soils
LIMITED

Basement Impact Assessment

at
163 Sumatra Road, London NW6 1PN

for
Drawing and Planning Limited

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This is not a valid document for use in the design of the project unless it is titled Final in the document status box.

Current regulations and good practice were used in the preparation of this report. The recommendations given in this report must be reviewed by an appropriately qualified person at the time of preparation of the scheme design to ensure that any recommendations given remain valid in light of changes in regulation and practice, or additional information obtained regarding the site.

Commission

Soils Limited was commissioned by Drawing and Planning Limited to undertake a Basement Impact Assessment on a property at 163 Sumatra Road, London NW6 1PN. The scope of the investigation was outlined in the Soils Limited quotation reference Q20659, dated 23rd October 2018 and on the email to the Client dated 8th July 2020.

This BIA report must be read in conjunction with the Ground Investigation Report ref. 13291/GIR, dated December 2012 and with the Basement Impact Assessment report with ref. 13291/BIA_Rev1.02, dated August 2015, both undertaken by Soils Limited.

Soils Limited also produced a Preliminary Basement Impact Assessment report, ref. 17279/PBIA, dated December 2018. Due to changes to the London Borough of Camden policy with regards to the construction of basements and lightwells, to building conditions and to the proposed structural scheme a revised Basement Impact Assessment, report ref. 17279/BIA, dated October 2019, was produced.

This revised BIA report was produced to account for minor changes to the layout received in July 2020, was based on the findings presented in the above mentioned documents, which are now superseded, and provides the results of ground movement assessment and expected damage category for the to date final version of the proposed building layout.

No Phase I Desk Study was undertaken on the above site by Soils Limited, as this did not form part of the Client's brief.

Standards

The site works, soil descriptions and geotechnical testing was undertaken in accordance with the following standards:

- BS 5930:2015
- BS EN ISO 22476-3:2005+A1:2011
- BS EN 1997-1:2004+A1:2013 Eurocode 7
- BS EN ISO 14688-1:2002+A1:2013
- BS EN ISO 14688-2:2004+A1:2013
- BS 10175:2011+A1:2013
- BRE Digest 240
- NHBC Standards 2020
- CIRIA SP200 – Building response to tunnelling
- CIRIA C760 – Guidance on embedded retaining wall design.

- Burland J.B., et al (2001). Building response to tunnelling. Case studies from the Jubilee line Extension, London. CIRIA Special Publication 200.
- Gaba A.R., et al (2003). Embedded retaining walls – guidance for economic design. CIRIA Report C580.
- Camden geological, hydrogeological and hydrological study, Guidance for subterranean development, Issue01/November 2010
- Environment Agency Water Framework Directive
- Strategic Flood Risk Assessment (SFRA)
- Property Asset Register Public Web Map, Transport for London
- The Lost Rivers of London, Historical Publications Ltd, 1992, N Barton

The geotechnical laboratory testing was performed by K4 Soils Laboratories in accordance with the methods given in BS 1377:1990 Parts 1 to 8 and their UKAS accredited test methods.

For the preparation of this report, the relevant BS code of practice was adopted for the geotechnical laboratory testing technical specifications, in the absence of the relevant Eurocode specifications (ref: ISO TS 17892).

Trial hole is a generic term used to describe a method of direct investigation. The term trial pit, borehole or window sample borehole implies the specific technique used to produce a trial hole.

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Section 1 Introduction

1.1 General

Soils Limited was commissioned by Drawing and Planning Ltd to undertake an intrusive investigation and to produce a Basement Impact Assessment for a proposed development to take place at 163 Sumatra Road, London NW6 1PN in 2016.

As a general overview, the basement construction to the site was to a mid-terraced two storey house with party walls to the adjacent properties.

The construction of the basement was already under way in 2016, when underpinning works were carried out under the party walls but ceased when planning consent from the London Borough of Camden was refused in January 2017.

The works recommenced in early January 2018 but were stopped again in February 2018 because of the collapse of half of the front façade and of all the internal structures. The Client stated that the cause for the collapse was a grab lorry repeatedly hitting the façade during the collection of waste from the site.

Three rounds of emergency temporary works were put in place by the building contractor to a specific design carried out by the Client's structural engineering consultant to ensure the safety of the remaining structure and of the neighbouring/adjoining properties.

The temporary works design and implementation were agreed with representatives of London Borough of Camden Building Control officer, David Page Health and Safety Ltd, Martin Redston Associates, Yoop Architects, PW Surveyors, the sites neighbours and Insurers, after the collapse.

Consultations among the above-mentioned people pointed out the need for removing the remaining internal structures and rear wall from the collapsed building and the bracing of the party walls to be put implemented before restarting the basement works.

A new building proposal was therefore prepared to take into account the new proposed scheme and specific design must be carried out for the new planning application.

This Basement Impact Assessment (BIA) was required to be part of the new planning proposal and was prepared with reference to the revised proposed development provided by the Client to Soils Limited in July 2020, better defined from the documents mentioned in paragraph 2.3 of this report and presented in Appendix D. This BIA report, therefore, overwrites and supersedes all the previous versions already issued.

In particular, the report comprised the results of a Phase I Desk Study undertaken by Soils Limited, devoted to investigating the feasibility of the proposed development from the point of view of geology, hydrology, hydrogeology, flood risk and eventual presence of nearby underground structures, and the results of a specific intrusive investigation, comprising both site investigation and laboratory testing, to define ground conditions, soil mechanical properties and expected bearing capacity and settlements associated with the proposed construction.

Screening and scoping sections were also prepared for assessing the foreseeable issues associated with the build.

A detailed ground movement assessment (GMA) was prepared in agreement with the methods and procedures described within CIRIA SP200, CIRIA C580 and CIRIA C760. The subsequent evaluation of the expected damage category was carried out according to the Burland's damage categories, as described within CIRIA C580 and C760.

This report also included a non-technical summary of the proposed development and assessment, presented in paragraph 11.2, in agreement with the requirements of the *London Borough of Camden Development Policy DP27 – Basements and Lightwells*.

1.2 Objective of Investigation

This report comprises a Basement Impact Assessment which is in agreement with the *London Borough of Camden Development Policy DP27 – Basements and Lightwells* and the *LB Camden guidance document "Camden geological, hydrogeological and hydrological study – Guidance for subterranean development"* produced by Arup to describe a risk-based impact assessment with regard to hydrology, hydrogeology and land stability. This has been used as relevant background technical guidance to the development of the Basement Impact Assessment (BIA).

The objective of this investigation was to establish the impact and risk of the proposed basement at 163 Sumatra Road, London NW6 1PN. The assessment would determine the impact on the surroundings structures with respect to groundwater and land stability and in particular to assess whether the development will affect the stability of neighbouring properties, local and regional hydrogeology and whether any identified impacts can be appropriately mitigated by the design of the development.

It is recognised that any Basement Impact Assessment is a live document and that further detailed assessments will be ongoing, if appropriate, as the design and construction progresses.

1.3 Limitations and Disclaimers

This Basement Impact Assessment relates to the site located at 163 Sumatra Road, London NW6 1PN and was prepared for the sole benefit of Drawing and Planning Limited (The "Client") to the brief described in 1.2 of this report.

Soils Limited disclaims any responsibility to the Client and others in respect of any matters outside the scope of the above.

This report has been prepared by Soils Limited, with all reasonable skill, care and diligence within the terms of the Contract with the Client, incorporation of our General Conditions of Contact of Business and taking into account the resources devoted to us by agreement with the Client.

The report is personal and confidential to the Client and Soils Limited accept no responsibility of whatever nature to third parties to whom this report, or any part thereof, is made known. Any such party relies on the report wholly at its own risk.

The Client may not assign the benefit of the report or any part to any third party without the written consent of Soils Limited.

The ground is a product of continuing natural and artificial processes. As a result, the ground will exhibit a variety of characteristics that vary from place to place across a site, and also with time. Whilst a ground investigation will mitigate to a greater or lesser degree against the resulting risk from variation, the risks cannot be eliminated.

The investigation, interpretations, and recommendations given in this report were prepared for the sole benefit of the client in accordance with their brief. As such these do not necessarily address all aspects of ground behaviour at the site.

Current regulations and good practice were used in the preparation of this report. An appropriately qualified person must review the recommendations given in this report at the time of preparation of the scheme design to ensure that any recommendations given remain valid in light of changes in regulation and practice, or additional information obtained regarding the site.

The depth to roots and/or of desiccation may vary from that found during the investigation. The client is responsible for establishing the depth to roots and/or of desiccation on a plot by plot basis prior to the construction of foundations. Supplied site surveys may not include substantial shrubs or bushes and is also unlikely to have data on any trees, bushes or shrubs removed prior to or following the site survey.

Where trees are mentioned in the text this means existing trees, substantial bushes or shrubs, recently (within the last 20 years) removed trees and those planned as part of the site landscaping.

Ownership of copyright of all printed material including reports, laboratory test results, trial pit and borehole log sheets, including drillers' log sheets, remains with Soils Limited. License is for the sole use of the client and may not be assigned, transferred or given to a third party.

Section 2 Site Context

2.1 Location

The site address is 163 Sumatra Road, London NW6 1PN approximately centred at OS Land Ranger Grid Reference TQ 252 848 and falling within the administrative boundaries of the London Borough of Camden, in the area of West Hampstead.

The site location plan is given in Figure 1.

2.2 Site Description

The site comprised a terraced house with a small front yard and rear garden. The site was bordered by further residential properties, gardens and Sumatra Road to the north.

The rear garden was grass surfaced with bushes noted to the rear boundaries of the property. A mature silver birch tree was noted on the pavement to the front of the property.

The site sloped downwards to the south, with the wider topography sloping at a shallow gradient downward in a south / southwest direction, with an average gradient of <2°.

Looking at available online historic maps the site was open land until the present property was built on it, circa 1890s. No discernible change to the property was noted up to the present day.

It must be reported, based on a site walkover conducted by Soils Limited in 2018, that the building had undergone collapse of half of the front façade and of much of the internal structures because of the reported repeated impact of a grab lorry removing the waste from previous stages of the basement development. Underpinning was already present under the party walls to a depth of about 3.80m below ground level as a part of the previous basement development.

The existing basement excavation was partially filled with gravel and rubble, predominantly towards the front of the partially excavated basement. There was also standing water within the lowest parts of the excavation, when a representative from Soils Limited visited the site on 31st October 2018.

An aerial photograph of the site has been included in Figure 2.

2.3 Proposed Development

The information available at the time of reporting considered the demolition of the existing structures and the construction of a 3-storey residential building, with full basement, lightwells to both the front and the rear of the property, which was to be subdivided into flats, and communal gardens.

In compiling this report reliance was placed on drawing numbers SMROD-L101, SMROD-P100 TO SMROD-P105, SMROD-E101, SMROD-E102 and SMROD-S101 to SMROD-S104, all dated June 2020 and prepared by Drawing and Planning, which superseded 1817-S01 Rev.A to 1817-S03 Rev.A, dated July 2019 and prepared by RP Designs, SMTRD-S701-703 and P700-705, dated June 2015, on scaffolding drawing no. 18.072.TP-22, dated February 2018, and on emergency works drawings no. 18.165.TW-200 to 18.165.TW-206, dated May 2018. A preliminary structural scheme, released as a draft from Martin Redston Associates, including drawings no. 18.165.1 Rev.A to 18.165.6 Rev.A, dated May 2018 and not to be used for design and construction purposes, were also made available and showed the detail of underpinning foundations. All the documents were prepared and supplied by the Client. Any change or deviation from the scheme outlined in these drawings could invalidate the recommendations presented within this report. Soils Limited must be notified about any such changes.

The proposed development layout as provided by the Client are presented in Appendix D.

2.4 Topography

The site gently sloped downwards on a southerly direction. The average slope angle was estimated as <2°, estimated from topographical data on plans provided by the Client.

2.5 Published Geological Data

The 1:50,000 BGS map showed the site to be located on bedrock of the London Clay Formation with no overlying superficial geology recorded.

