Basement Impact Assessment

Site	31 St Mark's Crescent London NW1 7TT
Client	The Basement Design Studio
Date	February 2017
Our Ref	BIA/8084A

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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 31 St Mark's Crescent, London NW1 7TT. The local planning authority is the London Borough of Camden.

1.2 Chelmer Site Investigation Laboratories Ltd (Chelmer) was instructed in December 2016 by The Basement Design Studio to complete this report. The report has been prepared by 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 The structure of this report follows that of a BIA complaint with Camden Borough CPG4 (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 surface water, groundwater and ground movement. 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 31 St Mark's Crescent, London NW1 7TT and is located at approximate Ordnance Survey grid reference (OSNGR) 528383E, 183890N. The site comprises of a four storey (including lower ground floor level), semi-detached residential property with associated rear garden and front lightwell.

1.6 It is to our understanding that the proposed development involves the excavation and construction of a basement level beneath the lower ground floor. The basement will extend approximately 6 m out to the rear of the building beneath the rear garden and a reduction in the garden level to lower ground floor level will be completed from the existing property to approximately 10.5 m into the rear garden. Existing and proposed plans are presented in Appendix B.

1.7 A site inspection (walk-over survey) was undertaken on 8th February 2017 by Jake Solomon of Chemer, 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 Ground investigations were undertaken by Chelmer (2016 & 2017) in November 2016 and January 2017, the Factual Reports on the investigations are presented in Appendix F, and the findings are summarised in Section 3.0.

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

Sher + White Architects

Drawing 1701/EX.01 (Existing Lower Ground Plan)
Drawing 1701/EX.02 (Existing Ground Plan)
Drawing 1701/EX.10 (Existing Section A [Elevation A & D])
Drawing 1701/EX.11 (Existing Section A [Elevation B])
Drawing 1701/EX.12 (Existing Section A [Elevation C])
Drawing 1701/EX.13 (Existing Section A [Section A])
Drawing 1701/PL.00 (Proposed Basement)
Drawing 1701/PL.01 (Proposed Lower Ground Plan)
Drawing 1701/PL.02 (Proposed Ground Plan)
Drawing 1701/PL.10 (Proposed Elevations A & D])
Drawing 1701/PL.11 (Proposed Elevation B & Section B)
Drawing 1701/PL.12 (Proposed Elevation C)
Drawing 1701/PL.13 (Proposed Section A)

Croft Structural Engineers

Drawing 161202-SL-10 (Structural Scheme Design, marked up with loading)
Drawing 161202-SL-10 (Structural Scheme Design)
Drawing 161202-SL-20 (Structural Scheme Design)
Drawing 161202-TW-10 (Temporary Works Scheme Design)

Greenhatch Group Surveyors

Drawing 24742_01_P Rev.0 (Topographical Survey)
Drawing 24742_02_P Rev.0 (Existing Floor Plans)
Drawing 24742_03_ES Rev.0 (Existing Existing Elevations & Sections)

2.0 PROPERTY AND AREA DETAILS

2.1 The site represents the end semi-detached property on St Mark's Crescent before Gloucester Avenue. The rear garden of the property backs on to Regents Canal. A major railway line linked to Euston Station runs 75m east of the property. The site occupies approximately 0.02 ha and is centred on approximate Ordnance Survey National Grid Reference 528377E, 183893N. The site location plan is presented in Figure 1 below.

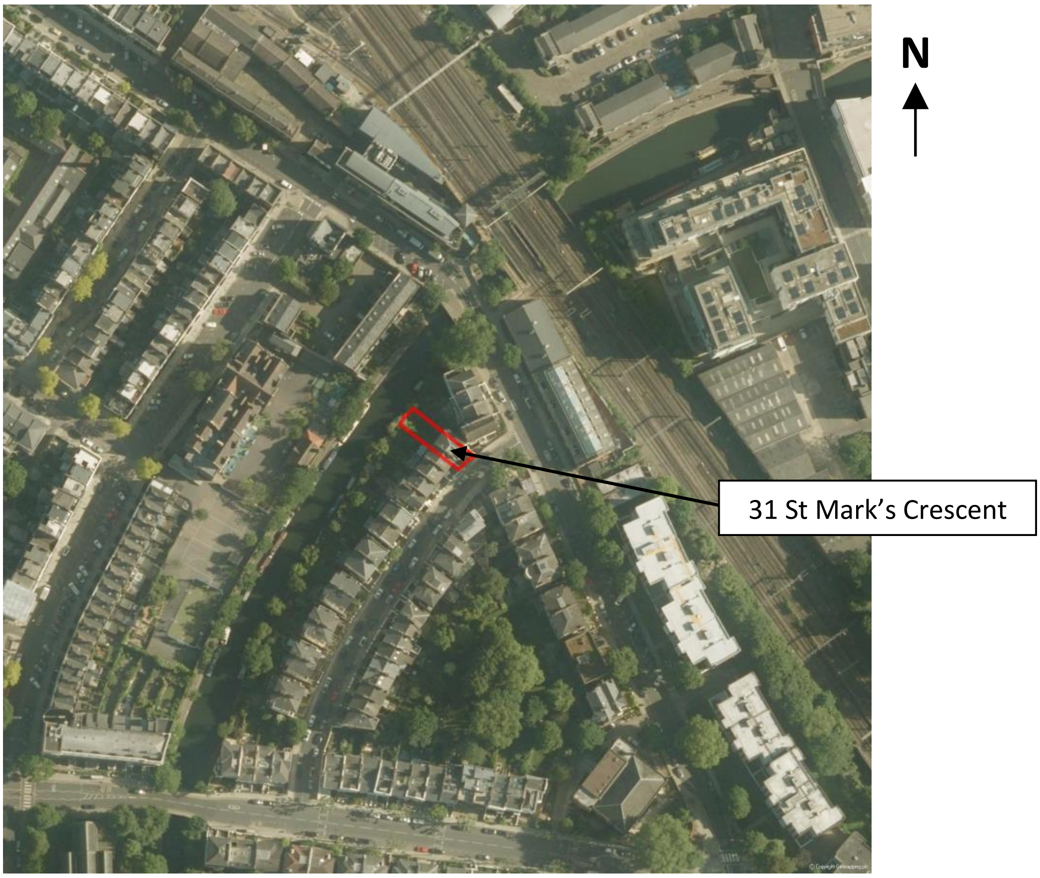

Figure 1. Site Location Plan

2.2 The site comprises 31 St Mark's Crescent which is a three storey with a lower ground floor, semi-detached, residential property with associated rear garden and front lightwell at the north-eastern end of St Mark's Crescent. The property is adjoined by No. 1 St Mark's Crescent to the west and neighbours No. 57 Gloucester Avenue to the east. The rear garden of the site backs directly onto Regents Canal.

2.3 A site inspection (walk-over survey) was undertaken on 8th February 2017 by Jake Solomon 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. It was noted that there was limited potential for surface water to drain on to site as the front of the property is bounded by a brick wall and raised step through the gateway, therefore surface water outside this catchment is likely to flow into and along St Mark's Crescent carriageway. The rear is bounded by a brick wall along the north-eastern perimeter, a brick wall and then Regents Canal to the rear and a wooden fence along the rear garden boundary with No. 1 St Mark's Crescent.

2.4 The existing property at 31 St Mark's Crescent has a lower ground floor at approximately 30.7 m AOD. The proposed development involves the excavation and construction of a basement level that extends the entire footprint of the current property and into the rear garden. The rear garden level will also be reduced by approximately 1.0 m to the same level as the lower ground floor. Existing and proposed plans are presented in Appendix B.

2.5 The proposed basement level will involve excavation beneath the lower ground floor to approximately 3.5 m below existing lower ground floor level (m blgl), including the depth of the slab as detailed in Drawing 161202-SL-02. The basement will be formed by reinforced concrete (RC) underpinning and RC retaining walls.

2.6 A search has been made of planning applications on the London Borough of Camden's 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 planning applications in the area only detailed minor modifications to existing basements, to the same depth as the existing lower ground floor level at No. 31 St Mark's Crescent. No excavations below this level have been identified in the vicinity of the proposed development.

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) are available from 1870 and these indicate that St Mark's Crescent and the current property were developed prior to 1870. The historic maps also identify very few new developments in the area since 1870.

3.2 Topography

3.2.1 The BGS Onshore GeoIndex indicates that the site is on relatively flat ground at approximately 33.1 mOD. This corresponds to topographical information displayed on Drawings 24742_01_P Rev.0 and 24742_02_P Rev.0 that indicate St Mark's Crescent is at approximately 33.1 mOD and the ground floor within 31 St Mark's Crescent is at approximately 33.6 mOD. The BGS Onshore GeoIndex indicates that the closest area with a slope is 600 m to the west of the site and represents Primrose Hill.

3.3 Hydrological Setting (Rivers and Watercourses)

3.3.1 The Groundsure Report identifies locations along the Regents Canal as the only surface water features within 250 m of the site; as discussed previously the rear garden of the site backs on to Regents Canal. The only surface water abstractions noted by the Groundsure Report within 1 km of the site are from Regents Canal and are located 165 m and 171 m to the northeast. The BGS Onshore GeoIndex identifies the nearest well as being located approximately 330 m to the south south-west of the property within the Chalk Group aquifer.

3.3.2 The book 'The Lost Rivers of London' (Barton, 1992) does not identify any lost rivers within the immediate vicinity of the site. The nearest underground stream is the River Fleet roughly 650 m to the east-northeast which flows from Hampstead Heath, under Kentish town, Camden, Kings Cross and Clerkenwell before entering the River Thames under Blackfriars Bridge. Two maps of the tributaries of the Thames and showing the approximate location of No. 31 St Mark's Crescent are presented in Figure 2 and Figure 3.

