Report

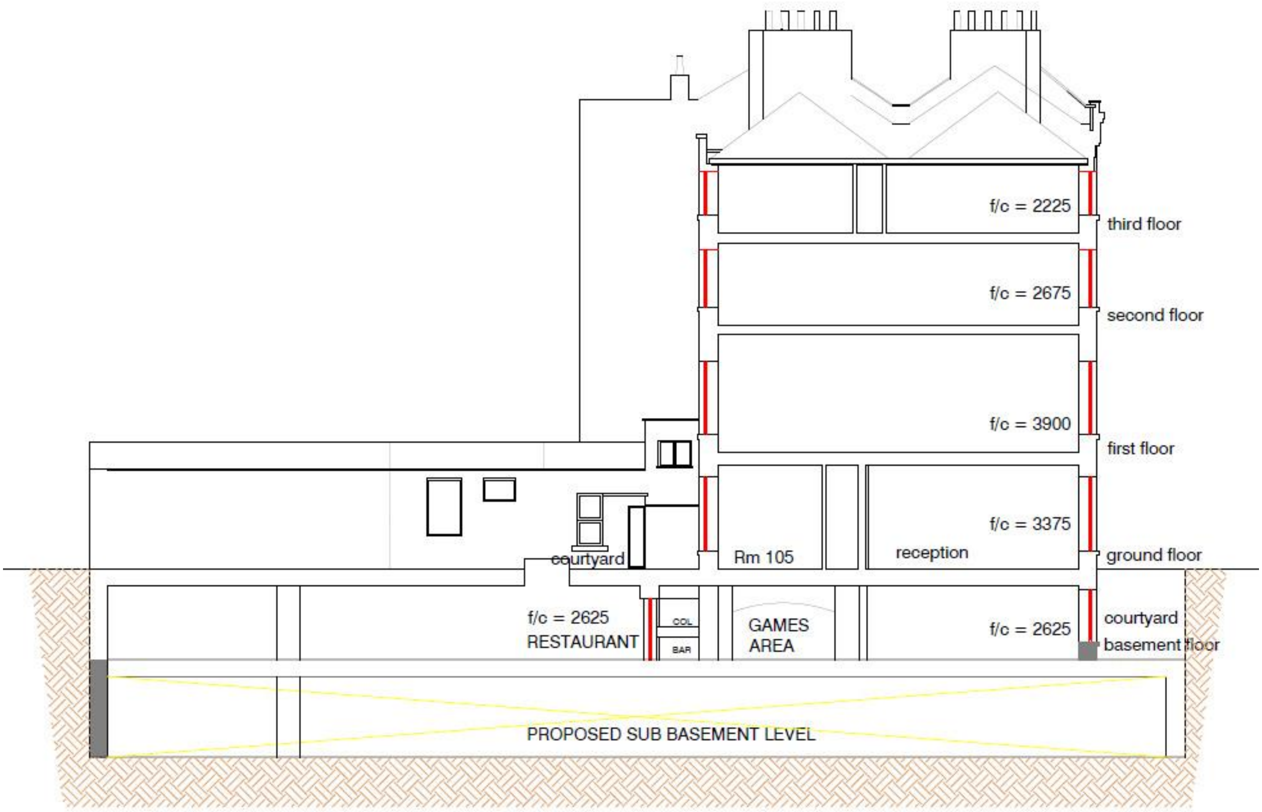
GabrielGeo Consulting

Basement Impact Assessment
('Geo-hydrology' Report)

6-7 Dorset Square, London, NW1 6QA

for

Mr I Chishty



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Basement Impact Assessment
('Geo-hydrology' Report)

Site: **6-7 Dorset Square, London, NW1 6QA**

Client: **Mr I Chishty**

Report Status: **FINAL**		
Role	**By**	**Signature**
Desk study, sitework, factual reporting and ground model by:	Alexander Goodsell BSc ACSM FGS	Alexander Goodsell
Impact Assessments by: Report checked & approved by:	Keith Gabriel MSc DIC CGeol FGS UK Registered Ground Engineering Adviser	K.R. Gabriel

Foreword

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

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

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

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

Contents Page

1. INTRODUCTION

1.1 This Basement Impact Assessment has been prepared to support a planning application to be submitted to the City of Westminster for the construction of a single storey sub-basement beneath Somerset Hotel, No's 6 & 7 Dorset Square, London, NW1 6QA. This report covers the matters required by Westminster's Basements SPD (October 2014) for a "*supplementary geo-hydrology report*" as part of the Structural Methodology Statement (SMS) in accordance with Policy CM28.1 of Westminster's City Plan (November 2016). This report is for planning and scheme development purposes and is not a design document. Appendix 1 to the Basements SPD sets out requirements for the contents of the SMS; this report addresses Sections A, C, part of D, E, F & G of those requirements.

1.2 Preparation of this assessment has been supervised/written by Keith Gabriel, a Chartered Geologist and UK Registered Ground Engineering Adviser with an MSc degree in Engineering Geology. The author has previously undertaken assessments of over 200 basement projects in nineteen London Boroughs.

1.3 **Desk Study:** Westminster's Basements SPD requires a "*thorough desk study*". Sections 2–6 herein present the information collected from various sources including borehole/well logs from the vicinity of the site (Appendix B), borough-specific flood reports and flood modelling, and geological data, environmental data and historic maps from Groundsure which are presented in Appendices C to E respectively. A preliminary inspection of the property (walk-over survey) was undertaken on 6th December 2018, followed by a more detailed inspection during the ground investigation, photos from which are presented in Appendix A.

1.4 **Ground Investigation:** Sitework for the ground investigation (borehole and trial pits) was undertaken on 30th April and 1st May 2019. The findings from the ground investigation are presented in Section 7.

1.5 **Impact Assessments:** The basement impact assessments, including the preliminary ground model, are presented in Section 8.

1.6 The following site-specific documents in relation to the proposed sub-basement have been considered:

- **NATION CONSTRUCTION Ltd:**

Drg No. 1512/01	Existing and Proposed Basement Plan
Drg No. 1512/02	Proposed Sub-basement Plan
Drg No. 1512/03	Existing Sections
Drg No. 1512/04	Proposed Sections
Drg No. 1512/05	Location and Block Plans

Minor discrepancies were found by MACE (see below) in the sub-basement dimensions so these plans should be considered as indicative.

- **Pearson Architects** (2011 drawings for a different scheme)**:**

  Drg No. DS/E08 Front Elevation as Existing

  Drg No. DS/E10 South Section as Existing

- **Michael Alexander Consulting Engineers (MACE):**

  Drg No. P4463 Existing Basement Loading Key Plan

  Drg No. P4463 New Sub-basement Loading Key Plan

  'P4463 working cad file 23-08-19' with two sections and three plans.

This report should be read in conjunction with all the documents and drawings listed above. No level survey has been available so all levels related to Ordnance Datum have been estimated by correlation with adjoining properties.

2. THE PROPERTY, TOPOGRAPHIC SETTING AND PLANNING SEARCHES

2.1 No's 6 & 7 Dorset Square consist of two large five-storey (including lower ground floor/basement) terraced properties, which together form Somerset Hotel, situated within the Dorset Square Conservation Area in the City of Westminster. Somerset Hotel is located on the east side of Dorset Square, which forms a short section of the A41 (Gloucester Place), extending from Melcombe Street to the south, to the adjoining No.8 Dorset Square to the north. To the rear (east), No's 6 & 7 are adjoined by No's 47 & 48 Chagford Street respectively, and to the south, No.6 Dorset Square is adjoined by No.5 Dorset Square, as shown in Figure 1.

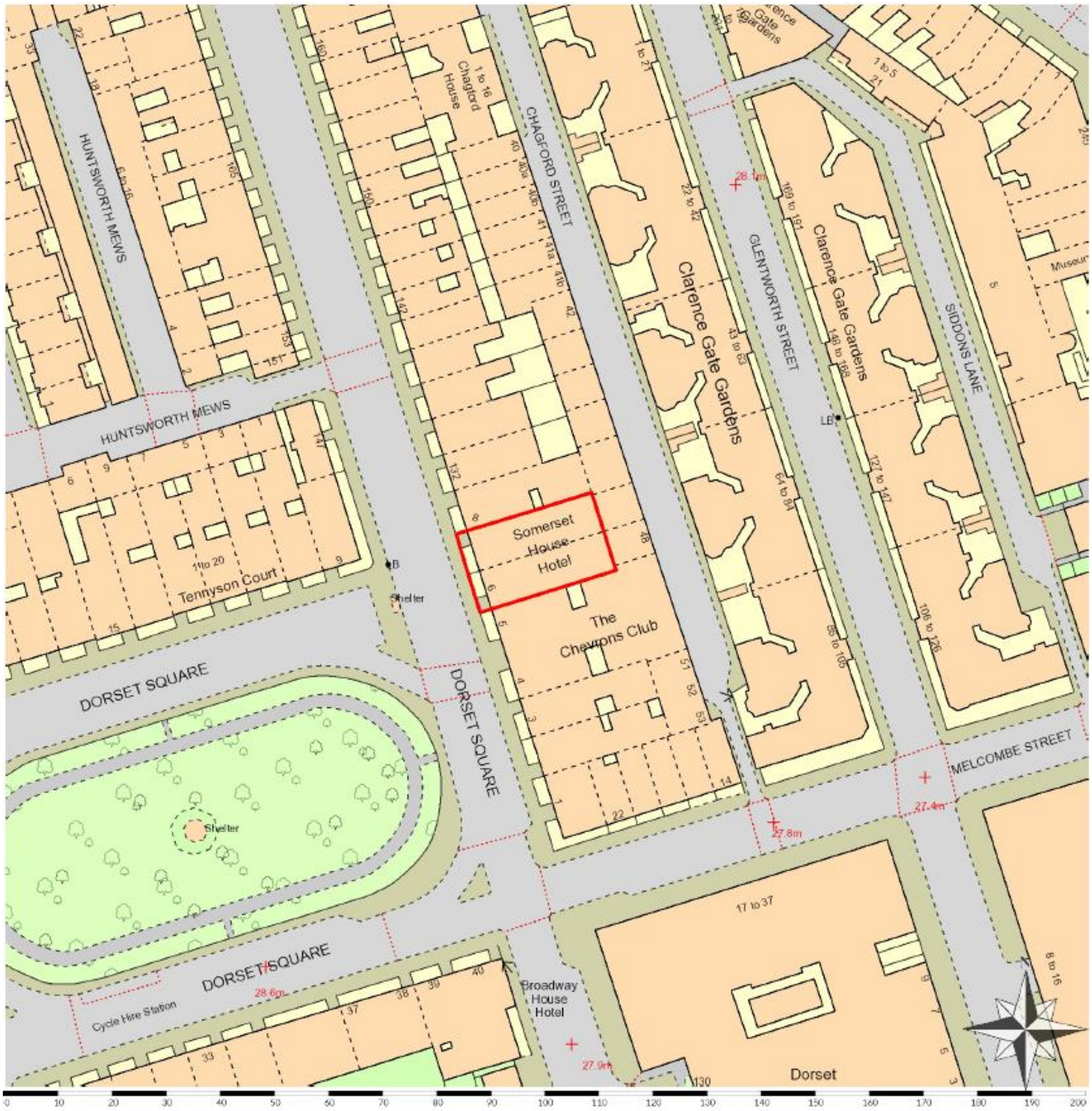

Figure 1: Extract from 1:1,250 OS map (not to scale) with the site outlined in red.

2.2 The properties on the east side of Dorset Square (including No's 6 & 7) are all of similar appearance, with existing lower ground floor levels and open lightwells at the front of the properties. At Somerset Hotel, a flight of metal steps provides access from the footway down to the front lightwell of No.6, with an opening between the two lightwells allowing access to No.7's front lightwell. The only other outside space within the site of Somerset Hotel is a small lightwell alongside the main rear wall of No.7. Additionally, there are vaults beneath the front footway, which can be accessed either internally, beneath the front entrance paths, or via the front lightwells.

2.3 Some structural damage was noted during the site inspection, including bulging of the main front wall of the property between the second and third floor windows, which is at its most severe at No.6 (see Photo 1 in Appendix A). It is understood that repairs/remedial measures have been implemented internally to pin back this area of the front wall.

2.4 The London County Council Bomb Damage Maps 1940-1945 (LTS, 2005) record that No's 6 & 7 Dorset Square sustained damage classified as '*Blast damage, minor in nature*' and '*General Blast damage – not structural*' respectively. Other details provided by the LTS maps (2005) include:

i. The adjoining No.8 Dorset Square was also classified as having sustained '*General Blast damage – not structural*', but immediately to the north, No's 132 & 134 Gloucester Place were classified as "*Seriously damaged, but repairable at cost*".

ii. On the opposite side of the Dorset Square/Gloucester Place (A41) carriageway, those properties at the eastern end of the terrace which fronts onto the northern side of Dorset Square (No's 9-12) are recorded as "*Seriously damaged, but repairable at cost*", with the exception of the rear part of No.9 Dorset Square, which is classified as "*Total destruction*";

iii. Properties at the eastern end of Huntsworth Mews were classified as '*Damaged beyond repair'*;

2.5 Dorset Square is shown on both Greenwood's 1830 map of London (drawn from a survey undertaken in 1824, 1825 & 1826), and Cross's London Guide for the Great Exhibition (dated 1851). These maps, along with the historic Ordnance Survey (OS) maps presented in Appendix E confirm that all of the surrounding road network and houses, including No's 6 & 7 Dorset Square, had been built before publication of the 1830 map,. The 1872 OS map, the earliest 1:1,056 scale OS map available from Groundsure (since the 1865 1:2,500 map appears blank), shows that the extensions to the rear of the properties had been built prior to this date, however they are not shown on Greenwood's map of London, which shows open space (rear gardens?) between the main five-storey part of No's 6 & 7 and the mews houses to the rear (No's 47 & 48 Chagford Street). The 1872 OS map also shows that although the surrounding road network had been built prior to this date, a number of roads have changed names, with Gloucester Place previously named Dorset Place, Melcombe

Street previously named New Street, and Chagford Street previously named New Street Mews.

2.6 Between publication of the 1876 and 1896 OS maps, Dorset Square was renamed to Upper Gloucester Place, and between publication of the 1916 and 1952 OS maps, the surrounding roads were re-named to their current names. Overall, relatively few significant changes can be seen within the near vicinity of Somerset Hotel, between publication of the 1872 and current OS maps. Exceptions to this however include Glenworth Street, located just to the east of Dorset Square, which was re-named from Park Street, and completely redeveloped to construct the Clarence Gate Gardens development, between publication of the 1897 and 1916 OS maps.

