathered) London Clay Formation, as mapped by the British Geological Survey (BGS). Soliflucted weathered CLAYS (Head deposits) which would have been formed from the underlying clays by slope movements during the Ice Age were found in all exploratory holes above the London Clay. Made Ground was also found overlying the natural strata across the site, this had a variable thickness of 0.33-0.50m across the site, and was predominantly variably gravelly, sandy CLAY.

9.13 No groundwater entries were recorded in BH1 during drilling. During the subsequent monitoring period, the standpipe remained dry after 6 days and a month later the water level was still only at 3.68m bgl, which is lower than would be expected in a London Clay site. These findings suggest that the permeability of the ground around the boreholes is very low.

9.14 High sulphate and Total Potential Sulphate concentrations were found in one sample of the Weathered London Clay. These must be taken into account when selecting the concrete mix design (see Section 10.4.14).

10. STAGE 4 – BASEMENT IMPACT ASSESSMENT

10.1 Conceptual Ground Model

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

- Made Ground: Made Ground was discovered within all of the exploratory holes and varied in thickness from 0.33m to 0.50m, with a proven maximum depth of 0.70m below ground level (bgl) in both the front and rear gardens. Descriptions of the Made Ground were typically "slightly sandy to sandy, variable gravelly CLAY" with included fragments of brick, mortar, charcoal, slate and flint; 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.
- Head Deposits/Soliflucted London Clay: Soft to firm CLAYS with variable amounts of fine to medium flint gravel, occasional polished shear surfaces and extensive disruption of the soil fabric were interpreted as Head Deposits or Soliflucted London Clay. These clays were found overlying the London Clay from which they were derived, and were recorded to the base of the three trial pits and to a depth of 1.0m in the borehole. No extensive, slope-parallel shear surfaces were seen (which typically occur at the base of these clays), though this does not preclude their presence. The extent and thickness of these Head Deposits is highly variable, with the ground investigations for No.67 and No.63 Goldhurst Terrace not recording any such deposits (though those boreholes and trial pits may not have been logged by anyone with experience in identifying such clays).
- Weathered in-situ London Clay: Firm to stiff, becoming very stiff with depth, mid brown to orange-brown CLAYS were found directly beneath the probable Head deposits in BH1, and extended to the base of the borehole 6.0m below ground level. Pockets and thin partings of fine sand were scattered through these clays, sometimes associated with selenite crystals (a form of gypsum which can be aggressive to buried concrete), and localised areas veining and mottling associated with decaying roots. These clays are fissured, which reduces their shear strength, and will undergo heave movements in response to unloading by the basement excavation.
- London Clay Formation ('un-weathered'): The boundary between the weathered and unweathered London Clay Formation was not found during this site investigation. The closest ground investigation to the site, 67 Goldhurst Terrace found the top of the unweathered London Clay at 8.4m bgl, however logs of other boreholes in the area show this at depths ranging between 7.62m to 10+m.

Hydrogeology

- Perched groundwater may be expected at least during the winter and spring seasons, however no groundwater was recorded during the site investigation. During the subsequent monitoring period, the water level in the standpipe was still only at 3.68m bgl some 37 days after drilling the borehole, which is lower than would be expected in a London Clay site in winter (in February/March 2018). These findings suggest that the permeability of the ground around the boreholes is sufficiently low to have prevented the groundwater level in the standpipe equilibrating with (ie: being representative of) the water pressure in the surrounding clays.
- Groundwater pressures are expected to be essentially hydrostatic within the depth of current interest in the London Clay except where modified by tree roots or artificial influences. Groundwater flow through these clays is likely to be limited to minor seepage through any of the silt/sand partings which are sufficiently interconnected. In BH1, occasional partings of fine sand were recorded throughout the Weathered London Clay; however, none were of sufficient size to warrant separate identification.
- 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.
- The walls of the cellar were found to be moist to very moist. The origin of this moisture was probably a combination of normal moisture in and perched above these high plasticity clays, together with water leaking from the defective soil/vent pipe at the rear of the house.

10.1.2 The hydrogeological regime outlined above will be affected by long-term climatic variations as well as seasonal fluctuations, all of which must be taken into account when selecting a design water level for the permanent works. No multi-seasonal monitoring data are available, so a conservative approach will be needed, in accordance with current geotechnical design standards which require use of 'worst credible' groundwater levels/pressures. See paragraph 10.2.8 for the recommended design groundwater level.

10.1.3 No railway tunnels are known to pass below or close to the site. The NW Storm Relief Sewer is understood (from Thames Water's drawings) to run beneath part of Goldhurst Terrace to the south-west of No.71. A services search has been undertaken and found no records of any adopted services or other infrastructure beneath the property, other than the services to this building. These searches will not identify any private services.

10.2 Subterranean (Groundwater) Flow – Permanent Works

10.2.1 The Made Ground comprises variably sandy, variably gravelly clays, so is generally low permeability and will permit little or no flow of perched groundwater. The existing foundations should prevent most downslope flow from rear to front of the site, although the aperture in the rear wall which was allowing water from the defective soil and vent pipe to flow under the building must be sealed. Thus, significant flow in the Made Ground, if any, will primarily occur where service trenches or granular pipe bedding facilitates flow. The suspected Head deposits, which extended to 1.0m bgl, may also be more slightly permeable than the clays that they are derived from. Groundwater in the backfill to footing trenches is typically static (until excavations are dug into/through the backfill).