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

- There is a surface water feature within 20m of the site (Regents Canal).
- There are two surface water abstraction licences within 175m 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. Location of River Fleet relative to 31 St Mark's Crescent (Extract from map posted on londonbygaslight.wordpress.com)

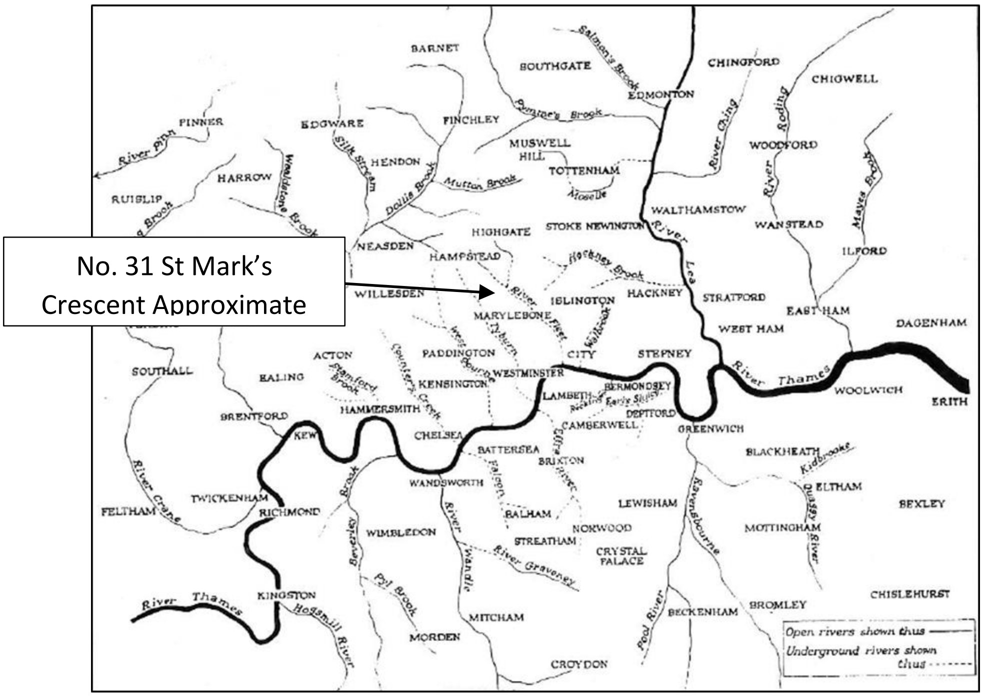

Figure 3. Tributaries of the Thames from Kingston to Erith identified in 'The Lost Rivers of London' (Barton, 1993)

3.4 Flood Risk

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. The EA website also shows that the property does not fall within an area at risk of reservoir flooding.

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 4 below; the property is identified as being at very low risk with the low risk area restricted to Regents Canal.

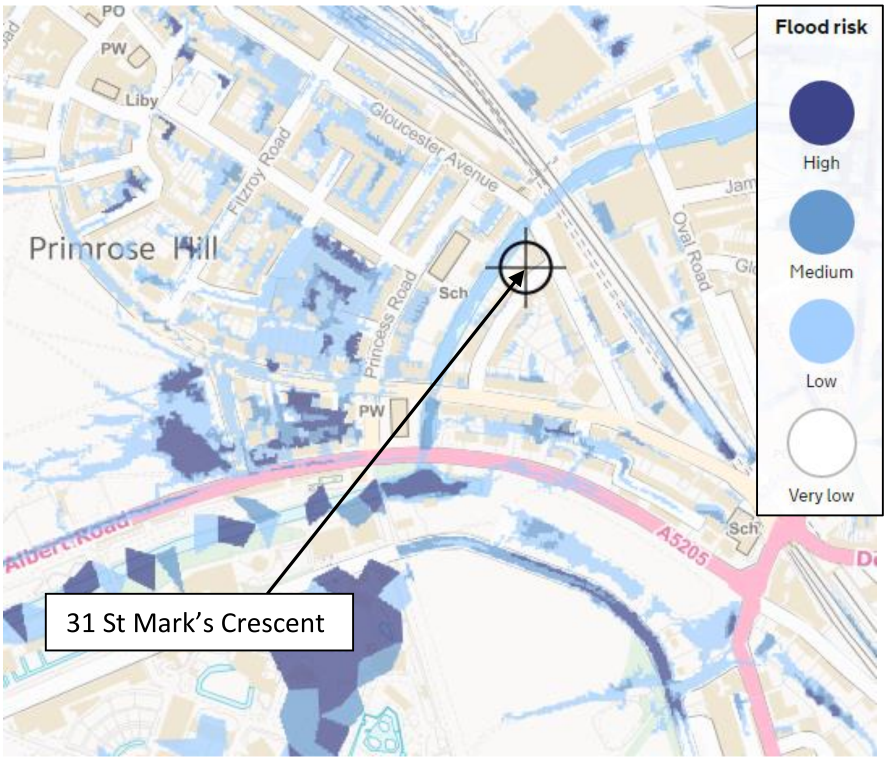

Figure 4. Flood Risk from Surface Water (Contains public sector information licensed under the Open Government Licence v3.0)

3.4.3 Figure 15 'Surface Water Flood Risk Potential' from the Camden Geological, Hydrogeological and Hydrological Study (GHHS) by Arup (November 2010) shows historic flooding close to the proposed development but the 'Floods in Camden' report by the London Borough of Camden (June 2003) does not indicate either of the 1975 or 2002 floods affected St Mark's Crescent and only the adjoining Gloucester Avenue was affected by the 1975 flood event. Figure 5 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.

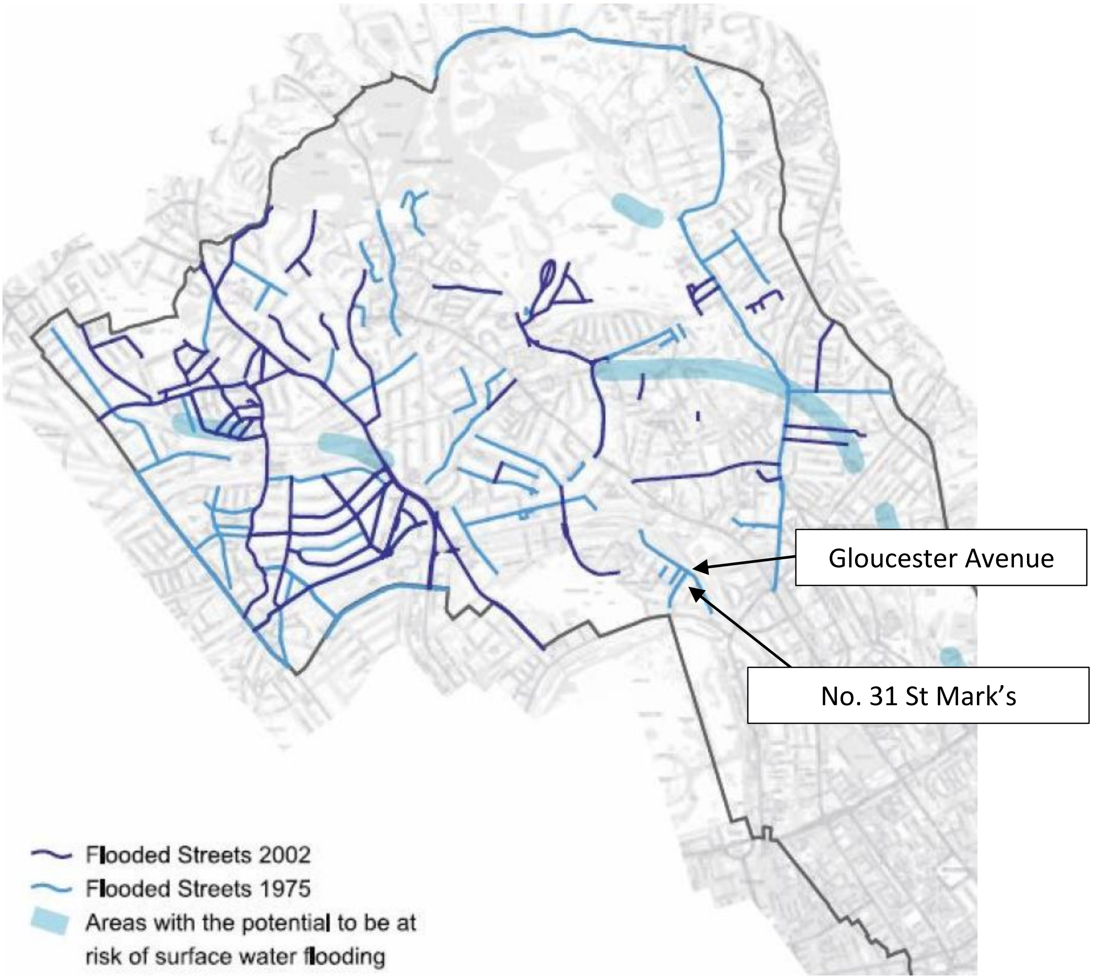

Figure 5. Extract from Figure 15 of the Camden GHHS (Arup, 2010)

3.4.4 Figure 5a of the London Borough of Camden Strategic Flood Risk Assessment (SFRA) by URS (2014) indicates the site is located in an area where there are no internal or external sewer flooding records. Extracts from Figures 5a and 5b of the SFRA is displayed in Figure 6 below.

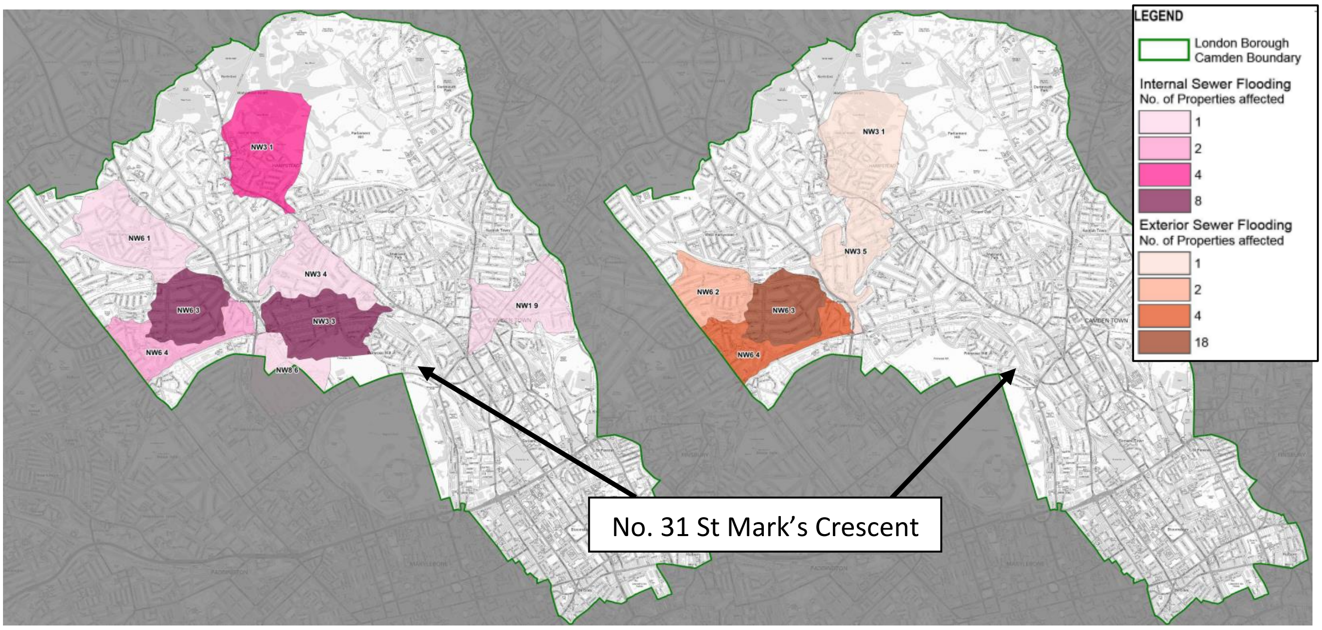

Figure 6. Extracts from Figures 5a and 5b of the SFRA (URS, 2014)

3.4.5 Figure 6 of the SFRA shows that the site is located in a Critical Drainage Area (CDA), along with the majority of the borough. A CDA is defined as *"A discrete geographic area (usually a hydrological catchment) where multiple and interlinked sources of flood risk (surface water, groundwater, sewer, main river and/or tidal) cause flooding in one or more Local Flood Risk Zones during severe weather thereby affecting people, property or local infrastructure."* It is also located within a Local Flood Risk Zone (LFRZ), which is defined as *"discrete areas of flooding that do not exceed the national criteria for a 'Flood Risk Area' but still affect houses, businesses or infrastructure. A LFRZ is defined as the actual spatial extent of predicted flooding in a single location."* An extract of Figure 6 is displayed in Figure 7 below.