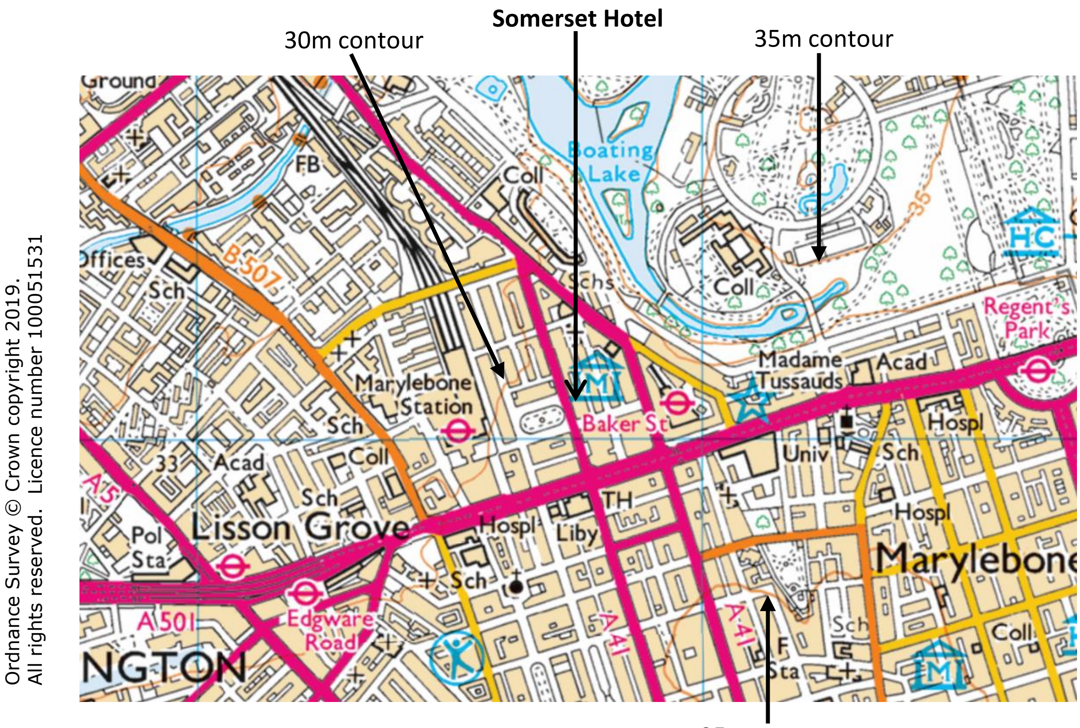

Figure 2: Extract from 1:25,000 Ordnance Survey map (not to scale) showing site location.

<u>Topographic Setting:</u>

2.7 Dorset Square is situated on a very gentle, broadly south to south-east facing slope, with the contours in Figure 2 indicating an average slope angle across the site of approximately 0.4-0.5° towards the south-east (calculated between the 30m and 25m contour lines). The shape of these contour lines indicate that the property lies on the south-west side of a weekly defined valley feature, which is consistent with the positon of the property relative to the Tyburn, one of the 'lost' rivers of London (see Section 5). Spot heights on the 1974 1:1,250 scale OS map in Appendix E indicate that this part of the Dorset Square (eastern end), as well as the adjoining Gloucester Place (A41) to the north, fall very gently southwards, with slope angles of around 0.15° to 0.30°, calculated from the spot heights located in front of No.164 Gloucester Place

(29.0m above Ordnance Datum, AOD), in front of No.7 Dorset Square (28.7m AOD), and just south of the junction between Dorset Square and Melcombe Street (28.3m AOD).

Planning Searches:

2.8 Searches were made of planning applications on the City of Westminster's online planning database on 8th April and 22nd August 2019 in order to obtain details of any other basements which have been constructed or are planned in the vicinity of Somerset Hotel. These searches found only:

- **No.5 Dorset Square (adjoining property):** Many applications for internal alterations, changes to windows etc (in conjunction with No's 3 & 4), mostly in connection with change from offices to residential use. Details of the lower ground floor were obtained from:
  - Application 02/03194/FULL (and 02/03195/LBC) for "*Internal and external works of refurbishment, including the rebuilding of the closet wing of No. 3, in connection with the conversion of the property into 14 self-contained flats*".
  - Application 04/00882/ADLBC for "*Details of skylights to the basements of all houses pursuant to condition 12b of listed building consent dated 10 September 2002 (RN 02/03195)"*.
  - Application 18/04485/LBC for "*Remedial damp proofing works to under-pavement vaults and installation of studded waterproof membrane and drainage channels*".
- **No.8 Dorset Square (adjoining property):** No applications found in period since 1996.
- **No.46 Chagford Street:** Application 11/10882/FULL for the "*Use as a single family dwelling. Excavation of basement floor and alterations to windows and doors to front elevation, including creation of integral garage. Installation of lantern light to main roof and alteration to roof at rear*." was granted permission on 17th Jan 2012. A 'Ground Report' was found on the website, but no site specific ground investigation was carried out.
- **No's 42 & 43 Chagford Street:** Application 16/02520/FULL for the "*Demolition of three storey office building with integral garage and basement and construction of two single family dwelling houses comprising basement, lower ground, ground and three upper floors with integral garages*." Was granted permission on 16th May 2016. A BIA by SAS was found on the website.

3. PROPOSED SUB-BASEMENT

3.1 Drawings by Nation Construction Ltd (see paragraph 1.6) show that the proposed sub-basement for which planning permission will be sought comprises a single-storey beneath the full footprint of the building, including beneath the front lightwells at existing basement level. The proposed sub-basement will not extend beneath the front vaults. This sub-basement level will only be accessible internally, via two flights of stairs.

3.2 Currently, Nation Construction's 'Proposed Sub Basement Plan' (Drg No. 1512/02) shows that the proposed sub-basement will not extend beneath the front entrance ways, either side of the open front lightwells. Revised proposals to this sub-basement plan include extending it beneath these areas, as shown on the drawings by Michael Alexander Consulting Engineers (MACE).

3.3 The finished floor level (FFL) in the sub-basement is proposed to be **4.00m below the FFL** in the existing basement/lower ground floor. This is slightly deeper than indicated on Nation Construction's 'Proposed Sections' (Drg No. 1512/04) which shows a headroom (floor to ceiling) of 2.90m throughout the proposed sub-basement.

3.4 Preliminary structural details for the proposed sub-basement have been provided by MACE. These include, in addition to the two drawings listed in paragraph 1.6, use of reinforced concrete underpins with 400mm thick base slabs and a matching thickness for the sub-basement slabs between the underpins in order to create a raft slab.

4. GEOLOGICAL SETTING

4.1 Mapping by the British Geological Survey (BGS) indicates that the site is underlain by the Lynch Hill Gravel Member (area 1 in Figure 3), over the London Clay Formation. Approximately 70m to the east of the site, the Langley Silt Member (area 2) can be found overlying the Lynch Hill Gravel Member, and around 200m to the north of the site, the London Clay Formation can be found at surface. Figure 3 shows an extract from Groundsure's GeoInsight report which illustrates the superficial deposits that overly the London Clay in relation to the road network for this part of London (see also Appendix C). In urban parts of London these natural strata are typically overlain by Made Ground.

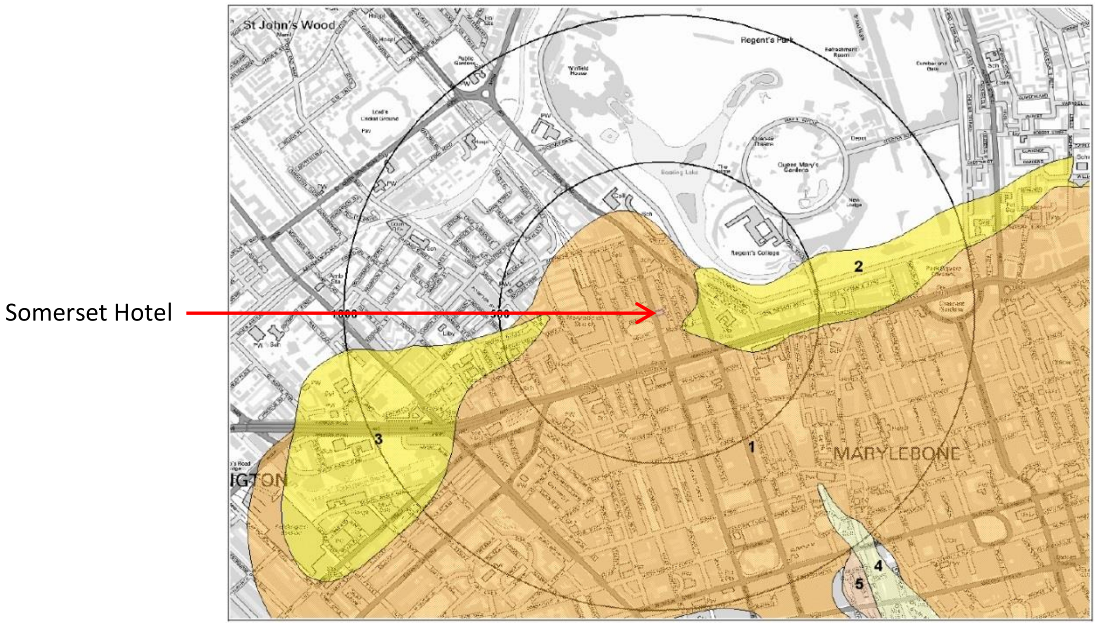

Figure 3: Extract from Groundsure GeoInsight report, illustrating the superficial deposits in the vicinity of the site and the surrounding areas (centred on Somerset Hotel).

4.2 The Lynch Hill Gravel Member is a River Terrace Deposit (RTD) associated with the River Thames and its tributaries. This deposit is described collectively with the other RTDs by the BGS memoir (Ellison et al., 2004) as "*variable proportions of sands and gravels*" along with local "*impersistent beds … of clayey and silty sand*". Subordinate clay horizons also occur at all levels within the River Terrace Deposits, while peats are rarely present (usually at the base of the sequence). The thickness of the Lynch Hill Gravel Member varies across London, and is known to reach 7.5m thick in the Paddington area. The thickness can also vary significantly locally over short distances owing to the presence of deep drift-filled hollows (sometimes called scour features).

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). These changes can occur seasonally, in response to normal climatic variations, to depths of up to 1.50m and to much greater depths in the presence of the trees whose roots abstract moisture from the clay, those these factors have little or no relevance where the London Clay is overlain by RTDs as in this case. The clay will also swell when unloaded by excavations such as those required for the construction of basements. The likely thickness of the London Clay Formation in this area exceeds the depth considered relevant to the proposed development, and as a result, the geology beneath the London Clay Formation is not considered further.

4.4 The Groundsure GeoInsight report (see Appendix C, Sections 4, 5 & 9) records:

- The historical surface ground working features within 250m of the site are variably recorded as 'lake', 'water body', 'pond' and 'boating lake'; all records refer to the same feature 226-230m north-east of the site in Regents Park (App. C, Section 4.1).
- Multiple historical underground working features within 1000m of the site. These are all tunnels, the closest of which recorded in Section 4.2 of the GeoInsight report are London Underground Ltd's (LUL) Metropolitan line tunnels at Baker Street Station, located 126-129m north-east of the site. However, this database does not include other closer LUL tunnels (see below).
- One historic 'mining' feature within 1000m of the site, this is an 'air shaft' located 647m to the north-west (App. C, Section 5).
- There are five underground railway lines within 250m of the site, these are London Underground's Jubilee, Bakerloo, Metropolitan, Circle and Hammersmith & City lines. The closest is the Jubilee Line, which passes 33m south of the site at a recorded depth of 25m below ground level. One additional railway tunnel is recorded at 223m east of the site which is probably another section of the Jubilee or Bakerloo line tunnels (App. C, Section 9.1).

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

4.5 The results of the BGS natural ground subsidence hazard classifications are provided in the Groundsure GeoInsight report (Appendix C); all indicated 'Negligible' or 'Very Low' hazard ratings on site. For an area 25m to the north-east, and further to the north, east and west of the site, 'Shrink - Swell Clay' was given a 'Moderate' hazard rating, which is likely to reflect a reduced thickness of the Lynch Hill Gravel in this area, near the boundary with the underlying London Clay Formation.

4.6 A search of the BGS borehole database was undertaken for information on previous ground investigations and any wells in the vicinity of the site, the locations of which are presented on the location plan in Appendix B. The strata depths in a selection of these boreholes are summarised in Table 1. Also summarised in Table 1 is the log of a borehole drilled at 42-43 Chagford Street, located just to the north-east of Somerset Hotel, which was drilled as part of a Basement Impact Assessment, found during the search of planning applications on the City of Westminster's online planning database. For full strata descriptions reference should be made to the logs in Appendix B.

Table 1: Summary of Strata in BGS and other Boreholes

Strata (abbreviated descriptions)	**Depths (m) and levels (m AOD) to base of strata**							
	42-43 Chagford Street		**TQ28SE/453**		**TQ28SE/322**		**TQ28SE/452**	
GL (mAOD)	Depth 15.0	**Level 23.30**	Depth 28.95	**Level 27.48**	Depth 15.24	**Level 27.55**	Depth 27.58	**Level 28.77**
Made Ground	0.70	**22.60**	0.15	**27.33**	0.46	**27.09**	0.91	**27.86**
Variably clayey gravelly SAND/sandy GRAVEL, to CLAY with occasional gravel (River Terrace Deposits: LHGM)	1.50	**21.80**	6.40	**21.08**	7.77	**19.78**	8.84	**19.93**
Stiff brown silty sandy CLAY (Weathered London Clay)	1.90	**21.40**	6.70	**20.78**	8.23	**19.32**	9.45	**19.32**
Stiff dark grey/blue, fissured silty sandy CLAY (Unweathered London Clay)	>15.0		>28.95		>15.24		>27.58	
Seepage/Strike	0.70	**22.60**	2.10	**25.38**	7.32	**20.23**	8.20	**20.57**
Groundwater Standing Level	-	-	-	-	-	**-**	7.60	**21.17**

5. HYDROLOGICAL SETTING (SURFACE WATER)

5.1 The site lies just to the west of the former alignment of the Tyburn, one of the 'lost' rivers of London (see Figure 4 and Barton & Myers, 2016), which is described as "flowing between Chagford and Glentworth Streets". This former river now flows within the sewerage system or in dedicated culverts, before discharging into the River Thames.

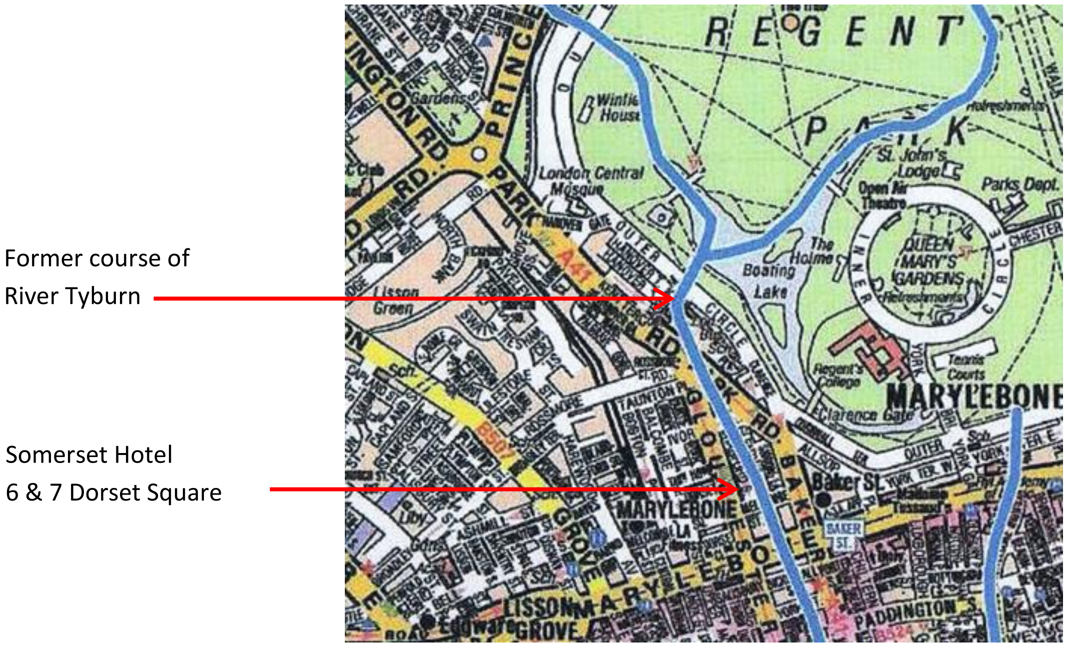

Figure 4: Extract from Map 15 of Barton & Myers' Lost Rivers of London (2016) – 'The Regent's Park section of the Tyburn'.

