- Temporary support will be required to all the new underpins and RC retaining wall panels, and must be maintained until the full permanent support has been completed, including allowing time for the concrete to gain adequate strength.

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

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

Geotechnical Design

10.4.9 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 also paragraph 10.4.10 below);
- Dead and live loads from the structures above (the existing building, entrance portico and steps, boundary walls), including loads from the adjoining property (No.35) which are carried on the party wall;
- Loads from the soil and surfacings placed over the roof of the basement below the front and rear gardens;
- A surcharge to allow for the flank wall of No.42 Woronzow Road;
- Vehicle loads on the front parking area and adjacent footways, and normal surcharge allowances elsewhere;
- Swelling displacements/pressures from the underlying clays;
- A provisional design groundwater level at 0.5m bgl or ground level, as recommended in paragraph 10.2.5;
- Precautions to protect the concrete from sulphate attack.

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

Made Ground (clays):	Unit weight, γ_b:	19.0 kN/m^3
	Effective cohesion, c':	0 kPa
	Angle of internal friction, φ':	25°
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: 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.

10.4.11 The formation level clays onto which the underpins/RC walls 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.

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 therefore be considered for the load-bearing walls which adjoin No.34.

10.4.13 The extent of the root protection areas for the trees in the vicinity of the basement should be identified; an arboricultural survey will therefore be required, if not already available, and the impact of the basement should then be reviewed.

10.4.14 The basement will be sufficiently deep to be below the influence of root activity from both the trees to be removed and those that will remain. However, the possibility of root activity from the trees which will remain affecting the ground below the footings to the adjoining No.35 should be reviewed using the criteria given in NHBC Chapter 4.2.

10.4.15 Movement joints should be inserted in the boundary walls at each location where they will straddle the edge of the basement, with additional piers to provide stability to these walls as required.

10.5 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 of the basement. These preliminary analyses have not modelled the horizontal forces on the retaining walls, so have simplified the stress regime significantly.

Table 2: Net changes of vertical pressure in PDISP Zones

ZONE	Net change in vertical pressure (kPa)		
#	Stage 1	Stage 2	Stages 3 and 4
1	-63.34	-63.34	-55.34
2	-46.89	-46.89	-38.89
3	-58.08	-58.08	-50.08
4	41.10	41.10	49.10
5	-54.20	-54.20	-43.20
6	40.31	40.31	48.31
7	87.06	87.06	95.06
8	43.29	43.29	51.29
9	16.06	16.06	24.06
10	0.00	-106.31	-98.31
11	0.00	116.88	124.88
12	0.00	-64.98	-52.98
13	0.00	94.66	102.66
14	0.00	-70.30	-58.30
15	0.00	-70.30	-58.30
16	0.00	218.04	226.04
17	0.00	-70.30	-58.30
18	-22.81	-22.81	-14.81
19	0.00	-70.30	-62.30
20	0.00	-90.84	-78.84
21	0.00	238.34	249.34
22	0.00	-70.30	-56.30

23	0.00	238.28	249.28
24	0.00	-77.73	-65.73
25	0.00	-70.30	-56.30

10.5.2 A load takedown for the proposed basement was provided by GSE and is reproduced as Figure D1 in Appendix D. Figure D2 illustrates the layout of the proposed underpins and basement slab, based on Green Structural Engineering's Drg No. 12686-GA/01. The maximum overall dimensions of the basement are 6.55-12.425m wide (typically 10.8m as at rear end) by 35.99m long.

10.5.3 Table 2 presents the changes in net vertical pressure for each of the zones used in the PDISP analyses, at the four major stages of the stress history of the basement's construction, as detailed in paragraph 10.5.6 below.

Ground Conditions:

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

10.5.5 The short-term and long-term geotechnical properties of the soil strata used for the PDISP analyses are presented in Table 3, based on this investigation and data from other projects.

Table 3: 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	4.7 30.0	100 158	50 190	30 114

Where:

Undrained shear strength, Cu assumed as Cu = 100 + 7.5z kPa
where z = depth below the top of the stratum
Undrained Young's Modulus, Eu = 500 * Cu
Drained Young's Modulus, E' = 0.6 Eu

PDISP Analyses:

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

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

10.5.7 The results of the analyses for the Stages 2 and 4 are presented as contour plots on the appended Figures D3 and D4 respectively.