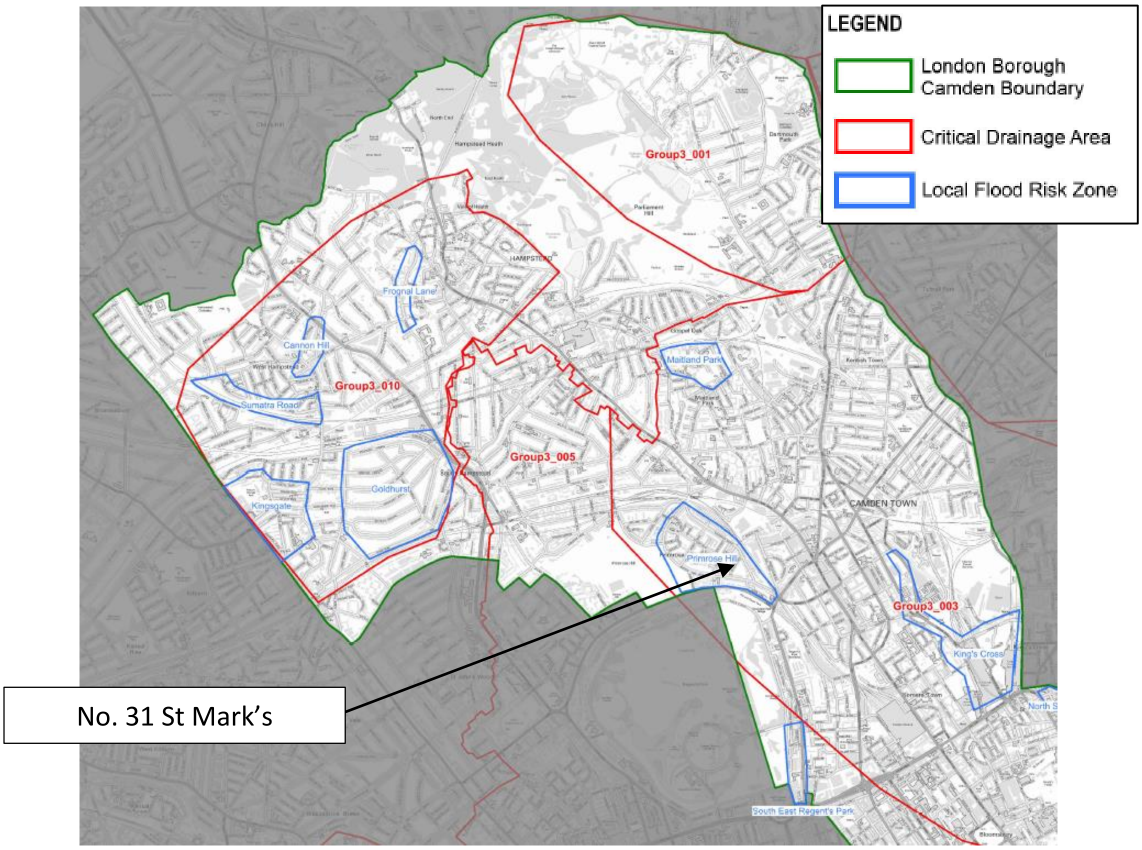

Figure 7. Extract from Figures 6 of the SFRA (URS, 2014)

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 London Clay Formation, with no overlying superficial deposits recorded. The BGS geological plan showing the site is presented in Figure 8 below. The BGS indicates the same geology is encountered for over a 1 km radius to the site. The Claygate Member and Bagshot Formation are present 2.0 km north-west of the site. Superficial Lynch Hill Gravel Member and Langley Silt deposits are present 1.6 km to the south of the site.

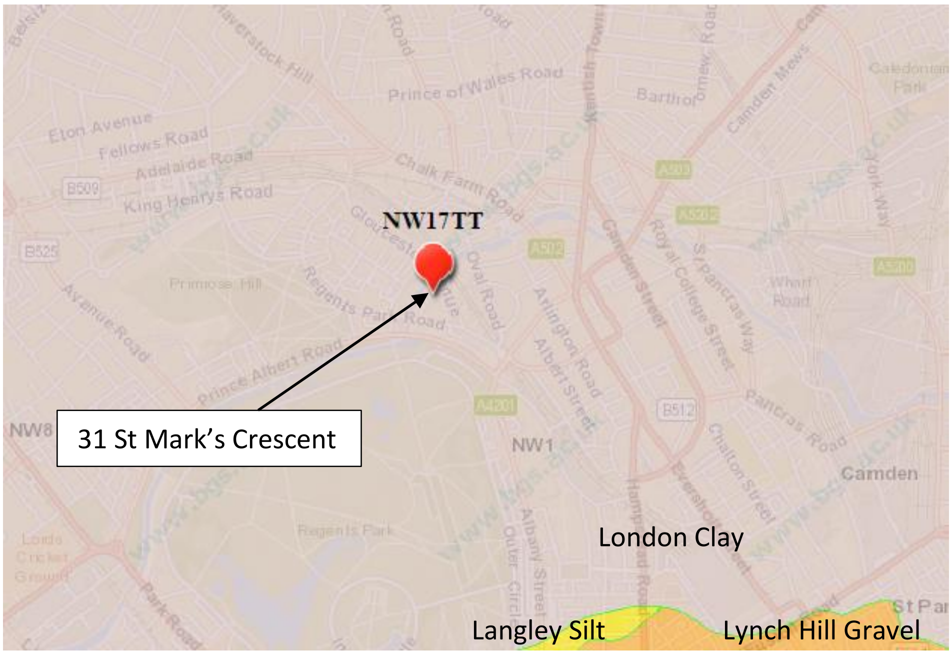

Figure 8. Site BGS Geological Plan (Contains British Geological Survey materials © NERC 2016. Base mapping is provided by ESRI)

3.5.2 The London Clay Formation consists of mainly dark blue-grey to brown-grey clay containing variable amounts of fine-grained sand and silt. The London Clay Formation generally weathers to an orange-brown colour with pockets of silty fine sand. The formation is particularly susceptible to swelling and shrinking when subjected to moisture content changes and is commonly intensely fissured. In addition, gypsum (selenite) crystals and pyrite nodules are commonly found throughout the formation.

When exposed to the weathering process the upper regions of the London Clay Formation oxidise to brown in colour. It usually contains selenite crystals, often grouped in bands or layers, which are thought to have originated from the decomposition of shell fragments. London Clay contains clay minerals in the form of illite, kaolinite and smectite. The presence of smectite renders the London Clay Formation particularly susceptible to changes in moisture content and is prone to shrinkage and swelling (settlement and heave) caused by alternate wetting and drying near the surface. In addition, weathering and possible slight transportation of semi-frozen material "en-masse" in glacial or peri-glacial regions is believed to have occured. This action often completely destroys the structure of the material and can involve a serious loss of strength. As the soil composition is derived mostly from materials local to the point of deposition, the lithology can be variable and reflects that of the parent strata.

3.5.3 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 9 below. The borehole logs are presented in Appendix D.

3.5.4 Five BGS boreholes were reviewed, with the deepest borehole extending to 90.53m below ground level (bgl). The boreholes typically showed Made Ground, comprising of a clay with brick and concrete fragments over London Clay Formation. The London Clay Formation is divided into weathered London Clay and the unweathered London Clay Formation. The weathered London Clay Formation consists of an orange and brown clay with occasional fine sand lenses which makes way to the unweathered London Clay Formation which comprises of a grey laminated fissured clay. The full depth of the London Clay Formation was only penetrated in the most south eastern reviewed borehole (TQ28SE7) which found it be underlain by Lambeth Group deposits at a depth of 44.0 m bgl. This in turn was underlain by Chalk bedrock at 65.5 m bgl, the Chalk was still present at the final borehole depth of 90.5 m bgl.

Groundwater levels recorded in the boreholes are detailed in Section 3.6.3 and presented in Table 2.

Figure 9. BGS Borehole Locations (Contains British Geological Survey materials © NERC 2016. Base mapping is provided by ESRI)

3.5.5 Two ground Investigations were completed by Chelmer (2016 & 2017). The initial (2016) comprised two C.F.A. boreholes (BH1 & BH2) both to 6.1 m depth. The second investigation (2017) comprised extending BH1 to a depth of 10.4 m blgl and the excavation of two hand dug trial pits (TP1 and TP2). BH1 was undertaken within the front lightwell at lower ground floor level and BH2 was undertaken in the rear garden at approximately 0.8 m above lower ground floor level. Made Ground was present in BH1 to a depth of 0.9 m blgl and 1.0 m blgl (1.8 m drilled depth (dd)) in BH2. A stiff light greenish brown silty clay with a pungent vinegar odour was present in both boreholes beneath the Made Ground to a depth of 1.8 m blgl in BH1 and 2.8 m blgl (3.6 m dd) in BH2. The London Clay Formation followed in both boreholes and consisted of a brown silty clay with rare partings of fine orangish yellow sand and rare disseminated selenite crystals to 5.3 m blgl (6.1 m dd). The extended BH1 encountered very stiff clay with partings of silt and fine sand from approximately 6.0 m blgl. No partings of silt and sand were recorded below 7.8 m depth in BH1. The base of the London Clay Formation was not proven at the maximum drilling depth of 10.4 m blgl. BH1 was proposed to go to 12.0 m blgl however the clay became too stiff to drill and at 10.4 m blgl no further progress was possible. TP1 and TP2 only recorded Made Ground beneath the concrete floor, to the maximum termination depth of 1.2 m blgl. Table 1 below presents a summary of the ground conditions encountered and the borehole records are presented within the Factual Reports in Appendix F.