5.2 The Environment Agency's 'Flood map for planning' website shows that this area is in Flood Zone 1, with the nearest Flood Zone 2 or 3 being approximately 2.65km to the south-east, close to Buckingham Palace. Zone 1 is defined as "*Land having a less than 1 in 1,000 (0.1%) annual probability of river or sea flooding*" and is described as having a **Low** probability of such flooding. This classification does **not** take into account flood defences or climate change.

5.3 The Environment Agency's 'Long term flood risk information' website shows that the Somerset Hotel is also in an area classified as having a **Very Low** risk of flooding from rivers or the sea which once again means that each year this area has a chance of flooding of less than 0.1%. This is from the EA's RoFRAS dataset and **does** take into account the effect of any flood defences in the area. Such defences reduce but do not completely stop the chance of flooding as they can be overtopped or fail, though are not relevant to Dorset Square. The same website shows that Somerset Hotel does **not** fall within an area at risk of reservoir flooding, with flood water in the "*maximum extent of flooding*" coming no closer than approximately 780m to the north of Somerset Hotel.

5.4 Some additional hydrological data for the site has been obtained from the Groundsure EnviroInsight report (see Appendix D), including:

- One surface water feature is recorded within 250m of the site, which is the closest section of the boating lake in Regent's Park at 232m north-east of the site (App.D, Section 6.11). There are no recorded 'Ordnance Survey MasterMap Water Network' entries within 500m of the site (App.D, Section 6.10).
- One surface water abstraction licence within 2000m of the site, this is an active licence 773m north-west of the site, and relates to abstraction from the Regent's Canal at St John's Wood (App.D, Section 6.4).
- There are no flood defences, no areas benefitting from flood defences and no flood storage areas within 250m of the site (App.D, Sections 7.4, 7.5 & 7.6).

5.5 Three major studies of flooding in the borough have been conducted in recent years:

- **Preliminary Flood Risk Assessment (PFRA):** 'Preliminary Flood Risk Assessment for City of Westminster' (Halcrow, Final Report, version 0.5, June 2011). This is part of the wider Drain London project, undertaken in compliance with the Flood Risk Regulations (2009) which implemented the EU Floods Directive in the UK. Such PFRAs are high level screening exercises which compile information on past and future local flood risk; local flood risk excludes flooding from the main rivers, the sea, large reservoirs and burst water mains.
- **Local Flood Risk Management Strategy 2017-2022 (LFRMS):** (City of Westminster, November 2017). This document outlines the general approach to managing local flood risk across the borough, consistent with the Flood and Water Management Act 2010 and the Flood Risk Regulations 2009.
- **Draft Strategic Flood Risk Assessment (SFRA):** (City of Westminster, June 2019). This has already superseded the 2010 SFRA, even though it is still a draft document.
    - The SFRA has been prepared in compliance with the National Planning Policy Framework (NPPF) by the Westminster City Council as a Lead Local Flood Authority, in consultation with the Environment Agency, Thames Water, the Canal and River Trust and Transport for London.
    - The SFRA "*seeks to form a holistic understanding of flood risk, factoring in the impact of climate change, and introduces a risk based management approach to flooding. This entails the assessment of all forms of flooding and associated risk, and to set out appropriate management measures*".
    - The SFRA is not in itself a policy document, but can be used as a source to inform policy and management. It forms part of the evidence base for informing strategic policies for flooding in Westminster's City Plan.

- This SFRA identifies areas at risk of surface water flooding and drainage issues, including manhole surcharge, and takes account of Westminster City Council's Local Flood Risk Management Strategy and enhanced surface water flood risk modelling (Westminster Initial Assessments, WSP April 2015). It identifies how these risks can be appropriately managed, taking account of location, site opportunities, constraints and geology. The risk of reservoir failure in Westminster is also considered in this SFRA.
- This SFRA shows that land available outside the flood risk areas is insufficient to accommodate all necessary development: development within areas potentially at risk of some form of flooding is therefore required. As such and to manage any potential flood risk for development the SFRA provides the information required for the application of the 'Exception Test' where appropriate. It is also acknowledged that the "*complexity of development in Westminster and the difficulty in meeting housing requirements makes a sequential approach to the location of highly vulnerable and more vulnerable uses outside of areas of surface water flood risk 'hotspots' difficult to achieve*" (paragraph 5.3.5).

Surface water flood risk was also considered in the City of Westminster's Supplementary Planning Document (SPD) '**Basement Development in Westminster**' (October 2014).

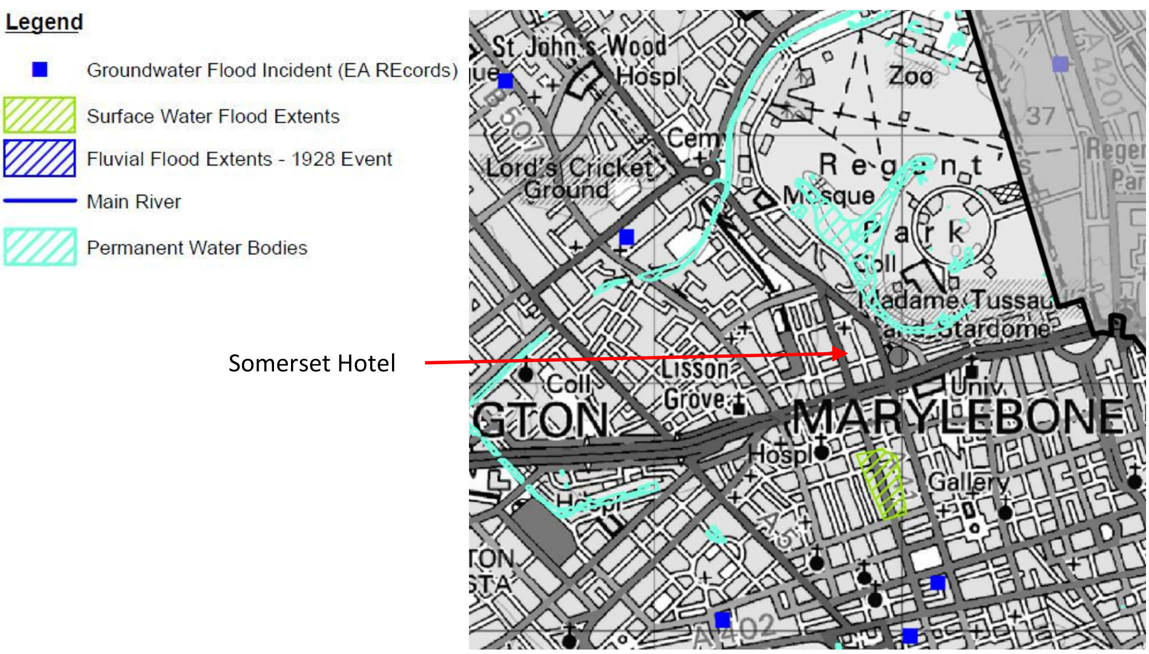

Figure 5: Extract from Map 4.1 in the PFRA (Halcrow 2011): 'Summary Map of Past Floods'.

5.6 The extent of past fluvial and surface water flooding within Westminster has been mapped in both Figure 1.1 and Map 4.1 of the PFRA (2011). Both show that Dorset Square does not fall within one of the areas of Westminster previously affected by **local** flooding. An extract from Map 4.1 is presented in Figure 5 above.

5.7 Figure 5 in the 'Basement Development in Westminster SPD' (October 2014) shows that Dorset Square does **not** fall within any of the 15 Surface Water Flood Risk Hotspots identified therein, the closest of which is 'Marylebone & Mayfair' (No.7), located approximately 190m to the south-east of the Somerset Hotel at its closet point. The same Flood Risk Hotspots appear in Figure 3.9 of Westminster's 2019 SFRA, but without any numbering or names.

5.8 The flood modelling included in the PRFA is of low resolution and also suffers from an indexation error between the flood model and the base map. It has been superseded by Westminster's 2019 SFRA and the latest Environment Agency flood model, so is not considered further.

5.9 The 'enhanced' flood modelling (by WSP) presented in Westminster's 2019 SFRA included surface water flood maps for a 1% annual probability storm plus 20% and 40% allowances for increase in peak rainfall intensity. Figure 6 presents an extract from the most onerous, 40% climate change allowance, model which appears to show that the depth of flooding in the Somerset Hotel site might be in the range from less than 75mm to 300mm. The low resolution of the flood map makes this a very approximate interpretation.

Figure 6: Extract from Figure 3.11 in Westminster's 2019 SFRA:
Flood model for 1% annual probability of surface water flooding, plus 40% climate change allowance.
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5.10 The Environment Agency (EA) published a new map of 'Flood Risk from Surface Water' in January 2014, and a more detailed version has since become available on the Government's 'Long Term Flood Risk Information' website, an extract from which is presented in Figure 7 below. This map identifies four levels of risk (high, medium, low and very low), and it appears to be based primarily on topographic levels, flood depths and flow paths. The EA's definitions of these risk categories are:

'Very low' risk: Each year, these areas have a chance of flooding of less than 1 in 1000 (0.1%).

'Low' risk: Each year, these areas have a chance of flooding of between 1 in 1000 (0.1%) and 1 in 100 (1%)

'Medium' risk: Each year, these areas have a chance of flooding of between 1 in 100 (1%) and 1 in 30 (3.3%).

'High' risk: Each year, these areas have a chance of flooding of greater than 1 in 30 (3.3%).

Figure 7: "Risk of Flooding from Surface Water": Enlarged extract from Environment Agency's website.

5.11 The EA's modelling is much clearer than that in the 2019 SFRA, but is still at a relatively low resolution. It shows an area at a 'Low' risk of flooding from surface water covering the whole of the Somerset Hotel site, which is more plausible than the variable depth of flooding predicted by WSP's modelling for the SFRA. The properties to the rear on

Chagford Street are also within this area, as is the adjoining No.5 Dorset Square to the south, and at least the southern part of No.8 Dorset Square to the north. Slightly further to the north, a localised area of 'Medium' and 'High' risk of flooding from surface water is shown opposite the junction between Gloucester Place and Huntsworth Mews, which suggests that some of the properties in that area may have deeper basements. The 'Low' risk area which includes the Somerset Hotel overlaps the adjacent roadway, which probably relates to the front vaults. The adjacent Dorset Square carriageway appears to be a 'Very Low' flood risk area of flooding from surface water, which is the national background level of flood risk (see Figure 7)

5.12 Table 3.1 'Sewer Flooding Incidents' in the 2019 SFRA shows that there have been no external sewer flooding events in the past 20 years in the NW1 postcode area. There are 5 records of internal sewer flooding in the past 10 years in this part of Westminster (with one property flooded at least twice) and 76 records in the past 20 years, most of which were probably during the 2007 flood event. If required, a site-specific 'Sewer Flooding History Enquiry' report could be obtained from Thames Water Utilities Ltd (TWU) to confirm whether there have been any incidents at this address.

6. HYDROGEOLOGICAL SETTING (GROUNDWATER)

6.1 The Lynch Hill Gravel Member which underlies the site is classified by the Environment Agency as a superficial 'Secondary A Aquifer – Permeable Layers' (part of the unconfined 'Upper Aquifer', see paragraph 6.3 below); while both the Langley Silt Member (see Figure 3) and the London Clay Formation are classified by the Environment Agency as 'Unproductive Strata'. Figure 8 shows the extent of the superficial Secondary A Aquifers in the vicinity of the site of current interest.

Figure 8: Extract from map of superficial aquifer designations as classified by the Environment Agency (centred on Somerset Hotel).

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

6.3 The Secondary Aquifers within the superficial River Terrace Deposits are collectively known as the 'Upper Aquifer'. The Upper Aquifer is typically unconfined, and generally occurs in the lower to middle part of the River Terrace Deposits (in this case the Lynch Hill Gravel Member). It is possible that multiple areas of perched groundwater may be present above the main groundwater table in the Upper Aquifer.

6.4 While the London Clay Formation is classified as an 'Unproductive Stratum', it can still be water-bearing. The water pressures within the clay in the depths of current interest are likely to be hydrostatic, which means they increase linearly with depth, except where they are modified by tree root activity or the influence of man-made changes such as utility trenches (which can act either as land drains or as sources of water and high groundwater pressures) or tunnels. Any silt or sand partings, laminations or thicker beds are likely to contain free groundwater and where these are laterally continuous they can give rise to moderate water entries into excavations. In most cases there will be only very limited or no natural flow in these silt/sand horizons.

6.5 Perched groundwater would typically be expected in any Made Ground, in at least the winter and early spring seasons. Variations in groundwater levels and pressures will occur in response to seasonal climatic changes and with other man-induced influences.

6.6 The groundwater catchment areas upslope of the Somerset Hotel are likely to differ for each of the main stratigraphic units:

- Made Ground: The catchment for any perched groundwater in the Made Ground is probably limited to the nearby areas of Made Ground where surface water infiltration can occur, except where the trenches for drains and other services provide greater interconnection.
- River Terrace Deposits (Lynch Hill Gravel Member): The catchment for the Upper Aquifer within the River Terrace Deposits will comprise recharge from both the overlying soils in the vicinity of the site, though this may be limited by the clays in the upper part of the sequence (see Section 7) and a wider subterranean area. The extent of the latter could be substantial due to the high lateral permeabilities of the soil types concerned.
- London Clay Formation: The catchment for the underlying London Clay will comprise predominantly recharge from the overlying soils in the vicinity of the site (mainly to the north of the site), plus potentially a wider area determined by the lateral extent of any interconnected silt/sand horizons.

6.7 Under the old groundwater vulnerability classification scheme, which now applies only to superficial soils, the site is within an area which is classed as 'Minor Aquifer High' groundwater vulnerability.

6.8 Other hydrogeological data obtained from the Groundsure EnviroInsight report (see Appendix D) include:

- No Source Protection Zones (SPZ's) within 500m of the site (App. D, Sections 6.6 & 6.7).
- The nearest active groundwater abstraction licence concerns two boreholes at Dorset House, 95m to the south-east of Somerset Hotel for "*Drinking, Cooking, Sanitary, Washing (Small Garden) – Household*" (App.D, Section 6.3). The next nearest active licence is at Abbey Lodge, 611m to the north-west of Somerset Hotel which, in common with the remaining 21 active groundwater abstraction licences within 2000m of the site (many of which are multiple records for the same site), are considered to be irrelevant to the proposed sub-basement.
- The nearest potable water abstraction licence is also at 95m to the south-east at Dorset House (App.D, Section 6.5) and is considered relevant to the proposed sub-basement (see Section 8.2). There are a further seven records for active potable water abstraction licences within 2000m of the site, none of which are considered relevant to the proposed sub-basement.

