Basement Impact Assessment

Site 26 West Hill Park
London
N6 6ND

Client Croft Structural Engineers
Date May 2017
Our Ref BIA/8417

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Foreword

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

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

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

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

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1.0 INTRODUCTION

1.1 This report presents the outcome of a Basement Impact Assessment (BIA) for the proposed development of 26 West Hill Park, London. The local planning authority is the London Borough of Camden.

1.2 Chelmer Site Investigation Laboratories Ltd (Chelmer) was instructed by Croft Structural Engineers in January 2017 to complete this report. The report has been prepared by Alexandra Ash MEng and Joel Slater BEng, and reviewed by Dr Martin Preene BEng PhD CEng FICE CGeol FGS CSci CEnv C.WEM FCIWEM. Dr Preene is a UK Registered Ground Engineering Adviser with 30 years' experience of geotechnical engineering.

1.3 This report presents a BIA that is compliant with Camden Borough CPG4 planning document (July 2015). As required by the CPG4, screening flow charts covering the three main issues (surface flow and flooding, land stability and groundwater flow) have been provided in Appendix A.

1.4 The BIA aims to identify any detrimental impacts the proposed basement may have to the local area or neighbouring properties through its potential impacts to groundwater and ground movement. At the request of the client the impacts to surface water have not been considered and assessed in this report. This has been performed by using the Stage 1 Screening assessment set out in CPG4 and completing the screening flow charts in Appendix A. Where Stage 1 identifies potential impacts these have been addressed in Appendix A, which refers to the relevant Conceptual Site Model sections in this report. The third stage of the BIA includes a site investigation and desk study; these are detailed in Section 3.0. The Conceptual Site Model, Section 4.0, evaluates the implications of the proposed development (Stage 4). Finally, a Ground Movement and Damage Category Assessment has been undertaken that identifies potential impacts to neighbouring properties (Stage 4).

1.5 The site comprises 26 West Hill Park, London and is located at approximate Ordnance Survey grid reference (OSNGR) 527905E, 186845N. The site comprises a three storey detached residential property, consisting of lower ground, ground and first floors. The property has front and rear gardens and a garage and driveway to the front. Mature trees and other vegetation are present across the site.

1.6 It is to our understanding that the proposed development involves extension to the lower ground floor to front and rear, including relocation of the swimming pool and extensions to both ground and first floors to the side of the existing property. A terrace is also proposed to the front of the property at ground floor level, above the lower ground floor extension. Existing and proposed plans are presented in Appendix B.

1.7 A site inspection (walk-over survey) was undertaken on 27th February 2017 by Alexandra Ash of Chelmer, photos from which are presented in Appendix C. Desk study data have been collected from various sources including borehole/well logs from the vicinity of the site from the British Geological Survey (BGS) (Appendix D) and geological data, environmental data and historic maps from Groundsure which are presented in Appendix E. Relevant information from the desk study and site inspection is presented in Sections 2.0 and 3.0.

1.8 A ground investigation was undertaken by Chelmer (2017) on 17th February and 2nd March 2017 and the findings are summarised in Section 3.0. The Factual Report from the ground investigation is presented in Appendix F.

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

London Development & Construction
Drawing 01 (Existing Lower Ground Floor Plan)
Drawing 02 (Proposed Sections A-A and B-B Lower Ground Floor Plan)
Drawing 03 (Existing and Proposed Ground Floor Plan)
Drawing 04 (Existing and Proposed First Floor Plan)
Drawing 05 (Existing and Proposed Façades)
Drawing 07 (Existing and Proposed Side Façade)

Croft Structural Engineers
Drawing SL-10 (Lower Ground Floor Plan)
Drawing SL-20 (Ground Floor Plan)
Drawing SD-11 (Structural Scheme Design - Sections)
Drawing TW-10 (Temporary Works Scheme Design)

CD Surveys Ltd
Drawing LDC/1609006 (Topographical Survey)

2.0 PROPERTY AND AREA DETAILS

2.1 The property is located on the west side of West Hill Park, between West Hill Park and Merton Lane. The site is approximately 1km south east of Kenwood House. The site occupies an area of approximately 840m^2 and is centred on Ordnance Survey National Grid Reference 527905E, 186845N.

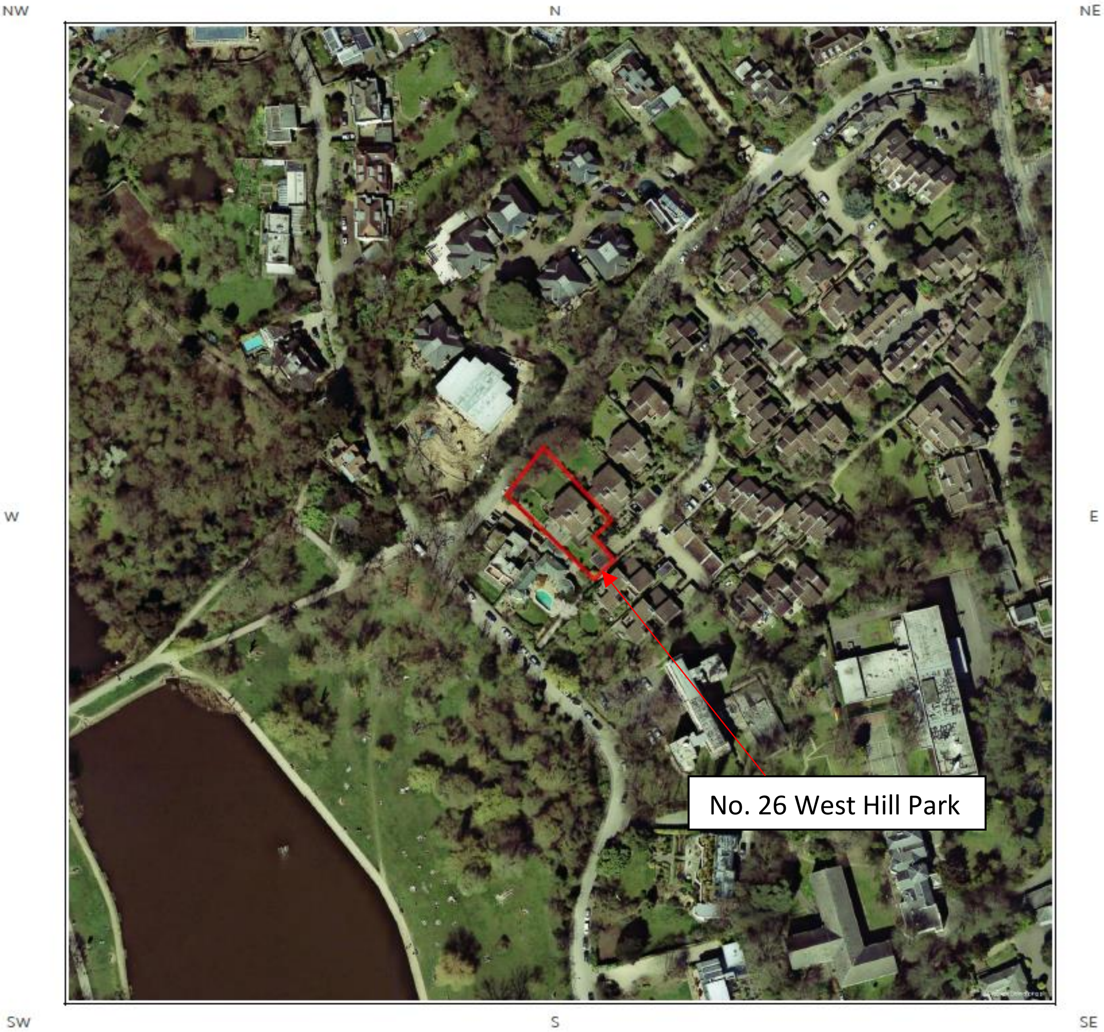

Figure 1. Site Location Plan (Groundsure)

2.2 The site comprises 26 West Hill Park, London N6 6ND which is a three storey detached residential property, consisting of lower ground, ground and first floors with the entrance at ground floor level. The property has front and rear gardens and a garage and driveway to the front at lower ground floor level. The front garden is at lower ground floor level and during the site walkover a small pond was observed alongside the south west boundary with No. 23 Merton Lane. The rear garden is terraced down from the north east boundary to the south west boundary. Mature trees and other vegetation are present across the site, including two large mature trees in the rear garden close to the boundary with Merton Lane. A further large mature tree was noted in the rear garden of No.25 West Hill Park. The property is neighboured by No. 25 West Hill Park to the north east, an alleyway and beyond that No.27 West Hill Park to the south east, No's 23 & 25 Merton Lane to the south west and the Merton Lane carriageway to the north west.

2.3 A site inspection (walk-over survey) was undertaken on 27th February 2017 by Alexandra Ash of Chelmer, photos from which are presented in Appendix C. The property appeared to be in a good state of repair during the site inspection visit, however a few of the walls around the site boundary had some cracking. No.25 West Hill Park was noted to be at a slightly higher level (<0.5 m) than No. 26. The south west boundary with No's 23 & 25 Merton Lane consisted of a retaining wall with a difference in ground level of approximately 1.5m to 2.5m. The difference in ground level between the rear garden and Merton Lane was approximately 1m to 1.5m.

2.4 The proposed development involves extension to the lower ground floor to front and rear, including relocation of the swimming pool and extensions to both ground and first floors to the side of the existing property. A terrace is also proposed to the front of the property at ground floor level, above the lower ground floor extension. Existing and proposed plans are presented in Appendix B.

2.5 The proposed lower ground floor extensions are anticipated to be set at depths of between approximately 2.3m and 3.8m below existing ground level (bgl) for the rear basement and 1.7m and 4.0m bgl for the front basement, given the changes in elevation across the site and allowing for the depth of the swimming pool in the rear. The rear basement is anticipated to be set at depths of 87.7m AOD and 86.2m AOD (for the swimming pool). The front basement is anticipated to be set at a depth of 87.0m AOD.

2.6 A search has been made of planning applications on the Camden website in order to obtain details of any other basements which have been constructed, or are planned, in the vicinity of the site. This search found a single planning application relating to a modern basement within the vicinity of the site at No.1 Haversham Place (Camden planning application no. 2012/01973/P) for 'extensions and alterations for the erection of a three storey (basement, ground and first floor) plus roof level side extension…'.

2.7 No information is available on the foundation depths of neighbouring structures. In this study we have assumed a conservative foundation level of 0.5 m bgl for the surrounding properties. However, given the sloping topography of the local area the ground level of No's 23 and 25 Merton Lane is approximately 1.5 m to 2.5 m below the south west boundary of No. 26 West Hill Park and the ground level of No. 25 West Hill Park is approximately 0.5 m above the northeast boundary of No.26.

3.0 PHYSICAL SETTING

3.1 Site History and Age of the Property

3.1.1 Historic maps (presented in the Groundsure Report in Appendix E) indicate the local area is predominantly occupied by large residential houses set within their own grounds from 1870. The site of No.26 West Hill Park is within the grounds of one of the houses. In the earliest maps Merton Lane is shown to border the site and Fitzroy Park and Millfield Lane are also present. The Highgate Pond chain on Hampstead Heath is present running from west to south of the site in a NW/SE orientation. Other small ponds are indicated 50m south east, 75m east and 175m west of the site. On the 1914-1915 map Nurseries are indicated 250m north of the site. On the 1935-36 map further residential development is indicated in the vicinity of the site and extensive residential development is indicated 250m east and south east of the site. A school is indicated to the north of the site, on the opposite side of Merton Lane, and a convent is indicated 100m east of the site. Another small pond is indicated 175m north of the site. The Nursery is no longer indicated. On the 1950-52 maps a small building is shown partially within the site, crossing the north east border, this is no longer present on the 1965 map but another small building is shown bordering the east of the site. On the 1962 map the small pond to the south east is no longer present. On the 1974 map the small building bordering the east of the site is no longer present, however a building is present adjacent to the south west site boundary, thought to be No.25 Merton Lane. The 1977 map shows the development of West Hill Park, including No.26. This map also no longer shows the convent or the small pond to the east. The 2002 map show the school on the opposite side of Merton Lane has been replaced by Haversham Place. The historic maps identify very few developments in the area since 2002.

3.2 Topography

3.2.1 The detached, three-storey building is located on the west side of West Hill Park, with Merton Lane to the rear (north west) of the site. The slope of the West Hill Park carriageway follows its direction to the lowest point outside No.28 West Hill Park. Merton Lane slopes down south-westwards. The BGS Onshore GeoIndex indicates that the surrounding land slopes down to the southwest approximately 6-7°. According to the Topographical Survey (Drawing LDC/1609006) the maximum level change across the site is approximately 3.5m from the north east boundary to the south west boundary, which corresponds to a slope across the site of approximately 10°.

3.3 Hydrological Setting (Rivers and Watercourses)

3.3.1 The site lies approximately 6.8 km to the north-west of the River Thames. The nearest surface water feature, identified in the Groundsure Report, is 122m south west of the site, with a further five surface water features identified within 250m of the site. The Groundsure Detailed River Network identified the Highgate Ponds (No.'s 3 and 4) and their associated culverts and drains within 250m of the site. At their closest pond No. 3 was identified 162m to the south west of the site. The surface water features identified are thought to be associated with the Highgate Ponds with the difference in distance due to an index error in the mapping. The BGS Onshore GeoIndex identifies the nearest well as being located approximately 880m to the southeast of the property.