Table 1: Summary of Ground Conditions Encountered		
Depth to top of stratum (m blgl)	**Depth to base of stratum (m blgl)**	**Description**
0.0	0.9 – 1.0 (>1.2 in TP2)	MADE GROUND: Dark brown to orange brown slightly gravelly silty CLAY with occasional brick and concrete fragments (BH1 & BH2). Silty gravelly SAND (TP1). Sand very gravelly clayey SILT (TP2).
0.9 – 1.0 / >1.2	1.8 – 2.8	Light greenish brown silty CLAY with pungent vinegar odour
1.8 – 2.8	6.00 – 5.3	LONDON CLAY FORMATION: Stiff brown silty CLAY with rare partings of fine orangish yellow sand and rare disseminated selenite crystals
6.00	7.80	LONDON CLAY FORMATION: Very stiff brown silty CLAY with rare partings of dark brown silt and orange fine sand
7.80	>10.40	LONDON CLAY FORMATION: Very stiff grey silty CLAY

Notes: No groundwater was observed during the ground investigations

3.5.6 As the BGS boreholes generally encountered the London Clay Formation with the Lambeth Group encountered beneath this at around 44 m bgl (approximately 41.7 m blgl using the stated 33.0 mOD ground level from TQ28SE7). Given that the site-specific ground investigations (Chelmer 2016 & 2017) only encountered the London Clay Formation, then ground conditions beneath the site can be assumed to be silty clay from the London Clay Formation to a significant depth.

3.6 Hydrogeological Setting (Groundwater)

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

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

- There are no groundwater abstraction licences are within 500 m of the site.
- Source Protection Zone 2 has been recorded 485m to the west of the site.
- There are no BGS groundwater flooding susceptibility areas within 50 m of the site and the site is not prone to groundwater flooding.

3.6.3 Groundwater information recovered from the BGS boreholes near the site (Figure 7) indicate groundwater was only encountered within the London Clay Formation in one of the five boreholes. Groundwater was recorded within borehole TQ28SE1829 (64.0 m to the north east of the site) at a depth of 1.9 m bgl as a seepage which rose to 1.7 m bgl after 30 mins within the London Clay Formation. Groundwater was encountered within the Chalk deposits in borehole TQ28SE7 and rose to 36.6 m bgl. Groundwater information from BGS boreholes near the site are detailed in Table 2 below.

Table 2: Summary of Groundwater Records from BGS Boreholes

Location	Date	Groundwater Standing Level (m bgl [m AOD])
TQ28SE1216	1990	*Groundwater not encountered during drilling*
TQ28SE1215	1990	*Groundwater not encountered during drilling*
TQ28SE1829	1995	*Seepage – 1.9 [25.6]* *Standing – 1.7 [25.8] after 30 minutes.*
TQ28SE637	1966	*Groundwater not encountered during drilling*
TQ28SE7	1850#	*Strike – 75.6* [-42.6]* *Standing – 36.6 [-3.6]*

Notes: # Numerous dates indicated, possibly completed prior to 1850
* This deep groundwater level is associated with the Chalk aquifer

3.6.4 No groundwater was observed during the drilling process of either ground investigation performed by Chelmer (2016 & 2017) where BH1 was drilled to 10.4 m blgl and BH2 to 5.3 m blgl. A groundwater monitoring standpipe was installed in BH1 to 10.0 m blgl. Three return monitoring visits have been completed, on 8th, 16th and 20th February 2017 and groundwater was recorded at depths of 4.67 m, 3.43 m and 3.08 m blgl (approximately 27.6 mOD).

3.6.5 The SFRA (URS, 2014) indicates that the site is not in an area with increased susceptibility to elevated groundwater, which is defined as an area *'where there is increased potential for groundwater levels to rise within 2m of the ground surface following periods of higher than average recharge'*. As presented in Figure 4e of the SFRA the nearest recorded groundwater flooding incident was approximately 650 m to the west-northwest of the site. An extract of Figure 4e of the SFRA is displayed in Figure 10 below.

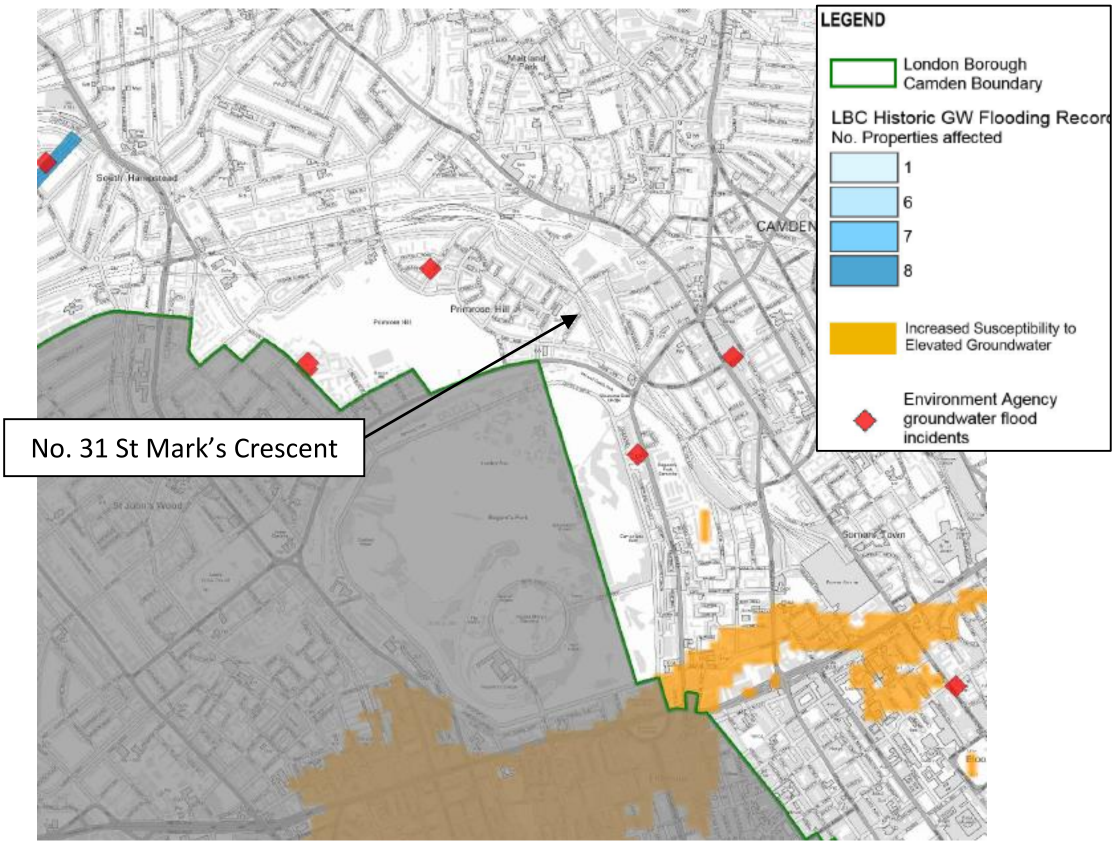

Figure 10. Extract from Figure 4e of the SFRA (URS, 2014)

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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 investigations were completed on 4th January 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 3.5 m blgl.
- The site is located on relatively flat lying land, that remains at the same level for over a 500 m radius around the site, except for the drop to the canal.
- The only surface water body within the vicinity of the site is Regents Canal that runs along the end of the rear garden.
- 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 (recorded up to 1.0 m blgl within the boreholes and a minimum of 1.2 m blgl in the trial pits), stiff silty clay of the London Clay Formation to 6 m depth and then very stiff to the base of the borehole drilled to 10.4 m blgl.
- The site is located above an unproductive stratum, formed by the clay of the London Clay Formation.
- Groundwater was not encountered during drilling of the on-site boreholes (BH1 & BH2) to a maximum depth of 10.4 m blgl, however, during three monitoring visits groundwater was recorded as high as 3.1 m blgl.

4.2 *Groundwater Flow Impact Assessment*

4.2.1 The site is located above an 'Unproductive' stratum formed by the clay of the London Clay Formation. No groundwater was observed during the drilling process of the ground investigation performed by Chelmer (2016 & 2017), where BH1 was drilled to 10.4 m blgl and BH2 to 5.3 m blgl (6.1 m below ground level). A monitoring standpipe was installed to 10.0 m blgl within BH1. Three return monitoring visits have been completed, on 8th, 16th and 20th February 2017 and groundwater was recorded at depths of 4.67 m, 3.43 m and 3.08 m blgl (approximately 27.6 mOD).

4.2.2 The permeability within the London Clay Formation at the site is expected to be very low due to the high clay content. 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 lowered lower ground floor level will be founded within the London Clay Formation. The monitoring performed in the on-site borehole (BH1) encountered groundwater level up to 0.4 m above the founding level of the proposed basement. The monitoring visits indicate that the groundwater level in the monitoring standpipe could still be rising, at a very slow rate due to a very low permeability bedrock. The anticipated very low permeability of the ground is likely to cause very little or no natural groundwater flow. Thus, the proposed basement is not anticipated to have any impact on the groundwater flows/levels and no significant impact on neighbouring properties would be expected. Due to the rising groundwater levels recorded during the monitoring visits, continued monitoring is recommended to identify a stable design height for the groundwater level.

4.2.4 The basement will be excavated below groundwater level so the basement will require waterproofing and groundwater control will be required during the basement construction works. The soils above formation level are expected to be of very 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.

4.3 Surface Water Impact Assessment

4.3.1 The site is in an area where flooding from rivers and seas is defined as very unlikely and the flood risk from surface water is very low. The only surface water feature recorded near the site is Regents Canal, which is detailed in the Camden GHHS (Arup, 2010) as being lined with puddle clay of a low permeability to prevent flow between the canal and surrounding ground. A brick wall that forms the rear boundary between the canal and the site provides a further barrier. Regents Canal is also considered to have a low flood risk as there are limited surface water inputs entering into the canal, not natural drainage channels. These inputs are of controlled inflows to maintain the water level. The very low risk of surface water flooding combined with no surface water features, except for the canal, can lead to the conclusion that conventional measures of managing surface water run-off should be sufficient to minimise any potential hydrological impacts.

4.3.2 The proposed development will extend into the rear garden, as shown on Sher White Architects Drawing 1701/PL.13 in Appendix B, which will result in an increased area of hardstanding approximately 4 m beyond the existing extent of hardstanding. Therefore, the basement would potentially result in an increase in impermeable surfacing; that should be mitigated. Potential mitigation options include implementing one of the following Sustainable Drainage Systems (SuDS), which must be designed formally to avoid any increase in the discharge of surface water to the mains drainage system. Any SuDS design should take account of groundwater conditions at the site. Potentially suitable SuDS systems include:

- Water butts or other temporary interception storage;
- Directing some roof water to a rain garden;
- Provision of other temporary intervention storage, such as rainwater harvesting.