- The BGS has classified the area within 50m of the site as having a '**Potential at Surface**' susceptibility to groundwater flooding, at a 'Moderate' confidence level (App.D, Sections 7.7 & 7.8). This classification relates to groundwater in the superficial deposits (i.e. the Upper Aquifer). Halcrow's mapping also places this site within an area of 'increased potential for elevated groundwater' from the superficial deposits as shown in Map 5.3 of the PFRA (2011) and was reproduced as Figure 3.12 of the 2019 SFRA. The implications of this classification for the proposed basement are considered in Section 8.3 below.

6.9 Groundwater (and other) flood incidents are included on Map 4.1 in the PFRA (2011), which presents a summary of past floods. An extract from that map is presented in Figure 5 (see Section 5). The nearest groundwater flood incidents were approximately 990m to the south, and 1.0km to the north-west of the site.

6.10 Details of the groundwater regime found by the site-specific ground investigation in April/May are presented in Section 7. In the borehole drilled just to the north-east of Somerset Hotel at 42 & 43 Chagford Street, a slight seepage was recorded at the boundary between the Made Ground and underlying SAND of the Lynch Hill Gravel Member.

6.11 Barton & Myers' Lost Rivers of London (2016), states that just to the south of Somerset Hotel, the Tyburn (see Section 5.1) "…*goes beneath the mansion flats in Brickenhall and York Streets. The blocks of apartments in this area have pumps to prevent flooding by groundwater unable now to flow into the river*".

7. GROUND INVESTIGATION

7.1 The ground investigation sitework was carried out on 30th April and 1st May 2019, and comprised one 'windowless' sampler borehole (BH1) drilled within the front lightwell, and six hand-dug trial pits (TPs 1-6). Logging of the recovered continuous 'core' samples from the 'windowless' sampler and the trial pit excavations were undertaken on site by Gabriel GeoConsulting Ltd (Alexander Goodsell). The factual findings from the investigation are presented in Appendix F, including an exploratory hole location plan (Figure GI-01), borehole and probe logs (GI-02 & 03), trial pit logs (GI-04 to GI-14) and laboratory test results.

7.2 Trial pits TP1 to TP6 were dug in order to investigate the foundations to No's 6 & 7 Dorset Square, and the soils beneath the footings, at their respective locations. A brief summary of the findings from these trial pits is summarised below. For further details, reference should be made to the logs in Appendix F.

TP No.	Section	Location and Footing Depth (m bgl)	Geology
TP1:	A	Rear/Party wall with No.47 Chagford Street = 0.28m	0.08m of concrete and flagstones; over 0.20-0.67m MADE GROUND (*variably clayey SAND*) over LYNCH HILL GRAVEL MEMBER (slightly sandy CLAY, over *variably clayey SAND*) to base of pit.
	B	7/8 Dorset Square Party wall = 0.28m (Pier below corner = 1.00)	
TP2:	A	5/6 Dorset Square Party wall = 0.37m	0.13m of wooden floor boards, bearers, concrete and flagstones; over 0.24m MADE GROUND (*ash and charcoal*) over LYNCH HILL GRAVEL MEMBER (locally *clayey SAND*) to base of pit.
	B	Rear/Party wall with No.48 Chagford Street = 0.37m (Pier below corner = 1.03m)	
TP3:	A	Main rear wall of No.6 = 0.44m	0.18m of wooden floor boards, bearers, ceramic tiles and concrete; over MADE GROUND (locally clayey, gravelly SAND) to base of pit.
	B	6/7 Dorset Square (party) wall = 0.44m	
TP4:	A	5/6 Dorset Square Party wall = 0.25m, with wall overhanging footing by 0.15m.	0.10m of floor tiles and concrete (2 layers); over 0.15m MADE GROUND (*slightly clayey, slightly gravelly SAND*) over LYNCH HILL GRAVEL MEMBER (*SAND*) to base of pit.
	B	Former lightwell wall (?) = 0.25m	

Continued…

TP5	A	Front wall of store room = 0.23m	0.09m of floor tiles and concrete; over 0.14-0.16m MADE GROUND (*CLAY, intermixed with odorous gravelly SAND*) over LYNCH HILL GRAVEL MEMBER (slightly clayey *SAND*) to base of pit.
	B	7/8 Party wall from store below entrance path: Footing depth = 0.23m	
TP6	N/A	Main front wall No.7: Footing depth = 0.35m.	0.10m of concrete; over 0.15-0.25m MADE GROUND (*slightly clayey, slightly gravelly SAND*) over LYNCH HILL GRAVEL MEMBER (slightly clayey *SAND*) to base of pit.

7.3 The geological sequence as found in BH1 (Figure GI-02 in Appendix F) may be summarised as below. The SANDS and GRAVELS of the Lynch Hill Gravel Member proved too dense to drill through, thus the borehole was aborted at 2.70m bgl. A dynamic probe test was able to be carried out below 2.70m, and was continued to a final depth of 10.0m bgl, the results of which are presented in Figure GI-03 in Appendix F and summarised in paragraph 7.5.

- Surfacing: Concrete with 'chicken wire' reinforcement, 0.20m thick at BH location.
- Made Ground: Comprised "*Slightly moist, orangey brown mottled reddish brown and greyish brown, locally clayey, slightly gravelly to gravelly SAND*". The gravel was found to be fine to coarse, and consisted of brick, flint, concrete, mortar and other assorted artificial fragments. A single decaying root fragment was also found within the Made Ground; below 0.40m the artificial fragments became less frequent.
- Lynch Hill Gravel Member (?): Recorded from the base of the Made Ground (0.60m) to the base of the borehole at 2.70m bgl, this River Terrace Deposit predominantly comprised multi-coloured, locally clayey SANDS, with occasional horizons of stiff, very sandy CLAY.
  Theses sands became moist below 1.50m bgl and between 1.80m and 2.00m bgl, were interbedded with thin laminae (<5mm) of very sandy CLAY to silty CLAY. Below 2.10m, the sand became gravelly, and from 2.40m to the base of the borehole, this River Terrace Deposit comprised sandy GRAVEL. The gravel was found to be fine to coarse, angular to rounded flint. This is taken to be the Lynch Hill Gravel Member (LHGM), as mapped by the BGS.

7.4 Standard Penetration Tests (SPTs) were carried out at 1.00m and 2.00m bgl in BH1. The resulting 'N' values (blows to drive the 300mm test length) indicated these sands to be Medium Dense, N = 22, becoming Very Dense, N = 51 (Figure GI-02).

7.5 Below 2.70m bgl, BH1 was extended by DPSH dynamic probing to 10.00m bgl. The resulting 'N_{10}' values (blows to drive each 100mm test length) are recorded on the dynamic probe log (Figure GI-03), which is also presented in Appendix F, and have also been plotted as a profile against depth in Figure 11 below. The dynamic probe results gave 'N_{10}' values ranging from 8 to 38 between 2.70 and 4.60m bgl, before decreasing to an 'N_{10}' value of 2 (the lowest value recorded) at 5.2-5.3m bgl, which is likely to represent the top of the London Clay. Below 5.30m bgl, the dynamic probe results show a consistent increasing trend with depth, up to a final 'N_{10}' value of 8 at 9.9-10.0m bgl.

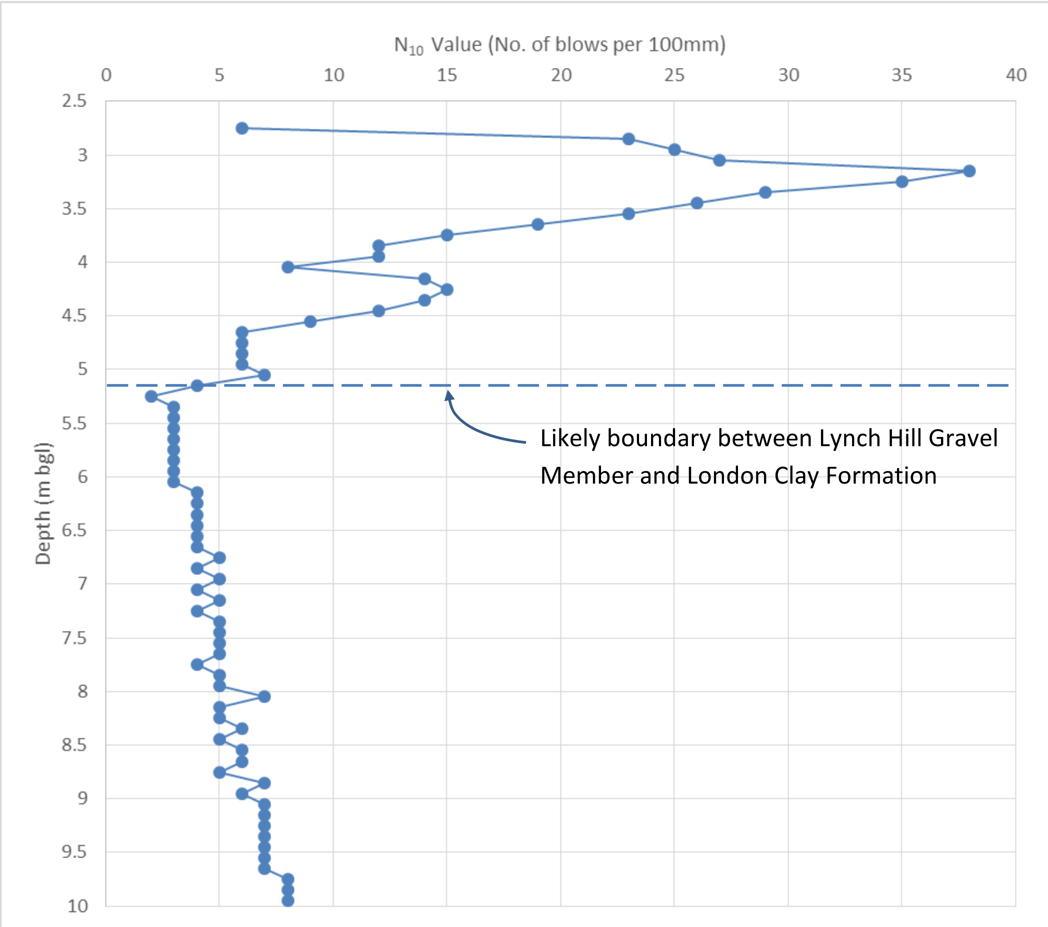

Figure 9: Dynamic probe N_{10} values with depth.

7.6 No roots were observed in any of the exploratory holes, with the exception of a single fragment of decaying root, which was found in the Made Ground in BH1.

7.7 No groundwater entries were recorded during the drilling of BH1, however the River Terrace Deposits were found to be moist below 1.50m bgl, and wet between 2.00m and 2.10m bgl. A standpipe was installed to a depth of 2.70m on completion, to allow the groundwater level within the borehole to be monitored, however the borehole was found to be dry during both of the subsequent groundwater monitoring visits on 15th May & 4th June 2019.

Laboratory Testing:

7.8 Laboratory tests on samples recovered from the borehole and trial pits were carried out by Geolabs. The testing comprised particle size distribution analyses (grading), and chemical testing to assess the potential for acid or sulphate attack on buried concrete. The results are presented in Appendix F.

7.9 The particle size distribution analyses (grading) tests were carried out on three samples recovered from BH1 between 1.10m and 2.60m bgl, one sample recovered from TP1 at 0.95-1.05m, and one sample recovered from TP4 at 0.40-0.50m bgl. These samples were found to consist of:

- BH1 1.10-1.30m: Clayey/silty, fine to medium SAND
- BH1 2.10-2.30m: Clayey/silty, fine to medium SAND & medium to coarse GRAVEL (gap-graded, which means that this deposit is vulnerable to internal erosion)
- BH1 2.40-2.60m: Slightly clayey/silty, very sandy, fine to coarse GRAVEL
- TP1 0.95-1.05m: Silty, clayey, fine to medium SAND
- TP4 0.40-0.50m: Slightly clayey/silty, slightly gravelly, fine to coarse SAND

7.10 The chemical tests were undertaken on eight samples in order to assess the potential for acid or sulphate attack on buried concrete, in accordance with BRE Special Digest 1 (2005). Three samples were recovered from BH1 between 0.60-2.60m, and from TP1 at 0.30-0.40m, TP2 at 0.15-0.25m, TP3 at 0.40-0.50m, TP5 at 0.20-0.25m and TP6 at 0.40-0.50m, so included Made Ground and Lynch Hill Gravel Member:

pH value: 7.4 – 9.5

Water-soluble sulphate: <10 – 1600mg/L

8. BASEMENT IMPACT ASSESSMENTS

8.1 Conceptual Ground Model

8.1.1 The desk study evidence together with the findings of the site-specific ground investigation at Somerset Hotel suggest a conceptual ground model, including hydrogeological model, for the site characterised by:

- Made Ground: Recorded in all of the exploratory holes; the Made Ground varied in thickness across the site, ranging from 0.23-0.37m in TP's 2, 4, 5 & 6, to 0.75m in TP1 (and the maximum thickness was not proved in TP3). The Made Ground generally consisted of variably clayey, variably gravelly SAND, with fragments of artificial material including brick, clinker, mortar, concrete and rare ash & charcoal, though other materials, as well as other soil types and greater thicknesses/depths are also likely to be present on site, owing to the inherent variability of Made Ground.
- Lynch Hill Gravel Member: Recorded from the base of the Made Ground to the base of BH1 at 2.70m bgl, the upper parts of this River Terrace Deposit generally comprised medium dense, multi-coloured, locally clayey SANDS, with occasional horizons of firm or stiff, very sandy CLAY. The Gravel content of these deposits was found to increase with depth, with the borehole terminating in very dense, very sandy GRAVEL. This was confirmed by the results of the grading analyses which also showed an increase in gravel content with depth (see paragraph 7.9). Below 3.2m the relative density of the sands/gravels reduced to dense then medium dense.
- London Clay Formation: Though not encountered in the borehole, stiff to very stiff, fissured, mid-grey, silty CLAYS of the London Clay Formation are expected beneath the Lynch Hill Gravel Member. Based on the results of the dynamic probe test, the top of the London Clay is likely to be at approximately around **5.2m bgl** (around **21.1m AOD**), though may vary across the site. This level for the top of the London Clay is also within the relatively consistent range recorded by other boreholes located nearby (19.78-21.80m AOD, as presented in Table 1, Section 4). The fissures in these clays may be sheared and slickensided which reduces their strength and they will undergo heave movements in response to unloading by the basement excavations. Elevated sulphate levels from selenite (a form of gypsum) are common in London Clay, and are aggressive to buried concrete.

  A thin weathered horizon of softer CLAY is often found at the top of the London Clay Formation, at the boundary with the overlying River Terrace Deposits, as recorded in all of the nearby boreholes presented in Table 1.
- Hydrogeology:
  - The moist and wet sands recorded in BH1 indicate that perched groundwater is likely to be present locally at shallow depth within the upper part of the River Terrace Deposits and possibly also in the Made Ground, supported on horizons of lower permeability; such perched

groundwater may be more widespread during the wetter winter and spring seasons.

