Chelmer
Consultancy Services
'Groundbreaking Services'

Basement Impact Assessment

Site 13/15 John's Mews
London
WC1N 2PA

Client JM13 Ltd
Date September 2015
Our Ref BIA/4507 Rev 3

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Report Status: **FINAL**		
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Foreword

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

This report is specific to the proposed site use or development, as appropriate, and as described in the report; Chelmer Site Investigation Laboratories Ltd accept no liability for any use of the report or its contents for any purpose other than the development or proposed site use described herein.

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

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

1.0 INTRODUCTION

1.1 This Basement Impact Assessment has been prepared in support of a planning application submitted to the London Borough of Camden (LBC) for construction of a single-storey basement beneath Nos 13 & 15 John's Mews, WC1N 2PA (application 2014/3330/P). The assessment is in accordance with the requirements of the London Borough of Camden (LBC) Development Policy DP27 in relation to basement construction, and follows the requirements set out in LBC's guidance document CPG4 'Basements and Lightwells' (September 2015).

1.2 Revision 3 (Ref: 15321/R1C) incorporates additional ground investigation findings and a revised pile foundation system prepared by Barrett Mahony. Some data from the previous structural engineering consultant, TS Consulting, has been used in this report.

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

1.4 A preliminary site inspection (walk-over survey) of the property was undertaken on Tuesday 19th August 2014. Photos from that visit are presented in Appendix A. Desk study data have been collected from various sources including borehole records (Appendix B) and geological data, environmental data and historic maps from GroundSure which are presented in Appendices D, E and F. Relevant information from the desk study and site inspections is presented in Sections 2–6, followed by the basement impact assessment in accordance with CPG4 Stages 1–4 in Sections 7–10 respectively.

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

FT Architects:

- Drg No. 200_32_01 Existing Ground and 1st Floors
- Drg No. 200_32_02 Existing Roof Plan
- Drg No. 200_32_03 Existing Sections
- Drg No. 200_32_04 Existing Elevations
- Drg No. 200_32_101 Proposed Basement + Ground Floor Plans
- Drg No. 200_32_102 Proposed First + Second Floor Plans
- Drg No. 200_32_103 Proposed Roof Plan
- Drg No. 200_32_104 Proposed Sections
- Drg No. 200_32_105 Proposed Elevations

Barrett Mahony, Consulting Engineers:

- Drg No. L14771/01-T1 GA: Lower Ground Floor & Ground Floor Plans
- Drg No. L14771/02-T1 GA: First and Second Floor Plans
- Drg No. L14771/04-T1 GA: Sections A and B
- Drg No. L14771/05-T1 GA: Sections 1-7 and Details
- Drg No. L14771/701-T1 Temporary Works + Method Statement Phase 1 Demolition Works
- Drg No. L14771/702-T1 Section 1 at Stage 1 and Stage 2 of Method Statement

Chelmer Site Investigations (CSI):

Factual results of the ground investigations including site plans, trial pit logs, borehole logs and gas/groundwater monitoring:

- Report Ref: 4507 (August 2014, including Factual 'Geotechnical Testing' Report Ref: CGL04233 by Chelmer Geotechnical Laboratories);
- Addendum Factual Report Ref: FACT/4507D Rev.1 (August 2015, including Laboratory Report Ref: CGL/4507D by Chelmer Geotechnical Laboratories).

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

1.6 Instructions to prepare this Basement Impact Assessment (BIA) were received by signed order on the 29th July 2015

2.0 THE PROPERTY AND TOPOGRAPHICAL SETTING

2.1 No's. 13 & 15 are two-storey former mews houses which are currently configured as a single unit combining a garage, workshop and offices. The property is on the east side of John's Mews, at the location shown in Figure 1. At the rear of the building there is a single-storey section, beyond which the gardens to Nos 23 and 24 John Street are approximately 1m higher than the floor level in No's 13/15 (as shown on FT Architects' Drg No.200_32_03).

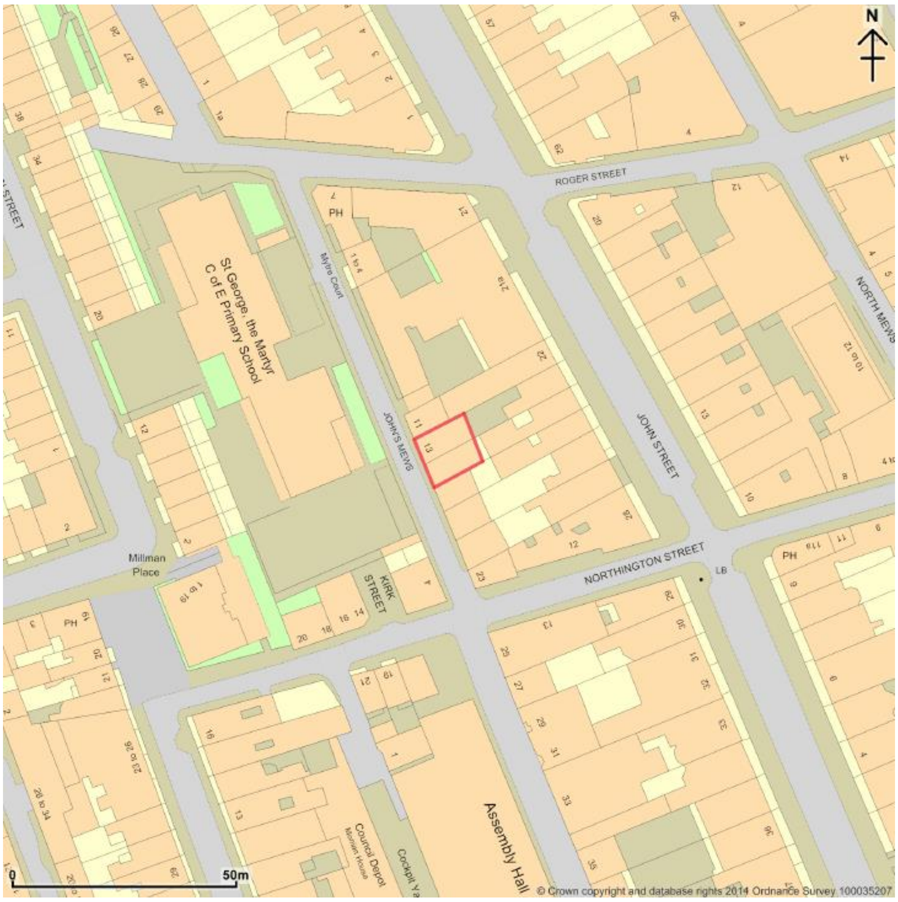

Figure 1: Extract from 1:1,250 OS map (not to scale) with the site outlined in red.

2.2 No's 13/15 share party walls with No.11 to the north and No.17 to the south, both of which have already had a third storey added (see cover photo and Photos 1 & 2 in Appendix A). Evidence of damp was visible in some of the walls, especially the 15/17 party wall. There was some broadly vertical cracking in the rear wall and a horizontal crack over No.13's garage door. Diagonal cracking in the front wall of No.11 suggested relative settlement of the 11/13 party wall.

2.3 The historic Ordnance Survey (OS) maps indicate that these buildings pre-date the first available map from 1875 (see Appendix F). By 1894 the single-storey rear section to No.15 had been built in the rear part of No.24 John Street's garden; the single-storey section behind No.13 did not appear on the OS maps till 1953.

2.4 Small mews-style houses formerly occupied the west side of John's Mews, fronting onto Robert Street; that area was re-developed for the primary school by 1973. The site to the north of No.11 was formerly occupied by a

Baptist Chapel and annex, sometimes labelled "Sun. School"; an aerial photograph from 1947 shows the chapel still standing but the 1951/52 OS map shows the site vacant and the Sunday(?) School as a ruin. The site had been redeveloped with what appears to be the current building (see Photo 2) by 1962.

2.5 John's Mews is on a north-facing slope which leads down to Roger Street. The loop in the 20m contour (see Figure 2) shows that Roger Street is located in the base of a shallow valley which was formed by a former tributary to the river Fleet, one of the 'lost' rivers of London. That tributary was orientated broadly west-east and was located below or close to Roger Street. The likely locations of the Fleet tributaries are considered further in Section 5. The 15m and 20m contours on Figure 2 define the Fleet's main (north-south) valley.

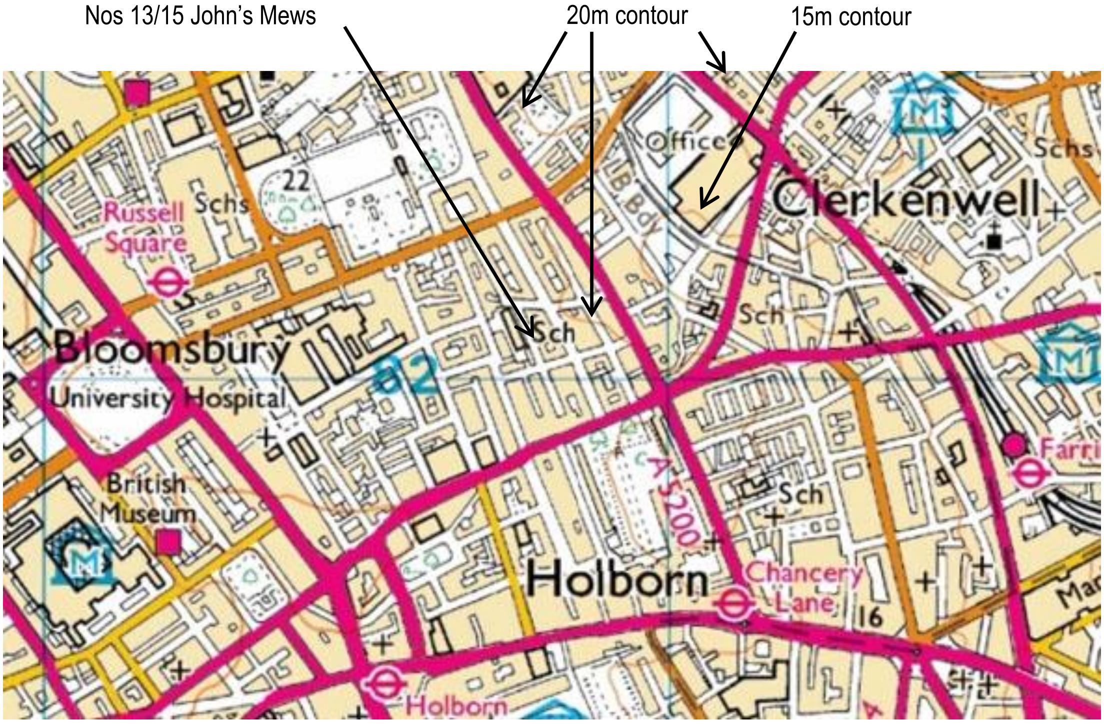

Figure 2: Enlarged extract from 1:25,000 Ordnance Survey map showing site location.

2.6 The commercial premises to the north of No.11 has a lower ground floor, and the vehicle access ramp falls steeply to an internal yard (Photo 3).