Heave/Settlement Assessment:

10.5.8 Excavation of the basement will cause immediate elastic heave in response to the stress reduction, followed by long-term plastic swelling as the underlying clays take up groundwater. 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 the 34/35 party wall is likely to undergo up to 4mm settlement while No.34's other walls may experience up to 10mm of heave. Heave movements beneath the basement slab (within the bases to the underpins/ RC retaining walls) were predicted to reach up to 13mm. The ranges of predicted short-term and long-term movements for each of the main walls are presented in Table 4 below. All the values quoted are approximate owing to the inherent simplification of the stress regime.

10.5.10 In general, the differential movements between different parts of the structure may be more significant than the total displacements. However, the PDISP model does not allow for the stiffness of the structure, so the range of displacements experienced may be somewhat less than predicted (the values given in Table 4 already ignore the least likely extremities of the predicted ranges of displacements).

Table 4: Summary of predicted displacements

Location	Stage 2 (Figure D3)	Stage 3	Stage 4 (Figure D4)
Front garden RC walls	1 – 4mm Heave	0 – 3mm Heave	1 – 8mm Heave
Front wall of house, including lightwell	3mm Heave – 1mm Settlement	2mm Heave – 2mm Settlement	5mm Heave – 3mm Settlement
Right flank wall	1 – 4mm Heave	1 – 3mm Heave	2 – 8mm Heave
34/35 party wall	0 – 2mm Settlement	1 – 2mm Settlement	1 – 4mm Settlement
Rear wall of house	0 – 3mm Heave	2mm Heave – 1mm Settlement	6mm Heave – 2mm Settlement
Basement slab (within underpins) below house	1 – 5mm Heave	0 – 4mm Heave	0 – 10mm Heave
Rear garden RC walls	0 – 4mm Heave	0 – 3mm Heave	0 – 8mm Heave
Basement slab (within RC walls) below front & rear gardens	3 – 7mm Heave	2 – 6mm Heave	5 – 13mm Heave

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 are relevant to the slab design. The analyses indicated that the maximum predicted post-construction displacements beneath the slab are likely to be about 8mm (total and differential).

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 as outlined in Section 10.4 above, then extensive past experience has shown that the bulk movements of the ground alongside the basement caused by underpinning for a single storey basement (typical depth 3.5m) should not exceed 5mm in either horizontal or vertical directions.

10.6.2 The excavation depth for the proposed underpins is 4.2m below the lower ground floor in No.34 (rounded from 4.18m, see paragraph 3.2), which is slightly greater than a typical single-storey basement. However, for the damage category assessment it is the depth below the underside of the footings of the adjoining or adjacent property which is relevant, and the adjoining No.35 was granted planning consent for underpinning (and various other works) in 2011. No details of the depth of underpinning were available on LBC's website, though aerial photos indicate that a major programme of work has already been undertaken. A conservative assumed founding level for the underpins at 1.5m below No.34's lower ground floor has been adopted, which means that the typical ground movements in response to the basement excavations would still apply.

10.6.3 A damage category assessment has been undertaken for the front wall of No.35, since the predicted settlement of the party wall is greatest below the front part of the party wall, and hence that represents the critical situation. In order to relate these typical ground movements to possible damage which an adjacent property might suffer, it is necessary to consider the strains and the angular distortion (as a deflection ratio) which they might generate. Ground movements associated with the construction of retaining walls in stiff clays have been shown to extend to a distance up to 4 times the depth of the excavation. So, the relevant geometries are as follows:

Assume underpin depth to No.35 = 1.5m

Depth of excavation = 4.2 – 1.5 = 2.7m

Zone of influence:	2.7 x 4 = 10.8m
Width No.35 (L) =	approx. 6.9m (front wall only).
Height (H) =	12.2m (max. height to eaves) + 1.5 =13.7m
Hence L/H =	0.50

Thus, for the typical 5mm horizontal displacement the strain beneath No.35 would, theoretically, be in the order of $\varepsilon_h = 4.63 \times 10^{-4}$ (0.046%).

The 4mm maximum settlement of the party wall predicted by the PDISP analysis may be added to the 5mm typical settlement caused by relaxation of the ground alongside the basement in response to excavation of the basement, giving a 9mm total predicted settlement of the ground at the assumed level of No.35's underpins. The settlement profile is expected to be convex with a worst case deflection, Δ = 17% of the predicted combined settlement , hence, Δ = 1.6mm, which represents a deflection ratio, $\Delta/L = 2.32 \times 10^{-4}$ (0.023%).