4.3.3 Due to the very low risk of surface water flooding then conventional measures of managing surface water run-off should be sufficient; such as up-stands to protect lightwells and a ground level difference at external doorways.

4.4 Ground Stability Impact Assessment

4.4.1 The site is located on relatively level ground, with no noticeable slope gradient, therefore slope stability will be highly unlikely to cause any problems with the proposed basement.

4.4.2 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.

4.4.3 The Groundsure Report (Appendix E) states there is a moderate hazard for shrink-swell clays at the property location.

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

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

4.4.5 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.

4.4.6 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.

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 1701/PL.00 provided by Sher+White Architects, and is presented in Figure 11 below. The proposed basement is approximately 16.8 m long by 7.8 m wide with excavation generally extending to a depth of approximately 3.5 m below existing ground level (bgl). The basement is understood to be constructed by RC underpins as detailed in Section 2.5.

5.1.3 The excavation depths for the basement have been modelled using Drawing 1701/PL.13 provided by Sher+White Architects to estimate the gross pressure reductions (unloading) across the development. Figure 12 below illustrates the layout of all load zones, positive and negative (unloading), used to model the proposed basement 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).

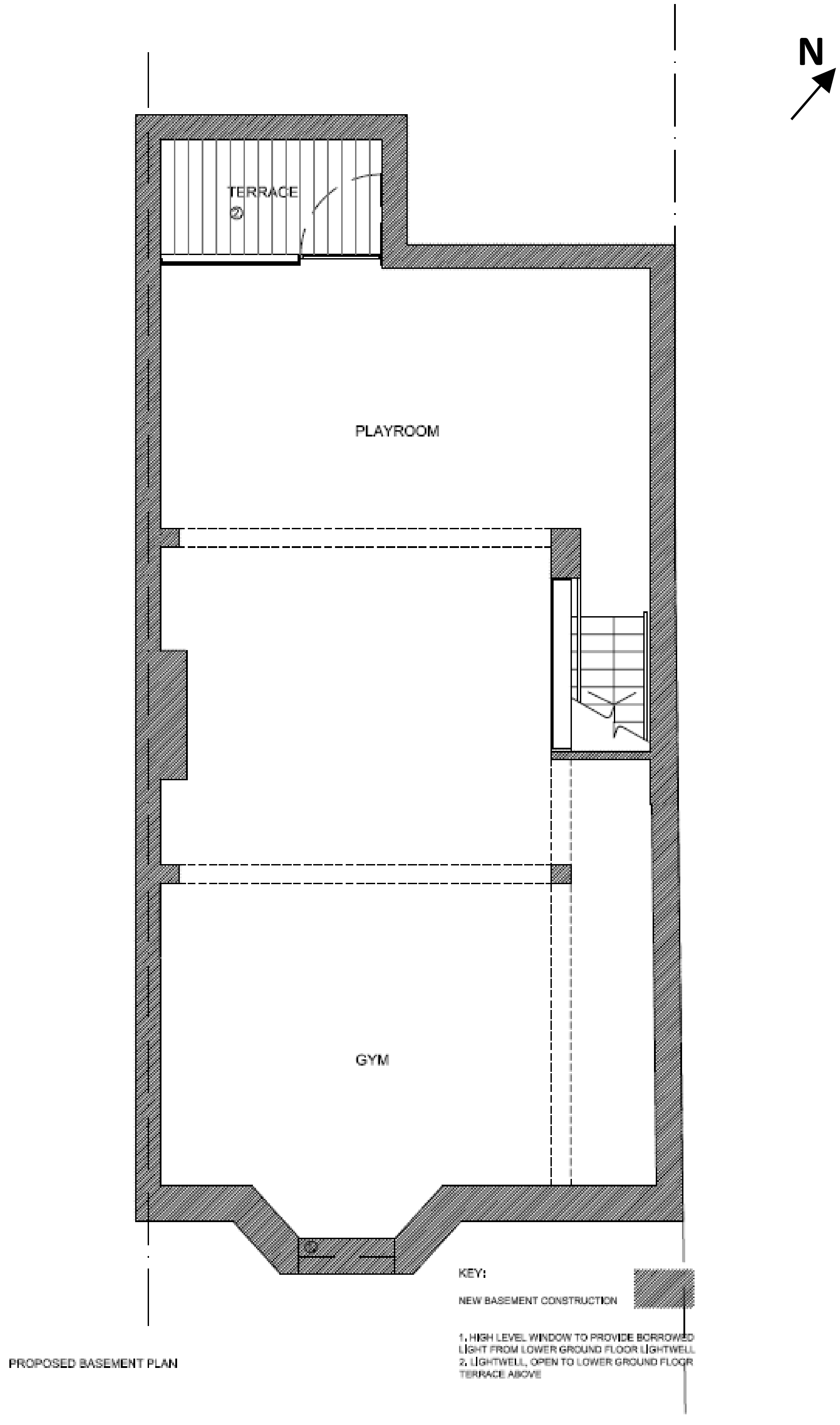

Figure 11. Layout of the proposed basement plan

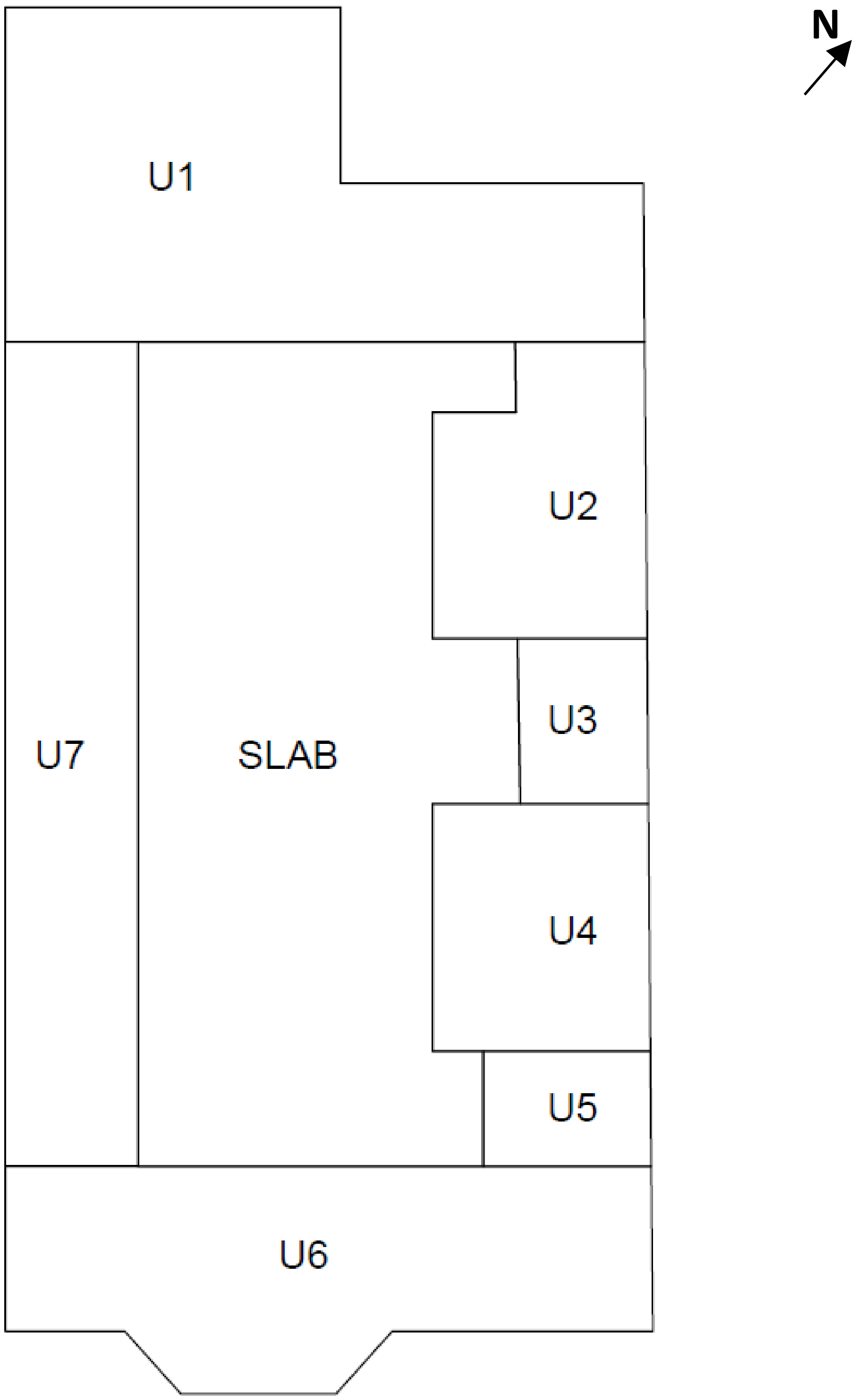

Figure 12. Detail of geometry introduced to PDISP
[U = Underpinning/retaining wall excavation and loads, Slab = Bulk excavation and slab loads]

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 (2016 & 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)*
London Clay Formation	3.5	41.5	24.9
	10.4	79.0	47.4

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 13, 14, and 15 respectively.