2.7 The bombsight.org website records no bombs falling on John's Mews, with the nearest being in Cockpit Yard, Roger Street and Great Ormond Street.

2.8 A search of planning applications on LBC's planning website found no records of applications for construction of basements beneath the neighbouring houses (Nos 11 & 17). Upslope of No's 13/15, permission has recently been granted (application 2013/5685/P) for a basement linking No.27 John Street and No.21 John's Mews. That scheme will involve extension of the existing basement to No.27 John Street rearwards beneath the rear courtyard to that property and creation of a new basement beneath No.21 John's Mews (so linking the two properties below ground level). The structural statement by SFK Consulting (Ref: RF/SD/13084, dated 19th August 2013, as available on the LBC website) states that this basement will be formed using underpinning techniques. No ground investigation had been undertaken when both the structural statement and the Basement Impact Assessment for that site were prepared. No evidence has been found for existing basements beneath the adjoining properties upslope (23 John's Mews and 12 Northington Street).

2.9 The BIA report for No.21 John's Mews records a Royal Mail tunnel 100m to the north of the site and a government communications tunnel approximately 40m to the west of No.21. There was limited confidence about the position of this tunnel so enquiries must be made to determine whether it is relevant to the proposed basement at No's 13/15

3.0 PROPOSED BASEMENT

3.1 The proposed development and basement for which planning permission will be sought, as shown in FT Architects' drawings, will comprise:

- Single-storey basement beneath the whole building/site;
- Re-building of the existing single-storey section at the rear of the site to include enclosed courtyards with rooflights, which will provide daylight to the basement below.

3.2 Barrett Mahony's drawings show that the basement will be constructed with minimum 330mm thick mass concrete underpins, and a structural box comprising a 400mm thick basement reinforced concrete (RC) slab supported on pile foundations, 250mm thick RC lining walls and a 225mm thick ground floor slab. The mass concrete underpins are shown as being constructed in two stages, with no footing for the 'upper lift' and bearing onto 250mm deep footings with widths to match the existing footings (shown 495-670mm wide).

3.3 The Finished Floor Level (FFL) in the basement beneath the main part of the house is shown in FT Architects' section (Drg No. 200_32_104) to be 3.3m below ground floor level, approximately 3.2m below the external ground level at the front of the house, while Barrett Mahony's drawings show them to be 3.20m below the internal floor level. With an allowance of 0.15m for the thickness of the insulation and floor finishes, the founding levels for the mass concrete underpins proposed by Barrett Mahony will be about 3.9m and 4.9m below the external ground levels at, respectively, the front and rear of the building.

3.4 The depth of excavation required will be approximately 4.1m below the existing internal ground-bearing floor slab.

4.0 GEOLOGICAL SETTING

4.1 Mapping by the British Geological Survey (BGS) indicates that the site is underlain by the Lynch Hill Gravel Member and possibly also the Hackney Gravel Member, which both overlie the London Clay Formation. Figure 3 shows an extract from Figure 5 of the Camden GHHS (Camden Geological, Hydrogeological and Hydrological Study by Arup, November 2010) which illustrates the site geology of the Holborn area. In urban parts of London, the natural geology is typically overlain by Made Ground.

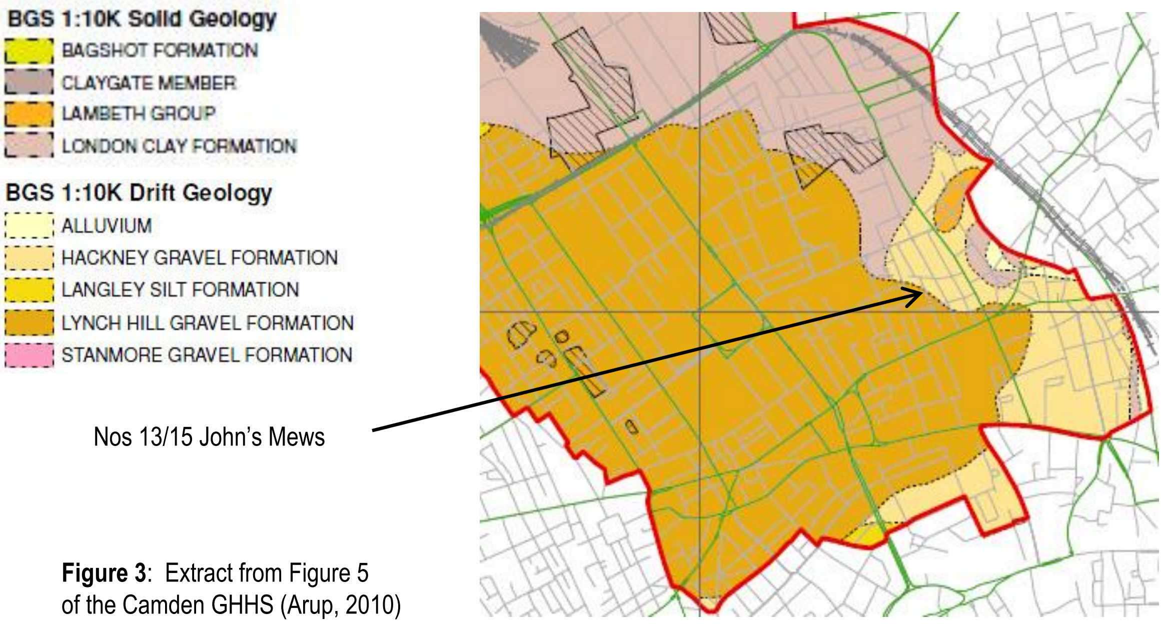

Figure 3: Extract from Figure 5 of the Camden GHHS (Arup, 2010)

4.2 The Lynch Hill Gravel Member (LHGMbr) and the Hackney Gravel Member (HGMbr) are two of the River Terrace Deposits associated with the river Thames and its tributaries. They were formerly classified as Formations, and before that were known as Terraces 3b and 3a respectively owing to their positions in the succession. Both are described by the BGS as 'Sand and gravel, locally with lenses of silt, clay or peat' (BGS Lexicon and Ellison et al, 2004). These are superficial deposits which formed in the Quaternary Period (up to 2 million years ago) when the local environment was dominated by rivers. The LHGMbr is the older deposit, so may extend underneath the HGMbr.

4.3 The London Clay is well documented as being a firm to very stiff over-consolidated clay which is typically of high or very high plasticity and high volume change potential. As a result it undergoes considerable volume changes in response to variations in its natural moisture content (the clay shrinks on drying and swells on subsequent rehydration). The clay will also swell when unloaded by excavations such as those required for the construction of basements.

4.4 The results of the BGS natural ground subsidence hazard classifications are provided in the GroundSure GeoInsight report (Appendix D); all indicated "Negligible hazard" to "Very low hazard".

4.5 An unknown natural cavity is recorded by the GeoInsight report at 389m to the north-west of the site (Appendix D, Section 3.6). Natural cavities are extremely rare in this geology, with the only plausible origin being a former ice-related feature associated with permafrost during the Ice Age. Alternatively it might have been a mis-identified man-made cavity.

4.6 The GeoInsight report also indicates that Chalk mining might have occurred in the area where Lambeth Group sediments sub-crop beneath the superficial soils in the bottom of the Fleet valley some 230m to the north of the site (Appendix D, Section 3.4). The likelihood of mining is rated 'Highly Unlikely'; the authors consider the probability is vanishingly small because of the extremely unfavourable conditions for mining in that area, where a deep scour feature has exposed the Lambeth Group.

4.7 Records have been obtained from the BGS borehole database for the nearest boreholes to the property. A location map is presented in Appendix B. The closest records available are summarised in Table 1.

Table 1: Summary of Strata in BGS Boreholes

Strata (abbreviated descriptions)	**Depths (m) and levels (m AOD) to base of strata in BGS Boreholes**									
	TQ28SW/ 743		**TQ28SW/ 157**		**TQ28SW/ 143**		**TQ28SW/ 2550**		**TQ28SW/ 266**	
GL (mAOD)	Depth	**Level 19.39**	Depth	**Level 24.63**	Depth	**Level 21.03**	Depth	**Level c.19.0**	Depth	**Level**
Made Ground and/or Topsoil	0.91	**18.48**	3.15	**21.48**	5.48	**15.55**	3.00	**16.0**	Records not available	
Soft to firm CLAY (Alluvium/RTDs)	-	**-**	3.21	**21.42**	-	**-**	6.20	**12.8**		
SAND and GRAVEL (River Terrace Dep's)	2.74	**16.65**	6.55	**18.08**	>6.39	**below 14.64**	-	**-**		
Soft brown CLAY (Weath'd London Clay?)	3.05	**16.34**	7.01	**17.62**	-	**-**	-	**-**		
Firm-to-stiff to very stiff CLAY (London Clay Fm)	17.37	**2.02**	>16.6	**-**	-	**-**	24.0?	**-5.0**		
Mottled CLAYS (Lambeth Group) Base of BH at:	>32.9	**-**	-	**-**	-	**-**	>30.0			
Groundwater standing level	In RTD	**?**	3.73	**20.90**	?	**?**	None?	**?**		

4.8 Logs from two boreholes at 11 John Street are also enclosed in Appendix B. Borehole WS1 was drilled in a lightwell so the 5.50m of Made Ground in WS2 is more comparable with the ground conditions at No's 13/15. Beneath the Made Ground these boreholes recorded 0.9-1.0m of 'Soft black peaty CLAY and brown clayey SILT' (WS1) and 'Soft to firm grey clayey SILT' (WS2).

5.0 HYDROLOGICAL SETTING (SURFACE WATER)

No's 13/15 John's Mews

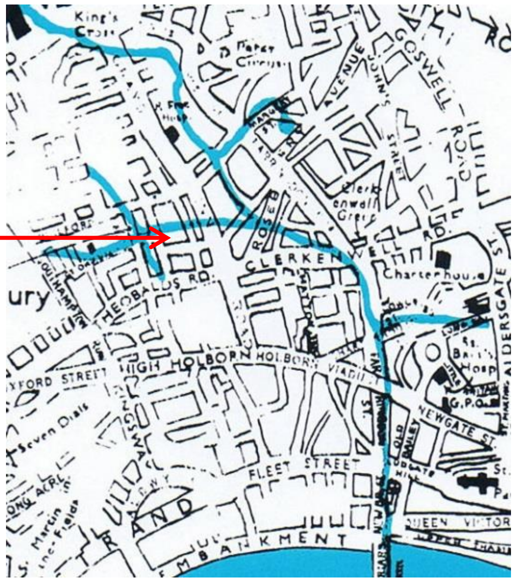

Figure 4: Extract from Figure 11 of the Camden GHHS (Arup, 2010) showing former watercourses, based on Barton (1992).