10.2.2 The lack of record of silt/sand horizons, or groundwater entries, in the weathered London Clay in BH1 (other than the thin partings, maximum thickness recorded was 50mm) indicates that these clays are likely to be towards the lower end of the permeability scale, so will permit little or no flow of groundwater. However, the lack of groundwater entries into boreholes from the London Clay during drilling does not necessarily mean that groundwater was absent, rather the low permeability of the clays merely means that the flow rate was too slow for groundwater entries to occur before the instrumentation was installed in the borehole, and any water in silt/sand partings was potentially sealed in by smearing of clays during the drilling process.

10.2.3 The basement will be founded at approximately 3.40-3.60m below internal floor level (see paragraph 3.3). It will therefore extend down through the Head deposits and into the underlying "stiff" clays. This in-situ London Clay contains thin partings of silt/sand; flow through these partings, if any, would only occur where the partings are sufficiently interconnected, which is generally rare, and even then is likely to involve very low flow rates and volumes. The proposed basement will not increase the width of the existing obstruction to flow (if any) created by the existing foundations and the cellar so, given the anticipated negligible flow in the London Clay, the proposed basement is considered acceptable in relation to groundwater flow.

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

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

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

10.2.7 The National House Building Council published new guidance on waterproofing of basements in November 2014 (now NHBC Standards, 2018, Chapter 5.4). Compliance would be compulsory if an NHBC warranty is required, otherwise it may provide a useful guide to best practice.

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

10.2.9 The basement structure must be designed to resist the buoyant uplift pressures which would be generated by groundwater at ground level. For the founding depths currently estimated, the uplift pressures would be up to 34kPa (factored).

10.3 Subterranean (Groundwater) Flow – Temporary Works

10.3.1 Despite the lack of any groundwater entries into the borehole, local groundwater entries into the excavations for the basement may occur, especially from the Made Ground on the upslope side of the basement, though, on current evidence, they should be manageable by sump pumping provided that they are not being fed by defective drains or water supply pipes. It would be prudent to ensure the external isolation stopcock is both accessible and operational before the start of the works. An appropriate discharge location must be identified for any groundwater removed by sump pumping.

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

10.3.3 The unloaded clays at/beneath formation level will readily absorb any available water which would lead to softening and loss of strength, so these clays must be protected from all sources of water, as described more fully in paragraph 10.4.8 below.

10.3.4 During the site inspection the existing cellar to No.71 was found to be moist (see Section 6.9), to a greater extent than is typical for such cellars in the London Clay. 'Blown' areas of replacement plaster provided further evidence of this moisture. A previous site inspection in the area, undertaken on 4th March 2014, reported that No.69 had experienced flooding in the cellar. The owners or occupiers of No.69 should be asked whether the cause of flooding to their cellar has been identified or rectified, and temporary works requirements should be reviewed in light of any answers given.

10.4 Slope and Ground Stability

10.4.1 With slope angles of approximately 1.0-3.0° upslope of this property, the proposed basement excavation raises no concerns in relation to slope stability.

10.4.2 It is understood from Green Structural Engineering Ltd (GSE) that the basement's perimeter retaining walls will be constructed using reinforced concrete (RC) underpinning techniques beneath the existing building. Where the basement extends beyond the existing building, for the front lightwell and in the rear garden, similar 'L' shaped cast-in-situ RC retaining walls, will be constructed in panels of limited width using the same 'hit and miss' methodology employed for the underpins.

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

10.4.3 Underpinning methods involve excavation of the ground in short lengths (not exceeding 1.0m is recommended) in order to enable the stresses in the ground to 'arch' onto the ground or completed underpinning on both sides of the excavation. Loads from the structure above will similarly arch across the excavation, provided that the structure is in good condition. Paragraphs 3.2 & 3.4 present the estimated founding (formation) level and depths of excavation which are likely to be required for the underpins, retaining walls and central basement slab.

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

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

A high quality of workmanship and use of best practice methods of temporary support are therefore crucial to the satisfactory control of ground movements alongside basement excavations (see 10.4.5 to 10.4.8 below). Any cracks in load-bearing walls which have weakened their structural integrity should be fully repaired in accordance with recommendations from the appointed structural engineer before any underpinning is carried out.

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

- It should be assumed that full face support will be required to the Made Ground, the probable Head deposits and any natural granular soils exposed in the excavations.
- Closely spaced support should be adequate in the stiff or very stiff clays of the London Clay Formation, depending on the degree of fissuring.
- Temporary support must also be installed to support all the new underpins, and must be maintained until the full permanent support has been completed, including allowing time for the concrete to gain adequate strength.

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

10.4.6 In accordance with normal health and safety good practice, the requirements for temporary support of any excavation must be assessed by a competent person at the start of every shift and at each significant change in the geometry of the excavations as the work progresses. London Clay is usually fissured; such fissures can cause seemingly strong, stable excavations to collapse with little or no warning. Thus, in addition to normal monitoring of the stability of the excavations, a suitably competent person should check whether such fissuring is present and, if encountered, should assess what support is appropriate.