2.5.1 London Clay Formation

The London Clay Formation comprises stiff grey fissured clay, weathering to brown near surface. Concretions of argillaceous limestone in nodular form (Claystones) occur throughout the formation. Crystals of gypsum (Selenite) are often found within the weathered part of the London Clay, and precautions against sulphate attack to concrete are sometimes required.

The upper boundary member of the London Clay Formation is known as the Claygate Member and marks the transition between the deep water, predominantly clay environment and succeeding shallow-water, sand environment of the Bagshot Formation.

The lower boundary is generally marked by a thin bed of well-rounded flint gravel and/or a glauconitic horizon. The formation overlies the Harwich Formation or where the Harwich Formation is absent the Lambeth Group.

In the north London area the upper part of the London Clay Formation has been disturbed by glacial action and may contain pockets of sand and gravel.

2.6 Available On-line Geology Data

A nearby borehole (BGS Reference: TQ28SE46), provided by the BGS website, records the London Clay Formation to a depth of approximately 74 m bgl, the Lambeth Group to approximately 88 m bgl and the Thanet Sand Formation to 96 m bgl, before reaching the Chalk Group.

2.6.1 Groundwater

Based on historic boreholes located within a radius of approximately 600m from the site, groundwater was reported as mainly absent, although in two boreholes it was encountered at depths ranging between 1.50m and 11.10m bgl in August 1981, within fissured layers of the London Clay Formation.

2.7 Hydrology

The nearest surface water feature was the Leg of Mutton Pond and associated spring line on West Hampstead Heath recorded 1.82 km northeast of the site. The site was recorded at an elevation of approximately 56 m AOD, and the Leg of Mutton Pond was at approximately 95 m AOD.

The site, however, lies within 100m of the Kilburn, one of the lost rivers of London, as reported in Figure 4, included within the Camden Geological, Hydrogeological and Hydrological Study produced by ARUP in 2010.

2.8 Hydrogeology

The Environment Agency has produced an aquifer designation system consistent with the requirements of the Water Framework Directive. The designations have been set for superficial and bedrock geology and are based on the importance of aquifers for potable water supply and their role in supporting water bodies and wetland ecosystems.

The London groundwater model was generally split into three aquifers, the Upper, Intermediate and Lower Aquifer.

> The Upper Aquifer was confined to the River Terrace Deposits, which were not anticipated onsite, overlying the London Clay Formation, which acts as an aquiclude.
>
> The Intermediate Aquifer was generally associated with granular layers within the Lambeth Group.
>
> The Lower Aquifer was principally associated with the Chalk but can include the Thanet Sand Formation.

Information presented by the Environment Agency classifies the London Clay Formation bedrock as unproductive strata.

Published geological data shows the site directly on the London Clay Formation, therefore the Upper Aquifer would not be present onsite. Any water infiltrating the London Clay Formation will generally tend to flow vertically downwards at a very slow rate towards the Intermediate and subsequently Lower Aquifer. Due to the predominantly cohesive nature of the soils, the groundwater flow rate is anticipated to be very slow. Published permeability data for the London Clay Formation indicates the horizontal permeability to generally range between 10^{-10} m/s and 10^{-8} m/s, with an even lower vertical permeability.

The Upper Aquifer, if present, was considered to be relevant to the proposed development and Basement Impact Assessment and must be confirmed via a ground investigation. The Intermediate and Lower Aquifers would not be affected in any way by the proposed works so were not considered further.

2.9 Flood Risk

The National Flood Information System considered the site not at risk of flooding for the action of rivers and sea, for breaches in reservoirs and for surface water. Areas at low risk of flooding for surface water, however, were recorded in the immediate surroundings of the house. Information from the NFIS was reported in Figure 5 to Figure 7.

The site lies within the Critical Drainage Area of West Hampstead (Group 3_010) and also within the Local Flood Risk Zone of Sumatra Road, according to the Strategic Flood Risk Assessment (SFRA) produced by URS in 2014, reported in Figure 8.

The Strategic Flood Risk Assessment also shows that Sumatra Road among a number of roads in Camden that were flooded in 1975 and 2002, as reported in Figure 9.

A site walkover was carried out on 31st October 2018 and the existing portion of the basement was found flooded, although it was unclear if the flooding was due to groundwater, surface water or other potential sources.

2.10 Underground Infrastructure

The railway line was located at about 25m to the south of the property. The nearest tunnel (Belsize Tunnels) was noted at about 600m to the east of the site.

Section 3 Screening

3.1 Introduction

The Ove Arup 2008 Scoping Study prepared for the London Borough of Camden requires that any development proposal that includes a subterranean basement should be screened to determine whether or not a full BIA is required.

A number of screening tools are included in the Arup document (Ref: Camden geological, hydrogeological and hydrological study, Issue01/November 2010), which includes a series of questions within a screening flowchart for three categories; surface water flow, groundwater flow and land stability. Responses to the questions are tabulated below.

3.2 Surface Flow and Flooding Screening Assessment

The response to the Surface Flow and Flood Screening Assessment is given in Table 3.1.

Table 3.1 – Surface Flow and Flooding Screening

Question	Response
1. Is the site within the catchment of the pond chains on Hampstead Heath?	No– It was located 1.82 km to the south-west and down-gradient of the nearest part of the pond chains on Hampstead Heath.
2. As part of the proposed site drainage, will surface water flows (e.g. volume of rainfall and peak run-off) be materially changed from the existing route?	No – Drainage will be taken to combined sewers in public highway.
3. Will the proposed basement development result in a change in the proportion of hard surfaced / paved areas?	**Yes** – The proposed development will comprise extension of the existing house and basement into the rear garden, which is currently soft landscaping.
4. Will the proposed basement development result in changes to the profile of the inflows (instantaneous and long term) of surface water being received by adjacent properties or downstream watercourses?	No – The increase of impermeable area to the rear of the house could increase the peak flow to existing surface water drainage, however there will be negligible impact to adjacent properties or downstream watercourses.
5. Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream watercourses?	No – All surface water will be taken to combined sewers in public highway not to a watercourse. Additionally, there were no Surface Water Features within a radius of 1.8 km, which could be affected by the development.
6. Is the site in an area known to be at risk from surface water flooding?	**Yes** – The NFIS reported the site as surrounded by areas with low risk of surface water flooding The Strategic Flood Risk Assessment prepared by ARUP shows that Sumatra Road was among a number of roads in Camden that were flooded in 1975 and 2002.

3.3 Subterranean (Groundwater) Screening Assessment

The response to the Subterranean (Groundwater) Screening Assessment is given in Table 3.2.

Table 3.2 – Subterranean (Groundwater) Screening

Question	Response
1a. Is the site located directly above an aquifer?	No –Geological maps show the site is located directly on bedrock of the London Clay Formation, an Unproductive Stratum.
1b. Will the proposed basement extend beneath the water table surface?	**Unknown**– It is considered unlikely given the setting of the site, but it may be that the proposed basement extends beneath the water table surface. It will need to be confirmed by a ground investigation.
2. Is the site within 100 m of a watercourse, well (used/ disused) or potential spring line?	**Yes** – The nearest Surface Water Feature a pond located ~1.82 km to the north-east, located at the south-western portion of Hampstead Heath. The site, however, lies within 100m from the Kilburn, one of the lost rivers of London.
3. Will the proposed basement development result in a change in the proportion of hard surfaced / paved areas?	**Yes** – The proposed development will comprise extension of the existing house and basement into the rear garden, which is currently soft landscaping.
4. As part of the site drainage, will more surface water (e.g. rainfall and run-off) than at present be discharged to the ground (e.g. via soakaways and/or SUDS)?	No – The area is not underlain by an aquifer, thus any increase will not impact upon groundwater flow or levels. Furthermore, drainage will be taken to combined sewers in public highway.
5. Is the lowest point of the proposed excavation (allowing for any drainage and foundation space under the basement floor) close to or lower than, the mean water level in any local pond or spring line?	**Unknown** – The nearest Surface Water Feature is a pond located ~1.82 km to the north-east, located in the south-western portion of Hampstead Heath. The site, however, lies within 100m from the Kilburn, one of the lost rivers of London, the elevation of which is not known.

3.4 Stability Screening Assessment

The response to the Stability Screening Assessment is given in Table 3.3.

Table 3.3 – Stability Screening

Question	Response
1. Does the existing site include slopes, natural or manmade, greater than 7°?	No – The site was noted to have a gentle fall from north to south of <2°. This was estimated from topographical data on plans provided by the Client.
2. Will the proposed re-profiling of landscaping at the site change slopes at the property boundary to more than 7°?	No – The proposed basement is not to alter existing site landscaping elevations.
3. Does the development neighbour land, including railway cuttings and the like, with a slope greater than 7°?	No – The wider area was noted to be generally flat and level with a gentle slope to south and south-west that was calculated to be <2° (4.3 m drop in elevation over a distance of 200 m in a north-east to south-west direction). No railway cuttings were in the close vicinity of the site.
4. Is the site within a wider hillside setting in which the general slope is greater than 7°?	No – The wider area was noted to be generally flat and level with a gentle slope to south and south-west that was calculated to be <2° (4.3 m drop in elevation over a distance of 200 m in a north-east to south-west direction).
5. Is the London Clay the shallowest strata at the site?	**Yes** – The London Clay Formation is recorded as the shallowest strata, to be confirmed by the ground investigation.

Question	Response
6. Will any trees be felled as part of the proposed development and / or are any works proposed within any tree protection zones where trees are to be retained?	No – It is understood that no trees will be felled during the development.
7. Is there a history of seasonal shrink-swell subsidence in the local area and / or evidence of such effects at the site?	Unknown – Anticipated geology was London Clay Formation, which would potentially be subject to shrink-swell subsidence. There was no visual evidence of subsidence at the surrounding structures.
8. Is the site within 100 m of a watercourse or potential spring line?	No – The nearest Surface Water Feature a pond located ~1.82 km to the north-east, located at the south-western portion of Hampstead Heath.
9. Is the site within an area of previously worked ground?	No – The relevant geological map did not show any Made Ground or Worked Ground within or in close proximity to the site.
10. Is the site within an aquifer?	No - Geological maps show the site is located on bedrock of the London Clay Formation, an Unproductive Stratum.
11. Is the site within 5 m of a highway or pedestrian right of way?	**Yes** – the site is adjacent to Sumatra Road to the north and a pedestrian pathway (Black Path) to the south.
12. Will the proposed basement significantly increase the differential depth of foundations relative to neighbouring properties?	**Yes** – the proposed basement is under an existing terraced house with properties to both sides.
13. Is the site over (or within the exclusion zone of) any tunnels, e.g. railway lines?	No – the site is located ~25 m to the north of a railway line. The nearest tunnel (Belsize Tunnels) was noted ~600 m to the east of the site.

3.5 Summary

Based on the screening exercise, further stages of the Basement Impact Assessment are required. A summary of the Basement Impact Assessment requirements has been provided in Table 3.4, Table 3.5 and Table 3.6

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Table 3.4 – Surface Flow and Flooding Screening

Item	Description
Q3	Proposed basement will increase the proportion of hard surfaced /paved areas.
Q6	Site is at risk from surface water flooding.

Table 3.5 – Subterranean (Groundwater) Screening Assessment

Item	Description
Q1b	It is considered unlikely that the basement will extend beneath the water table surface, but it will need to be confirmed by a ground investigation.
Q2	The site lies within 100m from the lost river Kilburn.
Q4	Proposed basement will increase the proportion of hard surfaced /paved areas.
Q5	It is unknown if the excavation will extend close or beneath the mean water level, however it must be confirmed following to specific investigation or assumed as per the worst-case scenario.

Table 3.6 – Stability Screening Assessment

Item	Description
Q5	The London Clay Formation is recorded as the shallowest strata at the site.
Q7	Anticipated geology was London Clay Formation, which would potentially be subject to shrink-swell subsidence. There was no visual evidence of subsidence at the surrounding structures.
Q11	The proposed basement is located within 5 m of a highway or pedestrian right of way.
Q12	The proposed basement may increase the differential depth of foundations relative to neighbouring properties.