5.1 Two former tributaries to the Fleet, one of the 'lost' rivers of London, were present close to No's 13/15's site, as shown in Figure 4. As already noted, the west-east orientated former tributary to the Fleet is believed to be located below or close to Roger Street. It may run in a culvert or, more likely, now flows in the Victorian 1372x1016 sewer beneath Roger Street (see extract from Thames Water's sewer plan in Figure 5). The location of the small tributary which flowed northwards to the Roger Street tributary is less clear because Barton's map does not show all the roads. It is possible that it is/was in the abandoned sewer which formerly ran below Robert Street to the west of John's Mews, as shown in Figure 5.

Abandoned sewer

Nos 13/15 John's Mews

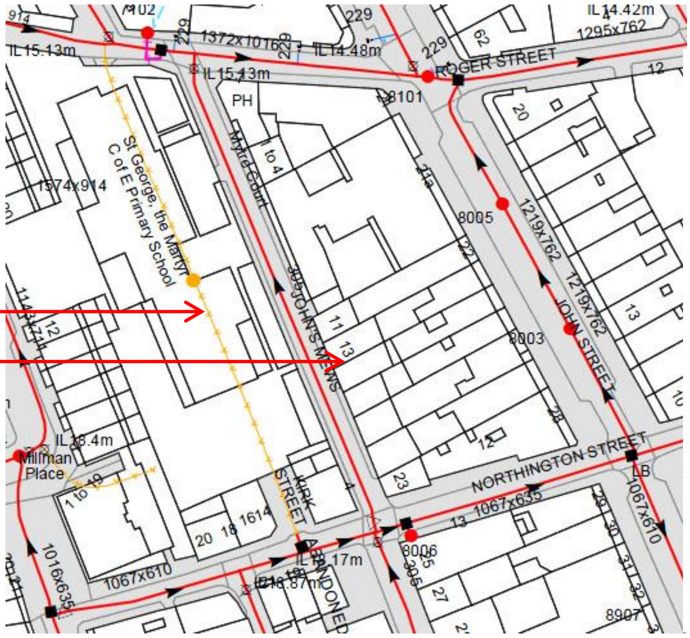

Figure 5: Extract from Thames Water's sewer plan.

5.2 Photo 4 in Appendix A shows that the front entrance to No.15 is raised above the public footway, and the continued northwards slope of the road results in No.13's garage threshold (and adjacent No. 11 door – Photo 3) being well above road level.

5.3 None of the lower part of the borough flooded in either the 1975 or the 2002 flood events, and John's Mews is remote from the 'Area with potential to be at risk of surface water flooding' associated with the Fleet, as shown on Figure 15 of the Camden GHHS (Arup, 2010).

5.4 The GroundSure EnviroInsight report records culverted rivers 340m and 426m to the east and south-west of the site respectively (Appendix E, Section 5.9). The former culvert carries the eastern branch of the Fleet, from the Highgate pond chain. No surface water features were recorded within 250m of the site (see Appendix E, Section 5.10).

5.5 Maps on the Environment Agency's website show that the site lies within Flood Zone 1, so is at negligible risk of flooding from rivers or the sea. The Environment Agency's website also shows that this area does not fall within an area at risk of reservoir flooding.

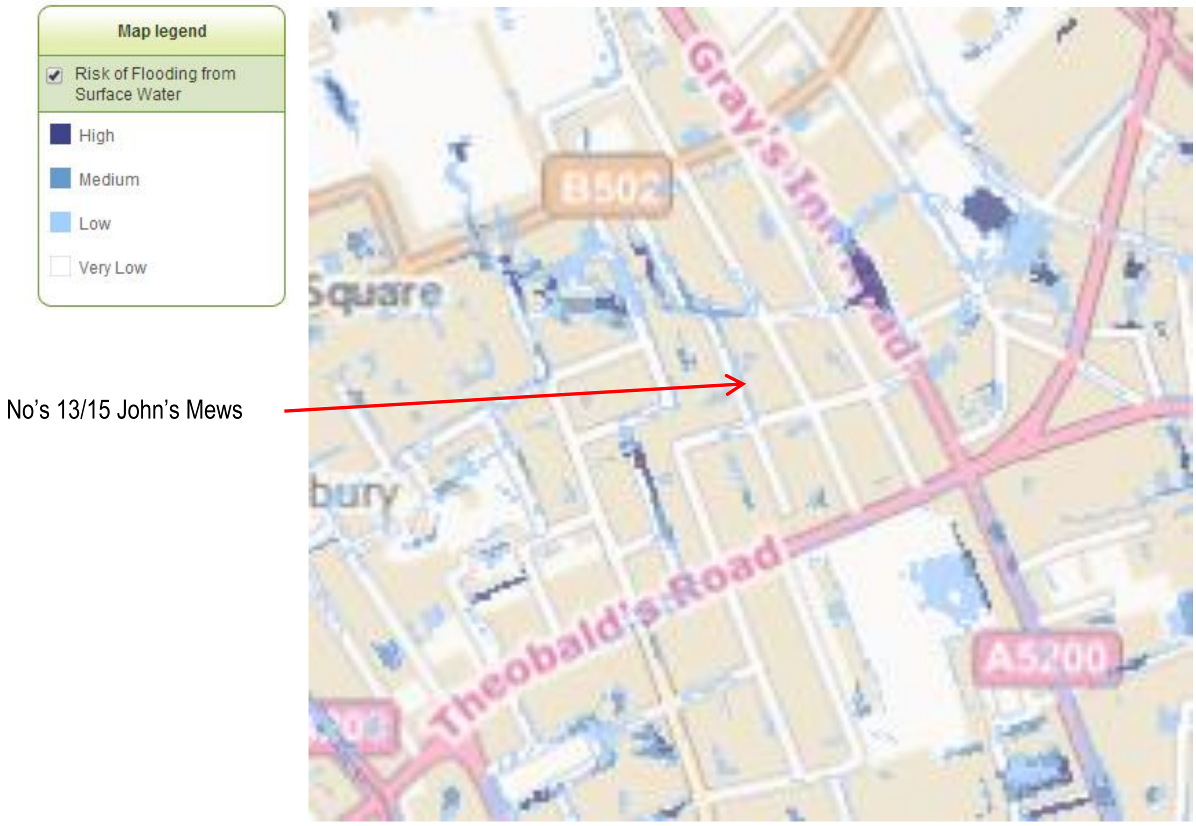

Figure 6: Extract from the Environment Agency's 'Risk of Flooding from Surface Water'.
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5.6 Modelling of surface water flooding has been undertaken by the Environment Agency and was published on its website in January 2014; an extract from their model is presented in Figure 6. While this map identifies four levels of risk (high, medium, low and very low) it understood that it is based at least in part on depths of flooding. This modelling shows a 'Very Low' risk of flooding (the lowest category for the national background level of risk) for No's 13/15's site itself and a ribbon of 'Low' risk along the east side of the carriageway to John's Mews (and in the much lower rear yard to the commercial building to the north of No.11). It is unclear why the ribbon of 'Low' flood risk is shown only on the east side of John's Mews given that it is a double-cambered road with highway gullies on both sides.

5.7 More recently, surface water flood modeling has been undertaken by URS as part of a Strategic Flood Risk Assessment for the London Borough of Camden, and was published in July 2014; an extract from their model is presented in Figure 7. As per the Environment Agency modeling, this map identifies the same four levels of risk (high, medium, low and very low), and shows a 'Very Low' risk of flooding for No's 13/15 and most of the surrounding area. It also shows a similar ribbon of 'Low' risk of flooding from surface water on the east side of the John's Mews carriageway, as shown in Figure 6.

5.8 Figure 7 also shows that John's Mews falls within Critical Drainage Area Group3_003.

Figure 7: Extract from the Environment Agency's map of 'Risk of Flooding from Surface Water'.

5.9 The implications from these flood models are discussed in Section 10.2.

6.0 HYDROGEOLOGICAL SETTING

6.1 The River Terrace Deposits are classified by the Environment Agency as a superficial 'Secondary A Aquifer'; this groundwater is usually unconfined and commonly referred to as the 'Upper Aquifer'. The underlying London Clay is an 'Unproductive Stratum'. Figure 8 shows the extent of the Secondary A Aquifer in the vicinity of the site of current interest.

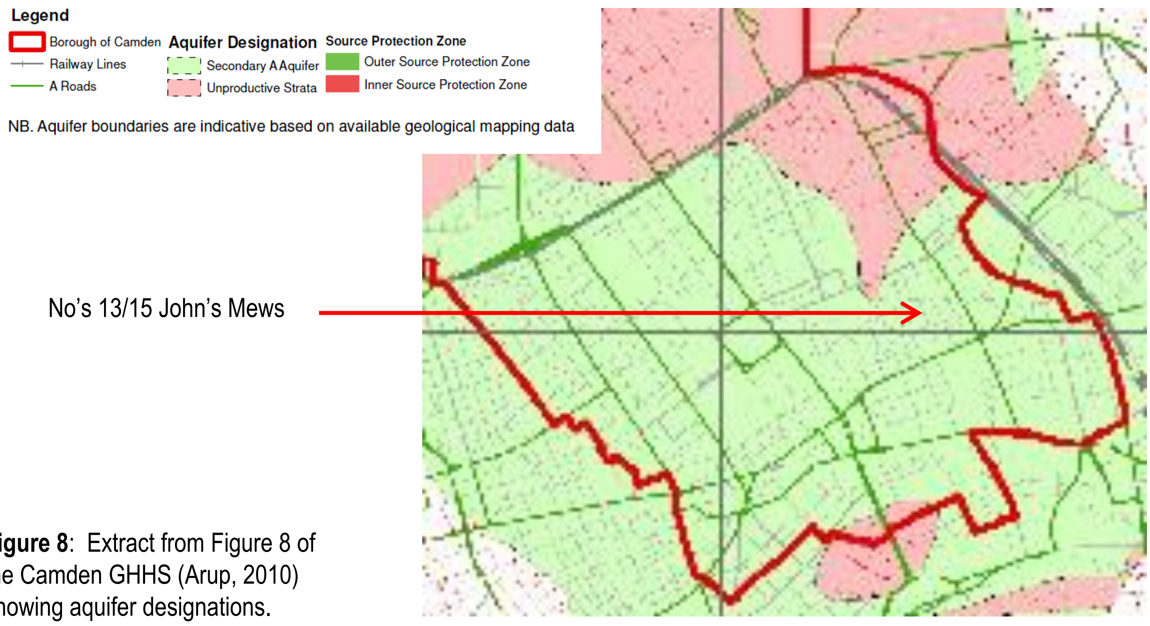
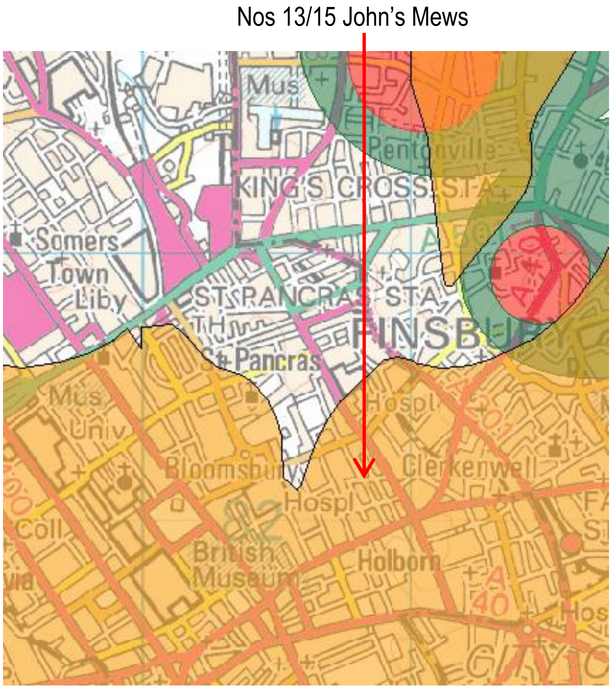


Figure 8: Extract from Figure 8 of the Camden GHHS (Arup, 2010) showing aquifer designations.