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

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

10.4.9 A preliminary construction sequence will be provided in the structural engineer's Construction Method Statement (by GSE). That can only be preliminary because the appointed contractor will be responsible for the temporary works and preparation of the final construction plan.

Design Considerations:

10.4.10 Geotechnical 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 10.4.11 below);
- Dead and live loads from the superstructure, including loads from the adjoining No's 69 & 73 which are carried on the party wall;
- Loads from all adjoining/adjacent walls in No's 69 & 73 which are founded within the relevant active earth pressure zone;
- Vehicle loadings on the forecourt and normal surcharge allowances elsewhere;
- Swelling displacements/pressures from the underlying clays;
- A design groundwater level at ground level, as described more fully in paragraph 10.2.8;
- Precautions to protect the concrete from sulphate attack; high sulphate levels were identified in the Weathered London Clay (Class DS-3), which is not unusual, while the oxidizable sulphides indicated that pyrite may not be present at significant levels.

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

Made Ground (clays):	Unit weight, γ_b:	17.0 kN/m^3
	Effective cohesion, c':	0 kPa
	Angle of internal friction, φ':	25°
Soliflucted London Clay/ Head:	Unit weight, γ_b:	19.0 kN/m^3
	Effective cohesion, c':	0 kPa
	Angle of internal friction, φ':	14°
London Clay Fm:	Unit weight, γ_b:	20.0 kN/m^3
	Effective cohesion, c':	0 kPa
	Angle of internal friction, φ':	22°

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

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

10.4.12 Normal good practice in foundation construction requires progressive stepping up between foundations of different depths beneath a single structure. Subject to agreement under the Party Wall Act negotiations, transitional underpins should be considered for all adjoining load-bearing walls in No's 69 & 73 except where existing cellars already provide adequate transition.

10.4.13 The basement will be founded sufficiently deep to be unaffected by the roots from the large Ash(?) in No.69's rear garden. The tree's canopy does not reach the rear of the proposed basement and extension, however an arboriculturalist should be asked to confirm whether the basement might have any impact on the tree's root protection area.

10.5 PDISP Heave/Settlement Assessment

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

10.5.2 Figure G1 in Appendix G illustrates the layout of the proposed basement based on drawings by Opera Architects (Drg No.17_27//2 Rev.03), along with the layout of PDISP zones used to model the underpins, lightwells and basement slab, based on information received from Green Structural Engineering (GSE). The load takedown data for the proposed building have also been provided by GSE, and are summarised on the annotated copy of Opera's 'Proposed Set: Ground Floor' (Drg No. 17_27_PR_1), an extract of which is presented in Figure G2.

10.5.3 The overall dimensions of the proposed basement are approximately 6.07m wide by 23.61m long (including the front lightwell) to the outside of the external walls of the basement. The basement levels and depths of excavation are given in Section 3. For the purpose of these analyses, the founding depth for the underpins was taken as 3.60m below Ground Floor level of No.71, and the depth of the basement slab was taken as 3.45m below the same level. This gave gross reductions of vertical stress (unloading) which ranged from 37.2kPa to 68.4kPa.

10.5.4 Table 4 presents the net bearing pressures for four main stages of the stress changes which will result from excavation and construction of the basement (see 10.5.8 below for details of those stages). The basement slab has been divided into two zones (Zones 11 & 12) to account for the existing step down towards the rear of the property. The front lightwell has been modelled with an assumed uniform slab thickness of 300mm (Zone 7). Superimposed Zones 10, 13 and 15 (coloured green) have been used to allow for the existing cellar beneath No.71, and Zone 14 (also coloured green) has been used to allow for the lower floor height in this section of the existing property.

Table 4: Net changes in vertical pressure for PDISP Zones			
ZONE	**Net change in vertical pressure (kPa)**		
#	**Stage 1**	**Stage 2**	**Stages 3 and 4**
1	-20.26	-20.26	-20.26
2	-31.47	-31.47	-31.47
3	-28.52	-28.52	-28.52
4	-30.62	-30.62	-30.62
5	11.18	11.18	11.18
6	20.79	20.79	20.79
7	-32.25	-32.25	-32.25
8	-1.19	-1.19	-1.19
9	-8.81	-8.81	-8.81
10	28.31	28.31	28.31
11	0.00	-65.55	-59.30
12	0.00	-63.84	-57.59
13	28.31	28.31	28.31
14	1.71	1.71	1.71
15	2.70	2.70	2.70

Ground Conditions:

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

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

10.5.7 The undrained shear strength, Cu, at the top of the stratum is based on a line of best fit through the SPT profile for the London Clay, which is compatible with the typical 7.5kPa increase in undrained shear strength per metre depth in the London Clay.

Table 5: 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	3.1 15.8	55 150	27.5 75	15.5 45
Where: Undrained Shear Strength, Cu profile is: Cu = 55 + 7.5z where z = depth below the top of the stratum (3.1m bgl) Undrained Young's Modulus, Eu = 500 * Cu Drained Young's Modulus, E' = 0.6 Eu				

PDISP Analyses:

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

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

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

Heave/Settlement Assessment:

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

10.5.11 The ranges of predicted short-term and long-term movements for each of the main parts of the proposed basement are presented in Table 6 below. These analyses indicated that the perimeter walls are likely to experience negligible to very minor heave/settlement movements in Stage 1, followed by minor heave movements in later stages, whereas the basement slab is likely to experience slightly greater heave movements. Only the party wall between No's 71 and 73 Goldhurst Terrace beneath the 'Main House' is likely to experience settlement after Stage 1; this is a result of the reduced excavation due to the existing cellar.