- The borehole at Glentworth Street (TQ28SE/453) recorded a seepage at 2.10m bgl and noted that water was sealed out once the casing reached the London Clay at 6.40m bgl. The other boreholes in Table 1 only recorded a limited depth of groundwater in the lower part of the River Terrace Deposits which form the Upper Aquifer. Thus, despite the 2.70m deep standpipe remained dry during the brief monitoring period, it remains possible that a significant depth of water may be present when the sub-basement is constructed. Additional readings should be taken from the standpipe next winter (during detailed design and/or just before the start of construction, as applicable) and groundwater in the Upper Aquifer must be allowed for in the design analyses (see Section 8.3).
- Groundwater pressures in the London Clay are expected to be essentially hydrostatic within the depth of current interest. Groundwater flow through these clays is likely to be minimal, in practice being limited to seepage through any silt/sand partings which are sufficiently interconnected.

- <u>Other influences on the Groundwater regime:</u>
  - 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.

8.1.2 The hydrogeological regime outlined above will be affected by long-term climatic variations as well as seasonal fluctuations, all of which must be taken into account when selecting a design water level for the permanent works. No multi-seasonal monitoring data are available, so a conservative approach will be needed, in accordance with current geotechnical design standards which require use of 'worst credible' groundwater levels/pressures. See paragraph 8.3.13 for the recommended provisional design groundwater level.

8.1.3 A services search has been undertaken. The responses did not identify any railway tunnels or plant owned by utility companies passing below the site. These searches will not identify any private services or lateral drains (which only became adopted when the law changed in 2011).

8.2 Hydrology - Surface Water and Flooding

8.2.1 The evidence presented in Section 5 has shown that:

- the former course of the 'lost' River Tyburn passed close to the east of the Somerset Hotel site, between Chagford and Glentworth Streets;
- the nearest surface water feature is a section of the Boating Lake in Regent's Park, located 232m to the north-east of the site;
- there are **no** records of past surface water flooding at Dorset Square in the documents reviewed;
- the site lies within the Environment Agency's **Flood Zone 1** which means that it is considered to be at **'Low'** risk of flooding from rivers or the sea (fluvial or tidal) or, when other factors are considered, the EA's Long-term Flood Risk website gives a **'Very Low'** risk of such flooding;
- Somerset Hotel does **not** fall within an area at risk of reservoir flooding;
- Dorset Square does **not** fall within any of the Surface Water Flood Risk Hotspots identified in the City of Westminster's Supplementary Planning Document (SPD) 'Basement Development in Westminster' (October 2014); the same hotspot map is reproduced with less information in Westminster's 2019 SFRA;
- flood modelling by WSP which is presented in Westminster's 2019 Draft SFRA indicates a potential depth of flooding in the Somerset Hotel's site which varies from <75mm up to 300mm (from rear to front, though the resolution of the mapping is very poor, see Figure 6 herein); this is considered implausible given the uniform level of the existing basement;
- flood modelling by the Environment Agency gives a **'Low'** risk of surface water flooding for the whole site including the front vaults and most of the immediately adjoining sites, which is considered to be more plausible, while the flood risk on the adjacent carriageway is **'Very Low'** which is the national background level of risk;
- for postcode area 'NW1', Table 3.1 in Westminster's 2019 SFRA records no external sewer flooding incidents in the past 20 years, 5 internal sewer flooding events in the past 10 years and 76 internal sewer flooding events in the past 20 years most of which were probably in the 2007 flood event.

<u>Change in Paved Surfacing, Surface Water Run-off & SuDS:</u>

8.2.2 The Somerset Hotel building, including the paved open front lightwells, occupies the whole of its site so there are no areas of soft landscaping where infiltration of surface water into the ground can occur.

8.2.3 The proposed sub-basement will underlie the full footprint of the existing Somerset Hotel building, excluding the front vaults, so it will not result in any change to the impermeable surface area. Despite this Westminster's Basement Development SPD expects compliance with Policy 5.13 'Sustainable Drainage' of the London Plan which identifies a 'drainage hierarchy' to minimise surface water run-off. The only SuDS items in the hierarchy which could be considered for implementation at the Somerset Hotel are:

"1 store rainwater for later use" or

"4 attenuate rainwater by storing in tanks or sealed water features for gradual release".

Alternatively, as covered in the supporting text (paragraph 5.57) to Policy 5.13, installation of a roof terrace including a green roof above the existing rear basement extension is an alternative form of SuDS which would comply with Policy 5.11 of the London Plan.

Surface Water (Pluvial) Flooding and Flood Resistance Measures:

8.2.4 The 'Low' risk of surface water flooding of the Somerset Hotel's site and the adjoining properties, as predicted by the Environment Agency's modelling (on the EA's website since January 2014), can be reduced further by incorporation of appropriate flood resistance measures in the proposed scheme as mitigation.

8.2.5 The Hotel's ground floor is raised above the adjacent carriageway by approximately 0.6m (0.61m at No.5) so is already protected. Thus, the only potential access points for surface water to the proposed sub-basement will be indirectly via the front lightwells, and directly via the internal lightwell, so flood resistance measures for the property in relation to surface water should include:

1. Ensuring that the existing upstand around the front lightwells is maintained and, if possible, raising the paving slab(s) slightly at the top of the steps into No.6's front lightwell in order to ensure that surface water on the pavement drains to the highway rather than into the lightwell;
2. Installing raised thresholds to the external doors which give access to the existing basement from the front lightwells. The lightwell thresholds must be of sufficient height to protect the basement from the maximum depth of flooding in the lightwell (predicted by appropriate calculations) or interception storage should be provided to ensure that the basement and the proposed sub-basement can never be flooded from the front lightwells.
3. Provision of a similar raised threshold at the access from the internal lightwell to the sub-basement, or suitably sized interception storage.
4. The existing basement and proposed sub-basement must also be protected from flooding by effluent (which includes surface water in the combined drainage system) when the adjacent sewers are running under 'surcharge', as described in more detail in the Sewer Flooding section below. It is likely to be beneficial to link the surface water storage volume in the lightwells with the temporary interception storage recommended in paragraph 8.2.9 (if required).

Sewer Flooding:

8.2.6 Drainage systems are designed to operate under 'surcharge' at times of peak rainfall, which means that the level of effluent in the sewers and associated manholes 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.

8.2.7 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 control and is largely outside the City Council's control too. Thus, given that Westminster's 2019 SFRA had only 5 recorded incidents of sewer flooding affecting properties in the NW1 postcode, the probability of future sewer flooding affecting Somerset Hotel is considered to be Low provided that the sewer system is well maintained and the flood resistance measures described below are implemented.

8.2.8 Non-return valves should therefore be fitted on the outfall from all drains serving the basement and the proposed sub-basement in order to ensure that water from the sewer system cannot enter the property when the sewers are operating under surcharge. Westminster's Basements SPD indicates that non-return valves represent the minimum acceptable form of resistance to sewer flooding; addition of a pumped above ground loop system on each outfall pipe should also be considered.

8.2.9 Once non-return valve(s) have been installed no surface water would be able to enter the sewer for most/all of the time that the flow in the sewer is sufficient to close the valve(s). The basement and sub-basement could then be vulnerable to flooding while the surcharged flows continue. Sufficient temporary interception storage should therefore be provided in order to hold temporarily the predicted maximum volume of discharge from all sources (roof water, the three lightwells and foul water as relevant) connected to the protected outfall for the duration of the surcharge flows. The same interception storage would be required with the above-ground loop system unless the design can demonstrate that the increased hydraulic 'head' created is sufficient to accommodate the predicted flows. This temporary interception storage will require formal design to ensure satisfactory performance.

8.3 Hydrogeology (Groundwater) – Permanent Works

8.3.1 The 2.70m deep standpipe installed within the Upper Aquifer in BH1 was found to be dry during the limited groundwater monitoring period though, as the standpipe could not be installed to the base of the River Terrace Deposits, groundwater is still likely to be present within the Upper Aquifer and must be allowed for in the design analyses.

8.3.2 Close to the north-east of the Somerset Hotel, groundwater seepage was recorded at 0.70m bgl (around 22.60m AOD) at No's 42 & 43 Chagford Street, at the boundary between the Made Ground and the underlying River Terrace Deposits, and was only sealed out once the drill casing reached the London Clay. Further to the east, a seepage was recorded in BGS BH TQ28SE/453 at 2.10m (around 25.38m AOD). To the south and south-west, groundwater seepages or standing levels were recorded near the base of the river terrace deposits in BGS BH's TQ28SE/322 & 452, at 7.32m and 7.60m bgl respectively (around 20.23m and 21.17m AOD).

8.3.3 The broad trend of natural groundwater flow in the Upper Aquifer, where unobstructed and not altered by artificial means, is expected to be broadly along the former course of the 'lost' river Tyburn, given its proximity, so in a south-south-easterly direction towards the Thames.

<u>Existing Basements:</u>

8.3.4 All the properties on the east side of Dorset Square have original lower ground floors/basements, including the Somerset Hotel, and the adjoining No's 5 & 8 Dorset Square. These are all understood to be at approximately the same level as the existing basement to Somerset Hotel at around 26.25m AOD (based on levels for No.5), since this part of Dorset Square falls only very gently southwards at around 0.15 to 0.30° (see paragraph 2.7).

8.3.5 Based on drawings gleaned from Westminster City Council's website, No's 47-50 Chagford Street located to the rear of Somerset Hotel do not have existing basements, and the ground floor levels of these properties are set approximately 0.90m above the basement level of Somerset Hotel, at around 26.4m AOD, as scaled from Famedean Limited's 'Section B-B as proposed' for No's 49-50 Chagford Street (Drg No.1086-08). The search of planning applications on Westminster City Council's website did however reveal that a single-storey basement is present beneath No.46 Chagford Street, which adjoins the north-east corner of Somerset Hotel; permeation grouting was specified for 0.50m alongside all the external walls of that basement, though there was no evidence of that grout inTP1. There is also a two-storey basement beneath No's 42-43 Chagford Street.

8.3.6 The single-storey basement beneath No.46 Chagford Street may form a small obstruction to groundwater flow in this area, however currently there is ample space for groundwater to flow both around, and beneath this basement. The two-storey basement beneath No's 42-43 Chagford Street is likely to form a more significant obstruction in that area, as it extends into the London Clay beneath the Upper Aquifer, thus groundwater can no longer flow beneath the property, however there is still ample space for groundwater to flow around this basement.

<u>Other Proposed Basements:</u>

8.3.7 No planning consents were identified by our searches for other new basements beneath any properties which are sufficiently close to Somerset Hotel to create a cumulative impact on groundwater seepage/flows.

<u>Proposed Basement at Somerset Hotel:</u>

8.3.8 The proposed basement will have a finished floor level (FFL) of 4.0m below the FFL of the existing basement. With allowance for 150mm of insulation, cavity drainage and floor finishes in addition to the 400mm thick base slab the proposed sub-basement will be founded at around **4.55m** below the existing basement's FFL, or approximately **21.7m AOD**, across the full footprint of the basement. This formation level will be:

- Approximately 0.6m above the expected top of the London Clay (in DP1);
- Approximately 3.0m below the highest record of moist/wet sands in BH1.

8.3.9 Based on the above founding depth, the proposed sub-basement is likely to create an obstruction to any seepage/flows in the Upper Aquifer, though groundwater will still be able to seep/flow around and below the sub-basement and the adjoining No.46 Chagford Street basement. Thus, at worst, this basement is expected to create a slight local increase in groundwater levels on the upstream side of the basement.

8.3.10 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 <u>could</u> enable groundwater flooding to occur and where groundwater <u>may</u> 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.

8.3.11 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."*

8.3.12 The BGS have also confirmed to the author (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.

8.3.13 Current geotechnical design standards require use of a 'worst credible' approach to selection of groundwater pressures. Both the perched groundwater in the middle/upper part of the River Terrace Deposits and the main Upper Aquifer in the lower part of the River Terrace Deposits will need to be allowed for in the design. The BGS classification of the area as 'Potential at Surface' for the risk of groundwater emergence need not be taken as justification for taking groundwater levels at ground level, for the reasons set out in 8.3.11 and 8.3.12 above. Some fluctuation in groundwater levels will occur, seasonally and in the longer-term, so use of a provisional design groundwater level at 1.0m below the ground level at BH1 (in the front lightwell) is recommended for the whole basement, subject to review based on the findings of the additional groundwater monitoring.

8.3.14 The basement structure should be designed to resist buoyant uplift pressure that would be generated by the 'worst credible' groundwater levels. For the provisional design groundwater level suggested above buoyant uplift pressures of up to 36 kPa (un-factored) would have to be accommodated.

8.4 Hydrogeology (Groundwater) – Temporary Works

8.4.1 Groundwater entries/seepages into the excavations for the basement should be expected. The coarsest of the three particle size distribution analyses (gradings) carried out on samples of the River Terrace Deposits recovered from BH1 (at 2.20-2.50m bgl) was found to consist of a slightly clayey/silty, very sandy, fine to coarse GRAVEL, so is likely to be of moderate permeability. If the coarsening-downwards sequence continues then high permeability gravels may be present in the lower part of the sequence, as is found elsewhere. Thus, on current evidence:

i. it is possible that pumping from sumps will be required to remove local entries of perched groundwater at shallow depths;

ii. it is likely that either sump pumping or well-pointing will be required to control water entries from the Upper Aquifer in the lower part of the river Terrace Deposits;

iii. if high permeability, water-bearing gravels are encountered then localised grouting around the perimeter of the basement using a low strength grout may be required, followed by pumping out of the 'trapped' water within the basement's footprint

It is recommended that, at the start of construction, a fully-shored trial excavation down to the base of the River Terrace Deposits should be undertaken near the centre of the basement in order to assess the rate of inflow and check the adequacy and efficiency of the chosen method of groundwater control. An appropriate discharge location must be identified for the groundwater removed by the dewatering.

8.4.2 All groundwater control measures should be supervised by an appropriately competent person. A careful watch must 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 area affected may need to be backfilled temporarily while advice is sought from a suitably experienced and competent ground engineer or dewatering specialist.

8.4.3 If the possible variations in level of the London Clay Formation are sufficient to bring the top surface up to or above the level at which the underpins and the basement slab will bear (approximately 21.7m AOD) then those clays must be protected from water because, once unloaded by the excavations, these over-consolidated clays would readily absorb any available water which would lead to softening and loss of strength. Thus, any clays exposed in the formation should be protected from all sources of water, with suitable channelling to sumps for any groundwater seeping into the excavations, and blinded with concrete immediately following excavation to final level and inspection.

8.5 Waterproofing

8.5.1 The proposed basement will need to be fully waterproofed in order to provide adequate long-term control of moisture ingress from the groundwater and infiltrating surface water. 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. Good workmanship will be crucial to the success of whatever system is selected.

8.5.2 The National House Building Council published new guidance on waterproofing of basements in November 2014 (now NHBC Standards 2019, Chapter 5.4). Compliance would be compulsory if an NHBC warranty is required, otherwise it may provide a useful guide to best practice.

8.5.3 Watertight seals will be required at all construction joints and service penetrations, including where any sumps extend below the basement slab.

8.6 Ground Stability and Bearing Capacity

8.6.1 With overall slope angles of <0.3° around and upslope of the site, the proposed basement excavation raises no concerns in relation to the overall stability of the slope, subject to normal precautions in supporting the ground around the basement.

Basement Retaining Wall Construction:

8.6.2 The section drawings provided by Michael Alexander Consulting Engineers (MACE) in electronic .dwg format indicate that the basement will be constructed using reinforced concrete (RC) underpinning techniques, with 594mm/608mm thick stems (locally 329mm stems) and 400mm thick bases. These underpins should be constructed in panels not exceeding 1.0m in width on a traditional '1-5, hit and miss' sequence. This method involves 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 calculated levels/depths of the underpins are given in paragraph 8.3.8.