6.2 The Chalk Principal Aquifer which occurs at depth beneath the London Clay, together with the secondary bedrock aquifer in the intervening Thanet Sand Formation, are not relevant to the proposed basement under current conditions. Groundwater levels/ pressures in these aquifers are now controlled by the GARDIT scheme and the London Catchment Abstraction Management Scheme (CAMS), which are managed by the Environment Agency, so this situation is likely to continue for the foreseeable future. There is also no evidence to suggest that the scour feature which has exposed the Lambeth Group to the north-east of the site has created a hydraulic connection between the Upper Aquifer and the Chalk Principal Aquifer, sufficient to affect groundwater levels below John's Mews. As a result the deep aquifers are not considered further.

6.3 Under the old groundwater vulnerability classification scheme, which now applies only to superficial soils, the site is in an area which is classed as 'Minor Aquifer High' groundwater vulnerability, as shown in Figure 9.

Figure 9: Extract from Environment Agency's map of Groundwater Vulnerability Zones and SPZs
(Zone 1 = red, Zone 2 = dark green).



6.4 Other hydrogeological data obtained from the GroundSure EnviroInsight report (see Appendix E) include:

- There are no Source Protection Zones (SPZs) within 500m of the site (Figure 9 above and Appendix E, Section 5.6);
- The nearest groundwater abstraction licence, which is also for potable use, is 839m to the north-east of the site (Appendix E, Sections 5.3 and 5.5). There are many other abstraction licences within 2km of the site, but none are likely to be relevant to the proposed basement.
- For an area within 50m of No's 13/15 the BGS has classified the susceptibility to groundwater flooding as '**Potential at Surface**'. This result is given a 'Moderate' confidence level (Appendix E, Sections 6.6 and 6.7). Such groundwater flooding is defined as "the emergence of groundwater at the ground surface or the rising of groundwater into man-made ground under conditions where the normal range of groundwater levels is exceeded". This classification relates to the groundwater in the River Terrace Deposits; the basis of this classification and guidance on interpretation are provided in Section 10.2. The proposed basement will, anyway, extend below the water table so must be designed to exclude groundwater (see paragraph 10.2.6).

6.5 The Upper Aquifer generally occurs in the lower part of the River Terrace Deposits and, from past experience of projects in these deposits, it is known that multiple areas of perched groundwater may be present above the main groundwater table in the Upper Aquifer. Two of the four nearby BGS boreholes considered by the desk study recorded groundwater within 2.7-3.7m of ground level (see Table 1).

6.6 Perched groundwater may occur in the Made Ground, at least in the winter and early spring seasons, where lower permeability materials are present. The Upper Aquifer is also known to extend up into the Made Ground in places. Variations in groundwater levels and pressures will occur seasonally and with other man-induced influences.

6.7 Other evidence from nearby ground investigations includes:

- King's Mews: Groundwater strike at top of the River Terrace Deposits (4.60m bgl); standing levels during monitoring 3.60-3.74m bgl (February-March 2007). Source: BIA report for No.21 John's Mews.
- King's Mews (different site): Groundwater standing within the River Terrace Deposits at 3.9-4.2m bgl (July 2012). Source: As above.

6.8 Details of what was found by the site-specific ground investigations in May to August 2014 and August 2015 are presented in Section 9.

7.0 STAGE 1 - SCREENING

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

7.2 Subterranean (groundwater) flow screening flowchart:

Question		Response, with justification of 'No' answers	Clauses where considered further
1a	Is the site located directly above an aquifer?	Yes	Carried forward to Scoping: 8.2, Sections 10.2 & 10.3
1b	Will the proposed basement extend beneath the water table surface?	Yes	Carried forward to Scoping: 8.2, Sections 10.2 & 10.3
2	Is the site within 100m of a watercourse?	No – There are no surface water features within 250m of site. Nearby former minor tributaries to the Fleet (CGHHS Fig.11) have been culverted since 1800's.	5.1 & 5.4
3	Is the site within the catchment of the pond chains on Hampstead Heath?	No – Site is in Holborn	
4	Will the proposed basement development result in a change in the proportion of hard surfaced/ paved areas?	No – The site has no external areas.	
5	As part of the site drainage, will more surface water (eg: rainfall and run-off) than at present be discharged to the ground (eg: via soakaways and/or SUDS)?	No – Roof/surface water will continue to be discharged to the mains drainage system.	
6	Is the lowest point of the proposed excavation (allowing for any drainage and foundation space under the basement floor) close to, or lower than, the mean water level in any local pond (not just the pond chains on Hampstead Heath) or spring line?	No – There are no surface water features within 250m of the site.	5.4

7.3 Slope/ground stability screening flowchart:

Question		Response, with justification of 'No' answers	Clauses where considered further
1	Does the existing site include slopes, natural or man-made, greater than 7°? (approximately 1 in 8)	No – Site is level and fully developed.	
2	Will the proposed re-profiling of landscaping at site change slopes at the property boundary to more than 7°?	No – No re-profiling is proposed.	
3	Does the development neighbour land, including railway cuttings and the like, with a slope greater than 7°?	No – adjoining sites are also believed to be broadly level, albeit at slightly different levels.	
4	Is the site in a wider hillside setting in which the general slope is greater than 7°?	No – Northwards fall on John's Mews is estimated at less than 2°.	
5	Is the London Clay the shallowest strata at the site?	No – the shallowest strata mapped by the BGS is the Lynch Hill Gravel Member	
6	Will any tree/s be felled as part of the proposed development and/or are any works proposed within any tree root protection zones where trees are to be retained?	No. There are no trees on the site.	
7	Is there a history of seasonal shrink/swell subsidence in the local area, and/or evidence of such effects at the site?	No. The structural cracking observed is attributed to differential settlement of foundations within Made Ground.	
8	Is the site within 100m of a watercourse or potential spring line?	No –see Q2 in subterranean flow screening above. There are no natural springs in the vicinity.	
9	Is the site within an area of previously worked ground?	(Yes) – The site is not in an area recorded by the BGS as having been worked (see Figure 3 and maps on pages 8 & 15 of the GeoInsight report, Appendix D), but the ground investigation found deep Made Ground and no (?) River Terrace Deposits.	4.1 Carried forward to Scoping: 8.3, Section 9.
10	Is the site within an aquifer? If so, will the proposed basement extend beneath the water table such that dewatering may be required during construction?	Yes	Carried forward to Scoping: 8.3, Sections 10.2 & 10.3
11	Is the site within 50m of the Hampstead Heath ponds?	No – Site is in Holborn	
12	Is the site within 5m of a highway or a pedestrian right of way?	Yes	Carried forward to Scoping:

			8.3, Section 10.4
13	Will the proposed basement substantially increase the differential depth of foundations relative to neighbouring properties?	Yes	Carried forward to Scoping: 8.3, Section 10.4
14	Is the site over or within the exclusion zone of any tunnels, eg railway lines.	No – Re railway tunnels. Unknown re other tunnels.	Carried forward to Scoping: 8.3, 10.1.3

7.4 Surface flow and flooding screening flowchart:

Question		**Response, with justification of 'No' answers**	**Clauses where considered further**
1	Is the site within the catchment of the pond chains on Hampstead Heath?	No – Site is in Holborn	
2	As part of the proposed site drainage, will surface water flows (eg volume of rainfall and peak run-off) be materially changed from the existing route?	No – All surface water will continue to be discharged to the mains drainage system.	
3	Will the proposed basement development result in a change in the proportion of hard surfaced / paved external areas?	No – The basement will be wholly beneath the existing building.	3.1
4	Will the proposed basement result in changes to the profile of the inflows (instantaneous and long-term) of surface water being received by the adjacent properties or downstream watercourses?	No –No run-off is received by the adjacent properties. The nearby historic natural watercourses have been culverted since the 1800's.	5.1
5	Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream watercourses?	No – There will be no significant change in types of surface generating run-off. None of the surface run-off from this property reaches a nearby watercourse.	3.1, 5.1
6	Is the site in an area known to be at risk from surface water flooding, such as South Hampstead, West Hampstead, Gospel Oak and King's Cross, or is it at risk from flooding, for example because the proposed basement is below the static water level of a nearby surface water feature?	No – the lower part of the borough did not flood in 1975 or 2002; the site is in flood risk Zone 1 and surface water flood modelling by the Environment Agency does not indicate any increase in flood risk for the site above the national background.	Section 5

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

The screening exercise in accordance with CPG4 has identified seven issues which need to be taken forward to Scoping (Stage 2); two are related to groundwater and five are related to ground stability. There are no issues related to flooding potential as identified by the screening questions, though some flood resistance and mitigation measures are recommended in Section 10.5.

8.0 STAGE 2 - SCOPING

8.1 The scoping stage is required to identify the potential impacts from the aspects of the proposed basement which have been shown by the screening process to need further investigation. A conceptual ground model is usually compiled at the scoping stage; however, because the ground investigation has already been undertaken for this project, the conceptual ground model including the findings of the ground investigation is described under Stage 4 (see Section 10.1).

8.2 Subterranean (groundwater) flow scoping:

Issue (= Screening Question)		Potential impact and actions
1a	Is the site located directly above an aquifer?	**Potential impact:** Increased hard surfacing would decrease infiltration of surface water into the aquifer. See also 1b below. **Action:** None in this instance, because there will be no change in ground surfacing.
1b	Will the proposed basement extend beneath the water table surface?	**Potential impact:** If basement extends below groundwater table it might affect groundwater levels and flows, will require increased waterproofing measures and would create an uplift force on the basement. **Action:** Ground investigation required; then impact assessment and appropriate design of both permanent basement structure and temporary groundwater control measures.

8.3 Slope/ground stability scoping:

Issue (= Screening Question)		Potential impact and actions
9	Is the site within an area of previously worked ground?	**Potential impact:** Backfilled workings may present unfavourable founding conditions and less stable ground for excavations. **Action:** Ground investigation required; then appropriate design of both permanent basement walls and temporary support to excavations.
10	Is the site within an aquifer? If so, will the proposed basement extend beneath the water table such that dewatering may be required during construction?	**Potential impact:** Dewatering increases the effective stress in the ground and may remove fines, both of which can cause settlement of the area affected. **Action:** Ground investigation required; then appropriate design of groundwater control.
12	Is the site within 5m of a highway or a pedestrian right of way?	**Potential impact:** Construction of basement causes loss of support to footway/highway and damage to the services beneath them. **Action:** Ensure adequate temporary and permanent support by use of best practice underpinning methods.