Section 4 Scoping

4.1 Introduction

The purpose of scoping is to assess in more detail the issues of concern identified in the screening process (i.e. where the answer is “yes” or “unknown” to any of the questions posed) to be investigated in the impact assessment. Potential hazards are assessed for each of the identified potential impact factors.

The scoping stage is furthermore to assist in defining the nature of the investigation required to assess the impact of the issues of concern identified in the screening process. The scope of the investigation must comply with the guidance issued by the London Borough of Camden Council and be a suitable basis on which to assess the potential impacts.

4.2 Potential Impacts

The following potential impacts were identified in Table 4.1.

Table 4.1 – Potential Impacts

Screening Flowchart Question	Potential Impacts	Discussion
Will the proposed basement development result in a change in the proportion of hard surfaced / paved areas?	Decrease recharge to the underlying ground. In areas underlain by aquifers this may impact upon groundwater flow/levels.	The geological map showed the site not to be underlain by an aquifer, however, this needs to be confirmed by ground investigation, comprising either trial pitting or borehole drilling to sufficient depth.
Is the site in an area known to be at risk from surface water flooding?	Property damage due to surface water either in the form of flash flooding due to surface run-off, rising groundwater, inadequate drain/sewer capacity or inadequate drain/sewer maintenance. Please note that as stated in “Camden Planning Guidance – Basement and Lightwells, CPG4”, Sumatra Road was among a number of roads in Camden that were flooded in 1975 and 2002.	The NFIS reported the site as surrounded by areas with low risk of surface water flooding The Strategic Flood Risk Assessment prepared by ARUP shows that Sumatra Road was among a number of roads in Camden that were flooded in 1975 and 2002.Developer to undertake a Flood Risk Assessment in accordance with PPS25.
Will the proposed basement extend beneath the water table surface?	Alteration of existing groundwater flow regime, which in turn could potentially cause local increase or decrease of groundwater levels.	It may be that the proposed basement extends beneath the water table, though this will need to be confirmed by a ground investigation, as locally perched pockets of groundwater could be present. Well installation and groundwater monitoring will be necessary.
Is the London Clay the shallowest strata at the site?	Potential for shrink-swell subsidence in ground surrounding proposed basement.	Ground investigation to establish soil conditions by means of boreholes and laboratory analysis (Atterberg Limit Tests). Effects mitigated at design stage.

Screening Flowchart Question	Potential Impacts	Discussion
Is there a history of seasonal shrink-swell subsidence in the local area and / or evidence of such effects at the site?	Changes to vegetation on site could adversely affect foundations of adjoining structures.	Ground investigation to establish soil conditions by means of boreholes. Effects mitigated at design stage.
Is the site within 5 m of a highway or pedestrian right of way?	Excavation of a basement could result in structural damage to the roads/ footways or buried services.	Site investigation to establish soil conditions and evaluation of expected movements and damages on the highway structures. Effects mitigated at design stage.
Will the proposed basement significantly increase the differential depth of foundations relative to neighbouring properties?	Basement construction can result in undermining of foundations of neighbouring properties and cause excessive ground movements resulting in structural instability.	Based on information supplied by the client, the properties adjoining the site do not include full basements. Therefore, they either have no basement, with foundations assumed to a depth of at least 1.0 m bgl or semi-basements (as the property on site). In both cases, given that the proposed basement levels are anticipated to be only at ~2.0 m bgl at its front (north) and ~1.0 m bgl at its rear (south) edge, the differential depth increase was not significant. Site investigation to establish soil conditions and details of existing foundations by means of hand excavated trial pit(s).
Is the site within 100 m of a watercourse, well (used/ disused) or potential spring line?	Potential flooding of the premises.	If confirmed by further investigation or assumed as per the worst-case scenario, the risk of flooding must be mitigated at design stage.
Is the lowest point of the proposed excavation (allowing for any drainage and foundation space under the basement floor) close to or lower than, the mean water level in any local pond or spring line?	Potential flooding of the premises.	If confirmed by further investigation or assumed as per the worst-case scenario, the risk of flooding must be mitigated at design stage.

Section 5 Intrusive Investigation

5.1 Proposed Project Works

The proposed intrusive investigation was designed to provide information on the ground conditions and to aid the design of foundations for the proposed residential development. The intended investigation, as outlined within the Soils Limited quotation (Q13124 Rev1.02, dated 18th October 2012), was to comprise the following items:

- 2No. windowless sampler boreholes and dynamic probes;
- 2No. groundwater monitoring wells;
- 2No. hand dug trial pits to expose existing foundations;
- Geotechnical laboratory testing.

5.1.1 Actual Project Works

The actual project works were undertaken on 1st November 2012 and comprised:

- 3No. windowless sampler boreholes (WS1 – WS3);
- 1No. dynamic probe (DP1);
- 3No. groundwater monitoring wells;
- 1No. trial pit for foundation exposure;
- Geotechnical laboratory testing.

The three windowless sampler boreholes (WS1 – WS3) were backfilled with gravel and bentonite following the installation of monitoring wells. The trial pit was backfilled with arisings.

Each well comprised a 38 mm diameter standpipe with a gravel filter surround. Slotted casing was used from 5.0-1.0 m bgl and plain casing with a bentonite seal, to prevent entry of surface water, from 1.0 m bgl to surface. A lockable 'top-hat' cover completed the installation.

All trial hole locations have been presented in Figure 3.

Following completion of site works, soil cores were logged and sub sampled so that samples could be sent to the laboratory for both contamination and geotechnical testing.

5.2 Ground Conditions

The scoping intrusive investigation was carried out by Soils Limited on 1st November 2012 and comprised three windowless sampler boreholes (WS1 to WS3), were drilled on site, at locations given by the Client, where access could be gained and no live services were identified, to a depth of 5.0 metres below ground level (m bgl). Given the difference in elevation between the front and rear gardens of the property, WS1-WS2 (front) were drilled at an elevation ~0.8 m higher than WS3 (rear).

Standpipe piezometers were installed in the boreholes to a depth of 5.0 m bgl to allow long term groundwater level monitoring following installation, as agreed with the Client. Groundwater monitoring was undertaken on 4No. occasions, the results of which are presented in Table 5.6.

A super-heavy (DPSH-B) dynamic probe (DP1) was driven adjacent to one of the boreholes (WS1), prior to its construction, to a depth of 6.0 m bgl.

One trial pit (TP1) was hand dug at a location given by the client, to a depth of 1.22 m bgl, to expose and record the former foundations to the original structure.

The trial hole locations are outlined in Figure 3, while Table 5.1 outlines the depths of each trial-hole.

Table 5.1 – Investigatory Depths of Trial Holes

Trial Hole	Final Depth (m bgl)
WS1	5.0
WS2	5.0
WS3	5.0
DP1	6.0
TP1	1.22

The soil conditions encountered were recorded and soil sampling commensurate with the purposes of the investigation was carried out. The depths given on the borehole logs and quoted in this report were measured from ground level directly adjacent to the boreholes.

The soils encountered from immediately below ground surface have been described in the following manner. Where the soil incorporated an organic content such as either decomposing leaf litter or roots or has been identified as part of the *in-situ* weathering profile, it has been described as Topsoil both on the logs and within this report. Where the soil has, in general, been found to have the same composition as the 'Topsoil' but also incorporated a minor constituent, e.g. less than an estimated 5%, of possibly non-naturally occurring material, or is of uncertain origin, the soil has been described as Topsoil/Made Ground both on the log and within this report. Where man has clearly either placed the soil, or the composition has been altered to a degree greater than an estimated 5% of a non-natural constituent, it has been referred to as Made Ground both on the logs and within this report.

For more complete information about the soils encountered within the general area of the site reference should be made to the detailed records given within Appendix A, but for the purposes of discussion, the succession of conditions encountered in the trial-holes, in descending order, were:

Made Ground (MG)
London Clay Formation (LC)

The ground conditions encountered in the trial holes are summarised in Table 5.2.

Table 5.2 – Ground Conditions

Strata	Epoch	Depth Encountered (m bgl) Top	Depth Encountered (m bgl) Bottom	Typical Thickness (m)	Typical Description
MG	Recent	GL	0.60-1.22[1]	0.90	Dark brown sandy silt/silty clay with brick and concrete fragments, ash, gravel and roots.
LC	Eocene	0.60-1.22[1]	5.00[1] (inferred to 6.00)	Not proven[2]	Orange brown to dark brown and grey mottled silty CLAY with occasional gravel horizons and very occasional selenite crystals and fine roots.

Note: [1] Final depth of trial hole. [2] Base of strata not encountered

5.3 Ground Conditions Encountered in Trial Holes

The ground conditions encountered in trial holes have been described below in descending order. The engineering logs are presented in Appendix A.1.

5.3.1 Made Ground

Made Ground was encountered from surface in WS3 or directly beneath a thin capping of concrete (0.04 m - 0.06 m) in WS1-WS2 and TP1 and comprised dark brown sandy silt/silty clay with brick and concrete fragments, ash, gravel and roots.

Made Ground was proved to depths ranging between 0.60 m bgl in WS3 and 1.10 m bgl in WS1 and was found for the full depth of TP1 to 1.22 m bgl.

The depth of Made Ground has been included in Table 5.3.

Table 5.3 – Final Depth of Made Ground

Trial Hole	Depth (m bgl)
WS1	1.10
WS2	0.95
WS3	0.60
TP1	1.22[1]

Note: [1] Final depth of trial hole.

5.3.2 London Clay Formation

The soils of the London Clay Formation were found directly beneath the soils of the Made Ground in each of the trial holes, except TP1, and comprised orange brown to dark brown and grey mottled silty CLAY with occasional gravel horizons and very occasional selenite crystals and fine roots.

The geological records indicate a thickness of about 74 metres of the London Clay Formation in this area.

The depth of London Clay Formation has been included in Table 5.4.

Table 5.4 – Final Depth of London Clay Formation

Trial Hole	Depth (m bgl)
WS1	5.00[1]
WS2	5.00[1]
WS3	5.00[1]
DP1	6.00[1]
TP1	-

Note: [1] Final depth of trial hole.

5.4 Roots

Roots were encountered in WS1 and WS2 to depths of 2.10 m and 1.50 m bgl, respectively, but were not encountered in WS3 or TP1, located to the rear of the property.

It must be emphasised that the probability of determining the maximum depth of roots from a narrow diameter borehole is low, thus a direct observation such as from within a trial pit is necessary to gain a better indication of the maximum root depth.

The depths of root penetration have been included in Table 5.5.

Table 5.5 – Depth of Root Penetration

Trial Hole	Depth (m bgl)
WS1	2.10
WS2	1.50
WS3	Not encountered
TP1	Not encountered

Roots may be found to greater depth at other locations on the site particularly close to trees and/or trees that have been removed both within the site and its close environs.

5.5 Groundwater

Groundwater was not encountered during the borehole drilling or the excavation of the trial pit; however, the speed of drilling may have masked any groundwater strikes.

Groundwater equilibrium conditions may only be conclusively established by means of a series of measurements made in piezometers installed in the ground after completion of site works.

Groundwater monitoring wells were installed in each of the boreholes to a depth of 5.0 m bgl.

Short-term and standing groundwater levels, where found, during the drilling of the boreholes and the groundwater monitoring are presented in Table 5.6. It must be noted that the groundwater readings were undertaken as part of the original intrusive ground investigation undertaken by Soils Limited in December 2012.

Table 5.6 – Groundwater Monitoring

Trial Hole	Depth to Water (m bgl)				
	1/11/12	13/11/12	26/11/12	19/12/12	15/1/13
WS1	Not encountered	4.10	3.09	2.43	2.23
WS2	Not encountered	2.12	1.97	2.09	2.06
WS3	Not encountered	No access	No access	No access	No access
TP1	Not encountered	-	-	-	-

Note: Given the different in elevation between the front and rear gardens of the property, WS1-WS2(front) were drilled at an elevation ~0.8 m higher than WS3 (rear).