3.3.2 The book 'The Lost Rivers of London' (Barton, 1992) identifies the lost River Fleet running west and south of the site, the current location of which is discussed in section 3.3.4 below. A map of the tributaries of the Thames and showing the approximate location of No.26 West Hill Park is presented in Figure 2 and the location of the Fleet relative to No.26 West Hill Park is presented in Figure 3.

3.3.3 Hydrological data has also been obtained from the Groundsure Report (see Appendix E), which indicates:

- There are no surface water abstraction licences within 2000 m of the site.
- There are no flood defences, no area benefitting from flood defences, and no flood storage areas within 250m of the site.

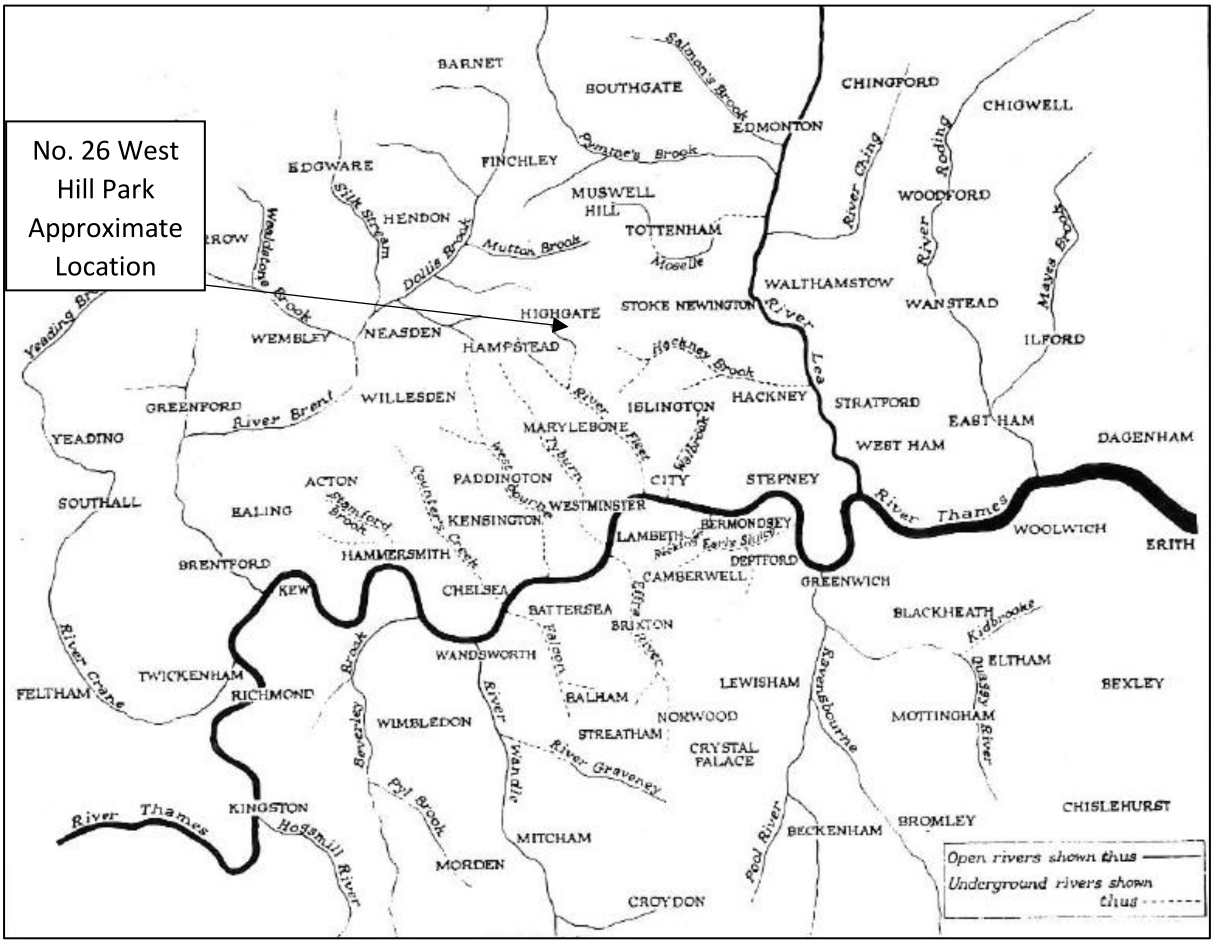

Figure 2. Tributaries of the Thames from Kingston to Erith identified in 'The Lost Rivers of London' (Barton, 1993)

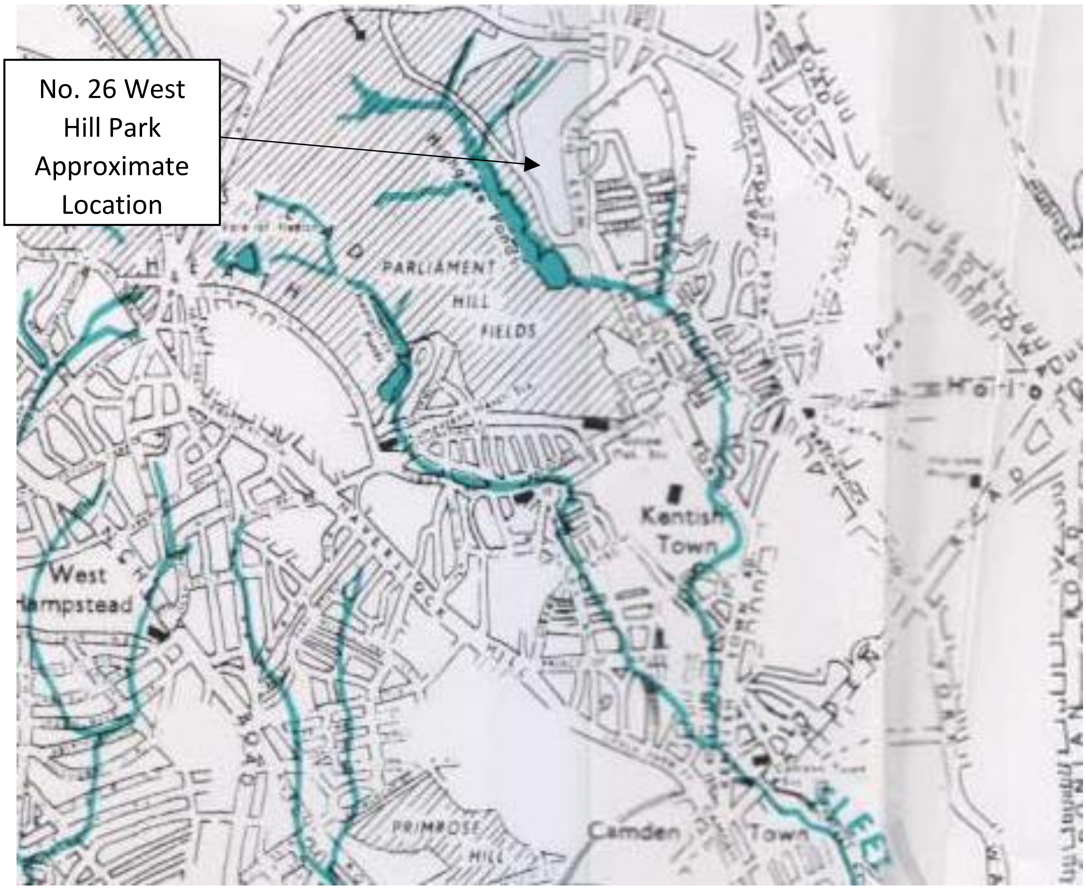

Figure 3. Location of River Fleet relative to 26 West Hill Park (Extract from map posted on londonbygaslight.wordpress.com)

3.3.4 Figure 2 'LB Camden Surface Waterbodies' of the London Borough of Camden Strategic Flood Risk Assessment (SFRA) by URS (2014) shows the now culverted River Fleet at the bottom of the Highgate Pond Chain, flowing south east through the borough. Figure 4 below shows an extract of Figure 2.

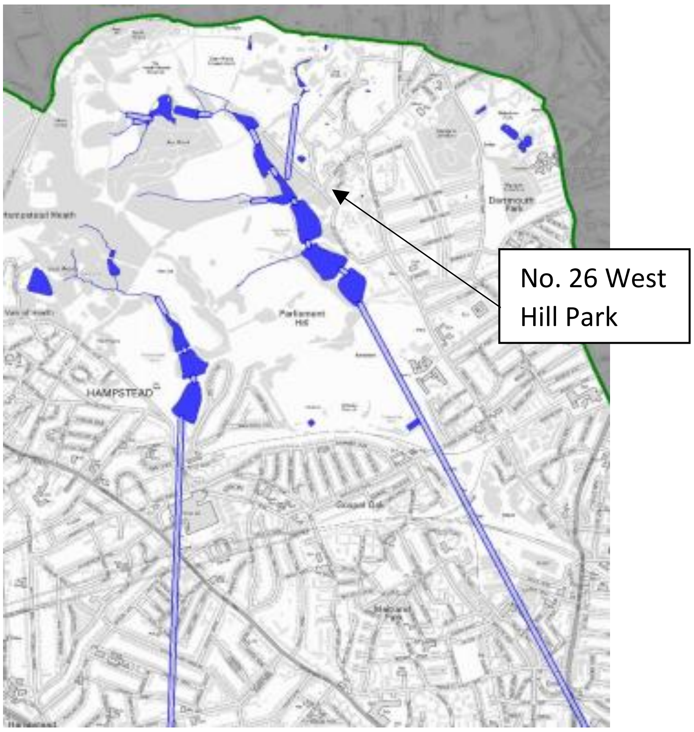

Figure 4. Extract from Figure 2 of the Camden SFRA, URS (2014) showing the culverted River Fleet from both the Hampstead (left) and Highgate (right) pond chains

3.4 <u>Flood Risk</u>

3.4.1 The Environment Agency (EA) website shows that the property lies within flood risk Zone 1 which is defined as areas where flooding from rivers and the sea is very unlikely, with less than a 0.1 per cent (1 in 1000) chance of such flooding occurring each year.

3.4.2 The Gov.uk website also identifies the area as being at a very low risk of flooding. The flood risk from surface water is presented in Figure 5 below; the property is identified as being at very low risk. In addition the maximum extent of flooding from reservoirs is presented in Figure 6 below.

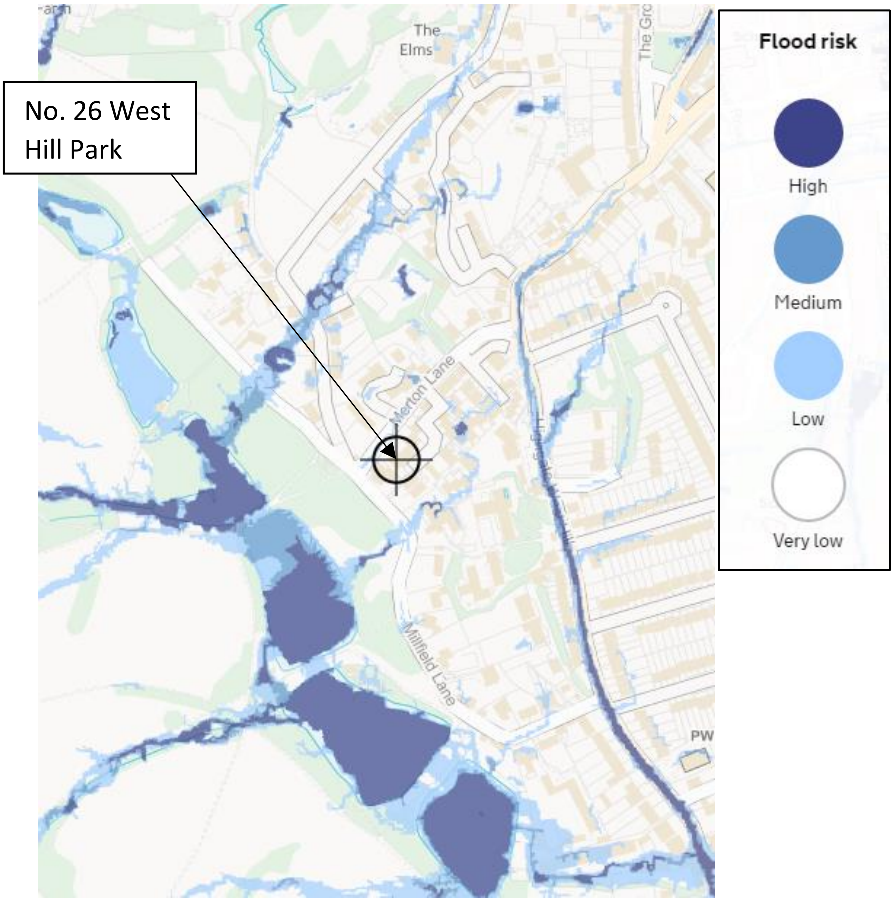

Figure 5. Flood Risk from Surface Water (Contains public sector information licensed under the Open Government Licence v3.0)

Figure 6. Flood Risk from Reservoirs (Contains public sector information licensed under the Open Government Licence v3.0)

3.4.3 Figure 14 of 'Hampstead Heath Surface Water Catchments and Drainage' from the Camden Geological, Hydrogeological and Hydrological Study (GHHS) by Arup (2010) shows the site lies within the Highgate Chain Catchment. Figure 7 below shows the extent of the catchment in relation to the site.