13	Will the proposed basement substantially increase the differential depth of foundations relative to neighbouring properties?	**Potential impact:** Differential movement, including loss of support to the ground beneath the foundations to neighbouring properties if basement excavations are inadequately supported. **Action:** Ensure adequate temporary and permanent support by use of best practice underpinning methods. Consider the need for transition underpinning.
14	Is the site over or within the exclusion zone of any tunnels, eg railway lines.	**Potential impact:** Stress changes on any tunnel lining, or even a physical conflict. **Action:** Undertake services search to check that there are no tunnels/services in the vicinity.

8.4 Non-technical Summary – Stage 2:

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

- A ground investigation is required (which has already been undertaken), followed by relevant impact assessments (presented herein).
- Appropriate design and construction of the permanent basement structure, allowing for both construction beneath the water table and the presence of deep Made Ground.
- Appropriate design and implementation of temporary groundwater control measures.
- Appropriate design and adequate implementation of temporary and permanent support to excavations, including use of best practice underpinning methods.
- Designer and contractor to take account of weakening of the structure caused by past movements.
- Consider the need for transition underpinning to mitigate differential foundation depths.
- Undertake a services search to ensure there are no deep tunnels/services including checking whether the known government communications tunnel might be affected by the basement.

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

9.0 STAGE 3 – GROUND INVESTIGATION

9.1 Two site-specific ground investigations have been undertaken by Chelmer Site Investigations (CSI), the first between between May and August 2014, and the second in August 2015. In 2014, trial pits TP1 and TP4 were logged on 22nd May; three attempts at drilling boreholes were made on the same day using a lightweight continuous flight auger (cfa) rig, but all were abandoned on obstructions at 0.9m below floor level. A second attempt at drilling a borehole (BH1A) was made on 3rd July; that hole reached 2.0m below floor level before encountering an impenetrable obstruction. The third attempt (BH1B) on 18th July used a crawler-mounted cfa rig. That borehole was completed successfully to a depth of 10.0m.

9.2 The factual findings from the investigations have been presented in separate reports by CSI, including site plans, trial pit logs, borehole logs and gas/groundwater monitoring. The results of the subsequent laboratory testing have been presented in Chelmer's Geotechnical Testing/Laboratory reports (see paragraph 1.4). Manuscript records for TPs 2 & 3 in 2014 were provided by TS Consulting Ltd and are included in Appendix B; that TP3 has been superseded by 2015 trial pit TP7.

9.3 Five additional trial pits, TP2, TP3 and TP5-TP7 were hand dug on 4th August 2015, and borehole BH5 was drilled using a 'cut-down' cable percussion drill rig on 18th August.

9.4 The 2014 trial pits to expose the foundations were aborted at depths of 1.00-1.25m because "large rubble" (operative's description) prevented further progress. As a result the founding levels of some of the footings were not proven. The footings exposed in the 2014 pits comprised:

11/13 party wall (TP1, Section B): 3 corbels onto a concrete footing which projected 375mm from face of wall. Concrete thickness >0.50m.

Front wall No.13 (TP1, Section A): 2 corbels onto a concrete footing which projected 300mm from face of wall. Concrete thickness >0.50m.

13/15 party wall (TP2, Section B): Founded at 0.85m below ground level (bgl); 2 corbels which projected 150mm from face of wall.

Rear wall No.15 (TP3, Section 2): Founded at 0.49m below internal GL; 1 corbel projecting 60mm.

15/17 party wall (TP3, Section 1): Founded in "compacted fill" at 1.17m; 2 corbels which projected 150mm from face of wall.

15/17 party wall (TP4, Section A): 2 corbels which projected 150mm from face of wall. Brickwork continued down below 1.25m depth.

Front wall No.15 (TP4, Section B): No corbels. Wall on concrete footing which projected 200mm from face of wall. Top of concrete immediately below floor slab at 0.20m depth; concrete thickness >0.80m.

9.5 The geometries of the footings exposed in the 2015 trial pits were:

No.13 internal wall at back of garage (TP2, Section A): Founded at 0.85m bgl; 3 'corbels', all of which projected 100mm from face of wall.

11/13 party wall (TP2, Section B): Founded at 0.80m bgl; 2 corbels, projecting 140mm from face of wall, over 200mm thick "brick/concrete foundation".

11/13 party wall, single-storey section (TP3 & TP5 Section A): Founded at 0.925m bgl in TP3, rising to 0.795m bgl in TP5; 3 corbels, projecting 170-180mm from face of wall (lowest corbel almost total immersed in the concrete footing), over 200mm thick concrete foundation projecting 0.37-0.48m from face of wall.

No.13's rear wall, party with No.23 John Street (TP5 Section B, & TP6): Founded at 0.795m bgl; 3 corbels, projecting 170mm from face of wall (lowest corbel almost total immersed in the concrete footing), over 200mm thick concrete foundation projecting 0.37m from face of wall.

No.15's rear wall, party with No.24 John Street (TP7, Section A): Founded at 1.075m bgl; 2 corbels, over 250mm thick "brick/concrete foundation".

15/17 party wall, single storey section (TP7, Section B): As Section A.

9.6 The site's geology as found by the ground investigation may be summarised as:

- Made Ground: Where seen, comprised assorted **demolition debris** (including brick and concrete rubble, clinker, ash, slate, broken slabs and 'granite' blocks) together with brown/dark brown/grey/black, sandy, silty or very silty **clays,** gravelly clayey **silts,** variably clayey, slightly gravelly **sands** and sandy fine to coarse **gravel**. In BH1B the artificial matter recorded was limited to occasional brick fragments, and the Made Ground was indicated to be in a medium dense state of compaction by Standard Penetration Testing (SPT) performed with a solid cone (CPTs). A pungent smell was also noted.
  In BH5, Made Ground was recorded to 3.0m bgl and SPTs indicated it to be loose (N = 7-9) to very loose and possibly voided (N = 0).
  BH1B recorded grey silty **clays** with occasional brick fragments from 4.4m to 5.9m bgl; above 5.4m these clays also contained gravel and gave an SPT blowcount of N = 35 ('dense'). This might represent disturbed alluvium rather than Made Ground.
- 'Reworked' Ground and Alluvium: In BH5 a 0.5m thick layer of Soft, black, silty **clay** was recorded beneath the Made Ground (3.0-3.5m bgl), overlying 0.5m of Firm, brown/grey, gravelly silty CLAY (3.5-4.0m bgl). These were attributed to 'Reworked' Ground and Alluvium respectively, with the former being similar to the Made Ground recorded below 4.4m in BH1B.
- River Terrace Deposits (Lynch Hill Gravel Member (LHGMbr) and/or Hackney Gravel Member (HGMbr)): Slightly silty, variably sandy GRAVELS were recorded from 4.0m to 5.7m bgl in BH5. These coarsened downwards and SPT tests indicated them to be dense or very dense.
- London Clay: Stiff, brown (mottled grey), silty CLAY with partings of silt and fine sand, and crystals (probably selenite) was recorded immediately below the Made Ground in BH1B (from 5.95m bgl). This clay became grey below depths of 7.5m and very stiff below 8.8m.
  In BH5, firm, brown/grey, slightly sandy, silty CLAYS were recorded from 5.7m to 7.0m. The uppermost 0.3m was slightly gravelly (probably derived from the overlying gravels) and contained selenite which suggested that these clays were probably weathered from the London Clay.

9.5 Standard Penetration Tests (SPT) taken in the London Clay in BH1B recorded blow counts which increased from N = 20 at 6.0m bgl to N = 40 at 9.0m bgl. In BH5 the SPT 'N' values increased from N = 23 at 7.00-7.45m to N = 52 at 11.50-11.95m. The uppermost part of the firm clays at 5.7-7.0m gave a relatively low N = 12, which was probably a further reflection of geological disturbance of these slightly sandy clays when the overlying gravels were deposited.

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

9.7 No groundwater entries were recorded in any of the trial pits. The Made Ground below 0.5m in BH1A was noted to be 'moist'. All the boreholes remained open (ie: stable) on completion. BH1B recorded a seepage at the base of the Made Ground (5.9m bgl) and a standing level on completion at 9.5m bgl. The lack of a ground water entry into this small diameter borehole in the clayey strata above this level did not necessarily mean that groundwater was absent; rather, the low permeability of the clays merely meant that the flow rate was too slow for groundwater entries to occur during drilling. For the same reason, the standing level on completion of BH1B reflects only the amount of water which had seeped into the borehole before installing instrumentation.

9.8 In BH5 a water strike was recorded at 4.5m; the water rose to 4.2m and was subsequently sealed out when the casing reached 6.0m.

9.9 A standpipe was installed to 8.0m bgl in BH1B. During the subsequent short period of monitoring in 2014, this standpipe recorded water levels at 3.39m and 3.27m bgl on 30th July and 10th August 2014 respectively. This groundwater might still have been rising, so may not be representative of the 'static/seasonal' groundwater levels/pressures in the surrounding ground. Monitoring of BH5 in the autumn of 2015 once again showed a gradual rise in groundwater levels, reaching 3.48m bgl on 14th September, when the level in BH1B was at 3.18m bgl.

9.9 <u>Laboratory Testing:</u>
Laboratory tests were carried out by Chelmer Geotechnical Laboratories and others on samples recovered from the BHs 1A, 1B & 5 and three of the 2015 trial pits. The testing comprised classification tests (moisture content, plasticity and particle size distribution grading analyses), and chemical analyses to assess the potential for aggressive attack on concrete.

9.10 Plasticity tests were performed on three samples of London Clay; the results indicated the sample of weathered (brown) London Clay to be of Very High Plasticity as classified by BS5930 (1999, 2010), whereas the samples from the underlying grey clays were found to have High Plasticity. All three samples had High volume change potentials, as defined by the NHBC (NHBC Standards, 2013, Chapter 4.2, Building near Trees).

9.11 Grading analyses were undertaken on two samples of the River Terrace Deposits from BH5. The results confirmed the coarsening downwards trend noted from the borehole log, with the samples shown to be:
4.00m: Slightly silty, very sandy (medium and coarse), fine to coarse GRAVEL;
5.00m: Slightly silty, slightly sandy, medium and coarse GRAVEL.

9.12 The results of the chemical tests on five samples gave:

Made Ground:	pH:	6.2-10.7
	Sulphate (SO_4):	510-4934mg/l
	Sulphur:	0.10-0.26%
London Clay:	pH:	7.9-8.1
	Sulphate (SO_4):	0.79-0.99g/l (790-990mg/l);

The sulphate concentrations classified the Made Ground specimens tested as Design Sulphate Classes DS-2 to DS-4, as defined in BRE Special Digest 1 (2005) 'Concrete in aggressive ground'. Using the sulphur value to calculate Total Potential Sulphate gave no higher classifications than these. The samples from the London Clay were classified as Class DS-2.