A site walkover was carried out on 31st October 2018, after the partial collapse of the front elevation and parts of the internal structure and installation of the temporary support works, and the lowest part of the existing portion of the basement was found to be flooded, although it was unclear if the flooding was due to groundwater, surface water or other potential sources.

Changes in groundwater level do occur for a number of reasons including seasonal effects and variations in drainage. The site investigation and the groundwater monitoring were conducted between November 2012 and January 2013, when groundwater levels should typically be approaching their annual maximum (i.e. highest in March) elevation.

Section 6 Discussion of Geotechnical In-Situ and Laboratory Testing

6.1 Dynamic Probe Tests

Dynamic probing (DPSH) was undertaken at one location (DP1) adjacent and prior to the drilling of WS1 to a depth of 6.00m bgl. The results were converted to equivalent SPT "N" values based on dynamic energy using commercial computer software (Geostru). The results were then interpreted based on the classifications outlined in Appendix B.1, Table B.1.1.

It should be noted that SPT 'N' values quoted within Table B.2.1, presented in Appendix B.2 and referred to within this report, are presented as corrected values in accordance with BS EN 22476 Part 3, to account for the rig efficiency, borehole depth, overburden factors etc. Further correction of the 'N' values should therefore not be necessary. Raw field data is archived within the Soils Limited project file and can be provided on request.

The London Clay Formation recorded equivalent SPT "N" values between 3 and >50, increasing with depth, classifying the cohesive soils as very low to very high strength and inferred undrained cohesive strength of 15kPa to >250kPa.

A full interpretation of the DPSH tests is outlined in Appendix B.2, Table B.2.1.

6.2 Atterberg Limit Tests

Atterberg Limit tests were performed on five samples obtained from the London Clay Formation. The results were classified in accordance with BRE Digest 240 and NHBC Standards Chapter 4.2.

The London Clay Formation was classified as medium to high volume change potential in accordance with both BRE Digest 240 and NHBC Standards Chapter 4.2.

A full interpretation of the Atterberg Limit tests is outlined in Table B.2.2, Appendix B.2 and the laboratory report in Appendix B.3.

6.3 Sulphate and pH Tests

Two samples were taken from the London Clay Formation (WS1:2.70m bgl; WS3:3.20m bgl) for water soluble sulphate (2:1) and pH testing in accordance with Building Research Establishment Special Digest 1, 2005, 'Concrete in Aggressive Ground'.

The tests recorded water soluble sulphate between 90mg/l and 130mg/l with pH values of 8.1 and 8.2.

The significance of the sulphate and pH Test results are discussed in Section 7.4 and the laboratory report in Appendix B.3.

Section 7 Foundation Design

7.1 General

An engineering appraisal of the soil types encountered during the site investigation and likely to be encountered during the redevelopment of this site is presented. Soil descriptions are based on analysis of disturbed samples taken from the trial holes.

7.1.1 Made Ground

The terms Fill and Made Ground are used to describe material, which has been placed by man either for a particular purpose e.g. to form an embankment, or to dispose of unwanted material. For the former use, the Fill and/or Made Ground may well have been selected for the purpose and placed and compacted in a controlled manner. With the latter, great variations in material type, thickness and degree of compaction invariably occur and there can be deleterious or harmful matter, as well as potentially methanogenic organic material.

The BSI Code of Practice for Foundations, BS 8004:2015, Clause 4.1.2.2 states, *'Spread foundations should not be placed on non-engineered fill unless such use can be justified on the basis of a thorough ground investigation and detailed design.'*

Made Ground was encountered from surface in WS3 or directly beneath a thin capping of concrete (0.04 m - 0.06 m) in WS1-WS2 and TP1 and comprised dark brown sandy silt/silty clay with brick and concrete fragments, ash, gravel and roots. Made Ground was proved to depths ranging between 0.60 m bgl in WS3 and 1.10 m bgl in WS1 and was found for the full depth of TP1 to 1.22 m bgl. The depths of Made Ground have been included in Table 5.3.

A result of the inherent variability, particularly of uncontrolled Topsoil, Fill and/or Made Ground is that it is usually unpredictable in terms of bearing capacity and settlement characteristics. Foundations should, therefore, be taken through any Topsoil and/or Made Ground and either into, or onto a suitable underlying natural stratum of adequate bearing characteristics.

7.1.2 London Clay Formation

The soils of the London Clay Formation were found directly beneath the soils of the Made Ground in each of the trial holes, except TP1, and comprised orange brown to dark brown and grey mottled silty CLAY with occasional gravel horizons and very occasional selenite crystals and fine roots. The geological records indicate a thickness of about 74 metres of the London Clay Formation in this area. The depth of London Clay Formation has been included in Table 5.4.

The results from DPSH testing inferred that the cohesive soils of the London Clay Formation were of a **very low to very high strength,** with undrained cohesions of between **15kPa and >250kPa**.

The results from Atterberg Limits tests showed that the soils of the London Clay Formation had **medium to high volume change potential** in accordance with both BRE Digest 240 and NHBC Standards Chapter 4.2.

Soils of the London Clay Formation are overconsolidated and are expected to display moderate to high bearing capacities with low to moderate settlement characteristics. The soils of the London Clay Formation were considered as a suitable foundation layer for the proposed development.

7.1.3 Roots

Roots were encountered in WS1 and WS2 to depths of 2.10 m bgl and 1.50 m bgl, respectively, but were not encountered in WS3 or TP1, located to the rear of the property.It must be emphasised that the probability of determining the maximum depth of roots from a narrow diameter borehole is low, thus a direct observation such as from within a trial pit is necessary to gain a better indication of the maximum root depth. Roots may be found to greater depth at other locations on the site particularly close to trees and/or trees that have been removed both within the site and its close environs.

7.1.4 Groundwater

Groundwater was not encountered during the borehole drilling or the excavation of the trial pit; however, the speed of drilling may have masked any groundwater strikes.

Groundwater equilibrium conditions may only be conclusively established by means of a series of measurements made in piezometers installed in the ground after completion of site works.

Groundwater monitoring wells were installed in the boreholes to a depth of 5.0 m bgl.

Short-term and standing groundwater levels, where found, during the drilling of the boreholes and the groundwater monitoring are presented in Table 5.6.

Changes in groundwater level do occur for a number of reasons including seasonal effects and variations in drainage. The site investigation and the groundwater monitoring were conducted between November 2012 and January 2013, when groundwater levels should typically be approaching their annual maximum (i.e. highest in March) elevation.

A site walkover was carried out on 31st October 2018 and the lowest part of the existing portion of the basement was found to be flooded, although it was unclear if the flooding was due to groundwater, surface water or other potential sources.

7.2 Foundation Scheme

The information available at the time of reporting considered the demolition of the existing structure and the construction of a 3-storey residential building, with full basement, lightwells to both the front and the rear of the property and communal gardens.

In compiling this report reliance was placed on drawing numbers SMROD-L101, SMROD-P100 TO SMROD-P105, SMROD-E101, SMROD-E102 and SMROD-S101 to SMROD-S104, all dated June 2020 and prepared by Drawing and Planning, which superseded 1817-S01 Rev.A to 1817-S03 Rev.A, dated July 2019 and prepared by RP Designs, SMTRD-S701-703 and P700-705, dated June 2015, on scaffolding drawing no. 18.072.TP-22, dated February 2018, and on emergency works drawings no. 18.165.TW-200 to 18.165.TW-206, dated May 2018. A preliminary structural scheme, released as a draft from Martin Redston Associates, including drawings no. 18.165.1 Rev.A to 18.165.6 Rev.A, dated May 2018 and not to be used for design and construction purposes, were also made available and showed the detail of underpinning foundations. All the documents were prepared and supplied by the Client. Any change or deviation from the scheme outlined in these drawings could invalidate the recommendations presented within this report. Soils Limited must be notified about any such changes.

Development plans provided by the client are presented in Appendix D.

7.2.1 Guidance on Shrinkable Soils

The Building Research Establishment (BRE) Digests 240, 241 and 242 provide guidance on 'best practice' for the design and construction of foundations on shrinkable soils.

The results from Atterberg Limits Tests showed that the London Clay Formation had **medium to high volume change potential** in accordance with both BRE Digest 240 and NHBC Standards Chapter 4.2.

High volume change potential must therefore be adopted where foundations pass through the London Clay Formation.

The BRE Digest 241 states: "An increasingly common, potentially damaging situation is where trees or hedges have been cut down prior to building. The subsequent long-term swelling of the zone of clay desiccated by the roots, as moisture slowly returns to the ground, can be substantial. The rate at which the ground recovers is very difficult to predict and if there is any doubt that recovery is complete then bored pile foundations with suspended beams and floors should be used".

The stated intention of the NHBC is to ensure that shrinkage and swelling of plastic soils does not adversely affect the structural integrity of foundations to such a degree that remedial works would be required to restore the serviceability of the building. It must be borne in mind that adherence to the NHBC tables and design recommendations may not, in all cases, totally prevent foundation movement and cracking of brickwork might occur.

The BRE Digest 240 suggests: *“Two courses of action are open:*

Estimate the potential for swelling or shrinkage and try to avoid large changes in the water content, for example by not planting trees near the foundations.

Accept that swelling or shrinkage will occur and take account of it. The foundations can be designed to resist resulting ground movements or the superstructure can be designed to accommodate movement without damage.”

The design of foundations suitable to withstand movements is presented in BRE Digest 241 “Low-rise buildings on shrinkable clay soils: Part 2”.

7.3 Foundation Scheme

Foundations **must not** be constructed within any Made Ground/Topsoil and Fill due to the likely variability and potential for large load induced settlements both total and differential.

Roots were encountered in four out of the five trial holes at depths ranging between 1.50m bgl and 2.10m bgl. If roots are encountered during the construction phase foundations **must not be placed within any live root penetrated** or desiccated **cohesive soils or those with a volume change potential**. Should the foundation excavations reveal such materials, the excavations **must** be extended to greater depth in order to bypass these unsuitable soils. Excavations must be checked by a suitable person prior to concrete being poured.

Considering the type of development, a shallow foundation solution within the basement was considered the most suitable.

The proposed development is likely to be both light and brittle. It is therefore considered that foundation design is undertaken using NHBC Standards Chapter 4.2.

7.3.1 Shallow Foundations within Basement

Foundations constructed within the basement excavation could be considered and the bearing capacity of such foundations is given below. If the foundation is to include lateral load from retained soil, then the distribution of loads on the foundation will be trapezoidal and the maximum pressure will be at the toe of the foundation. In such cases additional analyses must be requested by the client such that the appropriate analyse is undertaken.

If the wall is to have backfill placed on both sides, the backfill must be placed in shallow rises on both sides to maintain similar lateral forces on both sides of the wall.

A proposed basement excavation 2.50m deep would remove an overburden pressure of 45kPa, increasing to about 70kPa at 3.80m bgl, based on a unit weight of 18kN/m^3, for the overlying soil.

Based on a 5.00 by 1.00m strip foundation, using commercial software Table 7.1 shows the calculated “**net**” allowable bearing capacity and anticipated settlement characteristics.

Table 7.1 – Allowable Bearing Capacities within the London Clay Formation

Depth (m bgl)	Size (m)	Bearing Capacity (kPa)	Anticipated Settlement (mm)
2.50	5.00 x 1.00	95	25
3.00		135	25
3.80		150	25

The maximum bearing capacity within the London Clay Formation at basement formation level must not exceed 150kPa

Taking account of the removed overburden pressure the “**gross**” bearing value could be taken as **140kPa** if founding at a depth of 2.50m bgl, **180kPa** if the formation level was at 3.00m bgl and 220kPa if founding at 3.80m bgl.

For the allowable bearing value given above, settlements **should not** exceed **25mm**, provided that excavation bases are carefully bottomed out and blinded or concreted as soon after excavation as is possible and kept dry. Settlements may be taken as proportional to the applied foundation pressure for the given size of the foundations.

The use of reinforced trench fill foundations must be used to reduce the potential of differential settlement across foundations, which was anticipated to be up to 15mm.