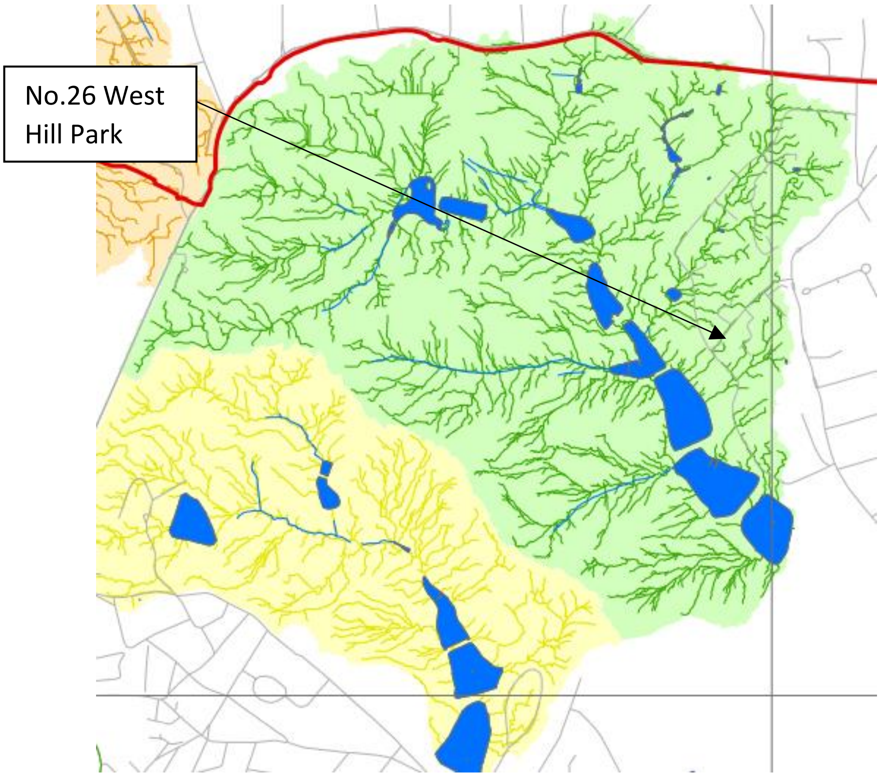

Figure 7. Extract from Figure 14 of the Camden GHHS, Arup (2010) showing Highgate Chain Catchment in green

3.4.4 Figure 15 of 'Surface Water Flood Risk Potential' from the Camden GHHS does not show any historic flooding on West Hill Park in either the 1975 or 2002 floods. Figure 8 below shows the extent of surface water flooding across most of the borough in both the 1975 and 2002 flood events and the potential at risk of surface water flooding.

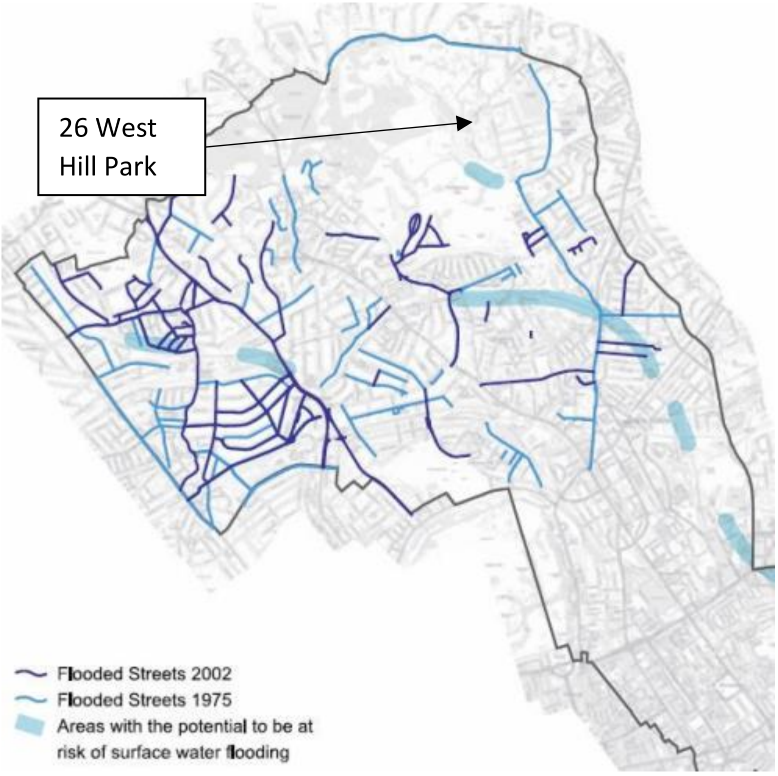

Figure 8. Surface Water Flood Risk Potential (Camden Geological, Hydrogeological and Hydrological Study, Arup (November 2010))

3.4.5 Figure 5a of the London Borough of Camden Strategic Flood Risk Assessment (SFRA) by URS (2014) shows that the site is not in an area affected by internal sewer flooding and Figure 5b shows the site is not within an area affected by external sewer flooding. Figures 9 & 10 below show extracts of figures 5a & 5b of the SFRA respectively.

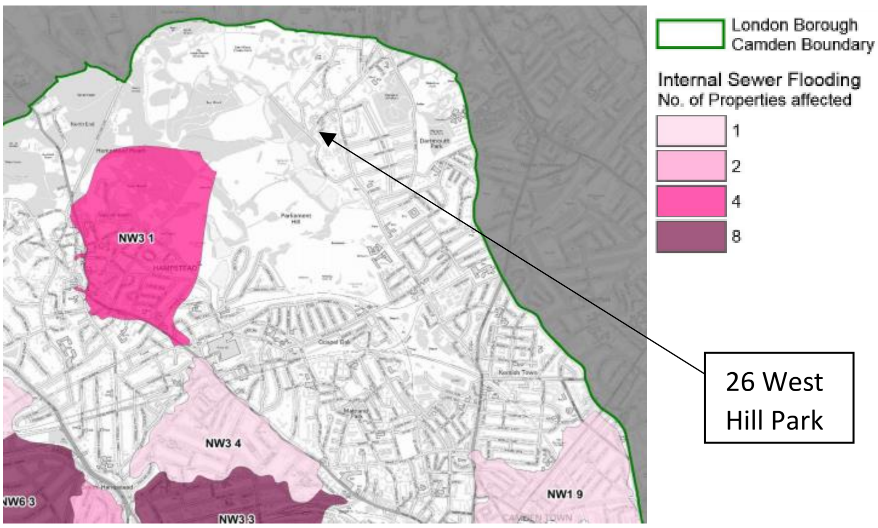

Figure 9. Extract of DG5 Internal Sewer Flooding (Camden Strategic Flood Risk Assessment (SFRA), URS (2014))

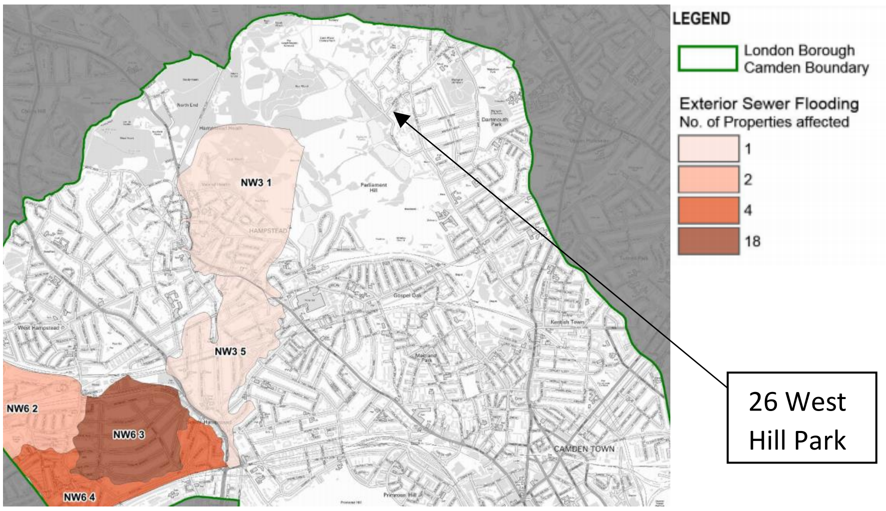

Figure 10. Extract of DG5 ExternalSewer Flooding (Camden Strategic Flood Risk Assessment (SFRA), URS (2014))

3.4.6 Figure 6 of the London Borough of Camden SFRA shows that the site is within Critical Drainage Area Group3_001.

3.5 <u>Geological Setting (Ground Conditions)</u>

3.5.1 Mapping by the British Geological Survey (BGS) indicates that the site is underlain by the Claygate Member, with no overlying superficial deposits recorded. The BGS geological plan showing the site is presented in Figure 11 below. The BGS indicates the London Clay outcrops within 1km of the site from the east through to the north west. At its closest, the London Clay outcrops approximately 41m southwest of the site according to the Groundsure Report (Appendix E). Similarly the Bagshot Formation is shown to outcrop approximately 238m north east of the site.

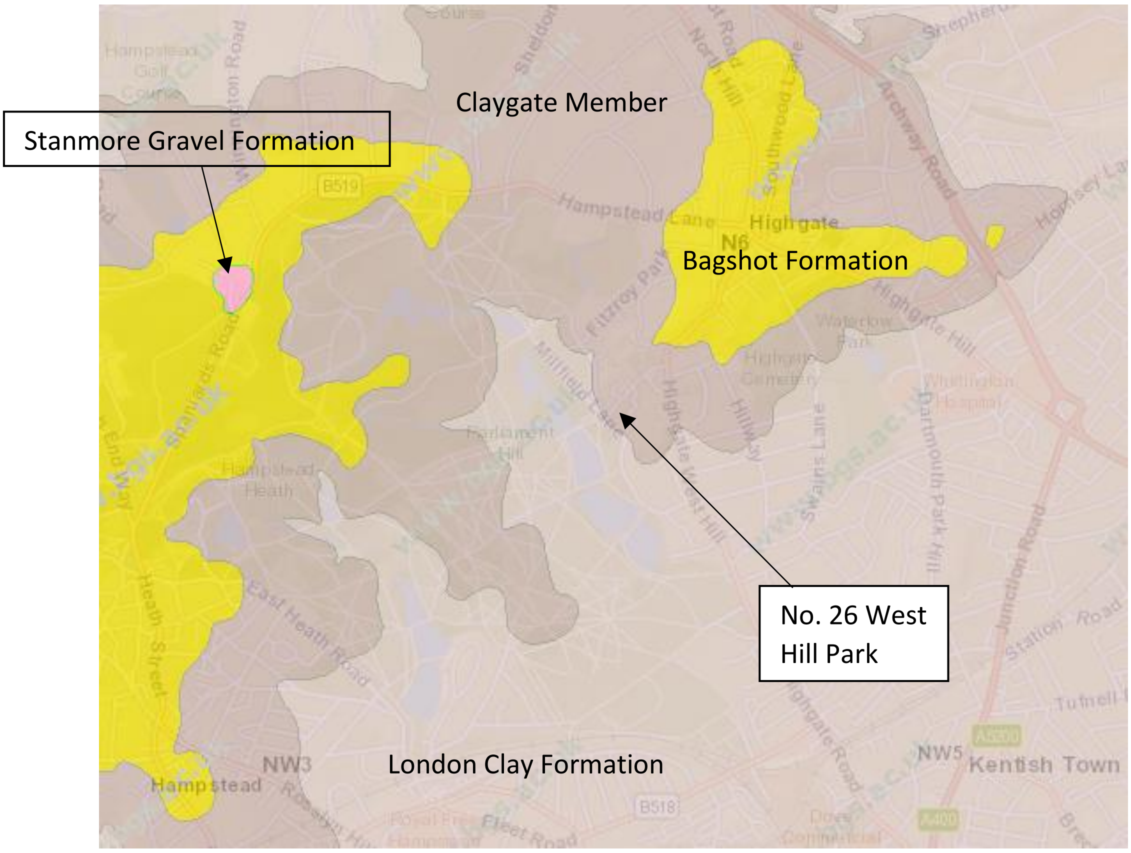

Figure 11. Site BGS Geological Plan (Contains British Geological Survey materials © NERC 2016. Base mapping is provided by ESRI)

3.5.2 The Claygate Member is a sedimentary bedrock formed approximately 34 to 56 million years ago in the Palaeogene Period. It comprises dark grey clays with sand laminae, passing up into thin alternations of clays, silts and fine-grained sand, with beds of bioturbated silt. Ferruginous concretions and septarian nodules occur in places. These rocks were formed in shallow seas with mainly siliciclastic sediments (comprising of fragments or clasts of silicate minerals) deposited as mud, silt, sand and gravel.

3.5.3 Figure 17 'Land Stability: Areas of Significant Landslide Potential (BGS)' from the Camden GHHS (Arup, 2010) shows an area of significant landslide potential in the area where the Claygate Member outcrops, therefore at the site. Figure 12 below shows this area relative to the site. However, the Groundsure Report states that there are no records of landslip within 500m of the site boundary and the hazard rating for the site is very low.