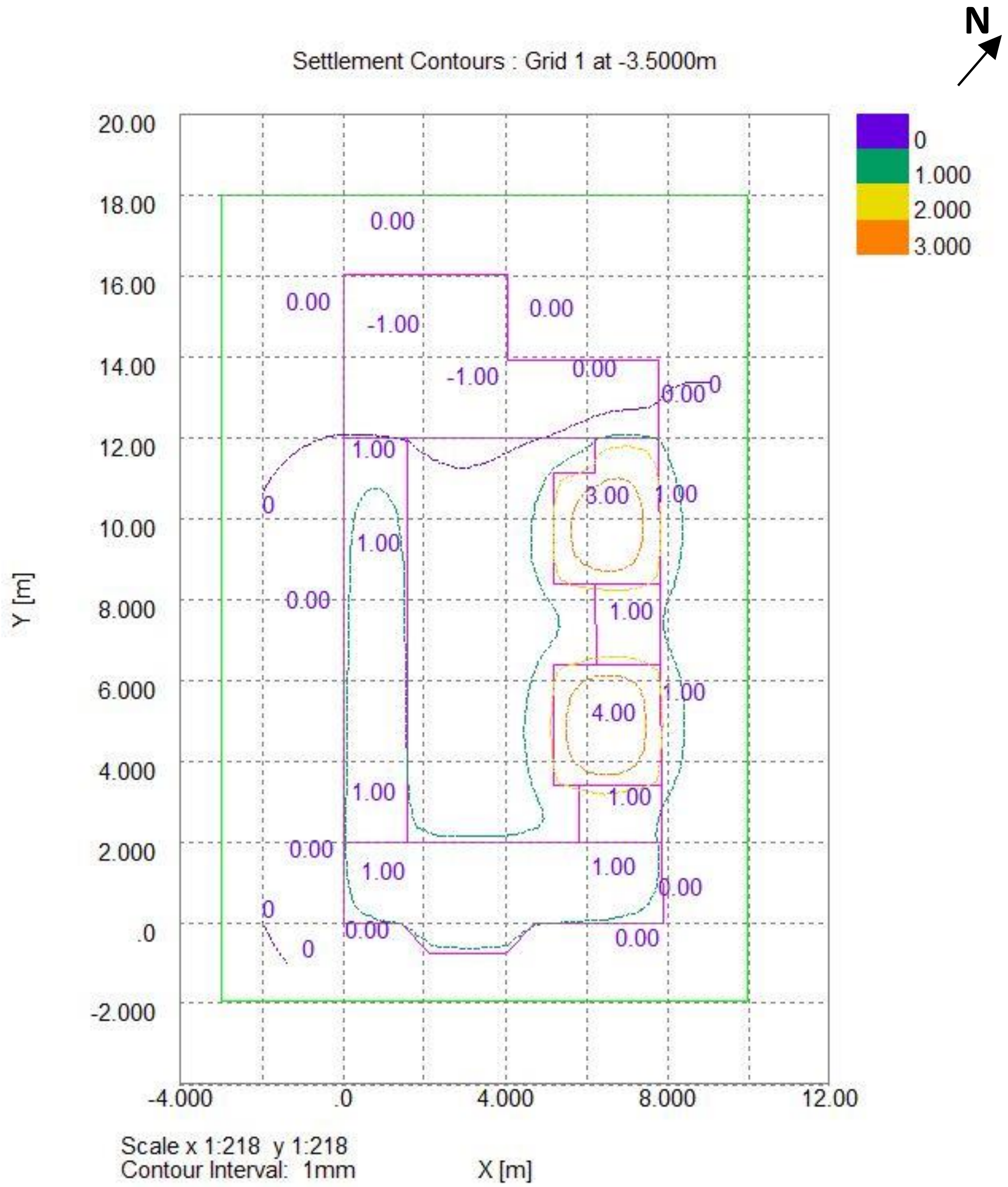

Figure 13. Stage 1 – Construction of underpins and retaining walls – Short-term (undrained) condition (1.0mm settlement contours)

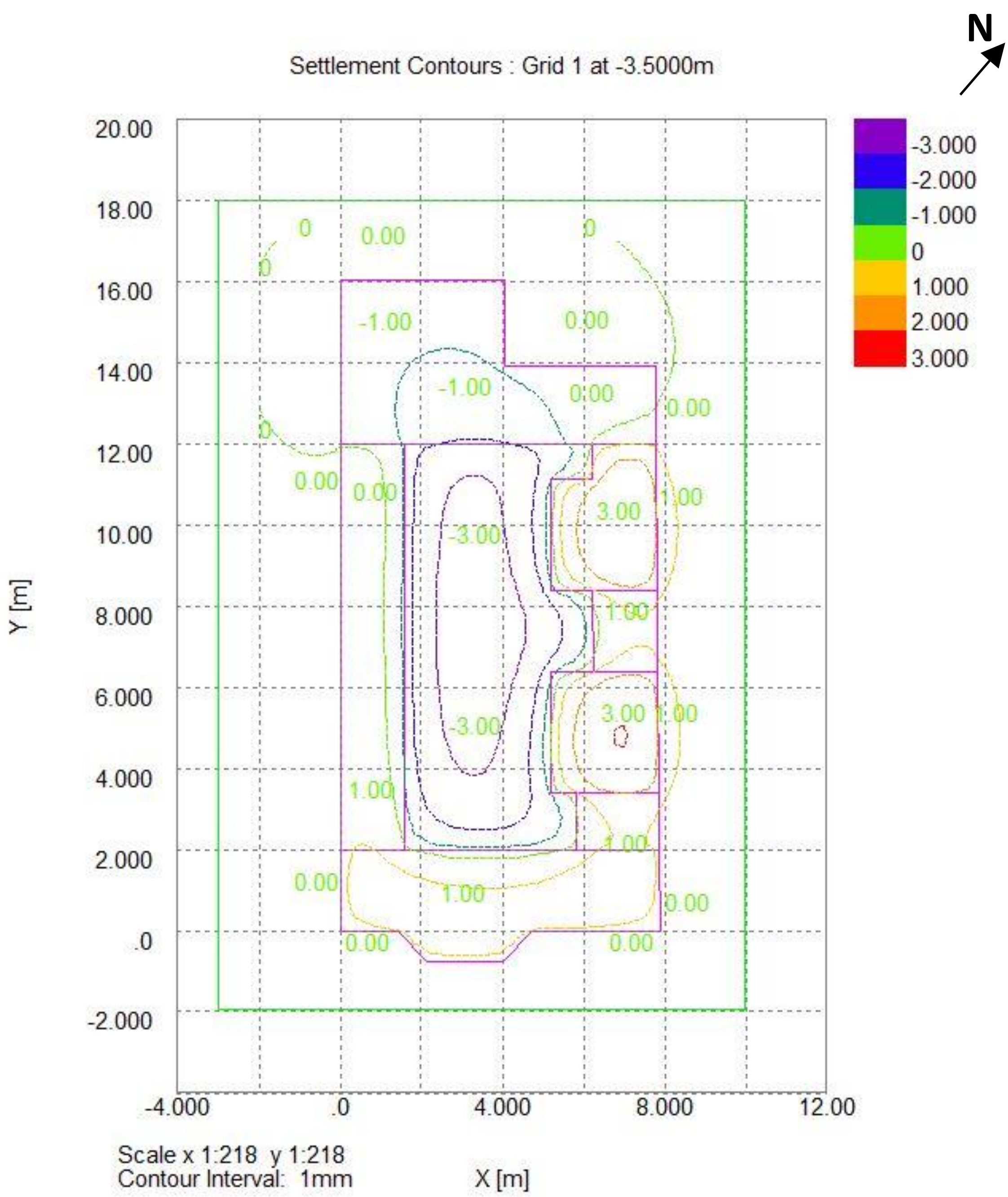

Figure 14. Stage 2 – Bulk excavation of central area and construction of the basement slab – Short-term (undrained) conditions (1.0mm settlement contours)

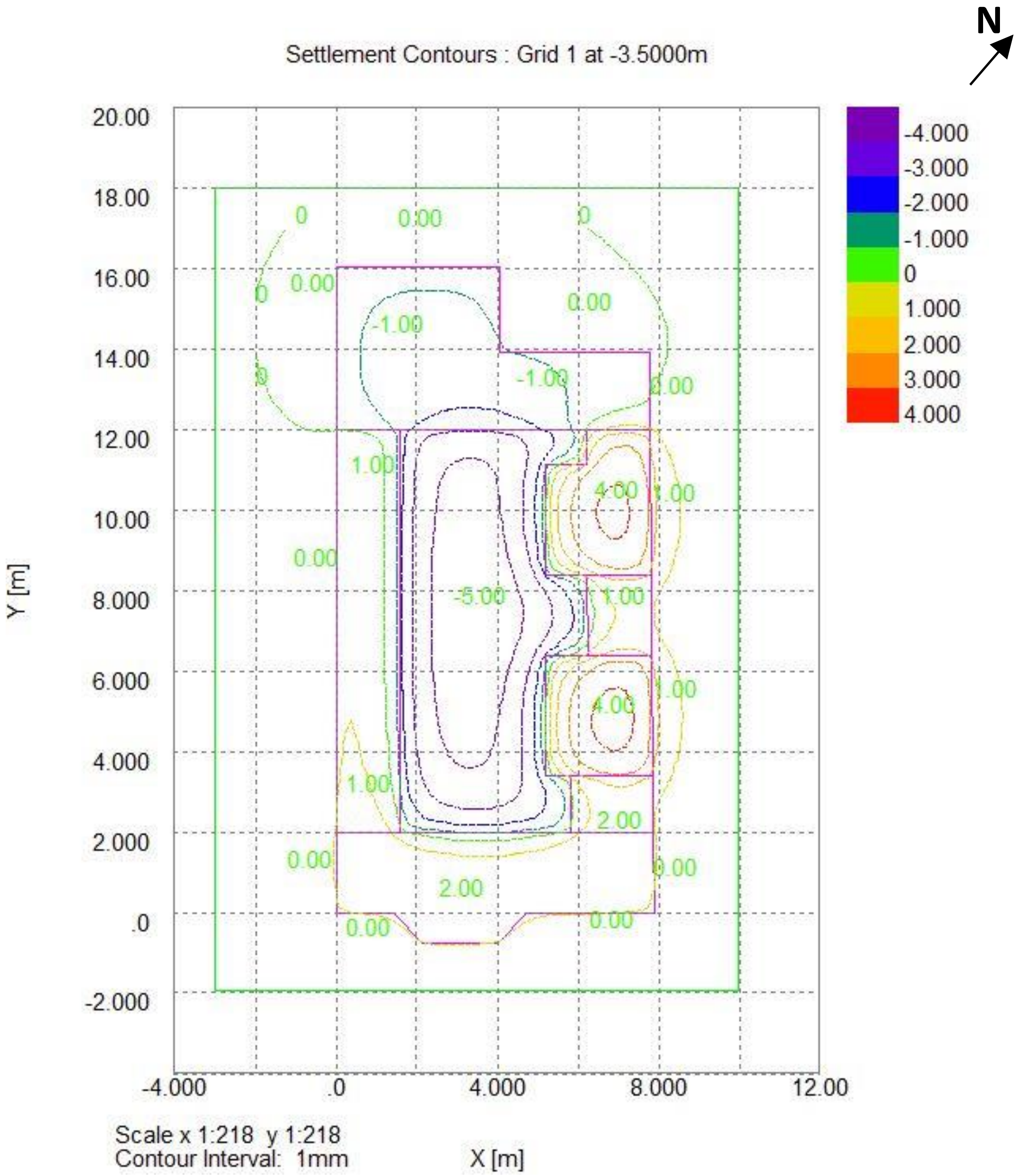

Figure 15. Stage 3 – Bulk excavation of central area and construction of the basement slab – Long term (drained) conditions (1.0mm settlement contours)

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 basement are presented in Table 4 below. These analyses indicated that the perimeter basement wall is predicted to undergo movements ranging from 1 mm heave to 3 mm settlement. The basement slab is predicted to undergo slightly greater displacements, from 2 mm to 5.0 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 4: Summary of Predicted Ground Movements from PDISP

Location / Building Element	***Stage 1 (short term)***	***Stage 2 (short term)***	***Stage 4 (long term)***
Northern perimeter of basement	0.0 – 1.0 mm Heave	0.0 – 1.0 mm Heave	0.0 – 1.0 mm Heave
Eastern perimeter of basement	0.0 – 2.0 mm Settlement	0.0 – 2.0 mm Settlement	0.0 – 3.0 mm Settlement
Southern perimeter of basement	0.0 – 1.0 mm Settlement	0.0 – 1.0 mm Settlement	0.0 – 2.0 mm Settlement
Western perimeter of basement	0.0 – 1.0 mm Settlement	0.0 – 1.0 mm Settlement	0.0 – 1.0 mm Settlement
Basement Slab	---	1.0 – 3.0 mm Heave	2.0 – 5.0 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 As identified in Section 2.6 the neighbouring properties in close proximity to the proposed development all have existing basements/lower ground floors to a similar level as No. 31's lower ground floor level. None of these properties have an additional basement level that is proposed at No. 31. Therefore, the closest structures, No. 1 St Mark's Crescent (SMC) and No. 57 Gloucester Avenue (GA) are considered the worst case scenarios for potential damage.