Settlements may be taken as proportional to the bearing capacity given for the same configuration of foundation.

Special care **must** be taken during foundation excavation in order to establish that any soft/loose spots found within the soils are removed from the base of excavations. These may well be found within the area at the front of the house where the proposed basement had already been excavated and standing open, with accumulated water within it.

Foundations must not be cast over foundations of former structures and other hard spots.

7.3.2 Stability Issues

The excavation of the basement **must not** affect the integrity of any adjacent structures beyond the site boundaries. Where there is a sufficient distance between the site boundary and the basement excavation, support may be permitted using a strip foundation to form an earth retaining structure. In other cases, the most suitable form of construction should be within a coffer dam structure using a sheet piles, secant or contiguous concrete piled wall around the periphery of the structure.

Cantilevered piled walls cannot be adopted in the current case because of the particular site conditions and proposed development layout. The adoption of traditional underpinning must be considered to transfer the loads to basement formation level and retain the surrounding soils.

7.3.3 Anticipated Heave

The evaluation of ground heave due to soil excavation was carried out in detail in paragraph 9.2.1 and 9.2.2. Immediate heave is likely to have a minimal effect as it would take place soon after excavation and any immediate heave is likely to be removed during the excavation of the basement slab in order to achieve the correct dig level prior to casting the slab.

It **must** be mentioned that it was assumed that excavations will be kept dry and either concreted or blinded as soon after excavation. If water is allowed for even a short time to enter excavations, not only will a greater heave be experiencing owing to the soil increasing in volume by taking up water, but the shear strength, and hence the bearing capacity, will also be reduced.

Notes: For the calculations of the immediate heave, the Ey (Young's Modulus) for uploading was taken as equal to the Ey for loading, which is considered to be a conservative approach. For the calculations of the long term swelling, the ratio of swelling index (Cs) compression index (Cc) was taken as Cs=Cc/5 *(Reference: Simon & Menzies, Foundation Engineering)*

7.4 Subsurface Concrete

Sulphate concentration measured in 2:1 water/soil extracts fell into Class **DS-1** of the BRE Special Digest 1 2005, *'Concrete in Aggressive Ground'.* Table C2 of the Digest indicated ACEC (Aggressive Chemical Environment for Concrete) site classifications of **AC-1**. The pH of the soils tested ranged between 8.1 and 8.2. The classification given was determined using the mobile groundwater case, as groundwater was recorded during groundwater monitoring. The laboratory results are presented in Appendix B.3.

Concrete to be placed in contact with soil or groundwater must be designed in accordance with the recommendations of Building Research Establishment Special Digest 1 2005, *'Concrete in Aggressive Ground'* taking into account any possible exposure of potentially pyrite bearing natural ground and the pH of the soils.

7.5 Excavations

Shallow excavations in the Made Ground and London Clay Formation are likely to be marginally stable in the short term at best.

Deeper excavations taken into the London Clay Formation are likely to become unstable as are those excavated through significant thickness of London Clay Formation or those taken below the groundwater table, where encountered.

Unsupported earth faces formed during excavation may be liable to collapse without warning and suitable safety precautions should therefore be taken to ensure that such earth faces are adequately supported or battered back to a safe angle of repose before excavations are entered by personnel.

Excavations beneath the groundwater table are likely to be unstable and dewatering of foundation trenches may be necessary.

Section 8 Preliminary Basement Impact Assessment

8.1 Mitigation of Adverse Effects

This section of the report addresses the potential impacts identified by the scoping study and the relevant findings of the ground investigation and mitigation measures, where required.

Will the proposed basement development result in a change in the proportion of hard surfaced / paved areas?

Potential Impacts: Decrease recharge to the underlying ground. In areas underlain by aquifers this may impact upon groundwater flow/levels.

Ground Investigation Findings: Windowless sampler borehole drilling revealed that the site was underlain by a thin capping of Made Ground over the soils of the London Clay Formation to 5.0 m bgl, which were established to comprise predominantly very low permeability CLAY. Therefore, the increased proportion of hard surfaced areas will not have an impact on groundwater flow/levels.

Mitigation: None required.

Is the site in an area known to be at risk from surface water flooding?

Potential Impacts: Property damage due to surface water either in the form of flash flooding due to surface run-off, rising groundwater, inadequate drain/sewer capacity or inadequate drain/sewer maintenance.

Ground Investigation Findings: The NFIS reported the site as surrounded by areas with low risk of surface water flooding The Strategic Flood Risk Assessment prepared by URS shows that Sumatra Road was among a number of roads in Camden that were flooded in 1975 and 2002.

Mitigation: Developer to undertake a **Flood Risk Assessment** in accordance with PPS25.

Will the proposed basement extend beneath the water table surface?

And

Is the lowest point of the proposed excavation (allowing for any drainage and foundation space under the basement floor) close to or lower than, the mean water level in any local pond or spring line?

Potential Impacts: Alteration of existing groundwater flow regime, which in turn could potentially cause local increase or decrease of groundwater levels.

Ground Investigation Findings: Windowless sampler borehole drilling revealed that the site was underlain by a thin capping of Made Ground over the soils of the London Clay Formation to 5.0 m, which were established to comprise predominantly very low permeable CLAY.

Furthermore, no groundwater was encountered either during the borehole drilling or the hand excavation of the trial pit.

The proposed basement slab levels are anticipated to be at ~2.0 metres below existing ground level (bgl) at its front (north) and ~1.0 metre at its rear (south) edge. The ground level at WS3 is -1.393 m (as shown in Fig. 3b), and ground level at WS1 and WS2 is reported as being ~0.8 m higher, i.e. -0.59 m. As such, the highest groundwater level recorded during the long-term groundwater monitoring in WS3 was -2.195 m (c.f. basement floor at -3.747 m) and the highest water level recorded in WS2 was -2.56 m (c.f. same basement floor level, almost up to the front lightwell). These levels have been plotted on to Figure 3b to illustrate the potential impact on the basement construction.

In addition, it must be mentioned that the groundwater monitoring was undertaken between November 2012 and January 2013, when groundwater levels are approaching their annual maximum (i.e. highest elevation typically in March). Therefore, it is likely that groundwater levels would increase to slightly higher levels than those recorded during this investigation, which would have a greater impact on basement construction.

Mitigation: Subject to the time of the year the basement excavation is to take place, dewatering is likely to be required to minimise the likelihood of constructing below the groundwater levels.

However, the magnitude of the change in water level ("damming effect") would be mitigated due to the following reasons:

a. The long axis of the footprint of the proposed basement is to be in alignment with the existing groundwater flow, therefore causing less deflection from its original path;
b. The absence of the "cumulative effect", which could have resulted by the existence of basements within the adjoining properties. As informed by the client, the adjoining properties do not include basements.

Is the London Clay the shallowest strata at the site?

Potential Impacts: London Clay Formation is the most prone to seasonal shrink-swell stratum from all the at-surface strata present in LB of Camden.

Ground Investigation Findings: Windowless sampler borehole drilling revealed that the site was underlain by a thin capping of Made Ground over the soils of the London Clay Formation to 5.0 m, which were established to comprise predominantly very low permeable CLAY.

The results of the Atterberg Limit testing indicated that the soils of the London Clay Formation fell into the BRE Digest 240 and the NHBC Standards Chapter 4.2 “medium to high volume change potential” classification.

Mitigation: The high volume change potential of the soils of the London Clay Formation must be taken into account in the design of the basement slab, in accordance with the relevant BRE Digest 240 and NHBC 4.2 Standards.

Is there a history of seasonal shrink-swell subsidence in the local area and / or evidence of such effects at the site?

Potential Impacts: Changes to vegetation on site could adversely affect foundations of adjoining structures

Ground Investigation Findings: Windowless sampler borehole drilling revealed that the site was underlain by a thin capping of Made Ground over the soils of the London Clay Formation to 5.0 m, which were established to comprise predominantly very low permeability CLAY. The hand excavation of a trial pit exposed the existing foundation that was **not** noted to have experienced any structural damage from heave or long-term swelling.

The results of the Atterberg Limit testing indicated that the soils of the London Clay Formation fell into the BRE Digest 240 and the NHBC Standards Chapter 4.2 “medium to high volume change potential” classification.

Mitigation: The high-volume change potential of the soils of the London Clay Formation must be taken into account in the design of the basement slab, in accordance with the relevant BRE Digest 240 and NHBC 4.2 Standards.

Is the site within 5 m of a highway or pedestrian right of way?

Potential Impacts: Excavation of a basement could result in structural damage to the roads/ footways or buried services.

Ground Investigation Findings: Construction of proposed basement will take place at a distance less than 5.0 m (~3.0 m) from Sumatra Road.

Mitigation: Design of permanent and/or temporary works to ensure induced ground movements are within tolerable limits and temporary works to prevent damage during construction

Will the proposed basement significantly increase the differential depth of foundations relative to neighbouring properties?

Potential Impacts: Basement construction can result in undermining of foundations of neighbouring properties and cause excessive ground movements resulting in structural instability.

Ground Investigation Findings: Based on information supplied by the client, the properties adjoining the site do not include full basements, which was further confirmed by the site investigation. Therefore, they either have no basement, with foundations assumed to a depth of at least 1.0 m bgl or semi-basements (as the property on site). The hand excavation of a trial pit exposed the existing foundation that was noted to extend to a depth of ~1.10 m bgl, which was assumed to be approximately the foundation depth of the adjoining properties.

Given that the proposed basement levels are anticipated to be only at ~2.0 m bgl at its front (north) and ~1.0 m bgl at its rear (south) edge, the differential depth increase was **not significant**.

Mitigation: Appropriate measures undertaken in design and construction phase**.** Close supervision will be made during the construction phase. Movement monitoring of neighbouring and nearby structures will be undertaken before construction starts and continued through the construction phase and for an appropriate period thereafter.

8.2 Effects of Basement Construction on Shallow Groundwater

The proposed redevelopment was to comprise the lateral extension of an existing basement of a 2/3-storey house, as well as the house itself. The proposed redevelopment was to have light wells at the front and rear adjoining the new basement areas.

The proposed basement slab levels are anticipated to be at ~2.0 metres below existing ground level (bgl) at its front (north) and ~1.0 metre at its rear (south) edge. The ground level at WS3 is -1.393 m (as shown in Fig. 3b), and ground level at WS1 and WS2 is reported as being ~0.8 m higher, i.e. -0.59 m. As such, the highest groundwater level recorded during the long-term groundwater monitoring in WS3 was -2.195 m (c.f. basement floor at -3.747 m) and the highest water level recorded in WS2 was -2.56 m (c.f. same basement floor level).

Given that the groundwater monitoring was undertaken between November 2012 and January 2013, when groundwater levels are approaching their annual maximum (i.e. highest elevation in March) there is potential for groundwater levels to increase slightly from those recorded.

The hydraulic gradient was shallow and flow rates would be very low and imperceptible as far as the development was concerned. Published data for the permeability of the London Clay Formation indicates the horizontal permeability to generally range between 10^{-10} m/s and 10^{-8} m/s, or a maximum of horizontal groundwater flow of the order 5mm a year.

The ARUP report raises the hazard of groundwater flow being impeded and creating a damming effect upslope. Subject to the time of the year the basement excavation is to take place, dewatering of limited scale maybe required to minimise the likelihood of constructing below the groundwater levels.

However, **the magnitude of the change in water level ("damming effect") would be mitigated** due to the following reasons:

a. The long axis of the footprint of the proposed basement is to be in alignment with the existing groundwater flow, therefore causing less deflection from its original path;

b. The absence of the "cumulative effect", which could have resulted by the existence of basements within the adjoining properties. As informed by the client, the adjoining properties do not include basements.

8.3 Surrounding Buildings

This section considers the potential effects of basement construction on nearby properties.

It must be noted that the party walls to the existing structure had already been underpinned, in addition a portion of the front elevation of the house and much of its internal structure to the front had collapsed. Temporary works had been designed and installed to retain the party walls, with bracing across the width of the structure roughly at former floor levels. No internal survey of the effects of the underpinning works undertaken pre collapse or post collapse and installation of temporary support works, were undertaken by Soils Limited and therefore, any damage to the existing party walls from the collapse or subsequent installation of the temporary works are not and could not be taken into consideration in the writing of this report.