Table 6: Summary of predicted displacements

Location	Stage 1 (Figure D3)	Stage 2 (Figure D4)	Stage 3 (Figure D5)	Stage 4 (Figure D6)
Front lightwell	0mm to 1.5mm Heave	0.5mm to 1.5mm Heave	0.5mm to 1.5mm Heave	0.5mm to 2.5mm Heave
Front Wall of No.71	0mm to 2mm Settlement	1mm Settlement to 2mm Heave	1mm Settlement to 1.5mm Heave	2mm Settlement to 3mm Heave
Party Wall between No's 71/73 beneath 'Main House'	0mm to 1.5mm Settlement	1mm Settlement to 3mm Heave	1mm Settlement to 2.5mm Heave	2mm Settlement to 4.5mm Heave
Party Wall between No's 71/73 beneath existing extensions	0mm to 1mm Heave	0.5mm to 3.5mm Heave	0.5mm to 3mm Heave	1mm to 5.5mm Heave
Rear wall of No.71	0.5mm to 1.5mm Heave	0.5mm to 3.5mm Heave	0.5mm to 3mm Heave	1mm to 5.5mm Heave
Boundary (fence) between No's 71/69	0.5mm to 1.5mm Heave	1mm to 4mm Heave	1mm to 3.5mm Heave	1.5mm to 6mm Heave
Party Wall between No's 71/69 beneath 'Main House'	0.5mm Settlement to 1mm Heave	0mm to 2.5mm Heave	0mm to 2mm Heave	0mm to 4mm Heave
Rear wall of 'Main House' (internal at basement level)	0.5mm to 1mm Heave	1mm to 4mm Heave	1mm to 3.5mm Heave	2mm to 6mm Heave
Basement slab	1.5mm Settlement to 1.5mm Heave	1mm to 5mm Heave	1mm to 4.5mm Heave	1.5mm to 8mm Heave

10.5.12 All the short-term elastic displacements would have occurred in Stages 1-3, before the concrete of the basement slab has set/cured, so, in theory, only the post-construction incremental heave/settlements (from Stages 3 to 4) should be relevant to the slab design, subject to the detailed construction sequence employed. The analyses indicated that the theoretical maximum predicted post-construction displacements beneath the slab would range from zero to 4mm 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 rigorous calculations of predicted ground movements are not practical. However, provided that the temporary support follows best practice, then extensive past experience has shown that the bulk movements of the ground alongside underpins for a single-storey basement should not exceed 5mm horizontally.

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

10.6.3 No evidence has been found on the London Borough of Camden's planning website for modern basements below the adjoining No's 73 & 69 Goldhurst Terrace, although these properties are likely to have existing cellars, similar in footprint to No.71's. The uniform founding level for the proposed basement means that the potentially critical locations will be determined by the displacements predicted by the PDISP analyses and the geometries of the adjoining buildings. For these damage category assessments, we are interested in the ground movements at the foundation level of the neighbouring buildings, whereas the empirical data for ground movements alongside excavations presented in CIRIA Report C580 (Gaba et al, 2003), concerns movements at ground surface (and presents data for embedded retaining walls, but, as no equivalent data exist for underpins, this data is the best available so must be interpreted very cautiously).

10.6.4 The worst case scenarios as predicted by the PDISP analysis will occur along the party wall between No's 71 and 73 beneath the 'Main House', and in the south-west corner of the front wall, where the analyses indicated that the settlements will be 1.50mm and 2.0mm respectively in Stage 1 (the greater heave movements in the later stages would be beneficial). There are no plans available for No.73, however existing plans for No.71 show an internal wall transverse to the zone of greatest settlement along the party wall, but opposite the cellar. So, even if this wall continues through the four properties in this section of the terrace, the depth of excavation alongside this wall below No.71's cellar will be relatively modest. The foundation depth of the cellar will also benefit the front wall of the house, which additionally benefits from being at the corner of the proposed basement and by not being a continuous wall (due to the front bays of these properties). For these reasons, only the main rear wall of No.73 (and its continuation into No.75) has been assessed as this wall is beyond the footprint of the cellar, so does not benefit from the additional foundation depth there.

10.6.5 The damage category assessment which has been undertaken for the worst-case scenario discussed above considered:

- ground movements arising from the vertical stress changes, as assessed by the PDISP analyses (see Section 10.5), including an allowance for the stiffness of the foundations;
- ground movements alongside the proposed underpins and retaining walls caused by relaxation of the ground in response to the excavations.

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

Rear Wall of No's 73 and 75 Goldhurst Terrace:

10.6.6 The relevant geometries are:

Depth of excavation (below ground level) = **3.30m**

Width of zone of affected soils = 3.30 x 4 = **13.2m**, so will extend the full width of the adjoining No.73 & 75, and beyond the break in the terrace properties into No.77.