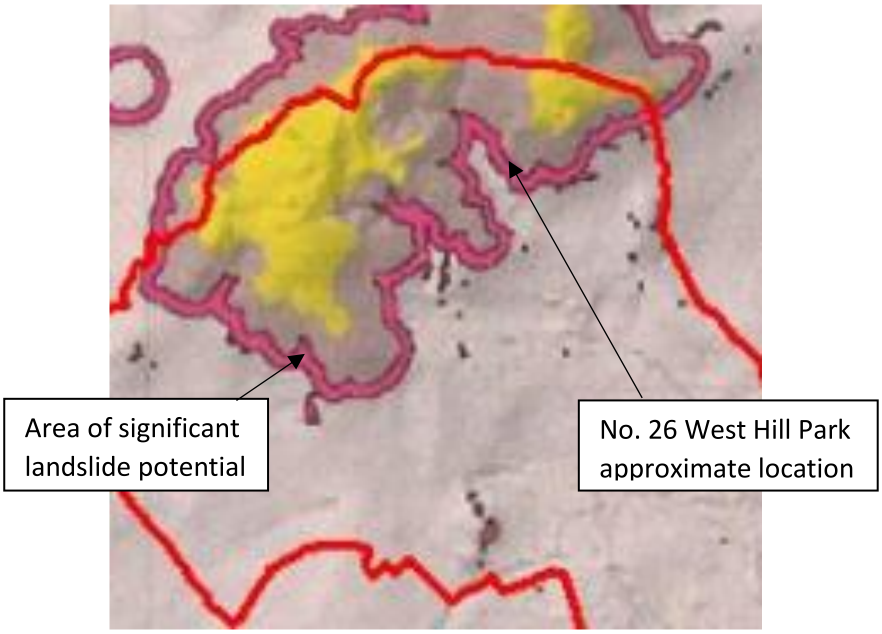

Figure 12. Extract from Figure 17 Areas of Significant Landslide Potential (BGS) of the Camden GHHS, Arup (2010)

3.5.4 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 approximate locations of which are presented on the location plan in Figure 13 below. The borehole logs are presented in Appendix D.

3.5.5 Eight BGS boreholes were reviewed, with the deepest borehole extending to 206m bgl. Some boreholes showed a thin stratum of Made Ground/Fill/Topsoil to a maximum depth of 2.6m bgl. Within the five shallow boreholes at location TQ28NE42 the Made Ground was underlain by Claygate Member, or this was encountered from ground level, to a maximum depth of 5.6m bgl. The Claygate Member was found to comprise stiff brown mottled sandy clay. In one of these shallow boreholes the Claygate Member was found to be underlain by the London Clay Formation. The other three boreholes encountered London Clay directly below the Made Ground/Fill. The London Clay comprised stiff to very stiff fissured brown and blue silty clay and was recorded to a maximum depth of 129m bgl and was underlain by the Thanet Sands. The Thanet Sands were recorded to a maximum depth of 147m bgl. The Thanet Sands were underlain by Chalk to the maximum borehole depth of 206m bgl.

Groundwater levels recorded in the boreholes are detailed in Section 3.6.3.

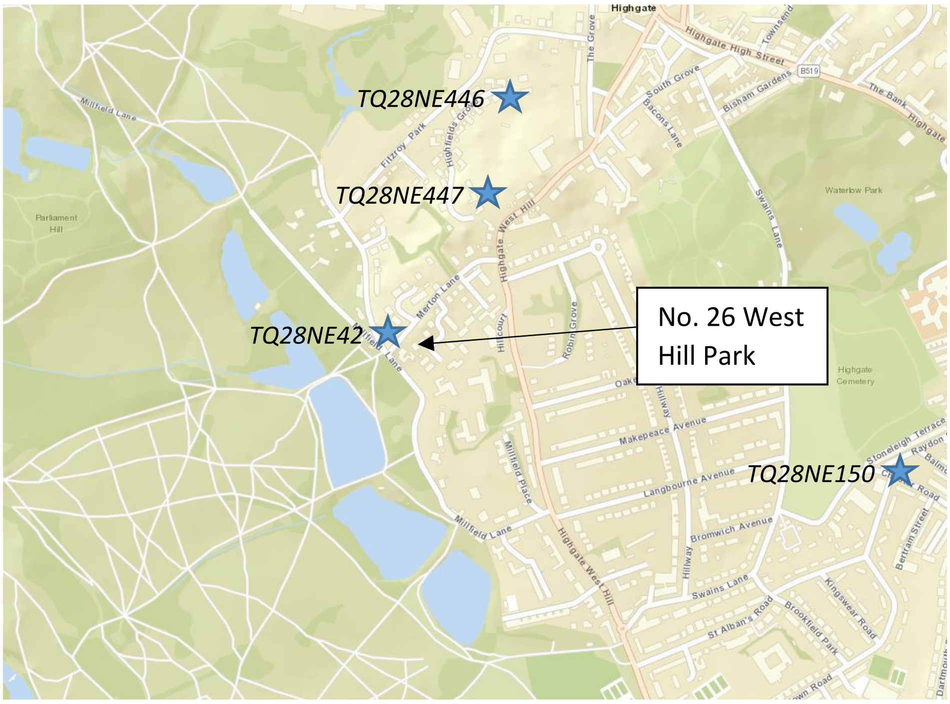

Figure 13. BGS Borehole Locations (Contains British Geological Survey materials © NERC 2016. Base mapping is provided by ESRI)

3.5.6 The ground Investigation completed by Chelmer (2017) comprised two continuous flight auger (c.f.a.) boreholes (BH1 & BH2) to 10.10 m bgl, to the west (rear) and east (front) of the existing building respectively, and two hand excavated trial pits (TP1 & TP2) to examine the current foundations of the property. The ground investigation indicated that the Claygate Member was present beneath Made Ground (or concrete in TP1) at depths of between 0.45m and 0.90m bgl. The Claygate Member generally consisted of a firm to very stiff brown becoming grey sandy silty CLAY. In TP1 the Claygate Member was recorded as firm thinly interlaminated silty CLAY and fine SAND to the base of the trial pit. The base of the Claygate Member was not proven at the maximum drilling depth of 10.10m bgl. Table 1 below presents a summary of the ground conditions encountered and the borehole and trial pit records are presented within the Factual Report in Appendix F.

Table 1: Summary of Ground Conditions Encountered		
Depth to top of stratum (m bgl)	**Depth to base of stratum (m bgl)**	**Description**
0.00	0.04/0.40	Paving / Concrete
0.00/0.40	0.35/0.90	Made Ground
0.35	0.45	Concrete (TP1)
0.45	0.65+	Claygate Member: *firm thinly interlaminated grey silty CLAY and orange fine SAND (TP1)*
0.80/0.90	4.80/6.00	Claygate Member: *firm to stiff brown sandy silty CLAY*
4.80/6.00	10.10+	Claygate Member: *stiff/very stiff dark grey sandy silty CLAY*

3.6 Hydrogeological Setting (Groundwater)

3.6.1 The Groundsure Report (see Appendix E) indicates that the Claygate Member which the property is situated on is classified as being a 'Secondary A' aquifer. The nearby London Clay Formation is classified as being 'Unproductive'.

3.6.2 Additional hydrogeological data obtained from the Groundsure Report, includes:

- There are no groundwater abstraction licences within 2000 m of the site.
- No Source Protection Zones (SPZs) have been identified within 500 m of the site.
- There are BGS groundwater flooding susceptibility areas within 50 m of the site relating to Clearwater flooding (associated with unconfined aquifers) with a limited potential for groundwater flooding to occur.

3.6.3 Groundwater information recovered from the BGS boreholes near the site (Figure 13) are detailed in Table 2 below.

Table 2: Summary of Groundwater Records from BGS Boreholes		
Location	**Date**	**Groundwater Standing Level (m bgl)**
TQ28NE42	*Not Specified*	*No Groundwater Record*
TQ28NE447	2014	138m*
TQ28NE446	2014	146m*
TQ28NE150	*Not Specified*	*'No groundwater encountered during boring'*

Notes: * These very deep groundwater levels are thought to be associated with the lower aquifer (Thanet Sand/Chalk) given the nature of casing within these boreholes.

3.6.4 During the drilling process of the ground investigation performed by Chelmer (2017), a groundwater seepage was recorded in BH2 at a depth of 6.8m bgl and a groundwater strike was recorded in BH1 at a depth of 7.0m bgl. Monitoring standpipes were installed to 10.0 m bgl within both boreholes and three return monitoring visits have been completed on the 15th and 22nd March and 12th April 2017. During the monitoring visits groundwater was recorded in BH1 at depths of 3.40m and 3.44m bgl and in BH2 at depths of 1.74m, 1.72m and 1.80m bgl.

3.6.5 Figure 4e 'Increased Susceptibility to Elevated Groundwater' of the London Borough of Camden Strategic Flood Risk Assessment (SFRA) does not show any Environment Agency groundwater flood incidents within 1 km of the site.

4.0 CONCEPTUAL SITE MODEL

4.1 *Basis of Conceptual Site Model*

4.1.1 The Conceptual Site Model has been built using desk study evidence together with the ground investigation findings, as outlined in Section 3 of this report. The ground investigation was completed on 2nd March 2017 (Appendix F).

4.1.2 The Impact Assessments contained in the sections below are based on the Screening Assessment in Appendix A and any concerns identified in Sections 2.0 and 3.0.

4.1.3 The Conceptual Site Model can be summarised as:

- The proposed basement excavation is to between 2.3m and 3.8m bgl for the rear basement and between 1.7m and 4.0m bgl for the front basement.
- The surrounding land slopes down to the south west approximately 6-7°.
- The nearest surface water feature identified is 122m south west of the site, with a further five features within 250m of the site, thought to be associated with the Highgate Pond Chain. The nearest well was identified as being located approximately 880m to the south east of the property.
- The site is an area where flooding from rivers and seas is reported as very unlikely, and the flood risk from surface water is reported to be very low.
- Ground conditions comprise, below a layer of Made Ground (maximum 0.9 m thick), firm to very stiff brown becoming grey sandy silty clay of the Claygate Member to the base of the boreholes drilled to 10.10 m depth.
- The site is located above a 'Secondary A' aquifer, formed by the Claygate Member.
- A groundwater seepage was recorded in BH2 at a depth of 6.8m bgl and a groundwater strike was recorded in BH1 at a depth of 7.0m bgl during the drilling process of the current investigation. During the three monitoring visits groundwater was recorded in BH1 at depths of 3.40m and 3.44m bgl and in BH2 at depths of 1.74m, 1.72m and 1.80m bgl.

4.2 *Groundwater Flow Impact Assessment*

4.2.1 The site is located above a 'Secondary A' aquifer comprising the Claygate Member. During the drilling process of the ground investigation performed by Chelmer (2017), a groundwater seepage was recorded in BH2 at a depth of 6.8m bgl and a groundwater strike was recorded in BH1 at a depth of 7.0m bgl. Monitoring standpipes were installed to 10.0 m bgl within both boreholes and three return monitoring visits have been completed on the 15th and 22nd March and 12th April 2017. During the monitoring visits groundwater was recorded in BH1 at depths of 3.40m and 3.44m bgl and in BH2 at depths of 1.74m, 1.72m and 1.80m bgl.

4.2.2 The permeability within the Claygate Member can vary significantly depending on the proportion of granular material present at a site. The sandy silty clay recorded over the depth of the boreholes is expected to have low permeability; the interlaminated clay and sand recorded in TP1 would be anticipated to have a higher permeability than the deeper Claygate Member;

however, this material is still indicated as being of low permeability. This hydrogeological regime (ie: groundwater levels and pressures) will be affected by long-term climatic variations as well as seasonal fluctuations and other man-induced influences, all of which must be taken into account when selecting a design water level for the permanent works. No long term, multi-seasonal groundwater monitoring data are available so a conservative approach will be needed, as required by current geotechnical design standards.

4.2.3 The proposed basement level will be founded within the Claygate Member. The monitoring performed in the on-site boreholes (BH1 & BH2) encountered groundwater up to 2.28 m above the founding level of the proposed basement. However, the anticipated low permeability of the ground indicates there is likely to be little or no natural groundwater flow. In the event that the interlaminated material in TP1 is more widespread, groundwater flows would be anticipated to flow around the basement. Thus, the proposed basement is not anticipated to have any impact on the groundwater flows/levels. Therefore, there would be no significant impact on neighbouring properties.

4.2.4 The basement will be excavated below groundwater level. The soils above formation level are indicated to be of low permeability, but groundwater seepages and localised inflows may be encountered. In ground conditions such as indicated at the site sump pumping is commonly used. Care should be taken to avoid water ponding on exposed clay surfaces at formation level.

4.3 Ground Stability Impact Assessment

4.3.1 The site is located on a south westerly slope with a slope gradient of approximately 6-7°. The slope across the site is approximately 10° and the boundaries with No's 23 & 25 Merton Lane and the Merton Lane carriageway consisted of retaining walls with ground level differences of approximately 1.5-2.5m and 1.0-1.5m respectively. The interlaminated clay and sand recorded in TP1 could be present across other areas of the site, which would be expected to have a higher permeability. Therefore, slope instability could cause a potential problem. The upslope perimeter basement walls must be designed to protect against this potential slope instability. This issue will be required to be assessed within the structural design.

4.3.2 Neighbouring properties could be affected by the excavation and construction of the proposed basement. This issue is addressed in the Damage Category Assessment section (Section 6.0) of this report.

4.3.3 The Groundsure Report (Appendix E) states there is a moderate hazard for shrink-swell clays at the property location. During the site inspection visit the property appeared to be in a good state of repair, however a few of the walls around the site boundary had some cracking.

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

- the geology;

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

4.3.5 A high quality of workmanship and use of best practice methods of temporary support are therefore crucial to the satisfactory control of ground movements alongside basement excavations. All cracks in load-bearing walls which have weakened their structural integrity should be fully repaired in accordance with recommendations from the appointed structural engineer before excavations for the underpinning works begin.