Using the graph for L/H = 0.5 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 2, as given in CIRIA SP200, Table 3.1, and illustrated in Figure 9 below.

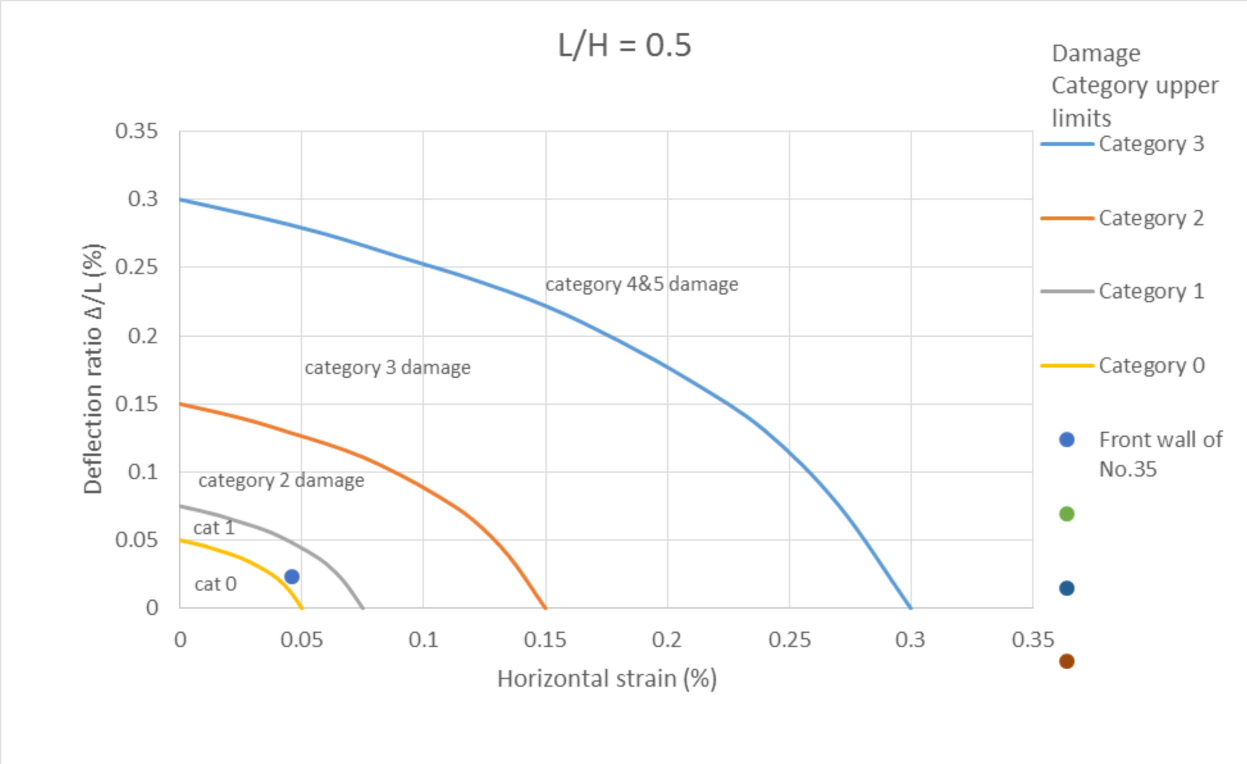

Figure 9: Damage category assessment for front wall of No.35.

10.6.4 If there is an internal wall in No.35 which is continuous from the party wall to the 4 storey side extension then the building geometry would be revised to L/H = 8.9/13.7 = 0.65. The strains calculated above would not alter. The graph used above for L/H = 0.5 remains the most appropriate, although the damage would plot slightly higher within Category 1 if a graph for L/H = 0.65 were to be created.

10.6.5 Use of best practice construction methods, as outlined in paragraphs 10.4.5 to 10.4.8, will be essential to ensure that the ground movements are kept in line with the above predictions.