Combined width (L) of No's 73 and 75 = approximately **11.5m**, measured from existing plans of No.71).

Assumed footing depth of rear walls = 0.7m, based on footing in TP2 (rear wall of No.71)

Height of wall, from footing to eaves (H) = 8.66m + 0.7m
= **9.36m.**

Hence, L/H = **1.23.**

10.6.7 Thus, for an anticipated 5mm maximum horizontal displacement, the strain beneath No's 73 and 75 would be in the order of $\varepsilon_h = 4.34 \times 10^{-4}$ (0.0434%).

10.6.8 The maximum settlement predicted by the PDISP analysis alongside the rear wall of No.73, in Stage 1 was 0mm (see Figure G3 in Appendix G), thus can be ignored. The typical settlement caused by relaxation of the ground alongside the basement in response to the 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) in the CIRIA Report C580. The settlement profiles are then summed to find the maximum deflection, Δ. The maximum Δ = 1.84mm, which represents a deflection ratio, $\Delta/L = 1.60 \times 10^{-4}$ (0.0160%)

10.6.9 Using the graphs for L/H = 1.0, these deformations represent a damage category of 'very slight' (Burland Category 1, ε_{lim} =0.05-0.075%), close to the boundary with Burland Category 0, 'negligible', as given in CIRIA SP200, Table 3.1, and illustrated in Figure 11 below.

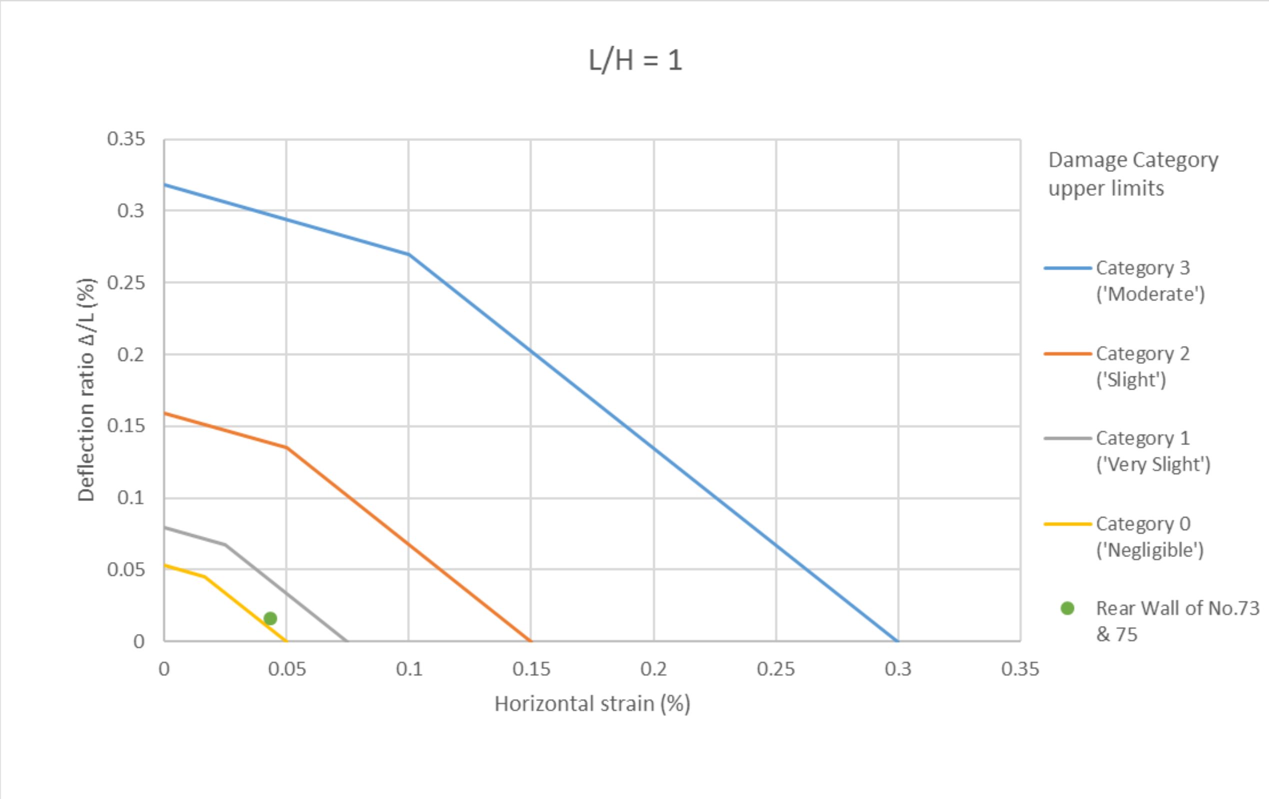

Figure 11: Damage category assessment for rear walls to No's 73 & 75.

10.6.10 For No.69 at the other end of this terrace, the shorter length of the walls (front, rear and internal) means that the L/H ratio will be more favourable, and the settlements predicted by the PDISP analyses are less. Thus, by inspection, it is clear that the damage category for No.69 will be the same as, or lower than, that found above for No's 73 (& 75).

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

10.7 Monitoring

10.7.1 Condition surveys should be undertaken of the neighbouring properties before the works commence, in order to provide a factual record of any pre-existing damage. Such surveys are usually carried out while negotiating the Party Wall Agreements and are beneficial to all parties concerned.