6.4 The uniform founding level for the proposed basement means that the potentially critical locations will be determined by the displacements predicted by the PDISP analyses and the geometries of the 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 front wall of No. 1 SMC, and the southern flank wall of No. 57 GA. 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 16 below.

Figure 16. 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 London Clay Formation. 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 3.5 m excavation (the approximate excavation depth for each assessed case) the total settlements immediately alongside the proposed basement walls due to the excavation of the soil would be 12.3 mm.

Front wall of No. 1 SMC:

6.9 The relevant geometries are as follows:

Depth of foundations = 1.0 m (as identified by trial pit TP1, see Appendix F)
Depth of excavation = 3.5 – 1.0 = 2.5 m
Width of affected ground = 2.5 x 4 = 10.0 m

Width of No. 1 SMC	=	6.3 m (scaled from Drawing 24742_01_P Rev.0), therefore, it will affect most of No. 2 SMC's front wall as well as No.1 SMC.
Affected width (L)	=	10.0 m
Height of No. SMH (H)	=	12.3 m (the same height as No. 31 SMC) + 1.0 m (footing depth) = 13.3 m
Hence L/H	=	0.8

6.10 The predicted 5 mm maximum horizontal displacement (see Section 6.1), reduces pro-rata over the horizontal zone of influence to 3.6 mm due to the depth of excavation. Thus, the horizontal strain beneath No. 1 SMC would, theoretically, be in the order of $\varepsilon_h = 3.6 \times 10^{-4}$ (0.036%).

6.11 The maximum settlement produced by the PDISP analysis beneath the location where the rear walls of the adjoining No. 1 SMC meet No. 31 SMC was in Stage 3 where 1.0 to 2.0 mm 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 12.3 mm (see Section 6.8) is reduced to 8.8 mm when the depth to No. 1 SMC's footings are taken into account. The total combined settlement of 10.8 mm, 8.8 mm predicted by the CIRIA methods plus the maximum 2.0 mm predicted by PDISP, is detailed as the point immediately alongside the proposed basement (0 m) in Figure 17 below. Figure 17 presents the settlement curve from the basement wall to the maximum distance of affected ground, 10.0 m (see Section 6.9).

6.13 The deflection along the front walls of No.'s 1 and 2 SMC is calculated as the difference between the tangent of the relevant width of the affected wall (10.0 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, Δ = 3.0 mm as displayed in Figure 17 below, which represents a deflection ratio Δ/L = 3.0 x 10^{-4} (0.030%).

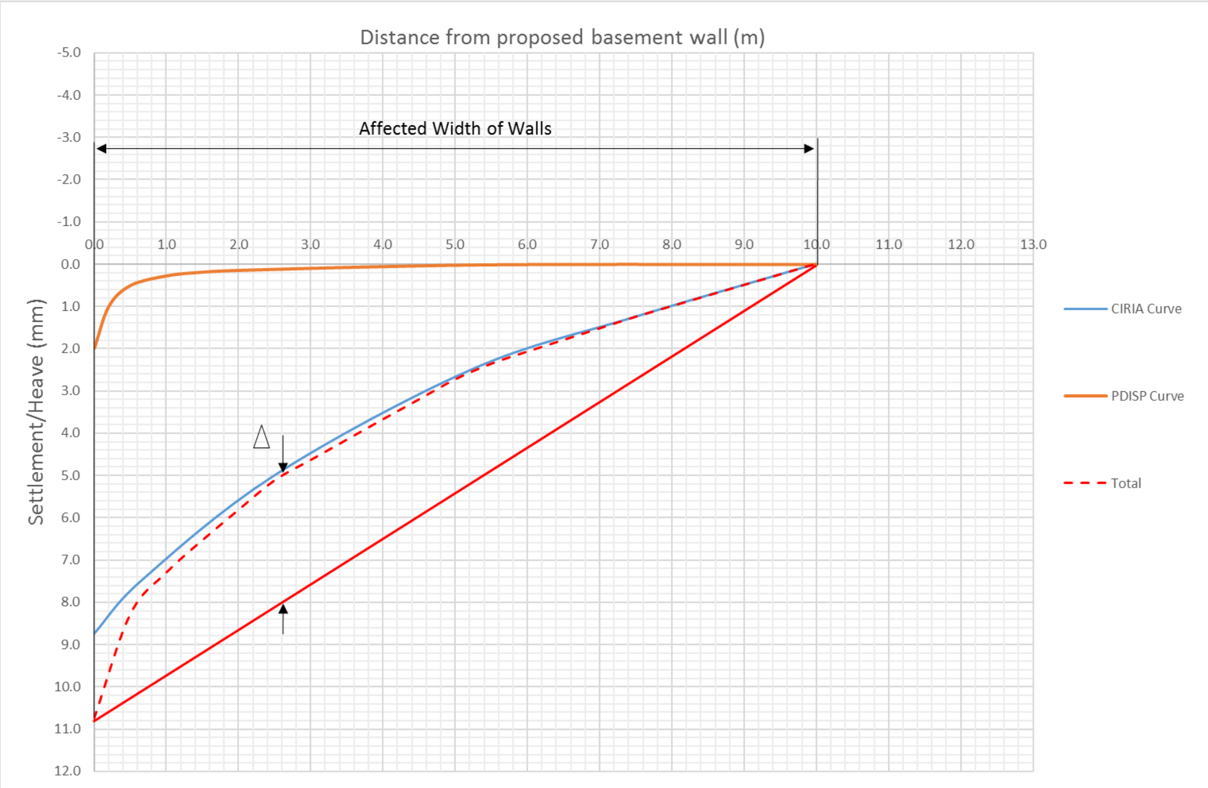

Figure 17. Combined displacements for No.'s 1 and 2 SMC front wall due to excavation of proposed basement

6.14 Using the damage category ratings and graphs given in CIRIA SP200, for L/H = 1.0 (A conservative value for the L/H of 0.8 defined in Section 6.9), these deformations represent a damage category of 'very slight' (Burland Category 1), as illustrated in Figure 18 below.

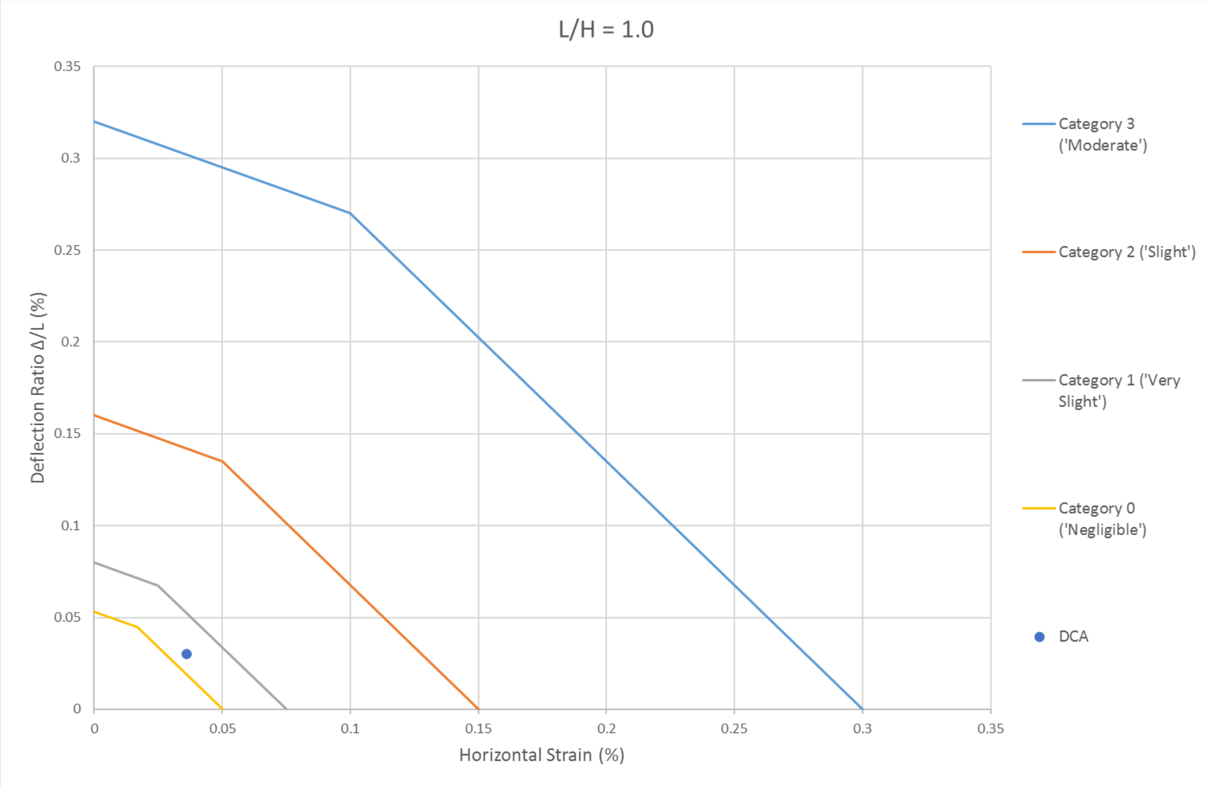

Figure 18: Damage category assessment for No.'s 1 and 2 SMC front wall

<u>Southern flank wall of No. 57 GA:</u>

6.15 The relevant geometries are as follows:

Depth of foundations = 0.5 m (conservative estimation)
Depth of excavation = 3.5 – 0.5 = 3.0 m
Width of affected ground = 3.0 x 4 = 12.0 m

Distance from No. 31 SMC = 0.5 m
Width of No. 57 GA = 12.5 m (scaled from Drawing 24742_01_P Rev.0)
Affected width (L) = 12.5 m (furthest point of wall from the proposed excavation due to their relative angles) – 0.5 m (distance to the wall) = 12.0 m
Height of No. SMH (H) = 12.3 m (the same height as No. 31 SMC) + 0.5 m (footing depth) = 12.8 m
Hence L/H = 0.9

6.16 In this case the pro-rata reduction of the 5 mm maximum horizontal displacement (see Section 6.1) has been ignored as the excavation is almost one storey. Thus, the horizontal strain beneath the wall would, theoretically, be in the order of $\varepsilon_h = 4.17 \times 10^{-4}$ (0.042%).