10.7 Monitoring

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

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 two levels at the following locations (as a minimum):

- Internally, at three locations on the 34/35 party wall;
- externally, on the front and rear walls to No.35, on the centreline of the party wall with No.34;
- externally, on the front and rear walls to No.35, at the front left and rear left corners of the house;
- if measurements confirm that the basement excavations will be sufficiently close to the flank wall of No.42 Woronzow Road to fall within the criteria for the Party Wall Act, then that wall should also be monitored close to its front and rear ends;

- at the client's discretion, since outside the Party Wall Agreement, it would also be sensible to monitor the front, flank and rear walls to No.34.

10.7.3 The accuracy of this system of monitoring is usually quoted as +/- 2mm. Thus, if recorded movements in either direction reach 5mm, then the frequency of readings should be increased as appropriate to the severity of the movement, and consideration should be given to installing additional targets. If the recorded movements in either direction reach 7mm, then work should stop until new method statements have been prepared and approved by the appointed structural engineer.

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

10.8 Surface Flow and Flooding

10.8.1 The evidence presented in Section 5 has shown that:

- the site lies within the Environment Agency's Flood Zone 1 which means that it is considered to be at negligible risk of fluvial flooding;
- the site is not at risk of flooding from reservoirs, as mapped by the Environment Agency;
- Queen's Road was not affected by the surface water flooding events in either 1975 or 2002;
- there are no surface water features within 250m of the site;
- the nearest former watercourse from any of the 'lost' rivers was a branch of the Tyburn, which was about 65m to the north-east of the site;
- the latest flood modelling by both the Environment Agency and by URS for the Camden SFRA (2014) showed a 'Very Low' risk of surface water flooding (the lowest category, which represents the national background level of risk) for No.34's site, with local areas of 'Low' risk of flooding on the adjacent carriageways (see Figures 6 & 7).

Change in Paved Surfacing & Surface Water Run-off:

10.8.2 Cranbrook basements' drawings show that the existing layout of the front garden/parking area will be re-created above the basement, with at least 1.8m of ground replaced over the roof of the basement. As the area concerned is already extensively paved, and the underlying geology is London Clay, no significant change in surface run-off quantity or direction is anticipated in this area.

10.8.3 The nature of the surfacing beneath the decking could not be determined, so is assumed to be unpaved and as a result the basement will cause a loss of permeable surfacing in that area. While replacement of 1.0m of soil over most of the remainder of the rear garden will provide useful soft landscaping, the proposed glazed skylights will also cause an increase in paved area. These anticipated increases in paved surfacing should be mitigated by the inclusion of one or more appropriate Sustainable Drainage Systems (SuDS) in the scheme, such as:

- Intervention storage;
- Rainwater harvesting;
- Directing a small quantity of roof water to rain gardens;
- Use of permeable paving in areas which are currently laid with impermeable surfacing.

Soakaways should not be used because the garden is underlain by London Clay.

Surface Water (Pluvial) Flooding:

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

1. Provision of upstands around all the proposed lightwells.
2. Provision of raised thresholds at the access doorways from the lightwells into the basement, of sufficient height to protect the basement from the maximum depth of flooding in the lightwell predicted by appropriate calculations. It is likely to be beneficial to link the surface water storage volume in the lightwells with the temporary interception storage recommended in paragraph 10.8.6.
3. Removal of any low airbricks in the existing outer walls and replacement with solid bricks.
4. Ensuring that surface water run-off from the main part of the rear garden drains away from the decked 'lower terrace'.

Sewer Flooding:

10.8.5 Thames Water has no records of flooding from public sewer affecting No.34 (see 5.11). 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 both the Applicant's and the Council's control. The probability of future sewer flooding affecting No.34 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.

10.8.6 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.

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

10.8.8 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, rear 'lower terrace', lightwells 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.

10.8.9 If a non-return valve is fitted with an 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 have been recommended in Sections 10.2-10.8:

- In the unlikely event that the basement excavations encounter either flow through the Made Ground alongside the house or a local deposit of more permeable soils within the London Clay, of sufficient thickness to permit significant flow, then an engineered groundwater bypass should be provided (10.2.2 & 10.2.4).
- 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 (10.4.5).
- Any measures recommended by the arboricultural report proposed herein (see 10.4.12).
- Insertion of movement joints in the boundary walls wherever they straddle the edge of the basement, together with the addition of piers where necessary to stabilize the walls (see 10.4.13).
- Subject to Party Wall Act negotiations, transitional underpinning blocks should be included beneath all load-bearing walls in No.35 which adjoining the basement (10.4.14).
- Provision of upstands around all lightwells; provision of raised thresholds at the doorways into the basement from the lightwells, and removal and replacement of any low air bricks (10.8.4).
- Non-return valves and/or 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 might be operating under surcharge (see paragraphs 10.8.7 to 10.8.9).