10.7.2 Precise movement monitoring should be undertaken weekly throughout the period during which the basement walls and slab are constructed, with initial readings taken before excavation of the basement starts. Readings may revert to fortnightly once all the perimeter walls and the basement slab have been completed. This monitoring should be undertaken with a total station instrument and targets attached at a minimum of two levels at the following locations:

- internally, at three equally spaced locations on the 69/71 party wall;
- internally, at five equally spaced locations on the 71/73 party wall;
- externally, on the front and rear walls of No.69, on the centreline of the 69/71 party wall;
- externally, on the front wall of No.69, on the line of the left flank wall, and on both rear corners of the rear projection;
- externally, on the front and rear walls of No.73, on the centrelines of the 71/73 and the 73/75 party walls;
- externally, on the rear corners of No.73's rear projection and rear extension;
- at the client's discretion, since outside the Party Wall Agreements, it would also be sensible to monitor all other load-bearing walls in No.71.

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 10.4.5).

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

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

10.8.1 The evidence presented in Section 5, paragraphs 5.10 & 5.11, has shown that:

- the site lies within the Environment Agency's Flood Zone 1 which means that it is considered to be at negligible risk of fluvial flooding (from rivers or sea);
- the area is not at risk of flooding from reservoirs;
- there are no flood defences, no areas benefitting from flood defences and no flood storage areas within 250m of the site.

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

10.8.2 There are no surface water features within 250m of the site (see paragraph 5.11).

10.8.3 The site is known to lie about 150m to the east of one of the former tributaries to the 'lost' river Westbourne (as described in Section 5 above). These tributaries have been culverted or diverted into the sewer system a century ago, so they are no longer able to receive surface water run-off. Whether the culverts remain connected hydraulically to the perennial surrounding groundwater is unknown, as discussed in more detail in paragraphs 5.1 to 5.4.

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

10.8.5 The latest flood models by both the Environment Agency, and by URS for the Camden SFRA, gave a 'Very Low' risk of surface water flooding, the lowest category which represents the national 'background' level of risk, for No.71's site, and for all other properties in the vicinity on Goldhurst Terrace (see Figures 6 & 7). The run-off route from the 'Low' risk of flooding on the upslope side of No's 39/41/43 Fairfax Close, which adjoin No.71's rear garden, is expected to be down Fairfax Close so will not require any additional precautions for the proposed basement. Thus, flood mitigation measures to protect the basement from local surface water flooding, may be restricted to:

- Providing upstands to the retaining walls around the lightwells in order to prevent surface water from the adjoining areas from draining into the lightwells. For the front lightwell, where the ground level slopes away from the lightwell, only a nominal upstand (minimum 50mm) will be required. For the rear lightwell with the access steps, the top of the upstand should be at the same level as the ground floor inside the house or 150mm above the adjacent ground level, whichever is the higher.
- Installing raised thresholds to the external doors in the lightwells.
- Installing temporary interception storage for surface water which might become trapped in the rear gardens (see 10.8.12 below).

<u>Changes to Hard Surfacing & Surface Water Run-off:</u>

10.8.6 The location for the front lightwell is currently occupied partially by the walled area adjacent to the front bay, formed of flint pebbles overlying concrete paving slabs, and partially by a parking area formed of concrete paving slabs bedded on sand. This area slopes towards the Goldhurst Terrace, and surface water run-off will discharge into the mains drainage via the highway gullies. Thus, the front lightwell will not alter the area of hard surfacing or the volume of water currently being discharged into the mains drainage.

10.8.7 The location for the rear extension, lightwell, staircase and 'walk-on glass' is currently occupied by the courtyard, which is predominantly formed of pea gravel overlying concrete or concrete tiles bedded in sand. It has been assumed that this concrete surfacing is continuous throughout the courtyard area, as it was recorded in both TP2 and TP3 (see Appendix F, Figures GI-04 and GI-05), with the exception of a narrow strip of former flower bed alongside the boundary fence (approximately 0.3m wide), where topsoil was found beneath the gravel and geosynthetic membrane. In addition, the proposed rear extension will overlap the decking area by 1.0m. The total increase in hard surfacing has therefore been estimated at approximately $8m^3$ (this assumes that the concrete surfacing does drain to a gully, as is typical for such Victorian rear courtyards, which is now covered under the pea gravel and woven geosynthetic).

10.8.8 The potential change in discharge to the mains drainage system that this very small increase in hard surfacing would potentially cause is virtually insignificant, but could be mitigated by the inclusion of one or more appropriate Sustainable Drainage Systems (SuDS) in the scheme. The options for simple SuDS in this circumstance, include:

- An allowance for soft landscaping elsewhere within the site – such as in the currently paved section of the front garden;
- Replacement of the concrete paving slabs in the front parking area with permeable paving;

- Inclusion of a green roof on the single-storey extension, although these provide only limited benefit once they become saturated in storm conditions, or frozen.