4.3.6 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.

4.3.7 Soil parameters, including the bearing capacity of the Claygate Member, are detailed in the Chelmer Geo-environmental Interpretative Report, ref. GENV/8522, dated May 2017.

5.0 GROUND MOVEMENT ANALYSIS

5.1 *Basement Geometry and Stresses*

5.1.1 Analyses of vertical ground movements (heave or settlement) arising from changes in vertical stresses caused by excavation of the basement have been undertaken using proprietary software (Oasys PDISP™). The analysis is based on Boussinesq's theory of analysis for calculating stresses and strains in soils due to vertically applied loads; the predicted ground movements are derived by integration of vertical strains derived from Boussinesq's equations. These preliminary analyses have not modelled the horizontal forces on the retaining walls, and so have simplified the stress regime significantly. In addition, consistent with Boussinesq theory, the soils are assumed to comprise semi-infinite isotropically homogeneous elastic medium.

5.1.2 The layout of the basement used within the analysis is based on Drawing 02 – "Proposed Lower Ground Floor Plan" provided by London Development & Construction, and is presented in Figure 14 below. The proposed development involves the construction of two detached basements, modelled in PDISP as two separate projects, identified by "Model1" and "Model2" label. The main basement (Model1), located under the north side of the existing building, is approximately 19.5 m long by 8.6 m wide with excavation generally extending to a depth included between 2.3m and 3.3 m below existing sloping ground level (bgl) for the living areas and to a depth of 3.8m bgl for the swimming pool foundation slab. The foundation level is anticipated to be set at approximately 87.7 m AOD for the living area and 86.2 m AOD for the swimming pool. The second basement (Model2), approximately 6.70 m long by 5.0 m wide, is located under the southeast side of the building, with excavation generally extending to a depth of approximately 4.0 m below existing ground level (bgl). The foundation level is anticipated to be set at approximately 87.0 m AOD. The basements are understood to be constructed by RC underpins and retaining walls as detailed in Section 2.5. The temporary works will include a phase of battering back the soil to a safe angle before construction of the RC walls, with the slope backfilled with compacted inert fill after RC wall construction.

5.1.3 The excavation depths for the basements have been modelled using Drawing 161206-SD-11 provided by Croft Structural Engineers to estimate the gross pressure reductions (unloading) across the development. Figure 15 below illustrates the layout of all load zones, positive and negative (unloading), used to model the proposed basements in PDISP. These include the excavation and loads on the underpins, the self-weight of walls, and construction of the concrete slab and excavation of central area from existing ground level.

5.1.4 The table in Appendix G presents the net changes in vertical pressure for each load zone for the four major stages in the sequence of stress changes which will result from excavation and construction of the basement (see 5.3.1 below for details).

Figure 14. Layout of the proposed basement plan

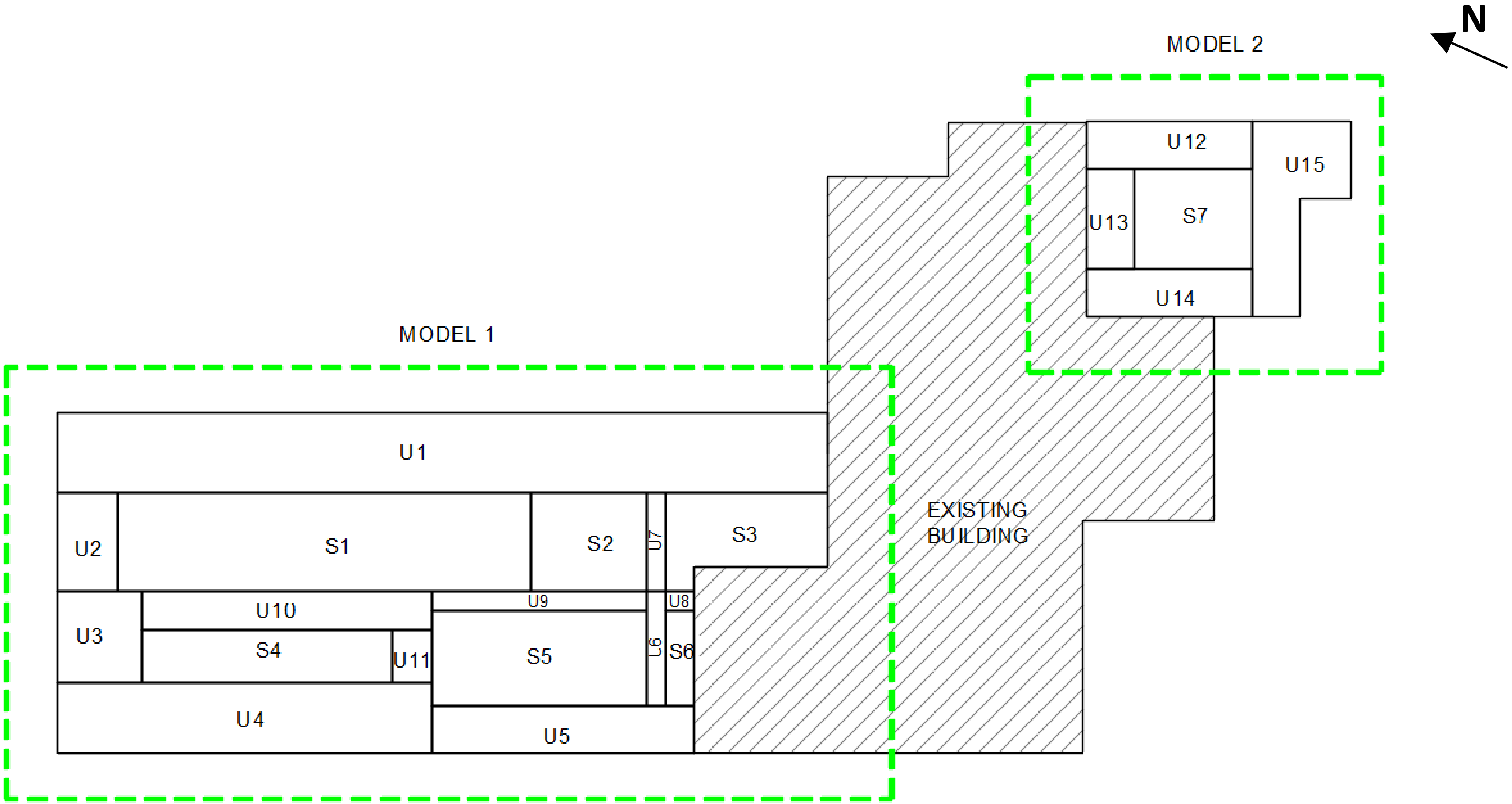

Figure 15. Detail of geometry introduced to PDISP
[U = Underpinning/retaining wall excavation and loads, S = Bulk excavation and slab loads]

5.2 Ground Conditions

The short-term and long-term geotechnical properties used in the analysis are summarised in Table 3 below. These were based on the Chelmer (2017) ground investigations, and on data from previous Chelmer projects in similar ground conditions. All Made Ground will be excavated and therefore only the change in vertical pressure, due to its excavation, is required for the PDISP analyses. Geotechnical parameters for the Made Ground are not used in the analysis.

Table 3 - Soil parameters for PDISP analyses

Strata	**Depth** *(m bgl)*	**Short-term, undrained Young's Modulus, E_u** *(MPa)*	**Long-term, drained Young's Modulus, E'** *(MPa)*
Claygate Member	2.0	40.0	24.0
	4.0	52.0	31.2
	10.0	60.0	36.0

Undrained Young's Modulus, $E_u = 500 * C_u$
Drained Young's Modulus, $E' = 0.6 * E_u$

Where no C_u data are available:
Undrained Shear Strength, C_u has been estimated by extrapolation previous data.

A global Poissons ratio of 0.5 has been adopted for the London Clay Formation over its modelled thickness.

5.3 PDISP Analysis:

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

- Stage 1 – Construction of underpins and retaining walls – Short-term (undrained) condition
- Stage 2 – Bulk excavation of central area and construction of the basement slab – Short-term (undrained) conditions
- Stage 3 – Bulk excavation of central area and construction of the basement slab – Long-term (drained) conditions

5.3.2 The results of the analyses for Stages 1, 2, and 3 are presented as contour plots on Figures 16 to 24 respectively.

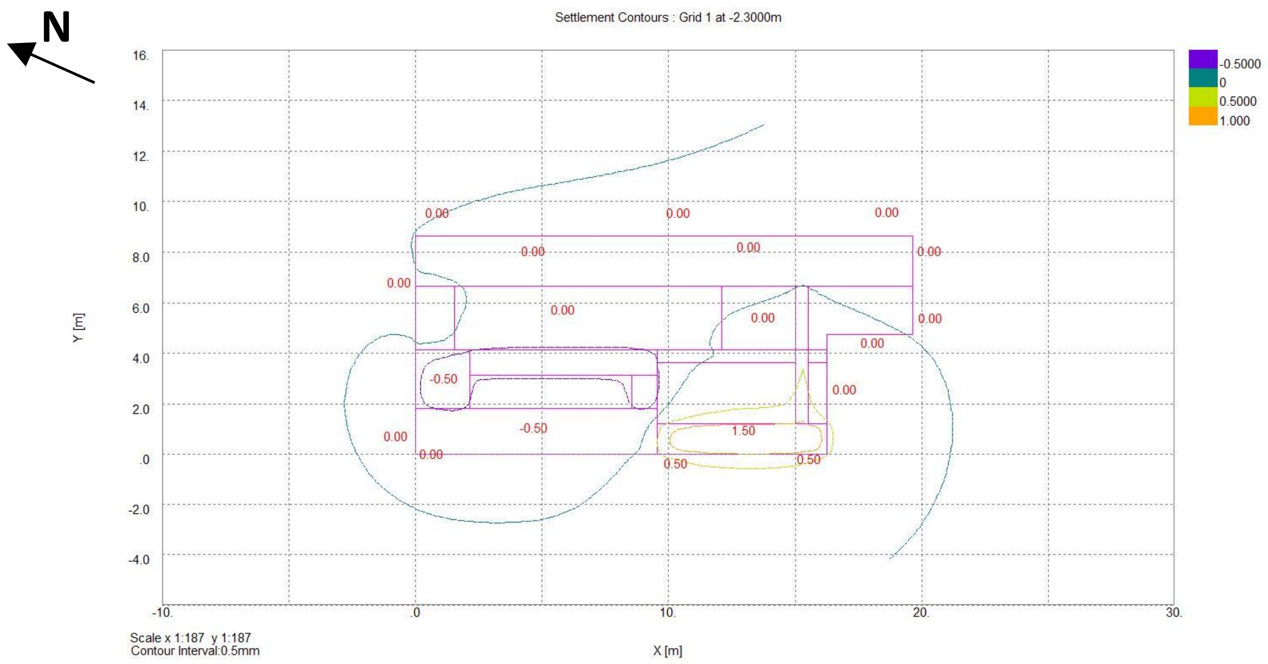

Figure 16. Stage 1 – Model 1 – Grid at 87.2m AOD - Construction of underpins and retaining walls – Short-term (undrained) condition (0.5mm settlement contours)

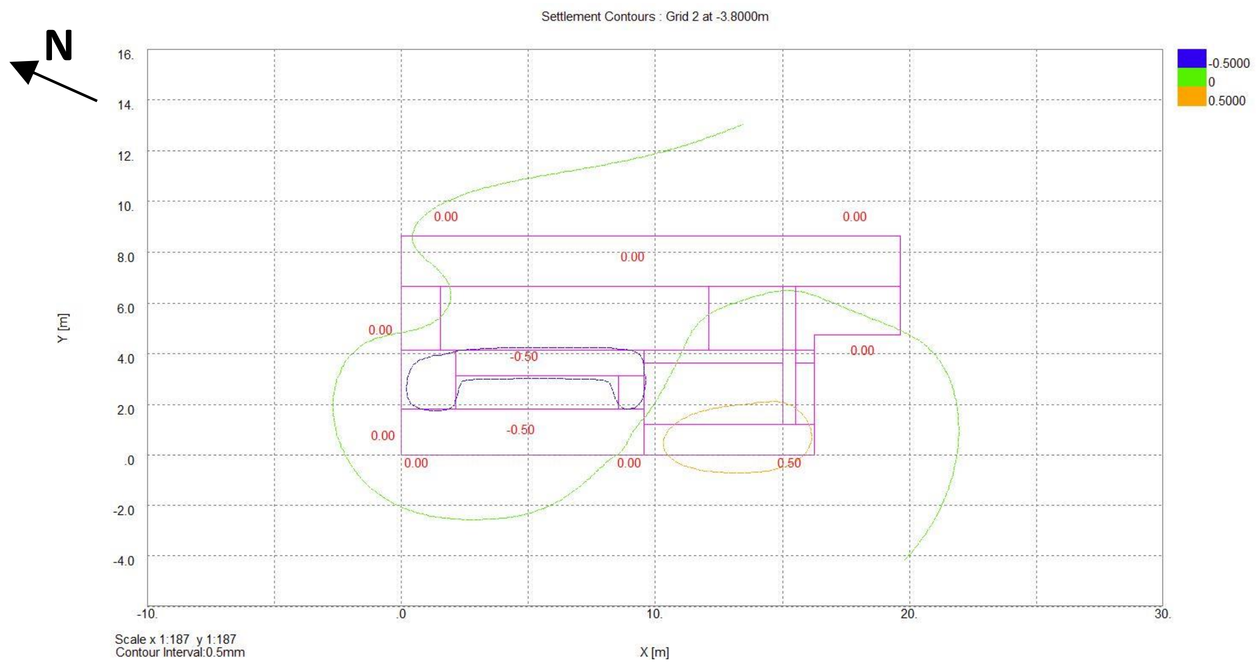

Figure 17. Stage 1 – Model 1 – Grid at 86.2m AOD - Construction of underpins and retaining walls – Short-term (undrained) condition (0.5mm settlement contours)