8.6.3 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 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 8.6.5 to 8.6.7 below). Any damage/cracks in affected load-bearing walls which have weakened their structural integrity should be fully repaired in accordance with recommendations from the appointed structural engineer before any underpinning is carried out.

8.6.4 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.

8.6.5 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. If clays of The London Clay Formation are encountered within the basement excavations, then influence of fissure/shears must be allowed for, because they can cause seemingly strong, stable excavations to collapse with little or no warning. Thus, in addition to normal monitoring of the stability of the excavations, a suitably competent person should check whether such fissuring is present and, if encountered, should assess what support is appropriate.

8.6.6 The minimum temporary support requirements recommended for the excavations for the proposed underpins at Somerset Hotel, subject to inspection and review as described in 9.6.5 above, are:

- It should be assumed that full face support will be required over the full depth of the excavations required for this basement, in the Made Ground and River Terrace Deposits (and the firm weathered uppermost layer of London Clay, if encountered).
- Temporary support must also be installed to support all the new underpins and must be maintained until the full permanent support has been completed, including allowing time for the concrete to gain adequate strength.

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

8.6.7 If clays are encountered at/beneath formation level, then they must be blinded with concrete immediately after completion of final excavation to grade, for the reasons explained in paragraph 8.4.3. Any unacceptably soft/weak areas must be excavated and replaced with concrete.

8.6.8 Normal good practice in foundation construction requires progressive stepping up between foundations of different depths beneath a single structure. Consideration should therefore be given during Party Wall agreement negotiations to the possible installation of transition underpins beneath the adjoining front, rear and internal transverse walls in No's 5 & 8 Dorset Square, and beneath the rear party wall between No.49 Chagford Street and No.5 Dorset Square, which adjoins the east corner of the proposed basement. Transition underpins are also recommended beneath the adjoining party walls between No's 47/48 and 48/49 Chagford Street, however since No.46 Chagford Street has already been underpins, they may not be required beneath either the 46/47 Chagford Street Party wall, or No.46's rear wall which is party with No.8 Dorset Square.

8.6.9 The construction sequence will be covered in the structural engineer's Structural Methodology Statement.

Geotechnical Design– Retaining Walls:

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

- Earth pressures from the surrounding ground (see paragraph 8.6.11 below);
- Dead and live loads from the superstructure, including loads from the adjoining No's 5 & 8 Dorset Square and No's 47 & 48 Chagford Street which are carried on the party walls;

- Loads from load-bearing walls in the adjoining properties which bear within the active pressure zone and would act on the rear face of the underpins.
- Higher ground levels in the adjoining Chagford Street mews houses.
- Vehicle loads on Dorset Square footway/carriageway and normal surcharge allowances elsewhere;
- Swelling displacements/pressures from the clays which underlie the River Terrace Deposits;
- A provisional design groundwater level at 1.0m below FFL in the existing basement (see paragraph 8.3.10) subject to taking additional groundwater readings during the coming winter;
- Precautions to protect the concrete from sulphate attack (see Section 8.10).

8.6.11 The following geotechnical parameters are applicable to the strata at this site, for use when calculating earth pressures acting on the basement's retaining walls.

Made Ground:	Unit weight, γ_b:	17.0 kN/m^3
	Effective cohesion, c':	0 kPa
	Angle of internal friction, φ':	25°
River Terrace Deposits (sands and gravels):		
	Unit weight, γ_b:	19.0 kN/m^3
	Effective cohesion, c':	0 kPa
	Angle of internal friction, φ':	32°
London Clay Fm (if higher than found in DP1):		
	Unit weight, γ_b:	20.0 kN/m^3
	Effective cohesion, c':	0 kPa
	Angle of internal friction, φ':	22°

Coefficient of earth pressure at rest, k_0: 1.0, after the likely existing higher stresses have been released by the excavations.

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

<u>Geotechnical Design – Bearing Capacity:</u>

8.6.12 For foundations bearing onto sands or gravels, the allowable bearing capacity is usually determined in relation to permissible settlements. The Standard Penetration Tests (SPTs) and the dynamic probe test indicated that the relative density of the River Terrace Deposits ranges from Medium Dense to Very Dense, but the sub-basement will be founded below the Very Dense soils so a characteristic SPT 'N' value of 20 would be reasonable for the lower part of this deposit. The underpin widths shown on MACE's foundation plan vary from 0.99m to 2.295m. For these foundation widths placed at shallow depth (as applies on the internal side of basement underpins), the allowable bearing capacity would be approximately 150kPa and 190kPa respectively for settlements up to 25mm (based on correlation by Peck, Hanson & Thornburn, 1974, see Figure 10 below). However, for a terraced property such as the Somerset Hotel settlements up to 25mm relative to the adjoining buildings would not be acceptable so the allowable net bearing pressure for underpins on granular strata should not exceed 125kPa and should be kept below that if possible in order to minimise differential settlements. In addition, the influence of the underlying clays must be considered as discussed below.

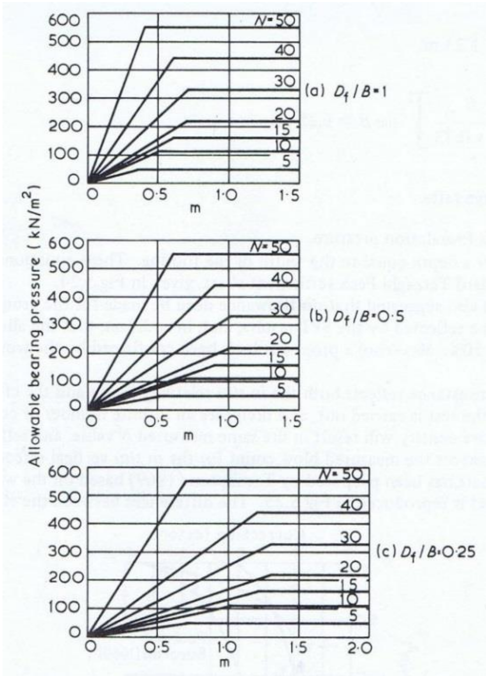

Figure 10: Correlation of allowable bearing pressure to give 25mm settlement to SPT 'N' value, After Peck, Hanson and Thornburn (1974).

8.6.13 The top of the underlying London Clay was indicated by the dynamic probe test to be approximately 0.6m below the basement's founding level at the position of BH1 in the front lightwell. Most of the nearby boreholes indicated that the consistency of the uppermost part of the London Clay is stiff or very stiff but the dynamic probe tests indicated that the undrained shear strength of the upper part of the London Clay below Somerset House is only about 40kPa, which is compatible with the conditions in borehole TQ28SE/453 in Glentworth Street. With allowance for load spreading through 0.6m of granular soils this would give an allowable net bearing capacity of 105kPa at formation level for settlements of up to 25mm. In the order to minimise differential settlements relative to the attached buildings, the following strategy is proposed:

- Restrict net bearing pressures to less than **80kPa** wherever possible;
- Where achieving a bearing pressure of less than 80kPa is not practical, undertake further ground investigation to check the depth and condition of the upper part of the London Clay beneath the underpins concerned and, if necessary, dig out the weaker soils at the top of the London Clay and replace them with concrete, under the supervision of a geotechnical engineer or engineering geologist

8.7 Heave/Settlement Assessment

Sub-Basement Geometry and Stresses:

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

8.7.2 Figure G1 in Appendix G illustrates the layout of the proposed sub-basement based on the 'P4463 sub-basement loading key plan' by Michael Alexander Consulting Engineers (MACE) together with the proposed underpinning plan by MACE in their 'P4463 working cad file 23-08-19'. The latter was modified by GGC in conjunction with MACE in order to reduce the net bearing pressures. The load takedown data for the proposed building were also taken from MACE's 'P4463 sub-basement loading key plan' which is reproduced in Figure G2. Zones 9, 14, 15 and 19-23 (coloured green on Figure G1) all model loads which are superimposed on the other zones to allow for loads from adjoining walls.

8.7.3 The overall dimensions of the sub-basement, as measured from MACE's CAD file, are 15.219m wide by 26.475m long (to the outer faces of the underpins). The sub-basement levels and depths of excavation are given in Section 3 and paragraph 8.3.8 above. The founding depth of the underpins and the raft slab was taken as 4.55m below the existing basement's FFL which gave a gross reduction of vertical stress (unloading) of 86.45kPa.

8.7.4 Table 2 presents the net/gross bearing pressures for four main stages of the stress changes which will result from excavation and construction of the sub-basement (see 8.7.7 below for details of the stages). For Stage 2 of these PDISP analyses, the bearing pressures in Table 2 are gross changes, because it was simpler to identify the unloading from the excavation separately for the whole of the sub-basement rather than modelling individually all the zones between the underpins. For Stages 3 and 4, by when the raft slab proposed by MACE will have been completed, the model was simplified into the three areas for the parts of the sub-basement beneath the front lightwells, the main building and the rear extension. I reality there will be some load-shedding from the central section into adjoining sections which has not been modelled, which makes this model conservative.

Table 2: Bearing pressure changes for PDISP Zones				
ZONE	**TYPE**	**Change in vertical pressure (kPa)**		
#		**Stage 1 Net changes**	**Stage 2 Gross changes**	**Stages 3 and 4 Net changes**
1	RC U/pin	-27.95	58.50	-27.95
2	RC U/pin	35.71	122.16	-
3	RC U/pin	60.79	147.24	-
4	RC U/pin	60.79	147.24	-
5	RC U/pin	-30.86	55.59	-
6	RC U/pin	-30.86	55.59	-
7	RC U/pin	2.60	89.05	-
8	RC U/pin	-13.69	72.76	-
9	Superimposed zone	109.68	109.68	-
10	RC U/P	16.13	102.58	-
11	RC U/P	6.66	93.11	-
12	RC U/P	82.00	168.45	-
13	RC U/pin	67.66	154.11	-
14	Superimposed zone	44.78	44.78	-
15	Superimposed zone	44.78	44.78	-
16	RC U/pin	81.79	168.24	-
17	RC U/pin	81.26	167.71	-
18	RC U/pin	81.26	167.71	-
19	Superimposed zone	63.58	63.58	-
20	Superimposed zone	63.58	63.58	-
21	Superimposed zone	94.58	94.58	-
22	Superimposed zone	77.99	77.99	-
23	Superimposed zone	77.99	77.99	-
24	Raft slab - main house	-	-86.45	23.71
25	Raft slab - rear ext'n	-	-86.45	-36.21

<u>Ground Conditions:</u>

8.7.5 The ground profile was based on the site-specific ground investigation by Gabriel GeoConsulting Ltd, as presented in Sections 7 and 8.1 above, and the desk study information.

8.7.6 The short-term and long-term geotechnical properties of soil strata were based on this investigation and the data from the previous projects nearby. The parameters used are summarised in Table 3.

Table 3: Soil parameters for PDISP analyses					
Strata	**Level** (m bgl)	**SPT 'N' value** (blows)	**Undrained Cohesion,** Cu (kPa)	**Short term, undrained Young's Modulus,** Eu (MPa)	**Long term, drained Young's Modulus,** E' (MPa)
River Terrace Deposits	4.55 5.10	20	-	40	40
London Clay	5.10 35.00	-	40.0 220.0	20 110	12 66

Where:

Undrained shear strength, Cu assumed as Cu = 40 + 6z kPa

where z = depth below the top of the stratum (5.1m bgl)

Undrained Young's Modulus, Eu = 500 * Cu

Drained Young's Modulus, E' = 0.6 Eu

<u>PDISP Analyses:</u>

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

- Stage 1 – Construction of underpins/retaining walls – Short-term condition
- Stage 2 – Bulk excavation of central area to sub-basement formation level – Short-term condition
- Stage 3 – Construction of sub-basement slab with full-moment connections to create raft slab – Short-term (undrained) conditions
- Stage 4 – As Stage 3, except – Long-term (drained) conditions.

8.7.8 The results of the analyses for the Stages 1 to 4 are presented as both contour plots and colour-blocked diagrams on the appended Figures G3 to G6 respectively (see Appendix G). These have annotated within PDISP with displacement values, though a few of those values appear incorrect relative to the contours.

<u>Heave Assessment</u>

8.7.9 Construction of the sub-basement will cause immediate elastic displacements in response to the stress changes, followed by long-term plastic deformations as the pore water pressures in the underlying over-consolidated clays adjust to the stress changes. 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. The sub-basement construction will need to be designed to enable it to accommodate the swelling displacements/pressures developed underneath it and the resultant distortions.

8.7.10 The PDISP analyses indicated that the largest settlements of up to 14mm are likely to develop beneath the underpins to the central 6/7 party wall, where that wall bears onto the underpins for the front wall. Other locations where the larger settlements occur were all predicted to be at the intersections of walls. Once the underpins have been linked up to create a full raft slab, as proposed by MACE, the analyses suggested that the maximum settlement would reduce to 5mm in the short-term and 8mm in the longer term, though it seems unlikely that the short-term reduction in settlements would be achieved. Beneath the rear extension's underpins heave movements of up to 10mm were predicted in the short-term, increasing to up to 12.5mm beneath the raft slab in the long-term. The ranges of predicted short-term and long-term movements for each of the main walls are presented in Table 4 below.

Table 4: Summary of predicted displacements

Location	**Stage 1 (Figure G3)**	**Stage 2 (Figure G4)**	**Stage 3 (Figure G5)**	**Stage 4 (Figure G6)**
Front lightwells	1mm heave to 6mm Settlement	0.5mm heave to 4mm Settlement	0.5mm heave – 1mm settlement	2.5mm heave – 2mm settlement
Front wall	2 – 14mm settlement	1 – 11mm settlement	Included in raft	Included in raft
5/6 party wall – main building	2 – 13mm settlement	1 – 9mm settlement	Included in raft	Included in raft
7/8 party wall – main building	2 – 12mm settlement	1.5 – 8mm settlement	Included in raft	Included in raft
Rear wall – main building	4 – 13mm settlement	1.5 – 9mm settlement	Included in raft	Included in raft
5/6 party wall – rear extension	10mm Settlement – 2mm Heave	6mm Settlement - 8mm Heave	Included in raft	Included in raft
7/8 party wall – rear extension	7mm Settlement – 3mm Heave	4mm Settlement - 6mm Heave	Included in raft	Included in raft
Rear wall – No's 47 & 48 Chagford Street	1mm Settlement – 1.5mm Heave	8mm Settlement – 1.5mm Heave	Included in raft	Included in raft
Internal footings – main building	9-13mm Settlement	9mm Settlement - 2mm Heave	Included in raft	Included in raft
Internal footings – rear ext'n	8mm Settlement – 2mm Heave	1.5 – 10mm Heave	Included in raft	Included in raft
Raft slab – main building	N/A	Up to 6.5mm heave (no slab present)	5mm Settlement – 2mm Heave	8mm Settlement – 4mm Heave
Raft slab – rear extension	N/A	Up to 13mm heave (no slab present)	1 – 7.5mm Heave	1.5 – 12.5mm Heave

8.7.11 The ranges of displacements quoted in Table 4 cover approximately the full range of predicted deflections, however the stiffness of the underpin bases is likely to reduce the range of displacements actually experienced.