6.17 The maximum settlement produced by the PDISP analysis beneath the location where the southern flank wall No. 57 GA is closest to the proposed basement was in Stage 3 where 2.0 mm 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.

6.18 The total predicted settlement (due to excavation) of 12.3 mm (see Section 6.8) is reduced to 10.5 mm when the assumed depth of foundations are taken into account. The total combined settlement of 12.5 mm, 10.5 mm predicted by the CIRIA methods plus the 2.0 mm predicted by PDISP, is detailed as the point immediately alongside the proposed basement (0 m) in Figure 19 below. Figure 19 presents the settlement curve from the basement wall to the maximum distance of affected ground, 12.0 m (see Section 6.15).

6.19 For the low stiffness ground support case (appropriate to underpinning), settlement is convex and gives a maximum vertical deflection, Δ = 2.0 mm as displayed in Figure 19 below, which represents a deflection ration Δ/L = 1.67 x 10-4 (0.017%).

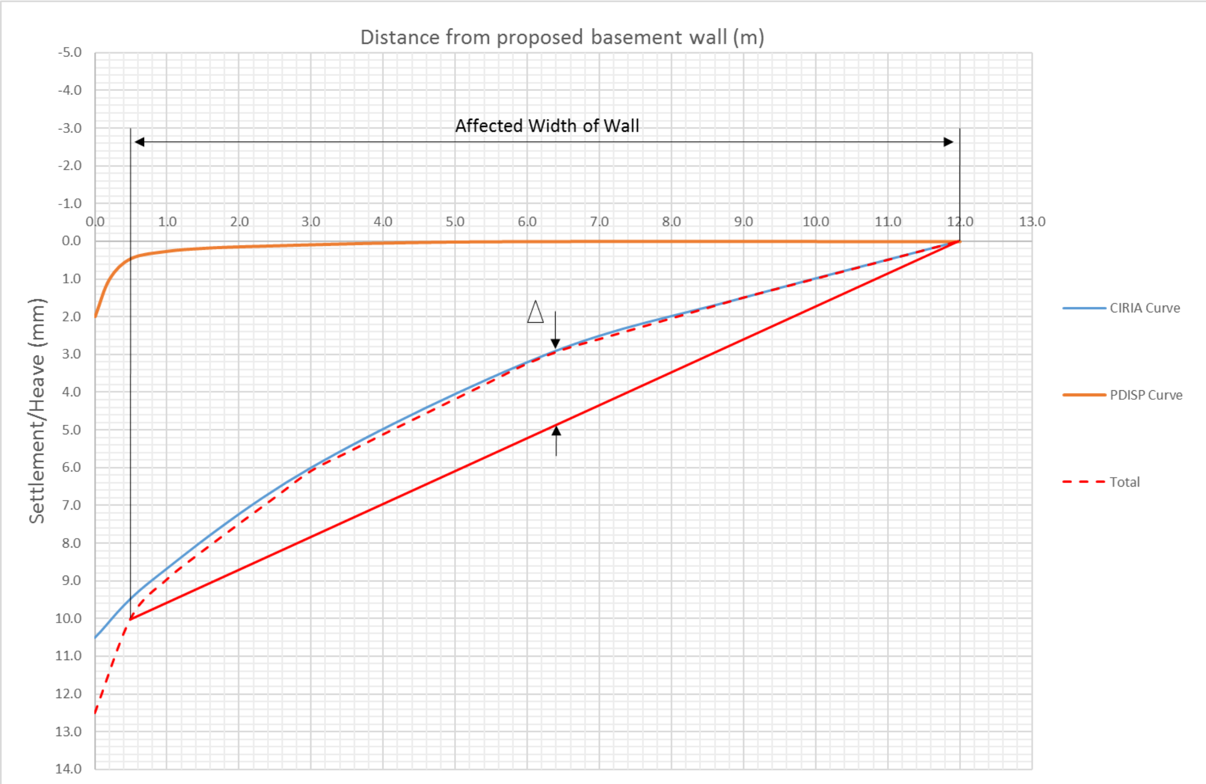

Figure 19. Combined displacements for No. 57 GA southern flank wall due to excavation of proposed basement

6.20 Using the damage category ratings and graphs given in CIRIA SP200, for L/H = 1.0 (A conservative value for the L/H of 0.9 defined in Section 6.9), these deformations represent a damage category of 'very slight' (Burland Category 1), as illustrated in Figure 20 below.

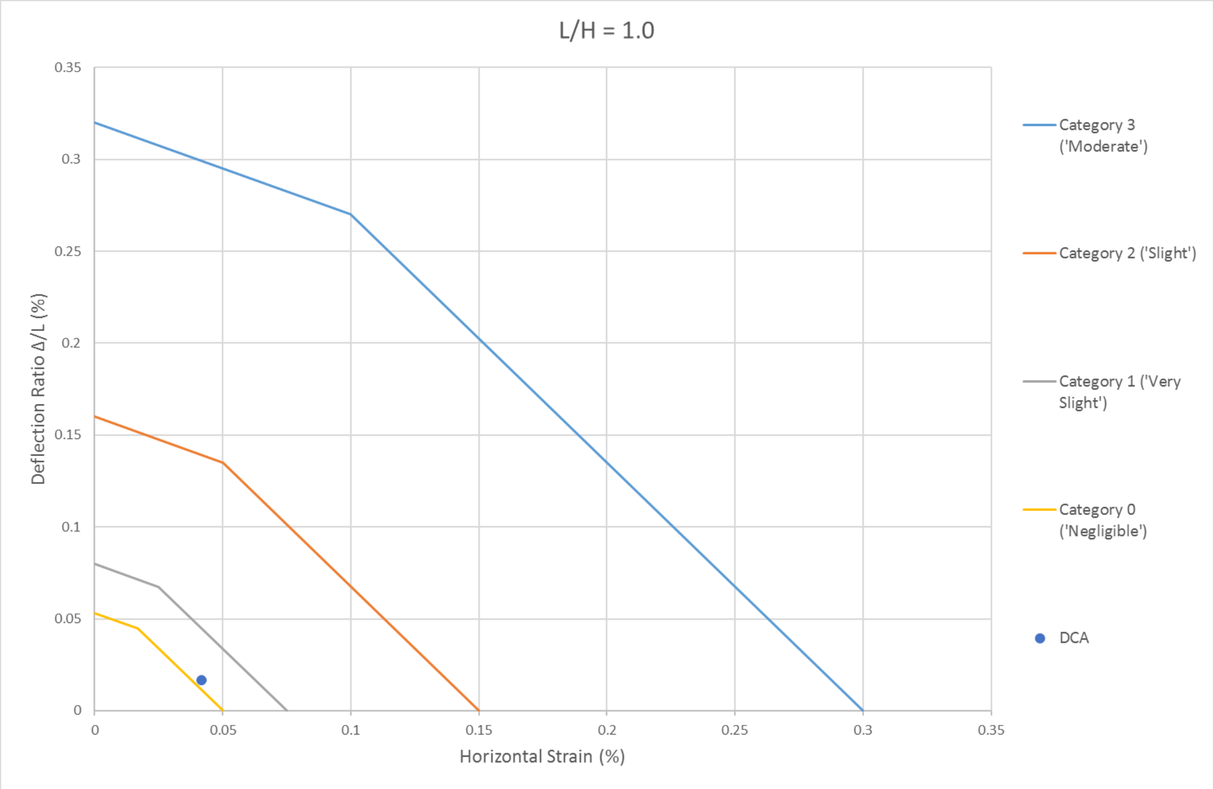

Figure 20: Damage category assessment for No. 57 GA's southern flank wall

6.21 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.

6.22 Due to the anticipated deeper foundation depths and increased distances from the proposed basement, other walls have not been assessed in detail. Therefore, the damage category assessment to all other walls within the affected zone are assumed to be Category 1 'very slight' or Category 0 'Negligible'.

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 in an area where flooding from rivers and seas is defined as very unlikely and the flood risk from surface water is very low. This combined with the lack of surface water features near the site, except for the lined Regent's Canal, can lead to the conclusion that conventional measures of managing surface water run-off (including Sustainable Drainage Systems (SuDS), if appropriate) should be sufficient to minimise any potential hydrological impacts.

7.3 The site is located above an 'Unproductive' stratum formed by the clay of the London Clay Formation. Groundwater has been encountered as high as 3.1 m blgl (0.4 m above the proposed founding level) during the monitoring visits and the basement will require waterproofing. However, the permeability of the London Clay Formation at the site is expected to be very low due to the high clay content and very little or no natural groundwater flow is anticipated. Thus, the proposed basement is not anticipated to have any impact on the groundwater flows/levels and no significant impact on neighbouring properties would be expected. Due to the rising groundwater levels recorded in the standpipe in BH1 during the monitoring visits, continued monitoring is recommended to identify a stable design height for the groundwater level.

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

7.5 The site is located on relatively level ground, with no noticeable slope gradient, therefore slope stability will be highly unlikely to cause any problems with the proposed basement.

7.6 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 13 – 15.

7.7 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.

7.8 In the assessed cases, the front wall of No. 1 St Mark's Crescent and the southern flank wall of No. 57 Gloucester Avenue, fell within Burland Category 1 'very slight' (as given in CIRIA SP200, Table 3.1). The damage category results have been plotted graphically in Figures 18 and 20 above.

7.9 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' or Category 1 'very slight'.

7.10 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.

References

Arup (2010). Camden Geological, Hydrogeological and Hydrological Study – Guidance for Subterranean Development, Issue01, November 2010.

Barton, N (1992). The Lost Rivers of London. Historical Publications, London.

Burland J.B., et al (2001). Building response to tunnelling. Case studies from the Jubilee line Extension, London. CIRIA Special Publication 200.

Chelmer Site Investigation Laboratories Limited (2016). Factual Report, 31 St Mark's Crescent, London NW1 7TT. Report FACT/8084.

Chelmer Site Investigation Laboratories Limited (2017). Factual Report, 31 St Mark's Crescent, London NW1 7TT. Report FACT/8084A.

Gaba A.R., et al (2003). Embedded retaining walls – guidance for economic design. CIRIA Report C580.

London Borough of Camden (2015). Camden Planning Guidance CPG4, Basements and Lightwells, July 2015.

London Borough of Camden (2013). Floods in Camden, Report of the Floods Scrutiny Panel, June 2003.

URS (2014). London Borough of Camden Strategic Flood Risk Assessment, Final Report, July 2014.

End of report

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