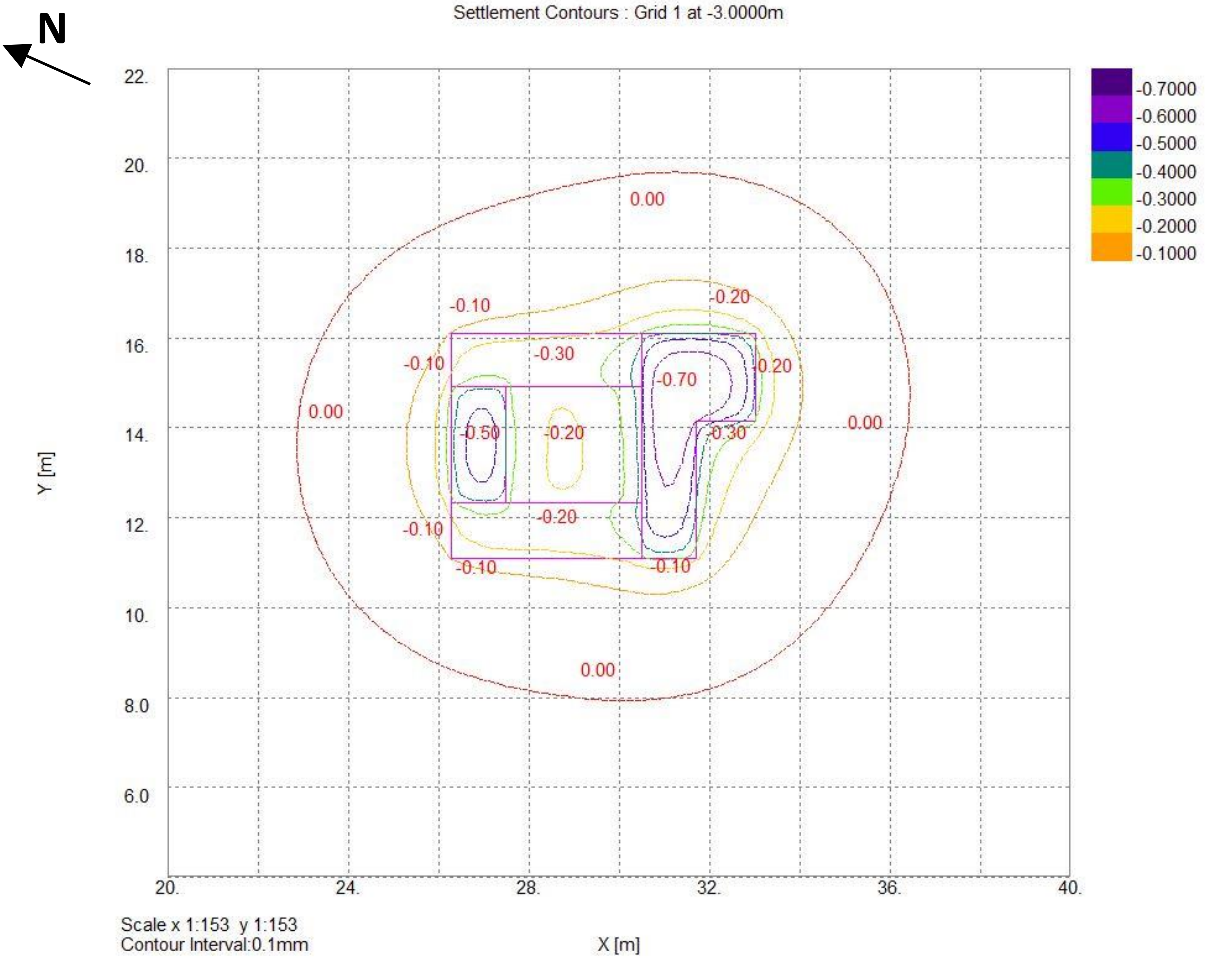

Figure 18. Stage 1 – Model 2 – Grid at 87.0m AOD - Construction of underpins and retaining walls – Short-term (undrained) condition (0.1mm settlement contours)

Figure 19. Stage 2 – Model 1 – Grid at 87.7m AOD - Bulk excavation of central area and construction of the basement slab – Short-term (undrained) conditions (1.0mm settlement contours)

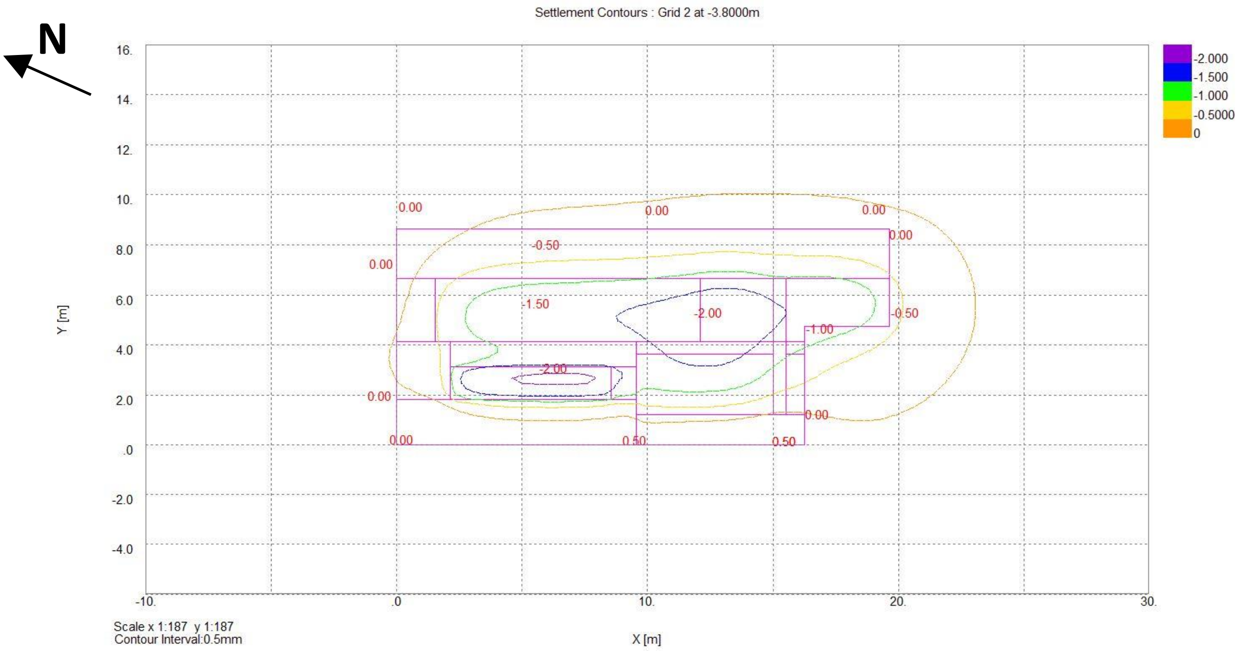

Figure 20 Stage 2 – Model 1 – Grid at 86.2m AOD - Bulk excavation of central area and construction of the basement slab – Short-term (undrained) conditions (1.0mm settlement contours)

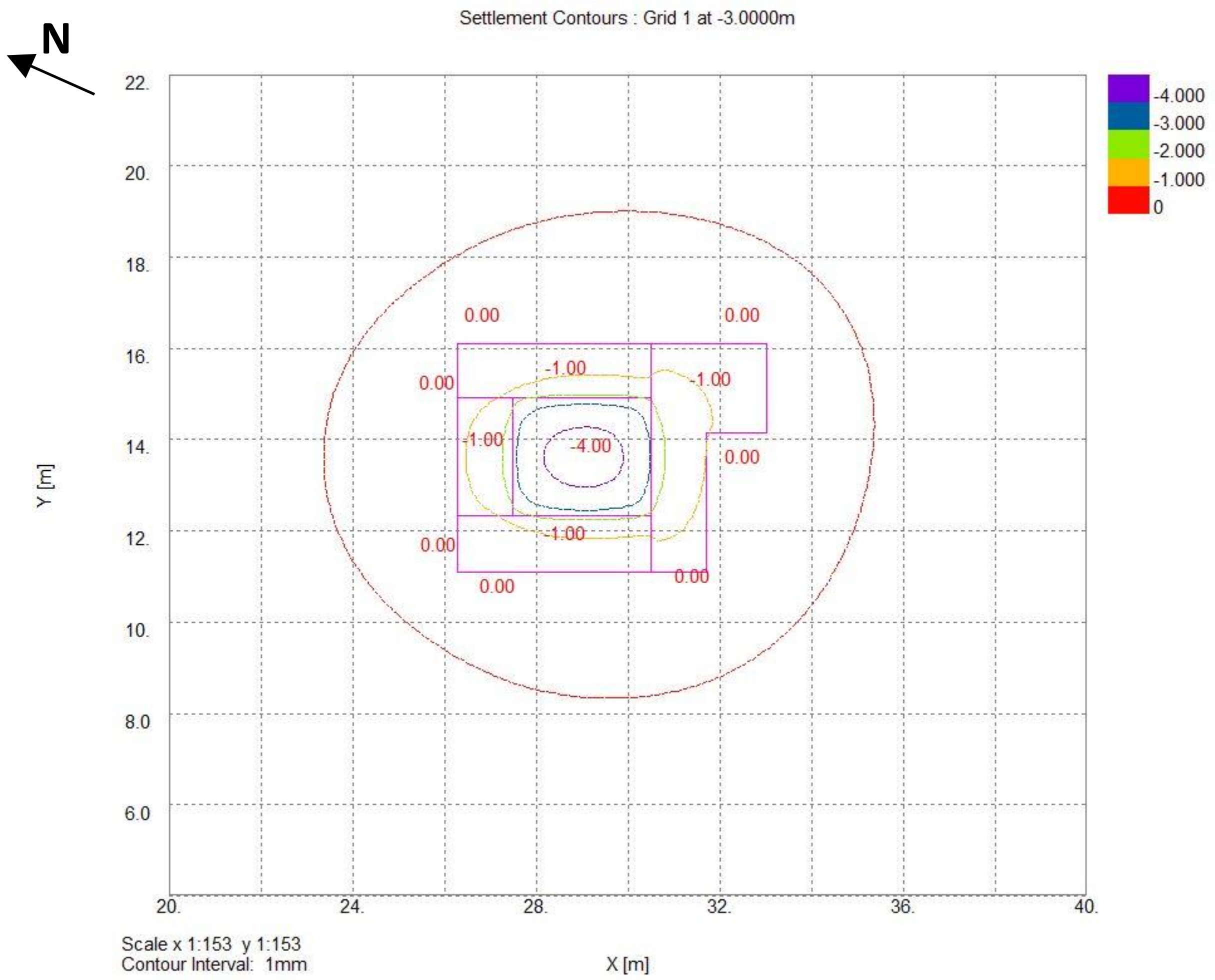

Figure 21. Stage 2 – Model 2 – Grid at 87.0m AOD - Bulk excavation of central area and construction of the basement slab – Short-term (undrained) conditions (1.0mm settlement contours)

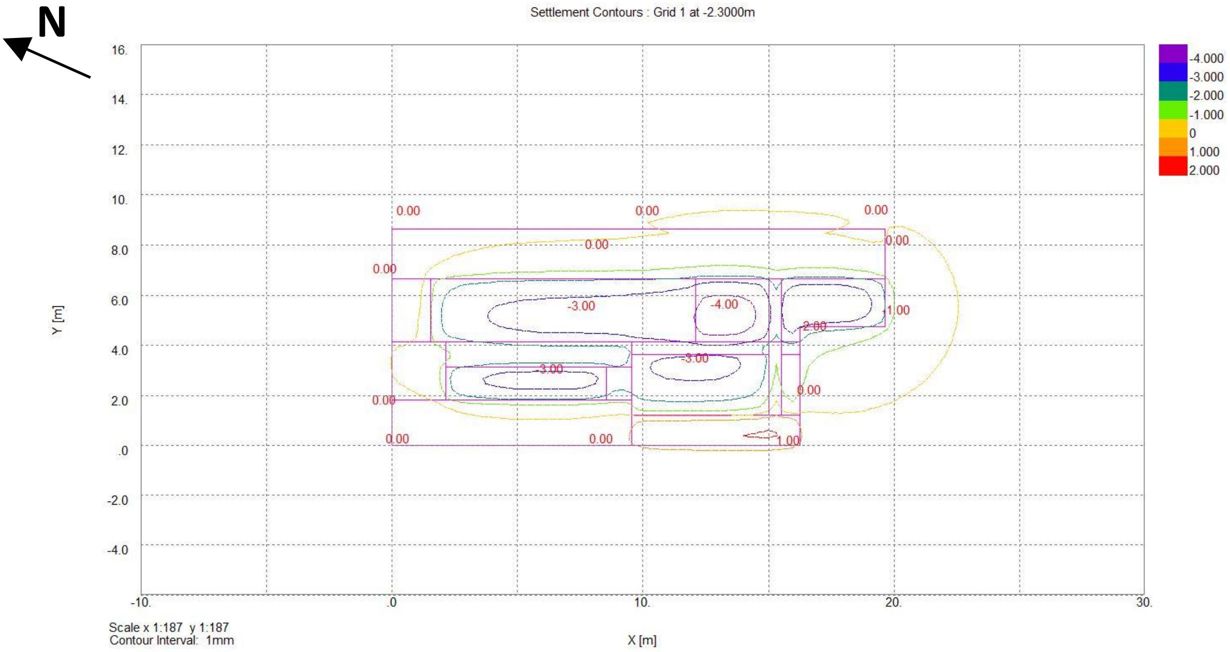

Figure 22. Stage 3 – Model 1 - Grid at 87.7m AOD - Bulk excavation of central area and construction of the basement slab – Long term (drained) conditions (1.0mm settlement contours)