Sewer Flooding:

10.8.9 Thames Water has no records of flooding from public sewers affecting No.71 (see 5.17). However, no drainage system can be guaranteed to have adequate capacity for all storm eventualities and all drainage systems only work at full capacity when they are properly maintained, including emptying gullies and regular checks of the sewers themselves for condition and blockages. Maintenance of the adopted sewers is the responsibility of Thames Water, so is outside the Applicant's control and largely outside of the Council's influence. Given the lack of any recorded history of sewer flooding affecting this property, the probability of future sewer flooding affecting No.71 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.

10.8.10 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.

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

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

10.8.13 If a non-return valve is fitted with a pumped above-ground loop, then the loop must rise high enough above ground level to create sufficient pressure head to open the valve when the sewer flow is surcharged to ground level, otherwise the basement would once again be vulnerable to flooding while the surcharged flow continues. If it is not possible to achieve a sufficient rise of the loop above ground level, then temporary interception storage should be provided as recommended above.

10.9 Mitigation

10.9.1 The following mitigation measures should be implemented:

- In the unlikely event that the basement excavations encounter a local deposit of more permeable soils, of sufficient thickness to permit significant flow, then an engineered groundwater bypass should be provided (see paragraph 10.2.5).
- Cracks in load-bearing walls which have weakened their structural integrity should be fully repaired, in accordance with recommendations from the appointed structural engineers, before any underpinning is carried out (10.4.4).
- Subject to Party Wall Agreement negotiations, transitional underpinning blocks should be included beneath the adjoining walls to No.69 & 73, except where existing cellars would provide sufficient transition (10.4.12).
- Provision of upstands at the top of the retaining walls around the lightwells and installation of raised thresholds to the external doors in the lightwells (10.8.5).
- Use of one or more simple type of SuDS as mitigation for the estimated $8m^3$ increase in hard surfacing: compensatory soft landscaping, permeable paving or green roof (10.8.8).
- Non-return valves and/or pumped above ground loop systems should be fitted to the drains serving the basement and lightwells in order to ensure that water from the sewer system cannot enter the basement when the mains sewer is operating under surcharge (see paragraphs 10.8.11 to 10.8.13).

11. NON-TECHNICAL SUMMARY – STAGE 4

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

11.2 The proposed basement is considered acceptable in relation to the likely negligible groundwater flow in the natural strata, while flow in the Made Ground around the house is likely to be limited to flow in backfill to service trenches or granular pipe bedding (10.2.1 to 10.2.3). No cumulative impact will be caused to groundwater flow because there are no adjoining/adjacent modern basements (10.2.4).

11.3 In the unlikely event that the basement excavations encounter a local deposit of more permeable soils of sufficient thickness to permit significant flow, then an engineered groundwater bypass would be required (10.2.5).

11.4 The basement will need to be fully waterproofed (10.2.6, 10.2.7). The design groundwater level should be taken at external ground levels. This means that the basement must be able to resist buoyant uplift pressures (un-factored) up to 34kPa (10.2.8, 10.2.9).

11.5 Water entries into the basement excavations are likely to be manageable by sump pumping (10.3.1). The clays onto which the underpins and the basement slab will bear must be blinded with concrete immediately following excavation and inspection (10.3.3 and 10.4.8). Enquires should be made to identify the cause of the (April 2014) flooding to the cellar below No.69, and the temporary works requirements for No.71's basement should be reviewed in light of the answers received (10.3.4).

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

11.7 The basement is expected to be constructed using a combination of RC underpinning beneath the existing building and cast in-situ RC retaining walls constructed in panels not exceeding 1.0m width. For all these excavations and both types of retaining wall, use of best practice methods and high stiffness temporary support systems, installed in a timely manner, will be crucial to the satisfactory control of ground movements around the basement (10.4.2 to 10.4.9)

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

11.9 Subject to agreement under the PWA negotiations, transition underpins should be considered for all load-bearing walls in No's 69 & 73 which adjoin No.71 except where existing cellars already provide adequate transition (10.4.12). An arboriculturalist should be asked to confirm whether the roots from the large Ash(?) tree in No.69's garden will be affected by the proposed basement.

11.10 The basement slab must be designed to accommodate swelling pressures generated by heave of the underlying clays (10.5.10). Preliminary heave/settlement assessments have been undertaken using PDISP software. The predicted displacements ranged from 2mm of settlement beneath the underpins to 8mm of heave below the central basement slab. However, only the preliminary predicted 0-4mm of post-construction incremental differential displacement is relevant to the design of the basement slab (Section 10.5).

11.11 The rear wall to No.73 (and No.75) was assessed to be the critical structure for displacements. A damage category assessment indicated that, provided best practice construction methods are employed, the worst case predicted deformation is likely to fall within Burland Category 1, termed 'very slight'. By inspection, it has been shown that No.69 is at a lower risk of potential damage from the excavation of the proposed basement (Section 10.6).

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

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

11.14 Goldhurst Terrace is close to a former tributary to the Westbourne, is within both a Critical Drainage Area and the Goldhurst LFRZ, and was recorded as having flooded during both the 1975 and 2002 events; however, this location is on a sloping part of the road (well above a topographic low point at the junction with Fairhazel Gardens). The latest flood modelling by the Environment Agency and Camden SFRA gave a 'Very Low' risk of flooding by surface water to No.71 and the surrounding area/highways. This is the lowest, national background level of risk. Appropriate minor flood mitigation measures are recommended (10.8.3 to 10.8.5).