8.7.12 All the short term elastic displacements would have occurred before the sub-basement slab is cast, so only the post-construction incremental heave/settlements are relevant to the design of the various sections of sub-basement slab between the underpin bases. The analyses indicated that the maximum predicted post-construction displacements could be approximately 12mm (total and differential) near the middle of the raft slab beneath the main building. The analyses for Stage 3 & 4 assumed a uniform distribution of loads across the slab, which is a further simplification, so if a more precise prediction of likely settlements is required for final design purposes then a finite element analysis (or similar) of the predicted design bearing pressures would be required by a suitably competent geotechnical engineer in order to model the soil-structure interaction.

8.8 Damage Category Assessment

8.8.1 When underpinning it is inevitable that the ground will be un-supported or only partially supported for a short period during excavation of each pin, even when support is installed sequentially as the excavation progresses. This means that the behaviour of the ground will depend on the quality of workmanship and suitability of the methods used, so calculations of predicted ground movements can never be rigorous. However, provided that the temporary support follows best practice as outlined in Section 8.6 above then extensive past experience has shown that the bulk horizontal movements of the ground alongside the sub-basement caused by underpinning for a single-storey basement (typical depth 3.5m) should not exceed 5mm.

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

8.8.3 In order to identify the locations where the potential for damage to the adjoining properties is greatest it is necessary to consider the settlement/heave predictions from the PDISP analyses, the location of laterally extensive load-bearing walls in the adjoining properties (short walls are less vulnerable than long walls of the same height), the height to length ratio of the adjoining walls, and any changes in ground/foundation levels in the adjoining properties. For the neighbouring properties in the same terrace as the Somerset Hotel:

- No plans have been found for the adjoining No.8, so its internal layout has been assumed to match that of No's 6 & 7, with no laterally extensive internal transverse walls. Aerial images show that its rear wall is significantly further back than that to the adjoining No's 132 & 134 Gloucester Place, whereas the front walls to all these properties are approximately aligned.
- The rear wall of No's 4 & 5 is not continuous (at ground and lower ground floor levels, at least, where the rooms in the small rear projections extend through the line of the main rear wall).
- The adjoining properties in this terrace all appear to be at approximately the same level.

Thus, damage category assessments are warranted for both the front and rear walls of the main part of the adjoining No.8 using the largest predicted settlements (in Stage 1) in order to cover the worst case scenarios of the varying factors.

8.8.4 The heave predicted by the PDISP analyses around most of the Somerset Hotel's rear extension will be beneficial in offsetting settlements caused by relaxation of the ground alongside the sub-basement excavations. Thus, for load-bearing walls adjoining the rear extension the predicted minimum heave or, if applicable, the maximum settlement will represent the worst-case scenario for a given wall geometry. Stage 1 will therefore be critical for the mews houses on Chagford Street to the rear of the hotel. No.46 is understood to have a modern basement, including a swimming pool on the corner adjacent to the rear left corner of the Somerset Hotel, though that basement wasn't evident in TP1. Drawings for 49-50 Chagford Street, to the rear of No's 4 & 5 Dorset Square, show the ground floor level in the mews house to be 0.90m above the lower ground floor level in No.4, and the same is likely to apply for No's 47 & 48 behind the Somerset Hotel. The greater length of the rear wall of No's 49-50 would make that more vulnerable than the shorter party walls between the mews houses, however that wall benefits from adjoining a corner of the sub-basement (where the horizontal displacements will reduce owing to the presence of the adjacent unexcavated ground). Thus, the worst case scenario for walls adjoining the hotel's rear extension will be the party wall between No's 47 & 48 Chagford Street where slight settlement is predicted beneath the adjacent part of this wall.

8.8.5 Three damage category assessments have been undertaken for the worst-case scenarios identified above; these assessments considered:

- ground movements arising from the vertical stress changes, as assessed by the PDISP analyses (see Section 8.7), including allowances for stiffness of the foundations;
- ground movements alongside the proposed underpins caused by relaxation of the ground in response to the excavations. Ground movements associated with the construction of retaining walls in sands/gravels have been shown to extend to a distance up to twice the depth of excavation (Figure 6.16 in CIRIA Report C760, Gaba et al 2017).

<u>No.8 Dorset Square & No.132 Gloucester Place: Front walls:</u>

8.8.6 The relevant geometries, based on the information in paragraphs 2.7, 3.3, 3.4 and 8.3.8, and Section 7, are:

Depth of foundations = **0.35m** (from TP6, assuming No.8 similar to No.6).
Height of No.8's front wall = 17.3m (by scaling from Pearson Architect's drawings – see paragraph 1.6).
Width of No.8 = approx. 7.4m.
Depth of Excavation = 4.55 – 0.35 = **4.2m** below No.8's foundations.
Width (L), zone of affected soils = 4.2 x 2= **8.4m**, so ground movements are likely to extend beneath the near side of No.132.
Height (H) = 17.3 + 0.35 = **17.65m** (wall height + foundation)
Hence L/H = **0.48**

8.8.7 Thus, for an anticipated 5mm maximum horizontal displacement, increased pro-rata to 6.0mm for the increased depth of excavation, the strain beneath No.8 would be in the order of $\varepsilon_h = 7.14 \times 10^{-4}$ (0.071%).

8.8.8 The maximum ground settlements alongside a supported excavation in sand are typically 0.3% of excavated depth (CIRIA Report C760, Figure 6.16), giving a predicted settlement of 12.6mm for the 4.2m depth of excavation beneath the foundations of No.8. This CIRIA settlement profile is based on monitoring of several retaining walls during construction and for short periods after construction, so will include all the short-term elastic ground movements but only a small portion of the long-term movements in the London Clay beneath the basement. Those long-term movements can be estimated by comparing the predicted displacements from Stages 3 & 4 of the PDISP analyses. At the centreline of the junction of No.8's front wall with the 7/8 party wall the settlement predicted by the PDISP analysis increased from 0.9mm in Stage 3 to 1.5mm in Stage 4 (see Figures G5 and G6). This 0.6mm of additional settlement is 0.014% of excavated depth which must be added to the 0.3% predicted by the PDISP settlement profile. Plotting the additional settlement onto Figure 6.16 of CIRIA Report C760 enabled the combined settlement profile to be identified which was then used to find the maximum deflection, Δ. In this case Δ = 2.8mm, which represents a deflection ratio, Δ/L = 3.33 x 10^{-4} (0.033%).

8.8.9 Using the graphs for L/H = 0.5, these deformations represent a damage category assessment of 'slight' (Burland Category 2, ε_{lim} = 0.075-0.15%) as given in CIRIA SP200, Table 3.1, and illustrated in Figure 11 below.

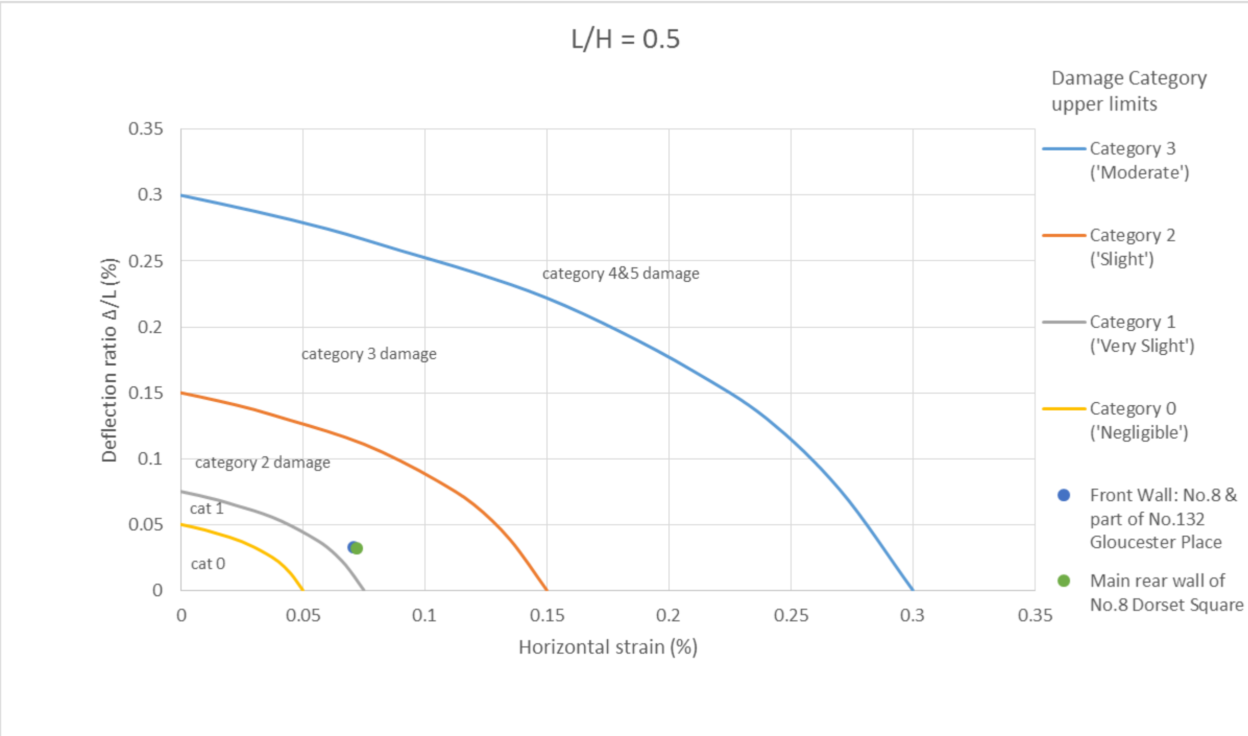

Figure 11: Damage category assessment for front walls of No.8 Dorset Square and part of No.132 Gloucester Place.

8.8.10 In order to prevent the ground movements reaching Category 2, it is recommended that, subject to Party Wall Act agreements, limited permeation grouting should be undertaken around the perimeter zone of the sub-basement, above formation level, before the excavations commence. A low strength grout should be used to facilitate subsequent excavation of the grouted soils (where they extend within the footprint of this sub-basement, or if the neighbours subsequently wish to add a basement beneath their properties).

No.8 Dorset Square: Main rear wall:

8.8.11 The relevant geometries, based on the information in paragraphs 2.7, 3.3, 3.4 and 8.3.8, and Section 7, are:

Depth of foundations = **0.44m** (from TP3, assuming No.8 similar to No.6)
Height of No.8's rear wall = 17.3m.
Width of No.8 = approx. 7.4m
Depth of Excavation = 4.55 – 0.44 = **4.11m** below No.8's foundations.
Width (L), zone of affected soils = 4.11 x 2= **8.22m**, so ground movements are likely to extend slightly beyond No.8's north flank wall.
Height (H) = 17.3 + 0.44 = **17.73m** (wall height + foundation)
Hence L/H = **0.46**

8.8.12 Thus, for an anticipated 5mm maximum horizontal displacement, increased pro-rata to 5.9mm for the increased depth of excavation, the strain beneath No.8 would be in the order of $\varepsilon_h = 7.18 \times 10^{-4}$ (0.072%).

8.8.13 Following the same procedure as used for the front wall of No.8 (see paragraph 8.8.8), the predicted settlement is 12.3mm for the 4.11m depth of excavation below the foundations of No.8's rear wall. At the centreline of the junction of No.8's rear wall with the 7/8 party wall there was no significant displacement predicted by the PDISP analysis between Stage 3 and Stage 4 (see Figures G5 and G6) so the standard settlement profile given in Figure 6.16 of CIRIA Report C760 is applicable without modification. Thus, the maximum deflection, Δ = 2.6mm, which represents a deflection ratio, $\Delta/L = 3.16 \times 10^{-4}$ (0.032%).

8.8.14 Using once again the graphs for L/H = 0.5, these deformations represent a very similar damage category assessment of 'slight' (Burland Category 2, ε_{lim} = 0.075-0.15%) as recorded for No.8's front wall, and as illustrated in Figure 11 above. Accordingly, the same recommendation for permeation grouting pre-treatment of the soils above formation level as given in paragraph 8.8.10 above applies equally here, subject to Party Wall Act agreements.

No's 47 &48 Chagford Street: Party wall:

8.8.15 The relevant geometries, based on the information in paragraphs 3.3, 3.4 and 8.3.8, Section 7 and Famedean Ltd's Section B-B through No's 49-50 Chagford Street (Drg No.1086-08) are:

Ground level difference relative to Somerset Hotel = **+0.9m**

Assumed depth of foundation = **0.3m** (assumed similar to 0.28m/0.37m deep footings to the rear wall of these mews houses, as seen in TPs 1 & 2, ignoring the local piers).

Height of party wall, ignoring apex = **6.58m** (by scaling the height of the parapet above internal ground floor level from Famedean's drawing).

Depth of Excavation = 4.55 + 0.9 – 0.3 = **5.15m** below party wall's foundations.

Width of zone of affected soils = 5.15 x 2= **10.3m**.

Length (L) of party wall to No's 47 & 48 = **8.2m**, so ground movements are likely to extend slightly beyond the front wall of these mews houses.

Height (H) = 6.58 + 0.3 = **6.88m** (wall height + foundation)

Hence L/H = **1.19**

8.8.16 Thus, for the typical 5mm maximum horizontal displacement, increased pro-rata to 7.4mm for the increased depth of excavation, the strain beneath No.8 would be in the order of $\varepsilon_h = 7.18 \times 10^{-4}$ (0.072%).

8.8.17 Following the same procedure as used for the previous two assessments (see paragraphs 8.8.8 & 8.8.13), the predicted settlement is 15.5mm for the 5.15m depth of excavation below the assumed shallow foundations of the party wall between No's 47 & 48 Chagford Street. At the junction of this party wall with the rear wall of the Somerset Hotel's rear extension (and proposed sub-basement) the PDISP analyses predicted an increase in heave from 4.4mm to 7.1mm between Stage 3 and Stage 4 (see Figures G5 and G6). This net 2.7mm of heave and the associated profile of net displacements beneath these mews houses must be combined with the profile given in Figure 6.16 of CIRIA Report C760 in order to determine the maximum deflection, Δ applicable to this party wall. From the combined displacement profile in Figure 12, using only the portion of the curve equal to the length of the party wall, the maximum deflection, Δ = 2.25mm which represents a deflection ratio, $\Delta/L = 2.74 \times 10^{-4}$ (0.027%).

8.8.18 Using the graphs for L/H = 1.0, these deformations once again represent a damage category assessment of 'slight' (Burland Category 2, ε_{lim} = 0.075-0.15%) as recorded for the front wall and main rear wall of No.8, and as illustrated in Figure 13 below. Accordingly, the same recommendation for permeation grouting pre-treatment of the soils above formation level as given in paragraphs 8.8.10 and 8.8.14 above applies equally here, subject to Party Wall Act agreements.