9.13 Non-technical Summary – Stage 3:

9.13.1 The ground investigation found a substantial thickness of Made Ground (3.0m and 5.9m at the two borehole locations) overlying the 'reworked' and in-situ soft/firm alluvial clays, then the anticipated sequence of gravels from the River Terrace Deposits overlying London Clay, the upper part of which was weathered. Where the Made Ground was deepest the gravels from the River Terrace Deposits were absent. Five other boreholes failed to penetrate through the Made Ground owing to obstructions. The trial pits showed the rear part of the building to be founded at 0.49-1.075m bgl, whereas they failed to identify the founding level of the footings beneath the front part of the building, owing to the amount of rubble included in the Made Ground and the greater depth of the footings, some of which appeared to have been underpinned.

9.13.2 Groundwater was recorded to within 3.27m of the internal floor level during the summer 2014 monitoring period (BH1B). This level may still have been rising so may not have fully equilibrated with water levels/pressures in the surrounding ground. Monitoring of BH5 in the autumn of 2015 recorded groundwater up to 3.18m bgl.

9.13.3 The laboratory testing has shown that the clay specimens from the London Clay were of High to Very High plasticity. Specimens from both the Made Ground and the London Clay gave high sulphate contents.

10.0 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:

- Foundations: The founding level of the footings varied, where proven, between 0.49m and 1.075m bgl. The footing depth below the main, two storey part of the building has only been identified (at 0.85m bgl) in the rear left corner of No.13. It is possible that at least the front wall to both properties and some of the 11/13 party wall have been underpinned.
- Made Ground & Reworked Ground: A deep layer of variable Made Ground, in excess of 5m thick, formed, where seen, of **demolition debris**, **clays, silts, sands or gravels** which proved to be impenetrable in places for hand dug trial pits of limited plan area and a light cfa drill rig. Other materials are also likely to be present, owing to the inherent variability of Made Ground. Chemical testing gave high sulphate and sulphur concentrations, which classified the specimens tested as Design Sulphate Classes DS-2 to DS-4.
  The lowest 1.5m of the Made Ground in BH1B comprised grey silty **clay** with occasional brick fragments. Similar soft, black, silty **clay** was recorded at 3.0-3.5m bgl in BH5. These clays might have been a disturbed, in-situ, alluvial clay from the river Fleet and/or the base of the River Terrace Deposits.
  The desk study found that a similar thickness (5.48m) of Made Ground was recorded in the closest of the BGS boreholes, at the junction of John Street and Northington Street, and again at 11 John Street (5.50m).
- Alluvium: 0.5m of in-situ, firm, alluvial clay with no evidence of disturbance was recorded in BH5 at 3.5-4.0m bgl. At 11 John Street the Made Ground was underlain by alluvium (soft peaty CLAY and soft to firm clayey SILT).
- Lynch Hill Gravel Formation: A coarsening downwards sequence of dense or very dense, slightly silty, variably sandy GRAVELS were recorded from 4.0m to 5.7m bgl in BH5. The reason why the sands and gravels were apparently removed beneath part of the site (BH1B) is not known, unless the site is located above or close to the south-north aligned minor sub-tributary to the river Fleet (see paragraph 5.1).
- Upper (Secondary A) Aquifer: Water from the Upper Aquifer has been shown to occur within this Made Ground, with groundwater recorded at levels up to 3.18m below floor level.
- Weathered in-situ London Clay: Stiff, silty CLAYS were found directly below the Made Ground or beneath the River Terrace Deposits. These clays are likely to be fissured, which reduces their strength, and will undergo heave movements in response to unloading by the basement excavation. The recorded "crystals" probably included selenite, which is aggressive towards buried concrete. Standard Penetration Tests in the London Clay recorded blowcounts which increased progressively with depth, as described in detail in Section 9.
- Other aspects of the site's hydrogeology:
  The hydrogeology may be complicated further by the backfill in service trenches and granular pipe bedding (where present) forming preferential groundwater flow pathways within the strata they pass through.

10.1.2 The hydrogeological regime outlined above will be affected by long-term climatic variations as well as seasonal fluctuations, all of which must be taken into account when selecting a design water level for the permanent works. Only limited, summer/autumn 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 provisional design groundwater level.

10.1.3 No railway tunnels are known to pass below or close to the site. The location of the known government communications tunnel in the vicinity of the site (see 2.9 above) must be checked. Other infrastructure (including tunnels), for sewers, cables or communications might be present within the zone of influence of the proposed basement, so an appropriate services search should be undertaken. If any such infrastructure is identified, then its potential influence on the proposed basement must be assessed. These searches will not identify any private services.

10.2 Subterranean (Groundwater) Flow – Permanent Works

10.2.1 The permeability of the Made Ground will depend on the degree to which voids in the rubble have been infilled with clays/clayey silts, the degree to which the areas of slightly more permeable soils are interconnected, and the extent of any other more permeable materials which presently remain undetected. The lack of a groundwater entry from the Made Ground in BH 1B, where monitoring subsequently showed that groundwater was present, suggests, though does not prove, that groundwater flow through these soils may generally be limited. The possibility remains however that more permeable materials are present within the Made Ground which might facilitate localised flow. Flow through the Made Ground may also occur where service trenches or granular pipe bedding facilitates channelled flow.

10.2.2 The proposed founding depth for this basement slab (including blinding) is approximately 3.8m below the internal floor level where the boreholes were drilled, while the underpins will be founded at approximately 4.05m below internal floor level (equivalent to 3.9m below the external ground level at the front of No.15, and 4.9m below the ground level in the gardens to the rear of the building). All these levels allow for 0.15m of insulation and floor finishes, as scaled from Barrett Mahony's sections. Thus, the basement's underpins as currently proposed will be founded partly in the Made Ground and partly in the dense sandy gravels of the River Terrace Deposits, and possibly partly in the alluvial clays. All of the basement slab and underpins will be below the groundwater level and some flow through the gravels should be expected. The groundwater strike at 4.5m in BH5, at least 0.9m below the groundwater level, and the apparently slow subsequent rise indicates that the permeability of the gravels is at the lower end of the expected range, although the grading curves would suggest relatively high permeabilities. As most of the gravels will remain in place below the underpins, the basement is not expected to have any significant adverse effect on groundwater flow.

10.2.3 The highest groundwater level reading from the standpipes during the limited monitoring periods was 3.18m bgl and the water level may still have been rising. The standpipes should be maintained so as to enable further groundwater readings to be taken before the start of the works.

10.2.4 The BGS has classified the susceptibility to groundwater flooding as 'Potential for groundwater flooding to occur at surface' which GroundSure has abbreviated to '**Potential at Surface**' (see paragraph 6.4). The 'Exploratory notes for users' prepared by the BGS for this dataset state that the "*data can be used to identify areas where geological conditions could enable groundwater flooding to occur and where groundwater may come to surface. Note: it is a susceptibility dataset and does not indicate hazard or risk*" (our underlining).

The classification is based on a theoretical model of "high groundwater levels" in areas where permeable strata are present at surface, which was then compared with a terrain model. It does not include any attempt to predict future changes so should reflect only the current groundwater situation.

10.2.5 The BGS exploratory notes also state that:

"The susceptibility data is suitable for use for regional or national planning purposes where the groundwater flooding information will be used along with a range of other relevant information to inform land-use planning decisions. It might also be used in conjunction with a large number of other factors, e.g. records of previous incidence of groundwater flooding, rainfall, property type, and land drainage information, to establish relative, but not absolute, risk of groundwater flooding at a resolution of greater than a few hundred metres. The confidence dataset will help in this assessment. The susceptibility data should not be used on its own to make planning decisions at any scale, and, in particular, should ***not be used to inform planning decisions at the site scale****. The susceptibility data cannot be used on its own to indicate risk of groundwater flooding."*

The BGS have also confirmed to the author (KRG, pers comm, 21/05/2014) that wherever there is local knowledge of groundwater conditions, that knowledge should be used in preference to the susceptibility model.

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 Given the pungent smell recorded in BH1B, and the soft alluvial clays in BH5 and elsewhere in the vicinity, consideration should also be given to making the basement gas-tight.

10.2.8 Current geotechnical design standards require use of a 'worst credible' approach to selection of groundwater pressures. Relevant evidence in addition to the on-site monitoring includes the lack of groundwater entries into the trial pits, the lower ground levels to the north of No.11, and groundwater levels at 2.7-4.2m bgl in the nearby boreholes reviewed for the desk study (though none of those readings were from long-term monitoring). As a result, use of a provisional design groundwater level at 1.0m below ground level is recommended, provided that further monitoring during detailed design and prior to the start of construction does not record a groundwater level above 2.0m bgl.

10.2.9 The basement structure must be designed to resist the buoyant uplift pressures which would be generated by groundwater at the design level. For the provisional groundwater level at 1.0m bgl recommended above, the uplift pressure would be up to 28 kPa below the basement slab and 31kPa below the underpins (both un-factored).

10.2.10 <u>Cumulative Impact:</u>

The proposed basement beneath No.21 John's Mews and the linking section to the existing basement beneath No.27 John Street (hereafter referred to as the 27JS-21JM basement), is directly upslope of No's 13/15 and was recently granted planning consent. That basement will almost certainly also be founded in Made Ground above the London Clay (because BGS borehole TQ28SW/143, which was very close to No.27 John Street, also recorded Made Ground to 5.48m bgl, similar to the thickness in BH1B beneath No.13). The 27JS-21JM basement would be significantly wider, cross-slope, than the basement currently proposed at No's 13/15 so no cumulative effect on groundwater flows would be anticipated if both basements are built.

10.3 Subterranean (Groundwater) Flow – Temporary Works

10.3.1 Some groundwater control will be required during the basement construction works. Water entries may be manageable by pumping from screened sumps installed (temporarily) below the excavation level. Use of several sumps will be required. However, lowering the groundwater level can lead to settlement because it increases the effective stress in the soils below the initial groundwater level. Thus, unless the party walls are supported temporarily off the piled foundations (see Section 10.4), detailed, precise monitoring of all walls to be underpinned and adjoining walls of the neighbouring buildings should be implemented, with readings taken daily for the first week of de-watering, and following any change in the dewatering regime (see also Section 10.7). If movements exceed certain trigger levels, which should be agreed during the negotiations required for Party Wall Act purposes, then pumping should be reduced or cease sufficiently to stabilise the affected area, and revised groundwater control measures would then need to be agreed.

10.3.2 An appropriate discharge location must be identified for the groundwater removed by sump pumping.

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

10.3.4 The formation level clays/clayey silts onto which some of the underpins and the basement slab will bear must be protected from water and physical disturbance, because they may be sensitive to softening and weakening. Thus, the formation should be blinded with concrete immediately following excavation and inspection.

10.3.5 A leaking water supply pipe to the property could increase significantly the volume of water entries, so it would be prudent to ensure the isolation stopcock is both accessible and operational before the start of the works.