11.15 The rear section of the basement will increase the area of hard surfacing slightly. While the potential impact of this increase on surface water run-off is likely to be minimal, options are provided for simple SuDS systems which could be used to mitigate the slight potential increase in surface water discharge to the mains drainage system (10.8.7, 10.8.8).

11.16 Thames Water has no records of flooding from public sewers affecting No.71, so the probability of future sewer flooding affecting No.71 is considered to be very low, provided that the sewer system is well maintained and appropriate flood resistance measures are implemented (10.8.9).

11.17 Non-return valves and above ground loop systems should be fitted to the drains serving the basement and gullies in the lightwells (10.8.11). The planned temporary interception storage should have sufficient capacity to hold the roof/surface water from an appropriate design period rainstorm; formal design would be required (10.8.12).

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

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APPENDIX A

Photographs

Project:

71 Goldhurst Terrace, London NW6 3HA

18672

Photo 1: Front elevation (looking south-east). No.71 is a three storey mid-terrace property, adjoining No.69 Goldhurst Terrace to the north-east and No.73 Goldhurst Terrace to the south-west. The property has a front parking area surfaced with concrete paving stones.

Photo 2: The parking area and path leading to the front door of No.71 slope slope towards the Goldhurst Terrace footway.

Title:	**Photographs - Sheet 1**					Sheet	**A1**
Date:	26th January 2018	Checked:	HB	Approved:	KRG	Scale :	NTS

Project:

71 Goldhurst Terrace, London NW6 3HA

18672

GabrielGeo Consulting

Path to No.71 entrance stepped up from paved parking area

Goldhurst Terrace footway

Planting area marking boundary between No.69 and No.71

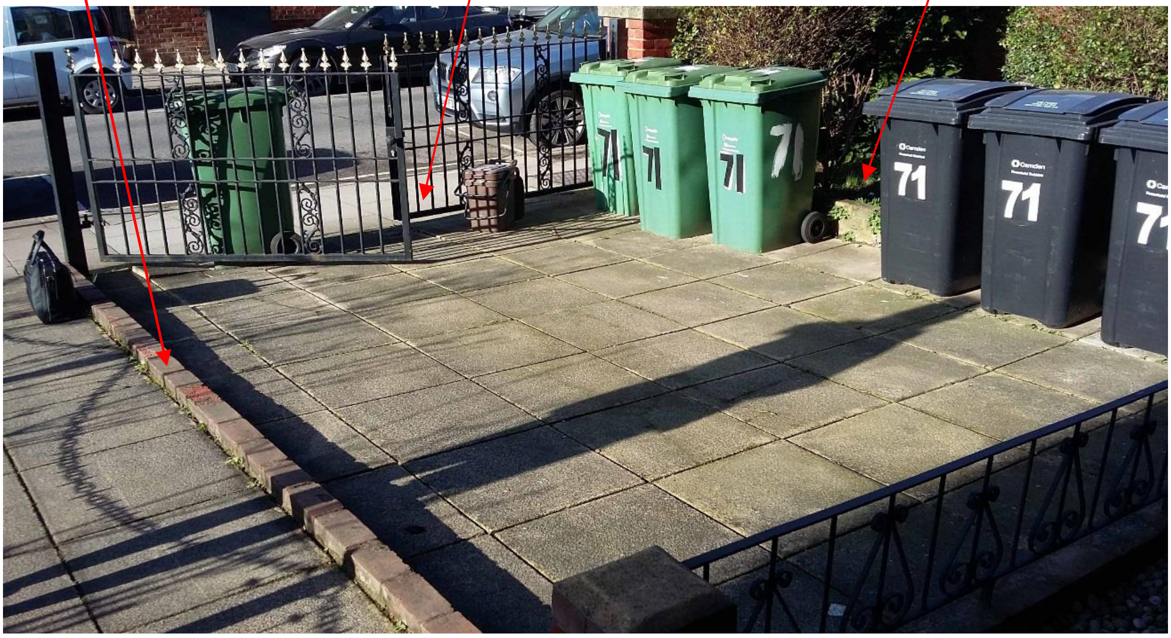

Photo 3: Front parking area/garden, looking north-west towards the Goldhurst Terrace footway and carriageway. The path leading to the front entrance of No.71 is stepped up from both the Goldhurst Terrace footway and the parking area.

Stairs leading to Ground Floor level

Cellar headroom: 1.25 - 1.27m

Photo 4: The existing cellar of No.71, looking north-east. The height of the existing cellar varies from 1.27m by the access stairs to 1.25m at the front wall, and was found to be very moist during the site inspection.

Title:	**Photographs - Sheet 2**					Sheet	**A2**
Date:	26th January 2018	Checked:	HB	Approved:	KRG	Scale :	NTS

Photo 5: Rear elevation of No.71, looking north-west, showing the 'courtyard' between the rear wall and rear projection.

Rear gravel covered 'courtyard' with irregular vegetation (weeds)

Single-storey rear extension

Small garden shed

Wooden decking

Paving lining both sides of lawn

Photo 6: Rear garden of No.71, looking north-west. The white projection of No.71 is a single storey rear extension which received planning permission in 1993. The wooden panel fencing marks the garden boundaries between the adjoining properties.