Figure 23. Stage 3 – Model 1 - Grid at 86.2m AOD - Bulk excavation of central area and construction of the basement slab – Long term (drained) conditions (1.0mm settlement contours)

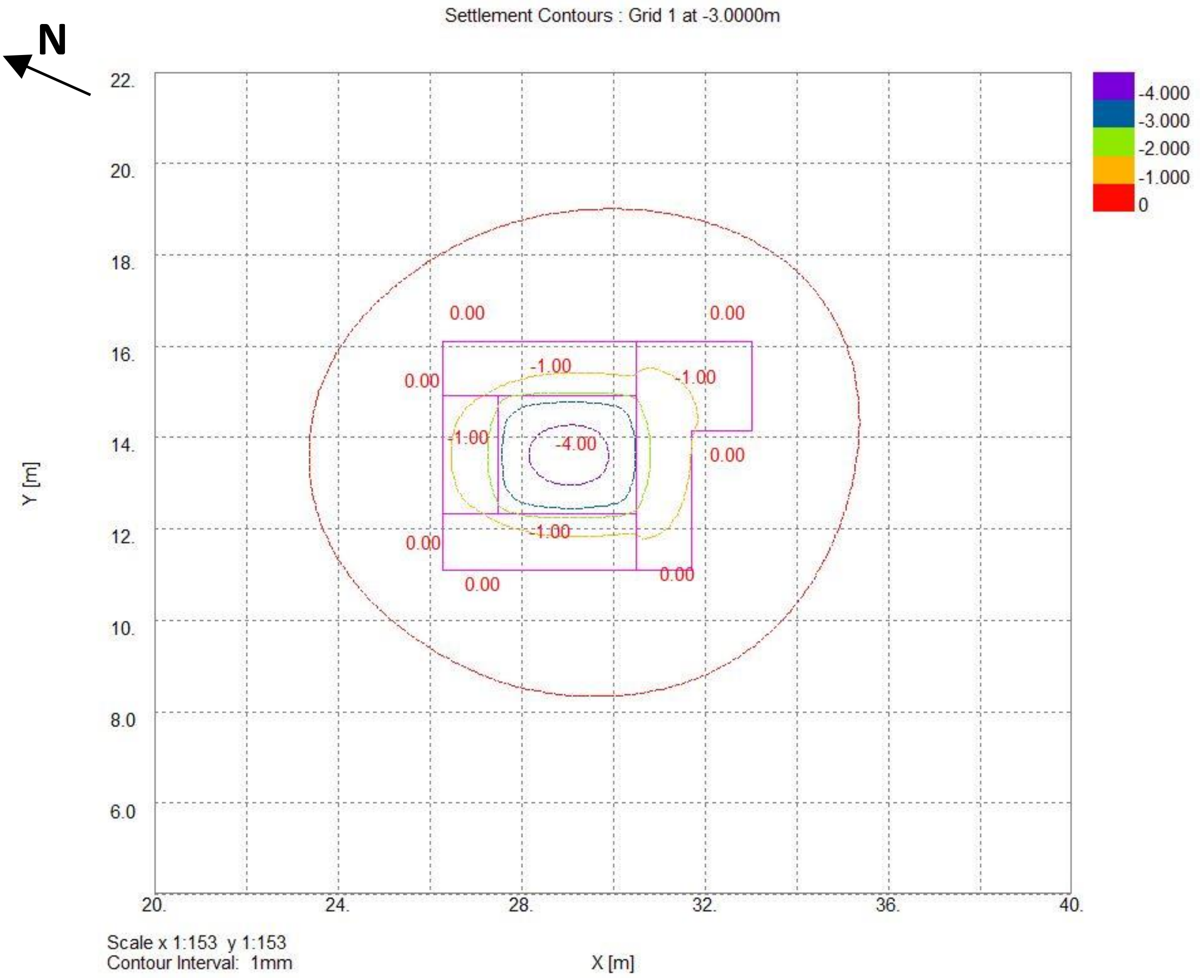

Figure 24. Stage 3 – Model 2 - Grid at 87.0m AOD - Bulk excavation of central area and construction of the basement slab – Long term (drained) conditions (1.0mm settlement contours)

5.4 Heave/Settlement Analysis

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

5.4.2 The ranges of predicted short-term and long-term movements for each of the main sections of the proposed basements are presented in Table 4a and Table 4b below. These analyses indicated that the perimeter basement walls are predicted to undergo movements ranging from 3 mm heave to 2 mm settlement. The basement slabs are predicted to undergo slightly greater displacements, from 1.0 mm to 4.0 mm heave. All values are approximate owing to the simplification of the stress regime and include only displacements caused by stress changes in the ground beneath the basement.

Table 4a: Summary of Predicted Ground Movements from PDISP – Model 1			
Location / Building Element	***Stage 1 (short term)***	***Stage 2 (short term)***	***Stage 3 (long term)***
Northern perimeter of basement	0.0 – 0.5 mm Heave	Negligible	0.0 – 1.0 mm Settlement
Eastern perimeter of basement	Negligible	0.0 – 0.5 Heave	0.0 – 1.0 mm Heave
Southern perimeter of basement	Negligible	0.0 – 1.0 mm Heave	2.0 mm Settlement to 3.0 Heave
Western perimeter of basement	1.5 mm Settlement to 0.5 Heave	0.0 – 1.0 mm Settlement	0.0 - 2.0 mm Settlement
Basement Slab at 2.30m bgl	---	1.0 – 3.0 mm Heave	0.0 – 4.0 mm Heave
Basement Slab at 3.80m bgl	---	1.0 – 2.0 mm Heave	1.0 – 3.0 mm Heave

Table 4b: Summary of Predicted Ground Movements from PDISP – Model 2			
Location / Building Element	***Stage 1 (short term)***	***Stage 2 (short term)***	***Stage 3 (long term)***
Northern perimeter of basement	0.0 – 0.5 mm Heave	0.0 – 1.0 mm Heave	0.0 – 1.0 mm Heave
Eastern perimeter of basement	0.3 – 0.7 mm Heave	0.0 – 0.5 mm Heave	0.0 – 1.0 mm Heave
Southern perimeter of basement	0.1 – 0.7 mm Heave	0.0 – 0.5 mm Heave	0.0 – 1.0 mm Heave
Western perimeter of basement	0.1 – 0.4 mm Heave	0.0 – 0.5 mm Heave	0.0 – 1.0 mm Heave
Basement Slab	---	1.0 – 2.5 mm Heave	1.0 – 4.0 mm Heave

5.4.3 All the short-term elastic displacements would have occurred before the basement slab is cast, so only the post-construction incremental heave/settlements (the difference from Stages 2, short-term, to 3, long-term) are relevant to the slab design.

6.0 DAMAGE CATEGORY ASSESSMENT

6.1 When underpinning it is inevitable that the ground will be un-supported or only partially supported for a short period during excavation of each pin, even when support is installed sequentially as the excavation progresses. This means that the behaviour of the ground will depend on the quality of workmanship and suitability of the methods used, so rigorous calculations of predicted ground movements are not practical. However, provided that the temporary support follows best practice, then extensive past experience has shown that the bulk movements of the ground alongside underpins for a single storey basement (of nominal depth 3.5 m) should not exceed 5 mm horizontally. This figure should be adjusted pro-rata for shallower or deeper basements.

6.2 In order to relate these predicted ground movements to possible damage which adjacent properties might suffer, it is necessary to consider the strains and the angular distortion (as a deflection ratio) which they might generate using the method proposed by Burland (2001, in CIRIA Special Publication 200, which developed earlier work by himself and others). A table displaying the classification of visible damage to walls and the relevant damage categories used in this assessment is provided in Appendix H.

6.3 No evidence has been found on the Camden's Council planning website that the adjoining properties (No. 25 West Hill Park and No's. 23 and 25 Merton Lane) have modern basements below them.

6.4 The different founding levels for the proposed basement and the sloping site means that the potentially critical locations will be determined by the displacements predicted by the PDISP analyses and the geometries and founding levels of the adjacent buildings. For these damage category assessments, we are interested in the ground movements at the foundation level of the neighbouring buildings, so it is the depth of the proposed excavation below foundation level of the neighbouring properties that must be considered.

6.5 The worst case scenarios for potential damage will be the west flank wall of No. 25 West Hill Park and the eastern wall of No. 23 Merton Lane. These walls are considered to have a higher potential for damage in comparison to others walls that make up the same structures due to the increased settlements predicted along them by PDISP, their geometries and proximities to the proposed excavation. The approximate distances and geometries are presented in Figure 25 below.

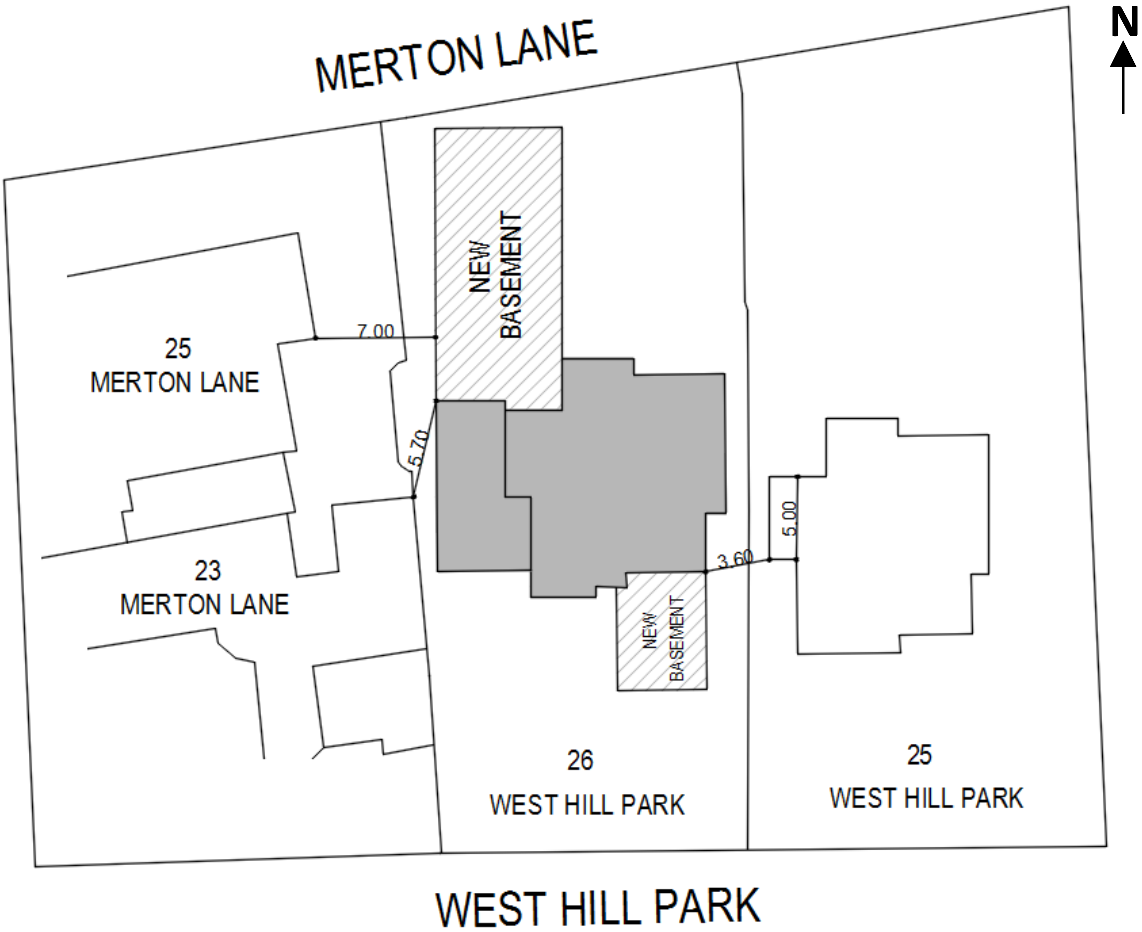

Figure 25. Approximate widths and distances of adjacent structures (Not to Scale)

6.6 The lateral extent of ground movements caused by relaxation of the ground alongside the basement excavation depends in part on whether the excavated soils are granular (mainly sands and gravels) or cohesive (clay). The ground investigation indicated that the excavation will predominantly be in the Claygate Member. Therefore, published data for ground movements associated with the construction of retaining walls in cohesive soils have been used for the damage category assessments.

6.7 The damage category assessments undertaken consider the following:

- ground movements arising from the vertical stress changes, as assessed by the PDISP analyses;
- ground movements alongside the proposed underpins and retaining walls caused by relaxation of the ground in response to the excavations.