10.4 Slope and Ground Stability

10.4.1 <u>Slope Stability</u>

With overall slope angles estimated at less than 2° upslope of this property, the proposed basement excavation raises no concerns in relation to the overall stability of the slope.

10.4.2 <u>Underpinning Methods and Ground Movements alongside the Basement</u>

Use of mass concrete underpinning techniques are proposed beneath the party walls, as shown on Barrett Mahony's drawings, together with a basement 'box' supported on pile foundations. Underpinning methods involve excavation of the ground in short lengths in order to enable the stresses in the ground to 'arch' onto the ground or completed underpinning on both sides of the excavation. The inherent variability of Made Ground means that it cannot be relied upon to behave consistently. So the proposed 1.0m length of the underpins must not be exceeded, and it may be necessary to provide additional temporary support to the wall either side of the underpin. The presence of dense gravels beneath other parts of the underpins will further increase the potential for differential movement of these underpins. Accordingly, it is recommended that temporary support for the party walls should be installed before the underpinning excavations start, using cantilevered needles off pairs of piles. Given that one SPT recorded a zero blowcount, which indicates extremely weak or voided conditions locally in the Made Ground, and Barrett Mahony propose two-stage underpinning with no footing to the first stage, this temporary support is considered to be essential.

10.4.3 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 the use of high stiffness temporary support systems, installed in a timely manner in accordance with best practice methods, are therefore crucial to the satisfactory control of ground movements alongside basement excavations (see also 10.4.6 below).

10.4.4 The minimum temporary support requirements recommended for the proposed underpins and retaining walls at No's 13/15, subject to inspection and review as described in 10.4.7 below, are:

- Full face support must be installed as the excavations progress against all faces of all excavations. If significant quantities of rubble are present in the Made Ground below the level of the existing footings then it may be difficult to maintain stable faces to the excavations without causing undue loosening. Pre-treatment of the ground would then be required using a weak grout (to aid permeation and to facilitate re-excavation) in order to maintain the stability of the ground around the excavations.
- Temporary support will be required to 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.

10.4.5 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.

10.4.6 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.

10.4.7 A construction method statement and temporary works details have been provided in Barrett Mahony's Drg No.14771/701-T1 & ~/702-T1; the sequence should be refined and expanded as follows:

- A new item should be added to cover full structural repair of the cracking in walls to be underpinned before any excavations are undertaken beneath those walls.
- The bearing piles should be installed before the underpin walls are constructed.
- It is possible that the piles will encounter obstructions in the Made Ground (all the boreholes which were attempted with a lightweight 100mm diameter cfa rig were abandoned on obstructions at 0.9m to 2.0m bgl, whereas the more powerful 150mm diameter cfa rig was able to penetrate the Made Ground without incident; the larger diameter piling rigs would be able to remove or penetrate some materials which are impassable for ground investigation rigs). Obstructions above the founding level of the existing footings could be dug out with an excavator and the ground then backfilled and compacted in layers before the pile is re-bored. Obstructions below the level of the existing footings may need to be broken up with a down-hole hammer or cored oversize.
- Temporary support to the party walls should be installed before excavations for the underpins begin (see 10.4.2 above).

10.4.8 Preliminary Damage Category Assessment

Provided that the temporary support follows best practice as outlined above, then extensive past experience has shown that the bulk movements of the ground alongside the basement caused by underpinning to this depth should not exceed 5mm in either horizontal or vertical directions. The detailed precise monitoring should be used to check the actual displacements and to adjust the working methods or even the design if greater than expected movements start to occur.

10.4.9 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 have been shown to extend to a distance up to 4 times the depth of the excavation. So:

Depth of excavation = 4.1m.

Width (L) = 4.1 x 4 = 16.4m, so the ground movements might theoretically extend into No.21 and to the ground below the access ramp on the north side of No.11).

Height (H) = 8.6m (to 2nd floor mansard roofs)

Hence L/H = 1.91

Thus, the maximum horizontal strain beneath adjoining properties would, theoretically, be in the order of $\varepsilon_h = 3.05 \times 10^{-4}$ (0.031%) and the maximum deflection ratio, with allowance for 2mm of heave (as per PDISP analysis, see Section 10.5) and a convex settlement profile, would be about $\Delta/L = 1.6 \times 10^{-4}$ (0.016%). For L/H = 2 (approx.) these represent a damage category of 'very slight' (Burland Category 1, ε_{lim} =0.05-0.075%), just above the boundary to 'negligible' (Burland Category 0, ε_{lim} = 0 - 0.05%) as given in CPG4 (and CIRIA Special Publication 200, Table 3.2).

10.4.10 Use of best practice construction methods, as outlined in paragraphs 10.4.3 to 10.4.6, will be essential to ensure that the ground movements are kept in line with the above predictions.

10.4.11 Geotechnical Design

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 10.4.12);
- The presence of Made Ground below the founding level of part of the basement (see paragraph 10.4.13 below);
- Dead and live loads from the superstructure, including loads from the adjoining houses which are carried on the party walls;
- Imposed loads from all load-bearing walls of the neighbouring properties which are within the potential zone of influence of active pressures acting on the basement walls;
- A surcharge on the front wall of the basement to allow for vehicle loadings on the footway and carriageway to John's Mews
- A surcharge to allow for the higher ground levels to the rear of the basement, and normal surcharge allowances elsewhere;
- Swelling displacements/pressures from the underlying clays;
- A provisional design groundwater level at 1.0m bgl (see paragraph 10.2.8);
- Precautions to protect the concrete from sulphate attack.

10.4.12 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°
Reworked Ground & Alluvial Clays:		
	Unit weight, γ_b:	18.0 kN/m^3
	Undrained cohesion, Cu:	20-40 kPa
	Effective cohesion, c':	0 kPa
	Angle of internal friction, φ':	25°
River Terrace Deposits (Sandy Gravels):		
	Unit weight, γ_b:	19.0 kN/m^3
	Effective cohesion, c':	0 kPa
	Angle of internal friction, φ':	32°

These parameters should be used in conjunction with appropriate partial factors dependent upon the design method selected. The actual shear strength or state of compaction of the formation soils must be checked by a suitably competent person before each underpin or slab is cast, and local soft spots must be dug out and replaced with concrete.

10.4.13 Made Ground is not normally considered to be a suitable founding stratum owing to its inherent variability. As the founding level for the proposed basement is partly within the Made Ground, it would be possible to design the bearing pressures imposed by the underpins such that they would give minimal or no net change in vertical effective stress (slight heave beneath the underpins would actually be beneficial in reducing the effect of settlement of the ground alongside the underpins, as shown by the heave assessment in Section 10.5). However, that would leave the basement vulnerable to changes in uplift forces with any fluctuation in groundwater levels, with the potential for on-going movement between No's 13/15 and the neighbouring properties. To prevent that possibility it is recommended that the whole basement should be supported on a piled slab, with the piles bearing into the London Clay and designed to accommodate the maximum uplift force on the basement. This recommendation is now implemented in the scheme drawings and is still considered valid, even though BH5 has shown that part of the basement will be underlain by dense/medium dense gravels.

10.4.14 The formation level clays onto which the underpins and the basement slab will be constructed must be protected from water and disturbance to prevent softening and loss of strength, as described in 10.3.4 above.

10.4.15 The width of the footings to the mass concrete underpinning is proposed "to match width of existing footing" (Barrett Mahony Drg No. L14771/04-T1). The existing footings exposed in the trial pits have widths which vary from about 0.5m to 1.25m (on the assumption that the walls are central on the footings), so this approach would give rise to very variable bearing pressures which would further increase the likelihood of local over-stressing of the variable Made Ground. For this reason, mass concrete underpins would not normally be used in these ground conditions, so consideration should be given to switching to reinforced concrete underpins with bases of sufficient width to allow the underpins to be supported by the piles. The number of piles would also need to be increased to one per underpin (as previously proposed), located closer to the perimeter of the basement and with adequate reinforcing in the piles to carry the eccentric load, before the central slab is cast.

10.4.16 <u>Cumulative Impact:</u>

Use of underpinning techniques is also planned for the proposed basement beneath No.21 John's Mews and the linking section to the existing basement beneath No.27 John Street (the 27JS-21JM basement), as noted in paragraph 2.8 above. The 27JS-21JM basement may have a similar impact on the ground beneath the adjoining properties as that predicted above for No's 13/15's basement, provided once again that best practice methods of underpinning are used. Construction of basements beneath both No's 13/15 and 27JS-21JM would have a cumulative impact on the 17/19 party wall, however that would be beneficial to either No.17 or No.19 relative to the likely impact if only one of the basements were to be built, because greater settlement of the 17/19 party wall would result in less differential settlement across whichever building (No.17 or No.19) would otherwise be closest to the one new basement.

10.5 Heave/Settlement Assessment

10.5.1 Basement Geometry and Stresses:

10.5.1.1 Figure C1 in Appendix C illustrates the proposed basement based on FT Architects' Drg No.200-32-18. The layout of the proposed underpins is presented in Figure C2 based on TS Consulting's Drg No. 1420_02.

10.5.1.2 Table 2 presents the co-ordinates used to input the main elements of the basement's geometry into PDISP, together with the net changes in overburden pressure resulting from a combination of the gross unloading from the excavation down to the basement founding level, the self-weight of the underpins and the maximum imposed loads from the superstructure, excluding live loads, as given by TS Consulting (see 'Load-01' sheet in Appendix C).

Gross unloading:

- Depth of excavation = 3.8m (paragraph 3.4)
- Estimated unit weight, γb = 17.0 kN/m^3.

Basement dimensions:

- 11.8m wide by 12.7m long, excluding strip footings (also taken from the TS Consulting's 'Load-01' sheet).

Table 2: Co-ordinates and loading detail of the underpin zones

	Dimension		Centroid		Angle with X-Axis	Net change in Bearing Pressure (kPa)
Zone	X (m)	Y (m)	Cx (m)	Cy (m)		
Wall A	2	8.755	1.48	6.35	6.43	6
Wall B	2	8.755	11.3	6.35	6.43	6
Wall C	11.8	2	5.9	11.7	0	-31
Wall D	11.8	2	6.88	1	0	-24
Wall E	2	8.755	6.39	6.35	6.43	14
Excavation 1	2.863	8.755	3.94	6.35	6.43	-65
Excavation 2	2.863	8.755	8.84	6.35	6.43	-65

10.5.2 Ground Conditions:

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

10.5.2.2 The geotechnical soil properties adopted for the analysis by PDISP are summarized in Table 3 below, based on the log of the borehole drilled by CSI and our previous experience of basement projects in the London Clay.

Table 3: Soil parameters for PDISP analyses

Strata	**Level** (m bgl)	**SPT blowcount** N	**Short term, undrained Young's Modulus,** Eu (MPa)	**Long term, drained Young's Modulus,** E' (MPa)
Made Ground	3.8-5.9	17	35	20
London Clay	5.9 27.5	20	40 120	25 70

Where:
Drained Young's Modulus = 2 x N
London Clay: Undrained shear strength, Cu assumed = 80kPa at 5.9m bgl
Eu = 500 * Cu Hence profile of Eu = 40 + 3.75z
Drained Young's Modulus was estimated based on E' = 0.6 Eu
where z = depth below the top of the London Clay stratum.