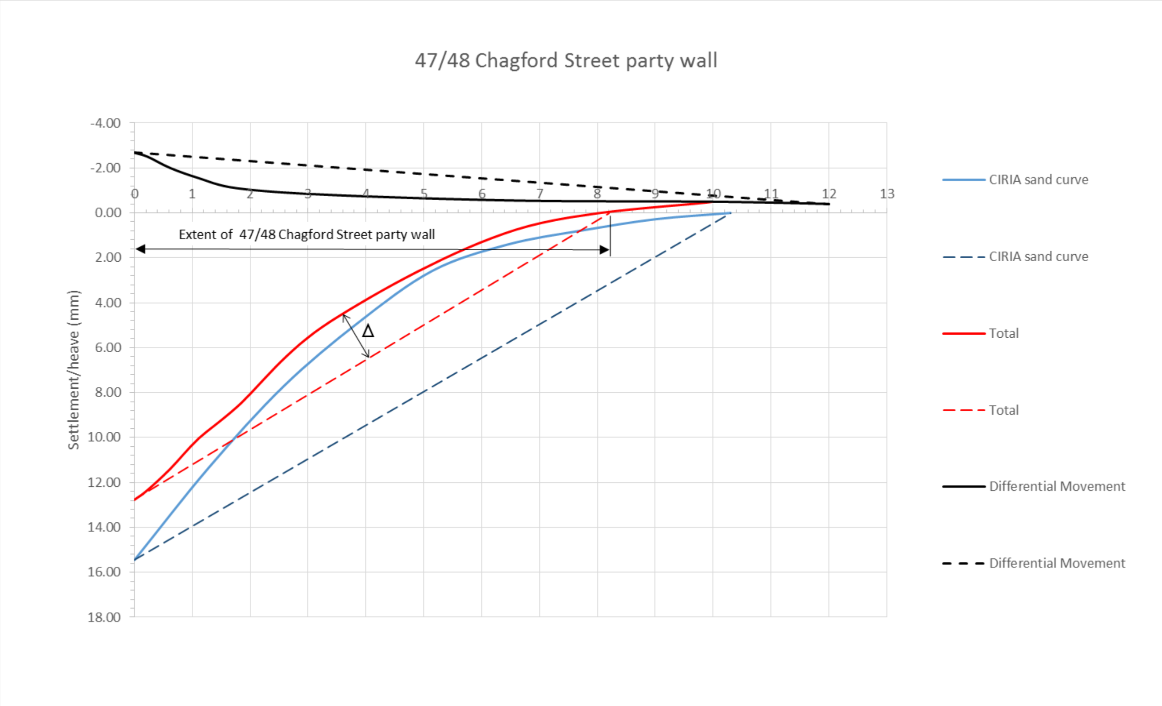

Figure 12: Ground movement assessment for the 47/48 Chagford Street party wall.

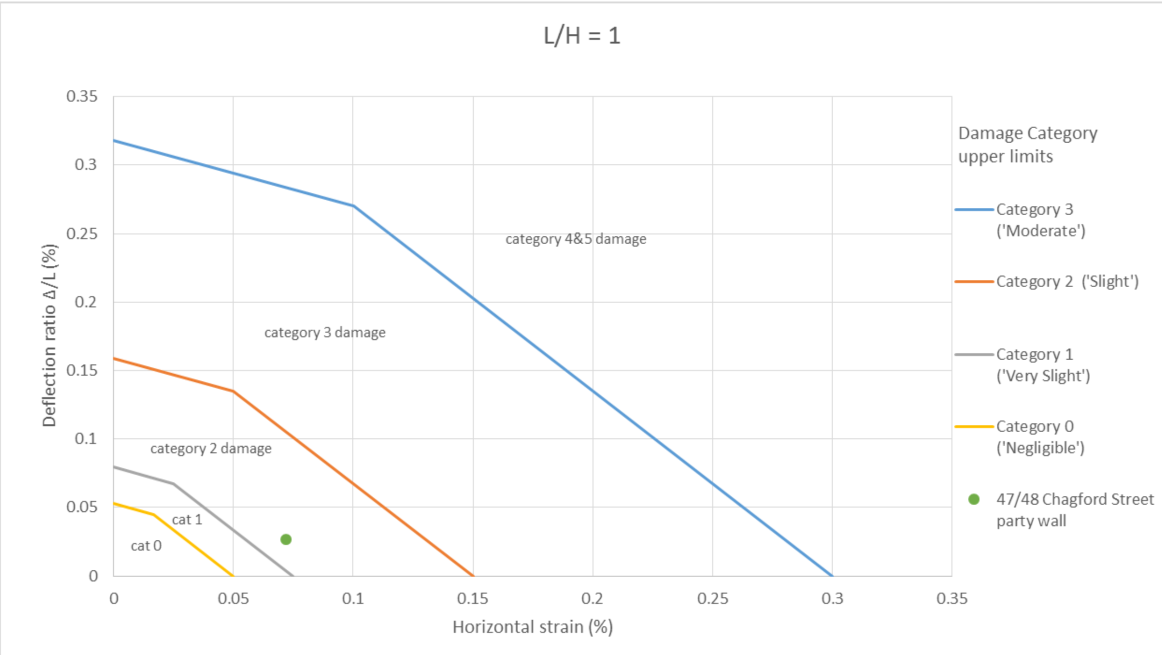

Figure 13: Damage category assessment for the 47/48 Chagford Street party wall.

8.8.19 Use of best practice construction methods, as outlined in Section 8.6 above, will be essential in order to ensure that the ground movements are kept in line with the above predictions.

8.9 Monitoring

8.9.1 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 a Party Wall Agreement and are beneficial to all parties concerned.

8.9.2 Precise movement monitoring should be undertaken weekly throughout the period during which the sub-basement walls and slab are constructed with three sets of initial readings taken before excavation starts in order to assess any on-going thermal or other movements in the structures concerned. Readings may revert to monthly for three months once all the perimeter walls and the sub-basement slab have been completed and the concrete has gained full strength. This monitoring should be undertaken with a total station instrument and targets attached at a minimum of two levels at each of the following locations:

- externally on the front walls of No's 5 & 8 Dorset Square, aligned with the 4/5, 5/6, 7/8 and 8/9 party walls;
- internally in the Somerset Hotel's existing basement at four equally spaced locations along the full length of the 5/6 and 7/8 party walls;
- internally in the Somerset Hotel's existing basement at three equally spaced locations along the rear party wall with No's 47 & 48 Chagford Street (the middle location being on the line of the 47/48 party wall);
- externally on the front walls of No's 47 & 48 Chagford Street, on the line of the 46/47, 47/48 and 48/49 party walls;
- at the client's discretion, since outside any Party Wall Agreement, it would also be sensible to monitor all other load-bearing walls within the Somerset Hotel that will be underpinned.

8.9.3 The wall movements detected by the monitoring exercise may be caused by rotation, flexing without cracking (especially for walls built using lime mortar) or lateral movements transverse to the plane of the wall. Movements such as these which occur without cracking would all fall within Burland's Category 0, so a twin-track approach to the monitoring will be required, combining both the target monitoring as proposed above and visual observations. Daily inspections of the subject property and the external walls of the adjoining buildings (where visible) should be made and recorded by a member of the contractor's staff. If any new structural cracks appear in the main loadbearing walls, then the appointed Structural Engineer should be informed and those cracks should be monitored using the Demec system (or similar) on the same frequency as the target monitoring. Additional targets might also need to be installed, at the engineer's discretion, depending on the location of the cracks. It will be important to ensure that all pre-existing cracks in affected load-bearing walls which have weakened their structural integrity should be fully repaired in accordance with recommendations from the appointed Structural Engineer before any underpinning is carried out (as recommended in paragraph 8.6.3 above).

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

8.10 Buried Concrete

8.10.1 The results from the chemical tests (see paragraph 7.10) indicate that the samples fell within the following Design Sulphate Classes, as defined by BRE Special Digest 1 (2005):

- DS-1: All samples from the Lynch Hill Gravel Member
- DS-1: Samples from Made Ground in TP3 and TP5.
- DS-3: Sample from Made Ground in TP2.

8.10.2 Groundwater on this site is mobile so, in accordance with BRE Special Digest 1 (2005), the design of buried concrete for use on this site should comply with the requirements the following 'Aggressive Chemical Environment for Concrete':

- ACEC Class: AC-3.

8.10.3 Normal good working practices should be implemented when handling, placing and compacting the concrete.

9. CONCLUSIONS

9.1 These conclusions consider only the primary findings of this assessment; the whole report should be read to obtain a full understanding of the matters considered.

Ground Investigation:

9.2 The site's geology has been confirmed to be broadly as mapped by the British Geological Survey, with River Terrace Deposits of the Lynch Hill Gravel Member overlying the London Clay Formation at approximately 5.1-5.2m below the level of the front lightwell (Section 7, paragraph 8.1.1).

9.3 There was evidence of perched groundwater at shallow depth in the River Terrace Deposits (moist and wet sands) but the standpipe (base at 2.7m below the front lightwell, approximately 23.5m AOD) remained dry during the brief monitoring period. Groundwater was found at 21.17-25.38m AOD in nearby boreholes so groundwater of the Upper Aquifer is likely to be encountered by the excavations for the proposed sub-basement. Further monitoring readings should be taken during the coming winter (Table 1, Section 7, paragraphs 8.1.1 & 8.3.2).

Basement Impact Assessment – Surface Water & Flooding:

9.4 No records of past surface water flooding in the vicinity of the Somerset Hotel were found in the flood studies reviewed (5.6, 8.2.1).

9.5 The site lies in flood risk Zone 1, so is considered to be at a Low risk of flooding from rivers or the sea, and only a Very Low risk under the Environment Agency's RoFRAS classification (5.2, 5.3, 8.2.1).

9.6 The Somerset Hotel is **not** in an area at risk of flooding following failure of a reservoir (5.3, 8.2.1).

9.7 The Somerset Hotel is not in or near any of the Surface Water Flood Risk Hotspots as identified in Westminster's Basement Development SPD and in Westminster's 2019 SFRA (5.7, 8.2.1).

9.8 The proposed sub-basement will not cause any increase in impermeable surface area so, as already proposed, will not create any increase in discharge of surface water run-off to mains drainage. SuDS options for compliance with Policy 5.13 'Sustainable Drainage' of the London Plan include storage of surface water for re-use, attenuation followed by gradual release to mains drainage, or inclusion of a roof terrace with green roof on the existing rear extension (8.2.2, 8.2.3).

9.9 Flood modelling presented in the 2019 SFRA appears to indicate a variable depth of flooding during a 1% annual probability storm with 40% climate change allowance, but the resolution is very low and the variable depth is implausible (5.5, 5.9, 8.2.1).

9.10 The Environment Agency's latest modelling predicted a uniform, Low risk of surface water flooding for the whole of the Somerset Hotel's site (which is considered to be more plausible than the 2019 SFRA's modelling). Recommendations are given for mitigation measures to maintain or increase the property's resistance to flooding (8.2.4, 8.2.5).

9.11 Thames Water have no records of sewer flooding affecting the Somerset Hotel, so the probability of future sewer flooding affecting the Hotel is considered to be Low provided that the sewer system is well maintained and the resistance measures identified below are implemented and maintained (8.2.7).

9.12 Non-return valves, possibly supplemented by pumped above-ground loop systems, should be fitted to the drains serving the property. Temporary interception storage may also be required (8.2.8, 8.2.9).

Basement Impact Assessment – Groundwater:

9.13 All the properties on the east side of Dorset Square including the Somerset Hotel have original lower ground floors/basements, and these are all understood to be at approximately the same level. The only deeper existing basements found by our searches are a single-storey basement at No.46 Chagford Street, which adjoins the rear left (north) corner of Somerset Hotel's site, and a 2-storey basement at No's 42-43 Chagford Street (8.3.3 to 8.3.6).

9.14 The proposed sub-basement will be founded at approximately 21.7m AOD, about 0.6m above the assessed base level of the River Terrace Deposits (in DP1), so is expected to extend below the groundwater level in the Upper Aquifer. Once the Somerset Hotel's sub-basement has been completed the anticipated south-south-easterly seepage/flow of groundwater is expected to continue flowing around and beneath the hotel's sub-basement and the adjoining small basement at No.46 Chagford Street (8.3.3, 8.3.8, 8.3.9).

9.15 Groundwater flooding at surface is considered unlikely, because local information takes precedence over the groundwater susceptibility data. A provisional design groundwater level at 1.0m below existing basement level is proposed, which means that the sub-basement must be able to resist buoyant uplift pressures (un-factored) ranging up to 36kPa (8.3.10 to 8.3.14).

9.16 Groundwater control may require sump pumping to remove local entries from the Made Ground, and is likely to require either sump pumping or use of well-points to control groundwater entries from the Upper Aquifer. Permeation grouting may be required if highly permeable gravels are encountered. A fully-shored trial excavation down to the proposed formation level is recommended at the start of construction in order to check the adequacy of the chosen method of groundwater control (8.4.1).

9.17 If clays are exposed at formation level then they must be protected from water and blinded with concrete immediately following excavation and inspection (8.4.3, 8.6.7).

9.18 The sub-basement will need to be fully waterproofed (Section 8.5).

Basement Impact Assessment – Ground Stability:

9.19 There are no concerns about slope stability (8.6.1).

9.20 Provided best working practices are followed when constructing the underpins, especially in relation to use of high-stiffness temporary support systems, with full face support, then ground movements and the resultant structural distortions can be kept within acceptable limits. All pre-existing structural damage to walls which are to be underpinned should be repaired before excavations for the underpins are started (8.6.2 to 8.6.7).

9.21 Consideration should be given to installation of transition underpins beneath the neighbour's load-bearing walls which adjoin the proposed sub-basement, subject to agreement under Party Wall Act protocols (8.6.8).

9.22 Various other guidance is provided in relation to the geotechnical design and construction of the sub-basement's perimeter walls (8.6.10, 8.6.11).

9.23 In order to minimise differential settlements relative to adjoining properties, the net bearing pressures should be restricted to less than 80kPa wherever possible because of the soft-firm condition of the upper part of the London Clay close beneath the founding level. If 80kPa maximum is not practical, then undertake further ground investigation at the locations concerned and, if necessary, dig out and replace the weak soils (8.6.12, 8.6.13).

9.24 Ground Movement Assessment (GMA): The sub-basement slab must be designed to accommodate swelling displacements/ pressures generated by heave of the underlying clays. A preliminary heave/settlement assessment has been undertaken using PDISP software which predicted between 14mm of settlement and 10mm of heave beneath the underpins and up to 13mm of heave below the centre of the sub-basement slab during the initial phase of construction. Once the underpins have been linked up to create a full raft slab, as proposed by MACE, the analyses indicated that the long-term displacements would range from 8mm settlement to 12.5mm of heave. More details are provided in Table 4 (page 41) and Appendix G (Section 8.7).

9.25 Damage Category Assessment (DCA): Three DCAs were undertaken for the locations assessed to be critical for displacements beneath adjoining buildings – the front wall and main rear wall of No.8 Dorset Square and the party wall between No's 47 & 48 Chagford Street. The other adjoining properties were assessed to be at lower risk of damage for the reasons set out in detail in paragraphs 8.8.3 and 8.8.4. All three DCAs indicated that, provided best practice construction methods are employed, the worst case predicted deformation is likely to fall within Burland Category 2, termed 'slight'. In order to reduce ground movements and the risk of damage to the adjoining buildings, and subject to Party Wall Act agreements, the soils around the perimeter of the sub-basement should be pre-treated by permeation grouting with a low strength grout before starting any excavations for the underpins (Section 8.8).

9.26 Monitoring: Condition surveys of the neighbouring properties should be commissioned. Requirements are given for a system of monitoring the adjacent structures, which should be established before the works start (Section 8.9).

9.27 Buried Concrete: The design of buried concrete for use on this site may be based on Design Sulphate Classes DS-1 to DS-3, and ACEC Class AC-3 (8.10.2 & 8.10.3).

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