Some ground movement is inevitable when basements are constructed. Ground movements associated with the construction of retaining walls in clay soils have been shown to extend to a distance up to 4 times the depth of the excavation, as detailed in Table 2.4 of CIRIA C580 (Gaba et al,, 2003).

6.8 For worst case 'low support stiffness' walls (which is appropriate to the underpinning construction method) the estimated vertical ground movements resulting from the excavation in front of the proposed basement wall would be as defined in Table 2.4 of CIRIA C580. This predicts a settlement 0.35% of the maximum excavation depth. Therefore, for a 4.0 m excavation (the conservative excavation depth for each assessed case) the total settlements immediately alongside the proposed basement walls due to the excavation of the soil would be 14.0 mm.

<u>Flank wall of No. 25 West Hill Park (WHP):</u>

6.9 The relevant geometries are as follows:

Depth of foundations = 0.35 m (estimated as similar to No 26 as identified by TP1)
Depth of excavation = 4.0 – 0.35 + 0.5 = 4.15 m (conservative assumption that No. 25 is founded 0.5m higher than No. 26, see Section 2.3)
Width of affected ground = 4.15 x 4 = 16.6 m

Distance from No 26 WHP = 3.6m
Width of No. 25 WHP = 5.0 m (estimated)
Affected width (L) = 5.0 m (furthest point of wall from the proposed basement due to their relative angles)

Height of No. 25 WHP (H) = 7.2 m (estimated the same height as No. 26 WHP) + 0.5 m (footing depth) = 7.7 m
Hence L/H = 0.6

6.10 Thus, for the predicted 5 mm maximum horizontal displacement (see Section 6.1) increased pro rata to 5.9mm, the horizontal strain beneath No. 26 WHP would, theoretically, be in the order of $\varepsilon_h = 3.55 \times 10^{-4}$ (0.036%).

6.11 The maximum settlement produced by the PDISP analysis beneath the location where the flank wall of the No. 25 WHP is closest to No. 26 WHP was in Stage 4 where no settlement was predicted. This must be added to the settlement profile presented in Figure 2.11(b) of CIRIA Report C580 for a worst case (low stiffness ground support) scenario, which is appropriate to the underpinning construction method.

6.12 The total predicted settlement (due to excavation) of 14.0 mm (see Section 6.8) is increased to 14.5 mm when the depth and the higher foundation level of No. 26 WHP's footings are taken into account. The total combined settlement of 14.5 mm, predicted by the CIRIA methods and no settlement predicted by PDISP, is detailed as the point immediately alongside the proposed basement (0 m) in Figure 26 below. Figure 26 presents the settlement curve from the basement wall to the maximum distance of affected ground, 16.6 m (see Section 6.9).

6.13 The deflection along the front walls of No. 25 WHP is calculated as the difference between the tangent of the relevant width of the affected wall (5.0 m) and the total predicted ground surface movements curve (from Figure 2.11(b) of CIRIA C580). For the low stiffness ground support case, settlement is convex and gives a maximum vertical deflection, $\Delta = 0.4$ mm as displayed in Figure 26 below, which represents a deflection ratio $\Delta/L = 0.8 \times 10^{-4}$ (0.008%).

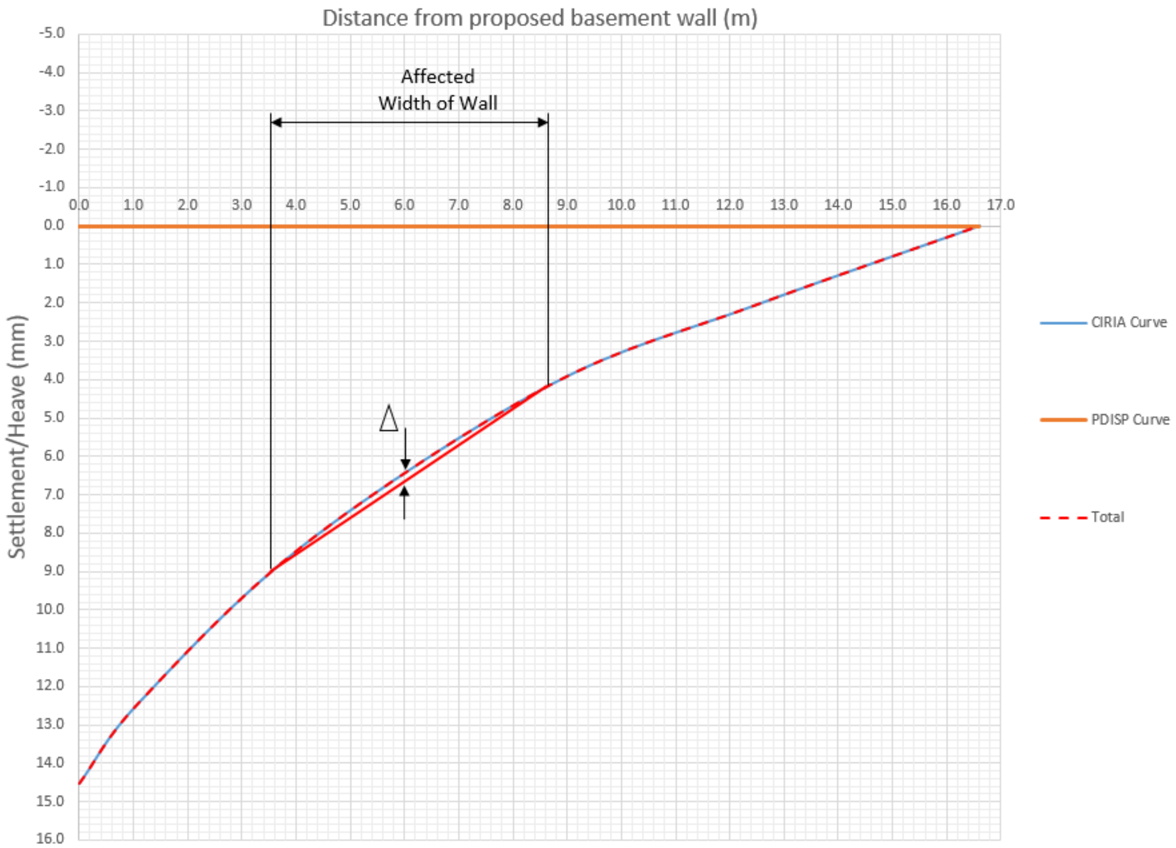

Figure 26. Combined displacements for No. 25 WHP flank wall due to excavation of proposed basement

6.14 Using the damage category ratings and graphs given in CIRIA SP200, for L/H = 0.5 (the closest value for the L/H of 0.6 defined in Section 6.9), these deformations represent a damage category of 'negligible' (Burland Category 0), as illustrated in Figure 27 below.

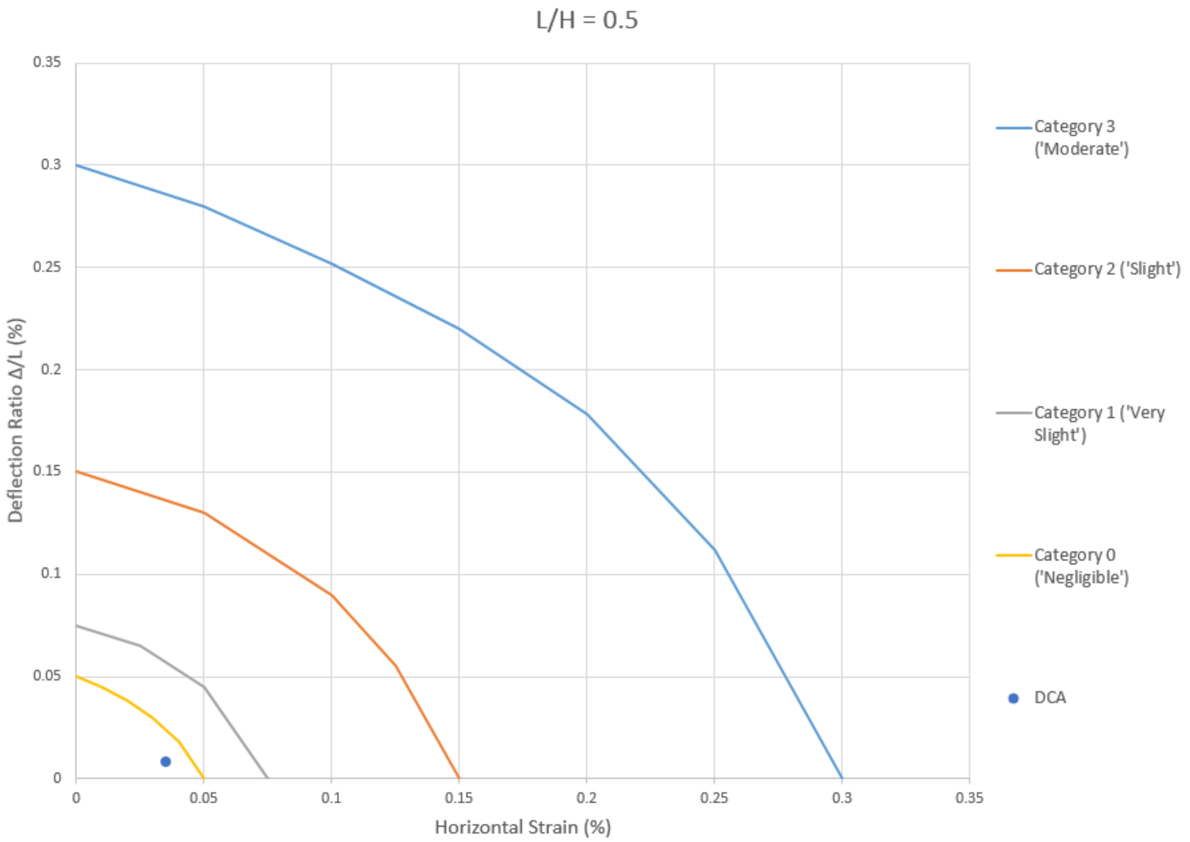

Figure 27: Damage category assessment for No. 25 WHP flank wall

Eastern wall of No. 23 Merton Lane:

6.15 The relevant geometries are as follows:

Depth of foundations = 0.5 m (conservative estimation)
Ground Level Difference = -3.2m (from the topographical survey)
Depth of excavation = 3.80 – 0.50 – 3.20 = 0.10 m
Width of affected ground = 0.1 x 4 = 0.4 m
Distance from No. 26 WHP = 5.7 m

Therefore, No 23 Merton Lane is not within the zone of influence of the proposed development due to its lower founding level.

6.16 Following the above conclusions (as section 6.15), No. 25 Merton Lane is also assumed not to be within the zone of influence of the proposed development given its anticipated founding level and the greater distance from the proposed basement excavations.

7.0 CONCLUSIONS

7.1 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.

7.2 The site is located on a 'Secondary A Aquifer' formed by the Claygate Member. A groundwater seepage was recorded in BH2 at a depth of 6.8m bgl and a groundwater strike was recorded in BH1 at a depth of 7.0m bgl during the drilling process of the current investigation. During the three monitoring visits groundwater was recorded in BH1 at depths of 3.40m and 3.44m bgl and in BH2 at depths of 1.74m, 1.72m and 1.80m bgl. The sandy silty clay recorded over the depth of the boreholes is expected to have low permeability; the interlaminated clay and sand recorded in TP1 would be anticipated to have a higher permeability than the deeper Claygate Member; however, this material is still indicated as being of low permeability. The anticipated low permeability of the ground indicates there is likely to be little or no natural groundwater flow. In the event that the interlaminated material in TP1 is more widespread, groundwater flows would be anticipated to flow around the basement. Thus, the proposed basement is not anticipated to have any impact on the groundwater flows/levels and therefore no impact on neighbouring properties.

7.3 The standpipes installed in BH1 & BH2 on site should be maintained so that further monitoring readings can be taken during the detailed design and prior to the start of construction.

7.4 The site is located on a south westerly slope with a slope gradient of approximately 6-7°. The slope across the site is approximately 10° and the boundaries with No's 23 & 25 Merton Lane and the Merton Lane carriageway consisted of retaining walls with ground level differences of approximately 1.5-2.5m and 1.0-1.5m respectively. The interlaminated clay and sand recorded in TP1 could be present across other areas of the site, which would be expected to have a higher permeability. Therefore slope instability could cause a potential problem. The upslope perimeter basement walls must be designed to protect against this potential slope instability. This issue will be required to be assessed within the structural design.

7.5 Contour plots of displacement in response to the changes in vertical pressure caused by the excavation and construction of the proposed basement are presented in Figures 16 – 24.

7.6 A Damage Category Assessment (DCA) was undertaken for the worst case scenario in the adjoining and adjacent properties, based on the maximum displacements predicted by the PDISP analyses, combined with the ground movements alongside the basement in response to the lateral stress releases, as predicted by CIRIA C580.

7.7 In the assessed cases, the flank wall of No. 25 West Park Hill, fell within Burland Category 0 'negligible' (as given in CIRIA SP200, Table 3.1). The damage category results have been plotted graphically in Figures 26 and 27.

7.8 No further damage category assessments have been carried out as the assessed cases are considered the worst case scenarios and therefore all other structures will be classified as Category 0 'negligible'.

7.9 Use of best practice construction methods will be essential to ensure that the ground movements are kept in line with the above predictions. Pre-construction condition surveys of neighbouring properties are also recommended and a system of monitoring adjoining and adjacent structures should be established before the works start.

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End of report

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