11.0 NON TECHNICAL SUMMARY – STAGE 4

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

11.2 A services search should be undertaken (10.1.3, 10.1.4).

11.3 The proposed basement is considered acceptable in relation to the likely limited or nil flow of groundwater through the essentially clayey Made Ground and the London Clay; although if more permeable soils and continual groundwater flows are encountered, then a groundwater bypass would probably be required (10.2.1 to 10.2.4).

11.4 Provisional design groundwater levels at 0.5-1.5m below the relevant external ground levels are proposed, which means that the basement must be able to resist buoyant uplift pressures (un-factored) which vary across the basement up to 56-47 kN/m^2 (10.2.5, 10.2.6). The basement will need to be fully waterproofed (10.2.7, 10.2.8).

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

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

11.7 The perimeter walls of the basement will be constructed using both underpinning techniques and RC retaining walls in panels of limited width. Use of best practice methods and high stiffness temporary support systems, installed in a timely manner, will be crucial to the satisfactory control of ground movements around the basement (10.4.2 to 10.4.7).

11.8 Various other guidance is provided in relation to the geotechnical design and construction of the basement's perimeter walls (10.4.9 to 10.4.12).

11.9 An arboricultural report is required regarding the trees in the vicinity of the basement, unless already available. Movement joints should be inserted in the boundary walls wherever they straddle the edge of the basement, together with the addition of piers where necessary to stabilize the walls (10.4.10, 10.4.15).

11.10 The basement slab must be designed to accommodate swelling displacements/pressures generated by heave of the underlying clays. A preliminary heave/settlement assessment has been undertaken using PDISP software which predicted between 4mm of settlement and 10mm of heave beneath the underpins and RC retaining walls, and up to 13mm of heave below the basement slab. However, only 8mm of post-construction incremental displacement (both total and differential) may be relevant to the design of the basement slab. All these estimated displacements are only approximate values owing to the simplification of the stress changes involved (Section 10.5).

11.11 A damage category assessment indicated that, provided best practice construction methods are employed, the predicted deformations in the front walls to the lightwells in the adjoining properties on both sides of No.2 are likely to fall within Burland Category 1, termed 'very slight' (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 proposed basement scheme will result in an increase in paved surface area. One or a combination of SuDS systems should be included in the scheme to mitigate this increase; suitable types of SuDS are listed (10.8.2, 10.8.3).

11.14 Recent modelling of the risk of flooding from surface water by both the Environment Agency and URS for the Camden SFRA showed a 'Very Low' risk for No.34's site; this is the lowest, national background level of risk. Appropriate flood mitigation precautions have been recommended (10.8.1 – 10.8.3).

11.15 Thames Water have no records of flooding from public sewers affecting No.34, so the probability of future sewer flooding affecting No.34 is considered to be very low, provided that the sewer system is well maintained and appropriate flood resistance measures are implemented (10.8.5).

11.16 Non-return valves or above ground loop systems should be fitted to the drains serving the basement and lightwells in order to protect the basement from all possible sources of flooding. Temporary interception storage is likely to be required with sufficient capacity for all sources of water discharged through the 'protected' drains (10.8.7, 10.8.8).

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

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b) Save for the client no duty is undertaken or warranty or representation made to any party in respect of the opinions, advice, recommendations or conclusions herein set out.
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Project:

34 Queens Grove, London, NW8 6HN

16453

Photo 1: Front elevation (street scene). No.34 Queens Grove is a large, Grade 2 listed, 4-storey semi-detached house, situated on the south-east side of Queens Grove.

Photo 2: The footway in front of No.34 falls gently towards the Queens Grove carriageway, away from the property.

Title:	**Photographs - Sheet 1**					Sheet	**A1**
Date:	24 April 2015	Checked:	AG	Approved:	KRG	Scale :	NTS

Project:

34 Queens Grove, London, NW8 6HN

16453

Photo 3: The Queens Grove carriageway falls eastwards towards a former tributary to the river Tyburn, one of the 'lost' rivers of London, which has created a weakly developed valley, broadly at the location of Avenue Road.