Detrimental effects from the ongoing construction of the basement post collapse and temporary works installation would be manifested as cracking and more serious structural damage. Many old buildings in London do exhibit signs of historic movement and repair. In practice, it is often difficult to attribute cracks visible in a structure to specific site construction activities unless a detailed survey of the affected structure and its founding strata had been undertaken before the construction works.

Any observed changes in the state of the building can then be causally linked to the works with more confidence and less debate than if no pre-works condition survey had been undertaken. Surveys require the cooperation of the property owners, as entry by surveyors into the property will be necessary. This would normally be undertaken in collaboration with the neighbour's party wall surveyors.

Close supervision will be made during the construction phase. Movement monitoring of neighbouring and nearby structures will be undertaken before construction starts and continued through the construction phase and for an appropriate period thereafter.

The data from the site investigation has established soil and groundwater conditions. The client's engineer can prepare working drawings and construction method statements that will mitigate adverse effects on nearby properties.

8.4 Residual Impacts

On completion of the scheme there will be no residual effect on the environment or on nearby properties.

The proposed basement extension will not be a hindrance against the possibility of future basement construction to adjoining properties.

Section 9 Basement Impact Assessment

9.1 Introduction

Underpinning of party walls was already completed, as reported within the Underpinning Report, prepared by Drawing and Planning Ltd and dated 13th January 2017. Basement excavation and underpinning of the front and rear walls were still to be completed due to the recent history of the site redevelopment, which led to a long interruption of construction works.

The Ground Movement Assessment and the estimate of the foreseeable damage levels to neighbouring structures, however, was carried out with regards to the initial conditions in order to evaluate the impact of the full construction process.

It must be noted that the party walls to the existing structure had already been underpinned, in addition a portion of the front elevation of the house and much of its internal structure to the front had collapsed. Temporary works had been designed and installed to retain the party walls, with bracing across the width of the structure roughly at former floor levels. No internal survey of the effects of the underpinning works undertaken pre collapse or post collapse and installation of temporary support works, were undertaken by Soils Limited and therefore, any damage to the existing party walls from the collapse or subsequent installation of the temporary works are not and could not be taken into consideration in the writing of this report.

It must be pointed out that no signs of disruption and/or damage on the neighbouring properties were noted at the time of the walkover undertaken by Soils Limited on 31st October 2018 or reported by the Client at the time of writing. A big, sub-vertical crack, however, was observed on the rear wall of the property, at the corner with the adjoining property at 165 Sumatra Road. The crack, showed in Appendix A.2, will be eliminated with the demolition of the existing structures and the wall replaced with a new one.

This section provides calculations to determine ground movements that may result from the construction of the basement level and to assess how these may affect the conditions of neighbouring buildings.

Movements are likely to occur through the following mechanisms:

9.1.1 Heave Movements

The excavation will unload the cohesive and overconsolidated soils of London Clay Formation and this may cause a degree of heave, and/or settlement after construction depending on the final ground loading.

9.1.2 Foundation Construction

Construction of foundations can lead to settlement due to the net increase in loading. The nature of final settlements depends on the level of loading achieved. Downwards movements (settlements) must be expected when the applied load is greater than the weight of soil removed. On the contrary, a certain degree of heave will remain in the long term when the applied load is lower than the weight removed. Settlement may potentially also occur where foundation loads are transferred to deeper, previously unloaded soil.

Where foundations are not shared, or the properties linked, workmanship will not affect the adjoining structure and will not be considered within the ground movement analysis.

9.2 Ground Movement Arising from Basement Excavation

Ground movements induced by the construction of the existing building can be considered as completed. No information, instead, can be obtained with regards to the development of ground movements induced by the underpinning of the party walls, the excavation of the basement already undertaken and the presence of standing water within the excavation, because data from eventual previous monitoring was not available. The analysis of ground movements, therefore, could only be carried out with reference to the original, undisturbed conditions and the ground movement assessment developed according the worst-case scenario.

The structural layout of the proposed building considered the demolition of the existing structure and the construction of a steel framed building. The steel columns were proposed to be set onto the top of the already built underpinning. The considerable stiffness of the vertically loaded retaining structures is likely to allow the spreading of the applied point loads over the wall strip foundation. It is unclear, however, if the top of the underpinning could satisfactorily allow the installation of the steel columns, therefore two limit cases were considered, respectively considering the structural loads applied to strip footings (Case 1) and the point loads applied to pad footings (Case 2).

The soils at formation level will be subject to stress relief due to the excavation, as up to about 3.80m overburden is to be removed respectively under the footprint of the existing building and for the construction of the front and rear lightwells according to the proposed plans. This is likely to give rise to a minimal degree of heave over both the short and the long term or settlement over the longer term as structural loads are reapplied. A proposed basement excavation of 3.80m deep would remove an overburden pressure of about 70kPa, based on a unit weight of circa 18kN/m^3, for the overlying soil.

A ground movement assessment has been undertaken using OASYS Limited PDISP (Pressure induced DISPlacement analysis) analysis software. PDISP assumes that the ground behaves as an elastic material under loading, with movements calculated based on the applied loads and the soil stiffness (E_u and E’) for each stratum input by the user. PDISP assumes perfectly flexible loaded areas and as such tends to overestimate movements in the centre of loaded areas and underestimate movements around the perimeters. Notwithstanding this, a raft was considered in the analysis because of the characteristics of the proposed development. In the case a different solution is adopted within the final design the ground movement assessment must be edited accordingly.

In order to maximise the effects on the neighbouring structures, the structural loads applied at basement formation level were considered equal to the calculated bearing capacity of basement foundations as described in paragraph 7.3.1 and considered as evenly distributed underneath the strip footings.

The mechanical characteristics of the soils involved in the analyses were defined based upon information gathered at the time of the intrusive investigation and compared with reference values from CIRIA SP200. It was chosen to use in the calculations the results of the intrusive investigation for the Made Ground and the soils of the London Clay Formation between 1.00m and 3.50m bgl. The reference parameters reported in CIRIA SP200, instead, were adopted for the soils of the London Clay Formation at greater depth, as more conservative than the corresponding ones from the site investigation.

Excavations will take place predominantly within Made Ground and the upper London Clay Formation, both presenting a cohesive behaviour.

The parameters adopted within the calculation were summarised in Table 9.1 and Table 9.2.

Table 9.1 – Soil Parameters – Undrained Conditions

Stratum	Depth to Top of Stratum (m bgl)	Undrained Cohesion (kPa)	Young's Modulus (MPa)	Poisson's Ratio
MG	0.00	15	2.8	0.50
LCF	1.00	40	8.3	0.50
	3.50	>70	50.0	0.50

Table 9.2 – Soil Parameters – Drained Conditions

Stratum	Depth to Top of Stratum (m bgl)	Friction Angle (ϕ°)	Effective Cohesion (kPa)	Young's Modulus (MPa)	Poisson's Ratio
MG	0.00	20	0	2.1	0.35
LCF	1.00	22	0	6.2	0.35
	3.50	24	0	40.0	0.33

The excavation of the proposed basement could induce movements and potential damages on the buildings located respectively to the west (159 Sumatra Road) and to the east (165 Sumatra Road). In addition, effects can be produced on the road structures of Sumatra Road to the north. The three scenarios were used for the undertaking of the Damage Category Assessment (DCA) under the two cases of strip foundations (Case 1) and point loads (Case 2).

Scenario SC1 considered the effects of excavation and construction on adjoining building at 165 Sumatra Road.

Scenario SC2 considered the effects of excavation and construction on adjoining building at 159 Sumatra Road.

Scenario SC3 considered the effects of excavation and construction on the road structures of Sumatra Road.

The ground movements considered for scenarios SC1 and SC2 were due to excavation (heave in the short and long term), to the application of structural loads and to workmanship errors. No workmanship error was considered for scenario SC3 because no application of dry pack was needed.

No information was made available to Soils Limited with regards to the structural layout of the adjoining buildings, therefore it was assumed to adopt a critical distance of 5000mm for both the cases in SC1 and SC2 due to the similarities with the building for the proposed development. The expected damage on the road structures of Sumatra Road (SC3) was calculated over a critical distance of 9000mm.

Information received from the Client with regards on the structural characteristics of the basement retaining structures reported the retaining walls were RC walls of a minimum thickness of 300mm, with a second moment of inertia of 225,000cm^4 per metre length.

It is the Client's responsibility to provide information on eventual changes to the layout and structural characteristics of the basement.

The critical sections considered in scenarios SC1 – SC3 were presented in Figure 10.

9.2.1 Short Term Heave

The underpinning of the front and rear walls and the excavation are still to be completed. The presence of standing water within the excavations and the softening of soils to be excavated created soft spots, the real behaviour of which remains unpredictable.

The soils at formation level will be subject to stress relief due to the excavation, as up to about 3.80m of overburden is to be removed under the footprint of the existing building and to the front and rear for the construction of lightwells according to the proposed plans. A proposed basement excavation 3.80m deep would remove an overburden pressure of circa 70kPa based on a unit weight of about 18kN/m^3 for the overlying soil.

Calculated short term heave, due to the removal of soils above the formation level, was evaluated by adopting the parameters in Table 9.1 and intended as deriving from the unloading of the soils of London Clay Formation.

The largest short-term heave across the footprint of the proposed development was predicted to be of a maximum of <-9mm (negative values indicate an upwards movement throughout this text). The movement decreases towards the boundaries of the excavation, along the boundary lengths of the basement. Heave was noted to occur within these areas ranging between -2mm and -5mm due to the net increase of surcharge load. A contour plot showing the variation of short-term movements across the entire basement footprint is presented in Figure 11.

9.2.2 Long Term Ground Movement

The maximum load applied by the construction of the structure was conservatively considered equal to the bearing capacity as defined in paragraph 7.3.1.

Long term movements generally depend on the development of the increase of heave (negative settlements) in the long-term due to the reduction in stiffness of the soils, with the dissipation of negative pore-water pressures, and the development of (positive) settlements due to the construction of the basement and the application of the loads from the upper structure to greater depths. Those movements develop contemporarily and generally cannot be distinguished, but an evaluation of the long-term heave, as independent values, was also reported for completeness on the contour plot in Figure 12. The maximum expected heave was calculated as <-15mm (negative values indicate an upwards movement throughout this text) within the footprint of the building.

Two different limit cases were considered for the application of structural loads, respectively considering the foundations of the proposed steel structures as strip footings (Case 1) or pad footings (Case 2).

For strip footings, the maximum overall long-term ground movements under the proposed building footprint were calculated as less than 10mm. Movements along the excavation boundaries ranged between 2.50mm and 7.50mm.

For pad footings, the maximum overall long-term ground movements under the proposed building footprint were calculated as less than 10mm. Movements along the excavation boundaries ranged between -2.5mm and 5.00mm.

It must be noted that site works have been interrupted for a long time and therefore it is likely that a good proportion of any heave have already occurred.

Contour plots with the variation of long-term movements across the basement footprint for cases 1 and 2 are presented in Figure 13 and Figure 14.

9.3 Ground Movement Due to Retaining Wall Lateral Deflection

The excavation of the proposed basement will comprise the construction of retaining structures to preserve the stability of soils and of the neighbouring structures. The depth of the excavation was considered to a maximum of 3.80m, in order to maximise the effects on the neighbouring buildings.

Information received from the Client with regards on the structural characteristics of the basement retaining structures reported the retaining walls were RC walls of a minimum thickness of 300mm, with a second moment of inertia of 225,000cm^4 per metre length.

No contribution to the second moment of inertia was considered because of the construction of liner walls. The retaining walls were considered as permanently propped at the base by the basement floor slab and at the top by the ground floor slab. Due to the methods adopted for the construction, the Client's engineers must ensure that the retaining structures are restrained at the top by means of the application of adequate bracing for ensuring the underpinning is permanently propped at the top.