10.5.3 PDISP Assessment:

10.5.3.1 Three dimensional 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 and underpinning of the relevant walls. These analyses used the basement geometry, loads/stresses and ground conditions outlined above. PDISP analyses have been carried out as follows:

- Stage 1 – Effect of underpin loads
- Stage 2 – Effect of excavation – Short-term condition
- Stages 3 & 4 – Construction of basement slab leading to Long-term conditions

10.5.3.2 The results of the short-term and long-term analyses are presented as contour plots on Figures C3 and C4 respectively in Appendix C.

10.5.3.3 The analyses indicated that small heave movements are likely to develop beneath the underpins to the perimeter walls, while slightly larger heave movements are predicted beneath the basement slab. The ranges of predicted short-term and long-term movements for each of the main walls are presented in Table 3 below. These values are approximate, so should be used as a general guide to possible movements rather than definitive values.

Table 3: Summary of predicted heave displacements		
Location	**Short-Term** (Figure 3)	**Long-Term** (Figure 4)
Front wall (Wall D)	2 - 5mm Heave	2 - 8mm Heave
15/17 Party Wall (Wall B)	2 - 5mm Heave	2 - 7mm Heave
Rear wall (Wall C)	2 – 5.5mm Heave	3 - 9mm Heave
11/13 Party wall (Wall A)	2 - 5mm Heave	3 - 7mm Heave
Central wall (Wall E)	3 - 5mm Heave	4 - 8mm Heave
Centre of basement slab	Max 7mm Heave	Max 11mm Heave

10.5.3.4 When the analyses were re-run including live loads, the heave magnitudes generally decreased by 1mm beneath the walls and 2mm beneath the central slab areas.

10.5.3.5 Excavation of the basement will cause immediate elastic heave in response to the stress reduction, followed by long term plastic swelling as the underlying over-consolidated clays take up groundwater (although minimal or none in the case of the alluvial clays). The rate of plastic swelling will be determined largely by the availability of water and as a result, given the low permeability of the London Clay, can take many years to reach full equilibrium.

10.5.3.6 All the short-term ground movement would have occurred before the basement slab is cast, so only the post-construction incremental heave is relevant to the slab. The maximum predicted heave beneath the slab is in the central area of excavation, where the maximum post-construction heave beneath the basement slab is predicted to be approximately 4mm.

10.5.3.7 Given the presence of Made Ground below the basement and the resulting importance of balancing, as far as possible, predicted heave and settlement magnitudes which will result from construction of the basement, it is recommended that further ground movement analyses must be undertaken during the design stage in order to assess further the likely range of heave/settlement magnitudes.

10.6 Surface Flow and Flooding

10.6.1 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 Environment Agency;
- John's Mews 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 latest flood models by both the Environment Agency and Camden's SFRA (July 2014) give a 'Very Low' risk of surface water flooding (the lowest category, which represents the national background level of risk) for this property (see Figures 6 & 7).

10.6.2 The site is also known to lie close to the former alignment of one of the Fleet's tributaries which has been culverted (as described in Section 5 above) so it is no longer able to receive direct surface water run-off, although the highway drains are probably connected to the culvert in Roger Street. Whether the culvert remains connected hydraulically to the perennial surrounding groundwater is unknown.

10.6.3 <u>Change in Paved Surfacing & Surface Water Run-off:</u>
The proposed basement will be entirely beneath the existing building, so there will be no change in the area of hard surfacing. Thus the surface water run-off will remain unchanged.

10.6.4 <u>Surface Water (Pluvial) Flooding:</u>
The latest surface water flood modelling shows a ribbon of 'Low' risk of flooding along the east side of the carriageway to John's Mews, which must represent a flow route when highway gullies are surcharged. No.13's garage opening and No.15's entrance door are already both raised above the gutter level by approximately 0.2m. The lower part of the new screen which will replace No.13's garage door should be designed and specified to be fully watertight. Further flood resistance could, optionally, be provided by the provision of watertight entrance doors although it is considered very unlikely that flood water would ever rise above the level of the thresholds under the modelled event.

10.6.5 The enclosed courtyards to be created at the rear of the new houses will receive only direct rainfall, so flood resistance measures should be limited to provision of suitably raised thresholds to the doorways giving access to those areas.

10.6.6 <u>Sewer Flooding:</u>
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.

10.6.7 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.

10.6.8 Non-return valves and/or pumped above ground loop systems should be fitted on the drains serving the basement and the enclosed courtyards, in order to ensure that water from the combined/foul sewer system cannot enter the basement or flood the courtyards when the public sewers are operating under surcharge. A battery powered reserve pump should be fitted to ensure that the system remains functional during power cuts.

10.6.9 If non-return valves are used without an above-ground loop, then no surface water would be able to enter the sewer whenever the surcharge in the main sewer is sufficient to close the valves. The basement could then be vulnerable to flooding while the rainfall continues. Sufficient temporary interception storage should therefore be provided if non-return valves are used, in order to hold temporarily the predicted maximum volume of surface water run-off from all sources (roof and courtyards) and foul water for the duration of a design storm. This temporary interception storage would require formal design to ensure satisfactory performance.

10.6.10 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/highway(road) 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.

10.6.11 Cumulative Impact:
No cumulative impact would be expected on surface water flooding from construction of both the proposed basement beneath No's 13/15 and the 27JS-21JM basement (No.21 John's Mews and the linking section to No.27 John Street).

10.7 Mitigation

10.7.1 The following mitigation measures should be implemented, as recommended in more detail in the preceding parts of Section 10:

- All structural crack damage in walls that are to be underpinned, which will have weakened the building's structural integrity, should be fully repaired in accordance with recommendations from the appointed structural engineers before any underpinning is carried out. Consideration should be given to stitching these cracks with resin-bonded tie bars (eg: Helifix bars) as part of this repair.
- Subject to Party Wall Award negotiations, consideration should be given to the inclusion of transitional underpinning blocks beneath the load-bearing walls to the adjoining properties, except where the existing foundations would provide sufficient transition.
- Installation of non-return valves and/or a pumped above-ground loop system to prevent flooding of the basement when the main sewer is operating under surcharge.

10.8 Monitoring

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

10.8.2 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 the following locations:

- internally, at intervals along both party walls;
- externally, on the adjacent front and rear walls to Nos.11 & 17;
- the front and rear walls to No's 13/15, and the internal former party wall.

This monitoring frequency should be increased to daily for a minimum of one week at the start of the dewatering operation, and at any change in the dewatering regime (see 10.3.1).

10.8.3 If any undue movements are recorded, the frequency of readings should be increased as appropriate to the severity of the movement and consideration should be given to installing additional targets.

10.8.4 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.

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 The site-specific ground investigations in 2014 and 2015 have found that the building has already been partially underpinned, mainly at the front, although the extent and depth of underpinning remains unclear and will require further investigation. The investigations also recorded Made Ground to depths of 3.0-5.9m (which is compatible with two other nearby boreholes), the lower part of which appeared to be disturbed alluvium. Where the Made Ground was deepest, the gravels of the River Terrace Deposits were absent (Section 9 & paragraph 10.1.1).

11.3 A services search should be undertaken, with particular enquiries regarding the known nearby government communications tunnel (10.1.3).

11.4 The proposed basement will be partially within the Made Ground and is considered acceptable in relation to the apparently limited flow of groundwater through the Made Ground, while any groundwater flow would be able to pass largely or completely unhindered through granular soils below the basement where it is constructed in or above the upper part of the River Terrace Deposits (10.2.1, 10.2.2).

11.5 The basement will be constructed below the water level, so will need to be fully waterproofed (10.2.2, 10.2.3, 10.2.6). Consideration should be given to making the basement gas-tight (10.2.7).

11.6 The standpipes should be maintained to allow further groundwater monitoring during detailed design and prior to construction (10.2.3). A provisional design groundwater level at 1.0m below ground level is proposed, which means that the basement must be able to resist buoyant uplift pressures (un-factored) of up to 28kPa/31kPa (10.2.8, 10.2.9).

11.7 Groundwater control will be required, probably by pumping from multiple screened sumps. As the buildings are founded in Made Ground over possible weak alluvium, precise monitoring of building movements should be carried out during the initial de-watering period and whenever the dewatering regime is altered, unless all the walls to be retained are provided with temporary support off the pile foundations (10.3.1). The clays onto which the underpins and the basement slab will be constructed must be blinded with concrete immediately following excavation and inspection (10.3.4).

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

11.9 Under present proposals, the perimeter walls to the basement will be constructed using mass concrete underpinning techniques; best practice methods using high stiffness temporary support systems will be required. Full face support must be allowed for all the excavations , and grouting may be required if the high rubble content makes it difficult to maintain stable faces (10.4.3 to 10.4.6).

11.10 The construction sequence provided by Barrett Mahony should be expanded to conform with the recommendations herein (10.4.7).

11.11 Preliminary damage category assessment calculations, for movements in the ground alongside the retaining walls, indicated that the damage, if any, could be expected to fall within Burland Category 1 – 'very slight', close to the boundary with Burland Category 0 'negligible' (10.4.8 to 10.4.10).

11.12 The basement slab will be supported on piles bearing into the London Clay and should be designed to resist the maximum uplift pressure from the groundwater (10.4.13).

11.13 Various other guidance is provided in relation to the geotechnical design and construction of the basement's perimeter walls (10.4.11 to 10.4.14).

11.14 The basement slab must be designed to accommodate swelling displacements/ pressures generated by heave of the underlying clays. PDISP ground movement analyses have indicated that heave in the order of 2-9mm could be expected beneath the underpins, with about 4mm post-construction incremental heave beneath the central slab areas, if the basement slab is constructed after the underpins (Section 10.5).

11.15 The basement will be wholly below the existing building, so there will be no change in the area of hard surfacing and hence no change in surface water run-off (10.6.3).

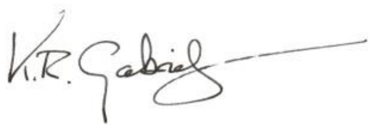
11.16 Flood resistance measures to protect the protect the property from the 'Very Low' risk of surface water flooding include: making the lower part of the screen which will replace No.13's garage door fully watertight, the possible provision of watertight front entrance doors, and provision of suitably raised thresholds to the rear courtyard access doors (10.6.4, 10.6.5).

11.17 Non-return valves or a pumped above-ground loop system should be fitted to the drains serving the basement and gullies in the lightwells (10.6.7).

11.18 If non-return valves are fitted, then temporary interception storage should be provided for the surface water accumulating from an appropriate design period rainstorm; formal design would be required (10.6.9, 10.6.10).

11.19 Mitigation measures should include repair of the structural cracking before any underpinning is carried out, and installation of non-return valves and/or a pumped above-ground loop system to prevent flooding of the basement when the main sewer is operating under surcharge (Section 10.7).

11.20 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 10.8).



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