Photo 4: The Woronzow Road carriageway immediately to the west of No.34 falls gently towards the south. Note the fall of the footway towards the carriageway, away from the property.

Title:	**Photographs - Sheet 2**					Sheet	**A2**
Date:	24 April 2015	Checked:	AG	Approved:	KRG	Scale :	NTS

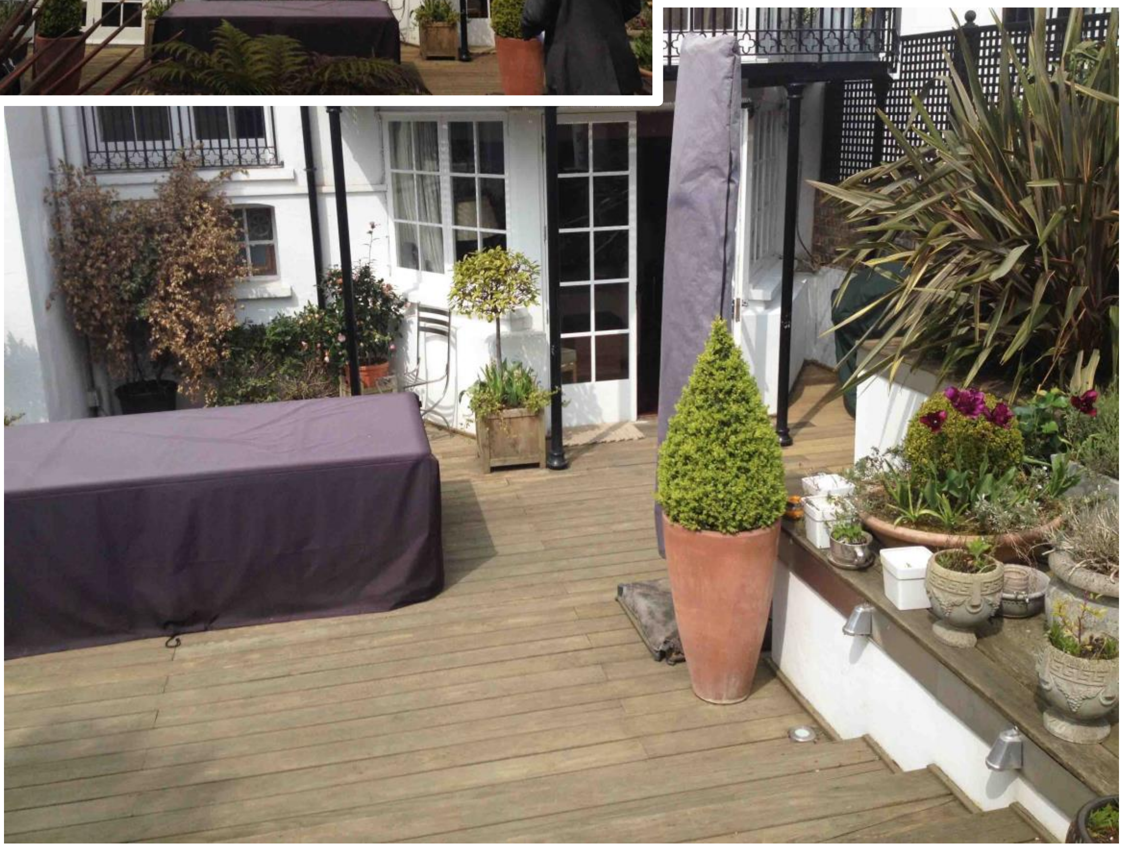
Project:

34 Queens Grove, London, NW8 6HN

16453

Photo 5 (left): Alterations to the property include the 4-storey rear projection, which is thought to have been built between publication of the 1962 and 1973 OS maps.

Photo 6 (below): Adjoining the rear of the house is a large area of wooden decking which is at the same level as the lower ground floor. A set of steps lead from this area, up to the rest of the rear garden which is at a higher level



Title:	**Photographs - Sheet 3**					Sheet	**A3**
Date:	24 April 2015	Checked:	AG	Approved:	KRG	Scale :	NTS

Project:

34 Queens Grove, London, NW8 6HN

16453

Photo 7 (above): The rear garden is mostly surfaced with paving slabs and gravel, with the exception of the perimeter soft landscaped area. Note the wooden fence which forms the south-eastern boundary to the rear garden.