No information was made available to Soils Limited with regards to construction sequence and propping of both the works undertaken before the suspension and of the currently proposed works. It must be reminded that the underpinning was already completed at the time of reporting and no signs of disruption and/or damage on the neighbouring properties were noted at the time of the walkover undertaken by Soils Limited on 31st October 2018 or reported by the Client at the time of writing. The calculated ground movements must therefore be considered as limit values for a satisfactory development and must not be exceeded.

In the absence of more precise information from the Client's engineers, temporary propping was considered at the top, the middle and the base of the underpinning for all the three scenarios.

Even if the construction of the underpinning was already completed at the time of reporting and no results were available from eventual previous assessments, it is recommended to undertake the monitoring of ground and structure movements to avoid the limit values to be exceeded. Soils Limited must be immediately notified in the case of unexpected large movements, or movements in excess of those presented.

Horizontal deflections were calculated using the dedicated software Wallap by Geosolve. Critical sections SC1 – SC2 considered the presence of a surcharge of 2kPa at the back of the wall for taking into account the neighbouring houses and their normal activities. A surcharge of 10kPa due to road traffic was considered applied along Sumatra Road. The same approach was used for both the cases of strip footings (Case 1) and pad footings (Case 2).

The maximum calculated horizontal deflections for both Case 1 and Case 2 were therefore not greater than 1.5mm for all the scenarios considered.

The calculated movements were summarised within Table 9.3 for Case 1 and Table 9.4 for Case 2 and the related ground movements identified on Figure 15 to Figure 20.

Table 9.3 – Summary of Movements – Case 1

Scenario	Critical Distance (mm)	Horizontal Movement (mm)	Vertical Deflection (mm)
SC1	5000	1.5	2.8
SC2	5000	1.5	2.9
SC3	9000	1.5	3.0

Case 1: strip footings.

Table 9.4 – Summary of Movements – Case 2

Scenario	Critical Distance (mm)	Horizontal Movement (mm)	Vertical Deflection (mm)
SC1	5000	1.5	2.0
SC2	5000	1.5	2.2
SC3	9000	1.5	2.7

Case 2: pad footings.

Section 10 Damage Category Assessment

10.1 Introduction

The ground movements reported in Section 9 were considered for assessing the expected potential damage category that the construction of a new basement was supposed to induce onto the adjoining properties. The assessment was carried out considering the method described in CIRIA Special Publication 200 (Burland et al., 2001) and CIRIA C580 (Gaba et al., 2003), based upon the method proposed by Burland et al. (2001) and taking into account the works by Burland and Wroth (1974) and Boscardin and Cording (1989).

The general categories of damage entity were summarised in Table 10.1.

Table 10.1 – Classification of Visible Damage To Walls

Category	Description
0 (Negligible)	Negligible – hairline cracks
1 (Very slight)	Fine cracks that can easily be treated during normal decoration (crack width <1mm)
2 (Slight)	Cracks easily filled, redecoration probably required. Some repointing may be required externally (crack width <5mm)
3 (Moderate)	The cracks require some opening up and can be patched by a mason. Recurrent cracks can be masked by suitable linings. Repointing of external brickwork and possibly a small amount of brickwork to be replaced (crack width 5 to 15mm or a number of cracks > 3mm).
4 (Severe)	Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows (crack width 15mm to 25mm but also depends on number of cracks).
5 (Very severe)	This requires a major repair involving partial or complete re-building (crack width usually >25mm but depends on number of cracks).

10.2 Summary of Ground Movements and Evaluation of Relative Deflection

The analysis of the ground movements reported in Section 9 allowed to estimate the relative vertical and horizontal deflections on the properties adjoining the building for the proposed development and on the road structures at Sumatra Road.

The evaluation of the vertical deflections for the cases and the scenarios considered was reported on Figure 15 to Figure 20.

The results of the assessment were reported within Table 10.2 and 3 and on Figure 21 were defined the expected damage categories on the neighbouring structures and highway according to the classification by Burland (2001) reported within CIRIA SP200 and CIRIA C760.

Table 10.2 – Expected Damage Category – Case 1

Section	Critical Distance (mm)	Horizontal Deflection (mm)	Vertical Deflection (mm)	Horizontal Strain ε_h (%)	Deflection Ratio Δ/L (%)	Damage Category
SC1	5000	1.5	2.8	0.030	0.056	1 (Very slight)
SC2	5000	1.5	2.9	0.030	0.058	1 (Very slight)
SC3	9000	1.5	3.0	0.017	0.033	0 (Negligible)

Table 10.3 – Expected Damage Category – Case 2

Section	Critical Distance (mm)	Horizontal Deflection (mm)	Vertical Deflection (mm)	Horizontal Strain ε_h (%)	Deflection Ratio Δ/L (%)	Damage Category
SC1	5000	1.5	2.0	0.030	0.040	1 (Very slight)
SC2	5000	1.5	2.2	0.030	0.044	1 (Very slight)
SC3	9000	1.5	2.7	0.017	0.030	0 (Negligible)

The ground movement assessment was carried out independently from the works already undertaken because of the lack of information with regards to construction sequence, propping, monitoring of ground and structural movements and softening of the soils not already excavated due to the ingress of water into the excavations and to the potential relaxation of the excavated faces.

The general approach from London Boroughs is to consider valid proposed developments inducing expected damage categories not beyond Category 2 (slight). It can be observed that the critical sections considered in this analysis presented expected Damage Categories of 0 (Negligible) to 1 (Very slight) for the three scenarios considered in both Case 1 and Case 2. The values reported within Table 10.2 and Table 10.3 are indicative of the stiffness adopted in the calculations and refer to the ground movements calculated within the report.

The construction of basements is generally acceptable when a maximum category damage 2 (slight) was achieved. However, when the expected damage exceeds a category damage 1 (very slight), the SPD produced by the London Borough of Camden required that mitigation measures must be applied in order to reduce this to a potential category damage 0 (negligible) to preserve the existing buildings, generally identified as brittle and extremely sensitive to ground movements. When suitable mitigation measures are applied, a further assessment must be carried out to confirm the improvement.

Considering that the maximum expected damage for the three scenarios considered did not exceed damage category 1, no mitigation measures must be provided and no reassessment has to be carried out.

It must be reminded that underpinning and basement excavation were already completed at the time of reporting and no evident signs of disruption on the neighbouring buildings were noted by Soils Limited during the walkover undertaken on 31st October 2018 or reported by the Client.

No information was provided to Soils Limited with regards to construction procedure, sequence and propping for the construction of the underpinning and the development of basement excavation. Furthermore, no results from building and ground movement monitoring were made available from the Client. The calculated ground movements must therefore be considered as limit values for a satisfactory development and must not be exceeded.

The retaining walls were therefore considered as temporarily propped at the top, the middle and the base, then permanently propped at the base by the basement floor slab and at the top by the ground floor slab. Due to the methods adopted for the construction, the Client's engineers must ensure that the retaining structures are restrained at the top by means of the application of adequate bracing for ensuring the underpinning is permanently propped at the top.

It is recommended to undertake the monitoring of ground and structure movements before, during and after the construction in order to avoid the limit values to be exceeded. Soils Limited must be immediately notified in the case of unexpected large movements, or movements in excess of those presented.

The above reported was specifically determined for the case considered and can be invalidated if changes are applied to building layout and structures.

Section 11 Conclusions and Recommendations of BIA

11.1 General

The findings of this report are informed by site investigation data and information on structures provided by the client. The underpinning was already completed at the rime of reporting, therefore the settlements induced by underpinning and basement excavation are already under development. No information was provided to Soils Limited with regards to construction methods, sequence and monitoring of building and ground movements to the time of reporting. The analysis was therefore developed with reference to the initial conditions and undertaken on the assumption of high-quality workmanship.

It must be noted that the party walls to the existing structure had already been underpinned, in addition a portion of the front elevation of the house and much of its internal structure to the front had collapsed. Temporary works had been designed by others and installed to retain the party walls, with bracing across the width of the structure roughly at former floor levels. No internal survey of the effects of the underpinning works undertaken pre collapse or post collapse and installation of temporary support works, were undertaken by Soils Limited and therefore, any damage to the existing party walls from the collapse or subsequent installation of the temporary works are not and could not be taken into consideration in the writing of this report.

The highest groundwater level recorded during the long-term groundwater monitoring in WS3 was -2.195 m (c.f. basement floor at -3.747 m) and the highest water level recorded in WS2 was -2.56 m (c.f. same as basement floor level), therefore groundwater would pose a risk during excavation. As a confirmation, the existing portion of the basement was found flooded at the time of the walkover carried out on 31st October 2018. Water ingress would have to be prevented during basement excavation. Given the cohesive nature of the London Clay Formation it would be recommend that dewatering with pumps from sumps introduced into the floor of the excavation must be considered.

Foundations must be constructed in accordance with a high-volume change potential when passing through the London Clay Formation, in accordance with for BRE Digest 240 and NHBC Standards Chapter 4.2.

An appropriate monitoring regime must be adopted to manage risk and potential damage to neighbouring structures, during and after construction onsite.

The basement construction would act as a barrier to the groundwater flow, due to extending through the shallow groundwater into the low permeability London Clay Formation. The composition of the Made Ground supporting the shallow groundwater was variable. Determining a permeability for further analysis would be based on rough assumptions. Therefore, calculating accurately the local groundwater level increase around the basement would be difficult. No basements were present underneath the neighbouring buildings, therefore cumulative effects on the rise of groundwater levels because of “damming effects” are considered as negligible.

The permanent works must be designed to ensure induced ground movements surrounding the site are within tolerable limits and temporary works sufficiently designed to prevent damage during construction. The Client's engineers must ensure that the retaining structures are restrained at the top by means of the application of adequate bracing for ensuring the underpinning is permanently propped at the top.

It was recommended monitoring of surrounding structure was undertaken during and after the construction.

Overall it was considered the proposed development could have a limited impact on neighbouring properties provided a suitable basement construction was selected and effective monitoring of ground movements put in place, to inform of eventual excessive movements that could require the undertaking of remedial measures. The statement is strictly related to the geotechnical results of this Basement Impact Assessment and refers to building structures in good conditions. No comments are or can be provided with regards to the structural conditions of the existing building and of the adjoining properties, especially considering the potential effects of the collapse of the front façade. A specific assessment must be undertaken by a structural engineer, who has to ensure the remaining structures are suitable to undergo the proposed development in safe conditions.

11.2 Non-Technical Summary

The construction of basements in Central London becomes more frequent and the London Borough of Camden developed a procedure for the authorisation of the construction based upon a series of stages for the estimation of the impact of the basement construction on the built environment.

A Basement Impact Assessment (BIA) must comprise five main stages:

1. Screening;
2. Scoping;
3. Site Investigation and study;
4. Impact Assessment;
5. Review and decision making.

The screening stage was based on a series of queries regarding issues as groundwater flow, land stability and surface flow and flooding and related flowcharts allowing to clarify if the development of a full BIA was needed.

The scoping stage was intended to evaluate the potential impact of the proposed scheme on the built environment in the surroundings sites.

The site investigation and study were intended to determine an understanding of the site and of its immediate surroundings. The understanding should also be based on the results of the screening and scoping stages, but in general comprises a desk study, site walkover, field investigation (including intrusive investigation), monitoring, reporting and interpretation. The site investigation must be able to determine the ground model to be used for further stages of the development of the site.

The basement impact assessment (BIA) was carried out as the proposed development introduced a considerable increase in the differential depth with regards to the foundations of the neighbouring buildings.

No basements were present underneath the neighbouring buildings, therefore cumulative effects on the rise of groundwater levels because of “damming effects” are considered as negligible.

The excavation and construction of the basement could potentially induce the development of damages on the neighbouring building due to the development of ground movements remote from the development site.

The geotechnical parameters of the soils involved in the development were evaluated on the basis of the results of the site investigation and on published data.

The geometry of the proposed development was provided by the Client and the loads considered with reference to the calculated bearing capacity. The reduction in load at the formation level due to the removal of soils was calculated by applying the soil densities derived from the site investigation to the removed volume of soil derived from the geometry of the development.