Photo 8 (left): Immediately adjacent to the south-eastern boundary to the rear garden, within the plot of No.42 Woronzow Road, is a large maple tree.

Title:	**Photographs - Sheet 4**					Sheet	**A4**
Date:	24 April 2015	Checked:	AG	Approved:	KRG	Scale :	NTS

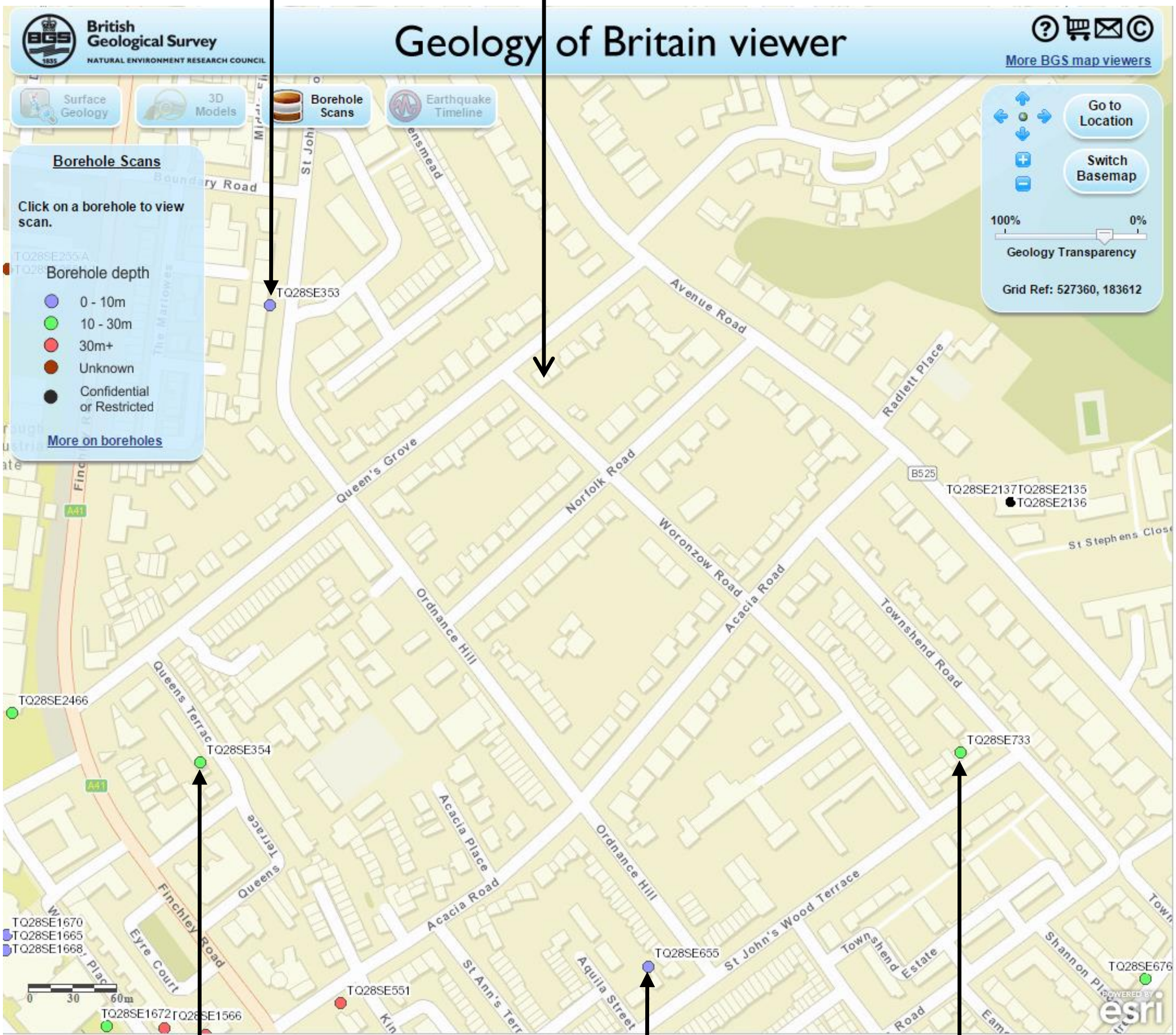
Project:

34 Queens Grove, London, NW8 6HN

16453

34 Queens Grove

TQ28SE/353



TQ28SE/354

TQ28SE/655

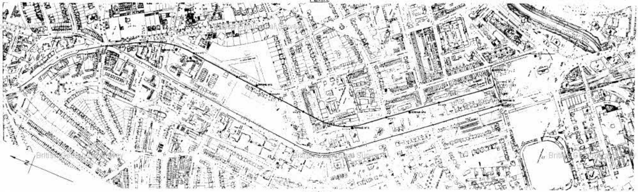
TQ28SE/733

Title:	**Location Plan of BGS Boreholes**					Sheet	**B1**
Date:	May 2015	Checked:	AG	Approved:	KRG	Scale :	NTS

LONDON COUNTY COUNCIL — MAIN DRAINAGE

TQ/2836/352-355
621-622.
590, 592
r 256

PLAN



SCALE:- 1/2500.

NOTES:-
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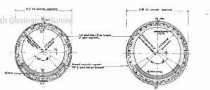
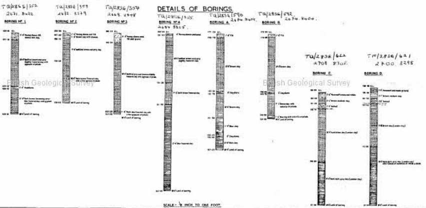
Based on the Ordnance Survey Map with the sanction of the Controller of Her Majesty's Stationery Office.

4'-9" INTERNAL DIAMETER SEWER. — 5'-3" INTERNAL DIAMETER SEWER.



SCALE:- 1/2 INCH TO ONE FOOT.

DETAILS OF BORINGS.



SCALE:- 1/8 INCH TO ONE FOOT.

KING'S SCHOLARS' POND SEWER

DIVERSION OF WELLINGTON ROAD BRANCH-STAGE II

SITE PLAN, SECTIONS OF SEWERS, AND BORINGS.

MD 546,65,1

CHIEF ENGINEER'S DEPARTMENT
MAIN DRAINAGE DIVISION

CONTRACT DRAWING No. 1

Kings Scholars Pond - Diversion of Wellington Road branch - stage II

TQ 28 SE/353
2672.8379

GEORGE WIMPEY AND COMPANY LTD., CENTRAL LABORATORY

BOREHOLE RECORD

256

B.H. No. 2

Ground level : +168.6 ft. 51.38m

Date started : 12.6.62.

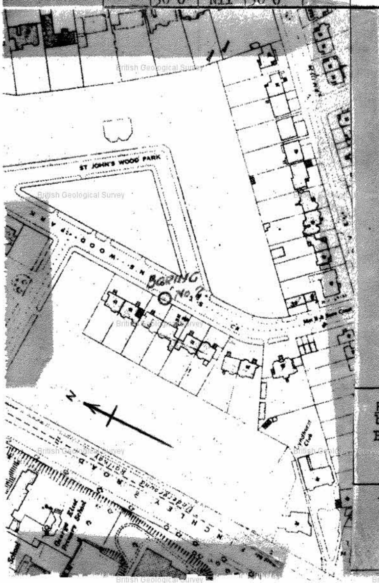
Type of boring : Shell and Auger

Date completed : 12.6.62.

Dia. of boring : 8"

Lining tubes :

Date	Depth of Boring	Water Level	Samples Depth	Samples Type	Change of Strata Legend	Change of Strata Depth	Change of Strata O.D. Level	Description of Strata
12.6.62		Nil				0.30m 1'0"	51.08m +167.6	Paving stones over fill
			2'6"	D		0.76m 2'6"	50.62m +166.1	Brown clay with ~~sances~~ stones
			4'0"	D				Mottled brown and grey clay
			9'0"	D		3.66m 12'0"	47.73m +156.6	
			14'0"	D				Dark brown fissured clay with a few gypsum crystals
			19'0"	D				
			21'0"	D				
	x		25'0"-30'0"	BD				
	30'0"	Nil	30'0"			9.14m 30'0"	42.24m +138.6	



Remarks : Note: Level of borehole determined by reference to T.B.M. at Intersection of Boundary Road and St. John's Wood Park.

Soils No : S/3030

Fig. No :