The existing underpinning and the excavation of the basement were already completed at the time of reporting and no evident signs of disruption were noted at the time of the walkover undertaken by Soils Limited on 31st October 2018 or reported by the Client.

Although the ground movements deriving from underpinning and basement excavation were already under development, the ground movement assessment was developed with reference to the status quo ante, in order to have a better understanding of the overall effects of every single stage of the development.

No information from the building and ground movements monitoring was made available to Soils Limited, therefore it is not possible to carry out a comparison between measured movements and the results of the ground movement assessment at the time of writing.

The calculated ground movements, therefore, must be considered as limit values for a satisfactory development and must not be exceeded.

Temporary propping was considered at the top, the middle and the base of the underpinning. The basement structures must be permanently propped at the base and at the top. The Client's structural engineers must ensure that adequate bracing is applied to the top of the underpinning for ensuring a similar behaviour. In the case different solutions are adopted, Soils Limited must be immediately notified as a reassessment of the BIA could be eventually required

It is recommended to undertake the monitoring of ground and structure movements during and after the construction in order to avoid the limit values of ground movements to be exceeded. Soils Limited must be immediately notified in the case of unexpected large movements.

The excavation of the basement unloaded the soils at the formation level. The presence of overconsolidated clays implied the development of a certain degree of heave, which was already completed in the short term (undrained conditions) and is now undergoing a further development in the long period (drained conditions).

As the construction proceeds, however, the application of the construction loads will interact with the heave developing in the long term and the fraction already developed will be recovered with time.

The evaluation of heave and/or settlements in the long and in the short term was carried out using the commercial software PDISP. In order to evaluate the vertical deflection induced by excavation and erection on the neighbouring buildings, the vertical movements were calculated along lines linking the outer face of the underpinning with the neighbouring buildings and road structures within the zone of influence of the development.

The maximum vertical deflection under the foundations of the neighbouring building must be evaluated in accordance with CIRIA C580 and CIRIA C760. For buildings with adjoining foundations, in general the deflection will be calculated on the movements profile identified from the outer face of the underpinning to the next closest bearing structure. In the case of detached buildings within the zone of influence, the deflection will be evaluated considering the movements profile under the neighbouring buildings themselves.

The software Wallap was used to evaluate the horizontal movements induced on the underpinning and the back of retaining structures, which induced further vertical movements due to soil relaxation. The process for the evaluation of the horizontal deflection on adjoining buildings considers the stages needed for the construction sequence.

The horizontal strain and the vertical deflection ratio were then calculated according to the procedures reported in CIRIA C580 and CIRIA C760 and combined with reference to the method proposed by Burland (2001) to allow for the evaluation of the expected damage induced by the development on the neighbouring buildings.

The construction of basements is generally acceptable when a maximum category damage 2 (slight) was achieved. However, when the expected damage exceeds a category damage 1 (very slight), mitigation measures must be applied in order to reduce this to a potential category damage 0 (negligible) to preserve the existing buildings, generally identified as brittle and extremely sensitive to ground movements. When suitable mitigation measures are applied, a further assessment must be carried out to confirm the improvement.

In the case assessed within this report, the expected damage induced on the neighbouring structures was estimated to a maximum Burland's Category 1 (very slight damage) and therefore no mitigation measures were needed. However, it was recommended the adoption of permanent propping and the undertaking of dedicated monitoring of ground movements and structures. Soils Limited must be informed of any changes to the layout or excessive ground movements, as this could invalidate the contents of this report.

It must be pointed out that the procedure is generally conservative and the real movements and damages induced by the development are typically lower than the calculated values.

Overall it was considered the proposed development could have a limited impact on neighbouring properties provided a suitable basement construction was selected and effective monitoring of ground movements put in place, to inform of eventual excessive movements that could require the undertaking of remedial measures. The statement is strictly related to the geotechnical results of this Basement Impact Assessment and refers to building structures in good conditions. No comments are or can be provided with regards to the structural conditions of the existing building and of the adjoining properties, especially considering the potential effects of the collapse of the front façade. A specific assessment must be undertaken by a structural engineer, who has to ensure the remaining structures are suitable to undergo the proposed development in safe conditions.

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Job Number 18511	**Project** 163 Sumatra Road, London NW6 1PN
Client Drawing and Planning Limited	**Date** July 2020

Figure 2 – Aerial Photograph

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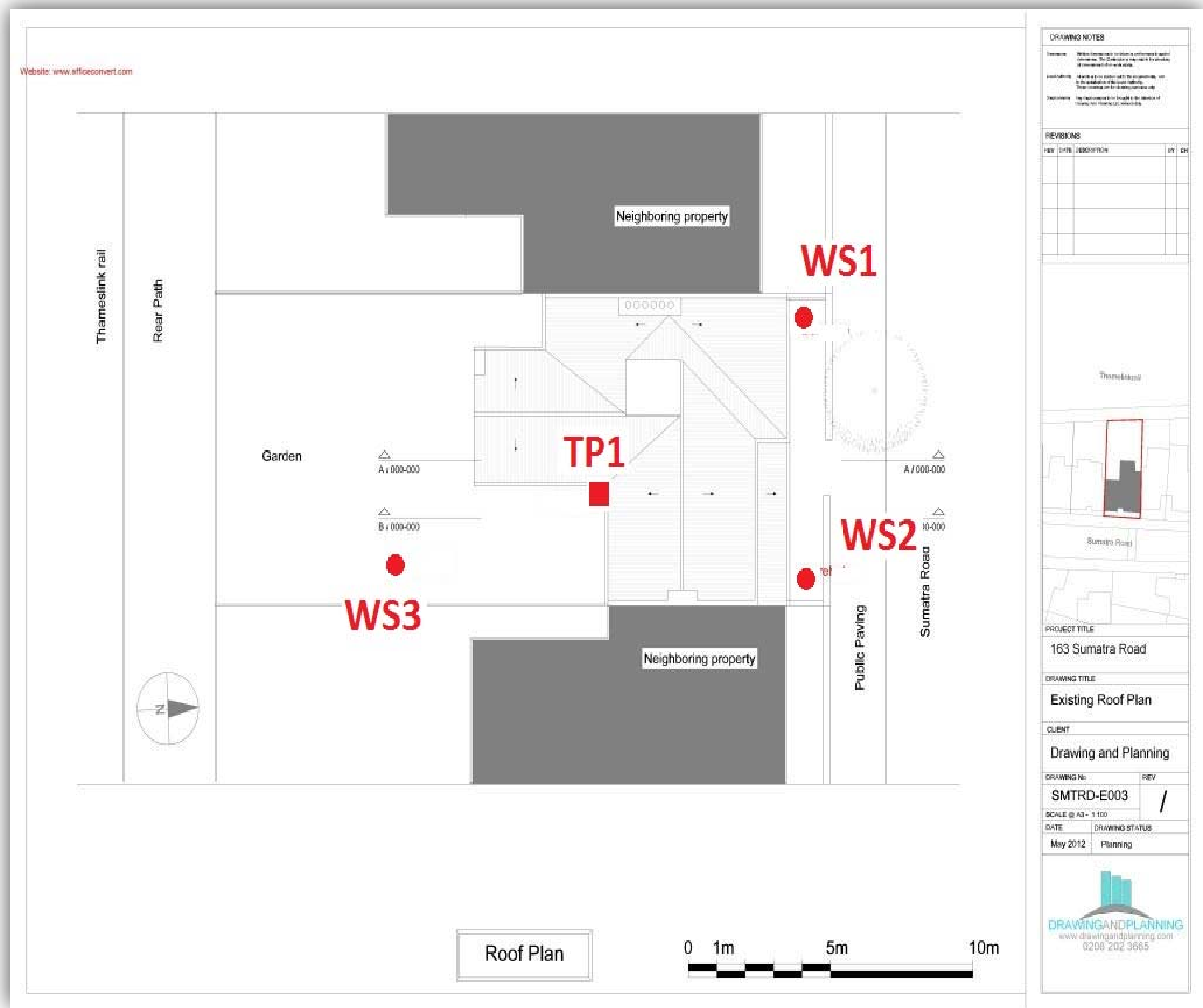

Figure 3 – Trial Hole Plan

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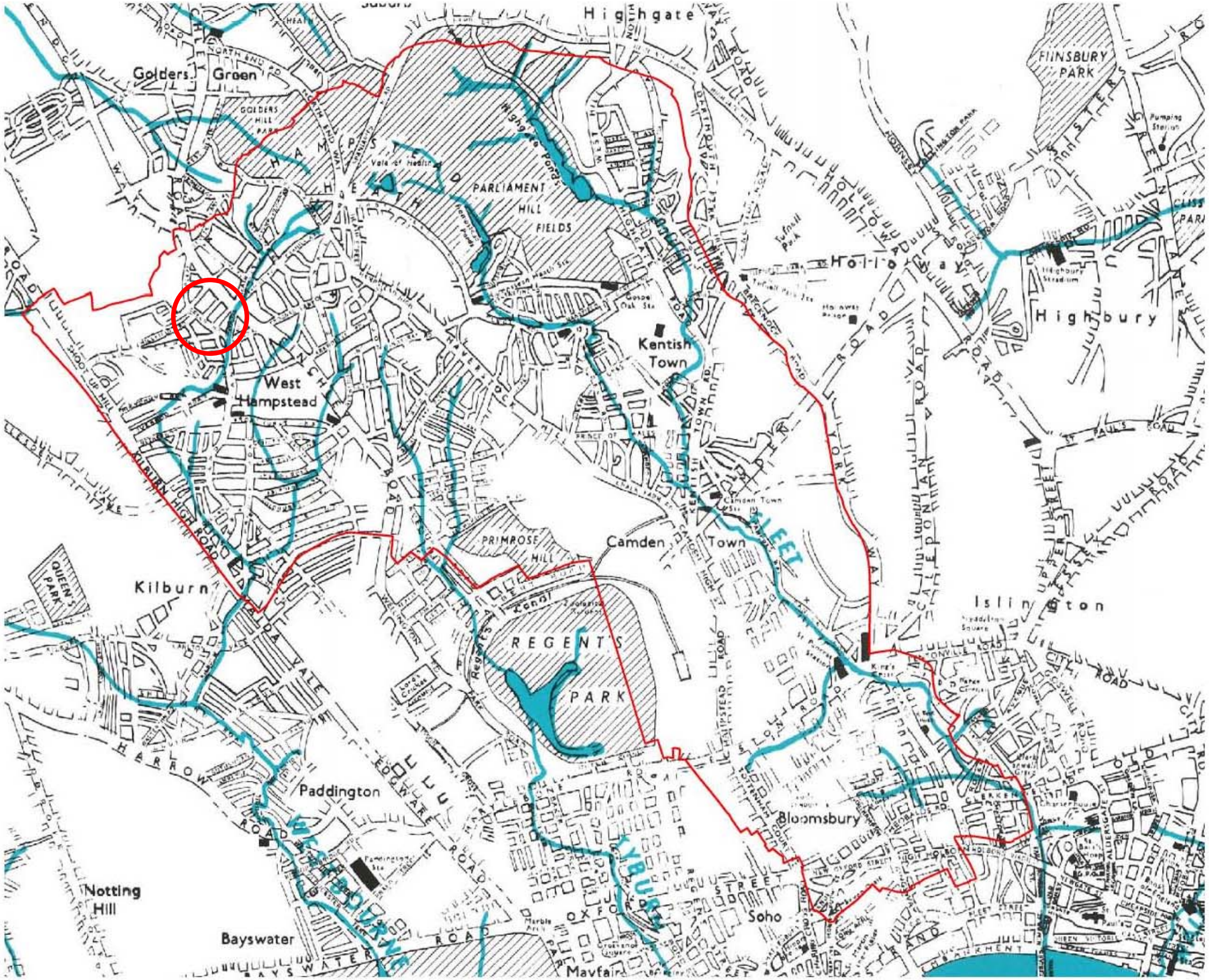

Figure 4 – Lost Rivers in Camden

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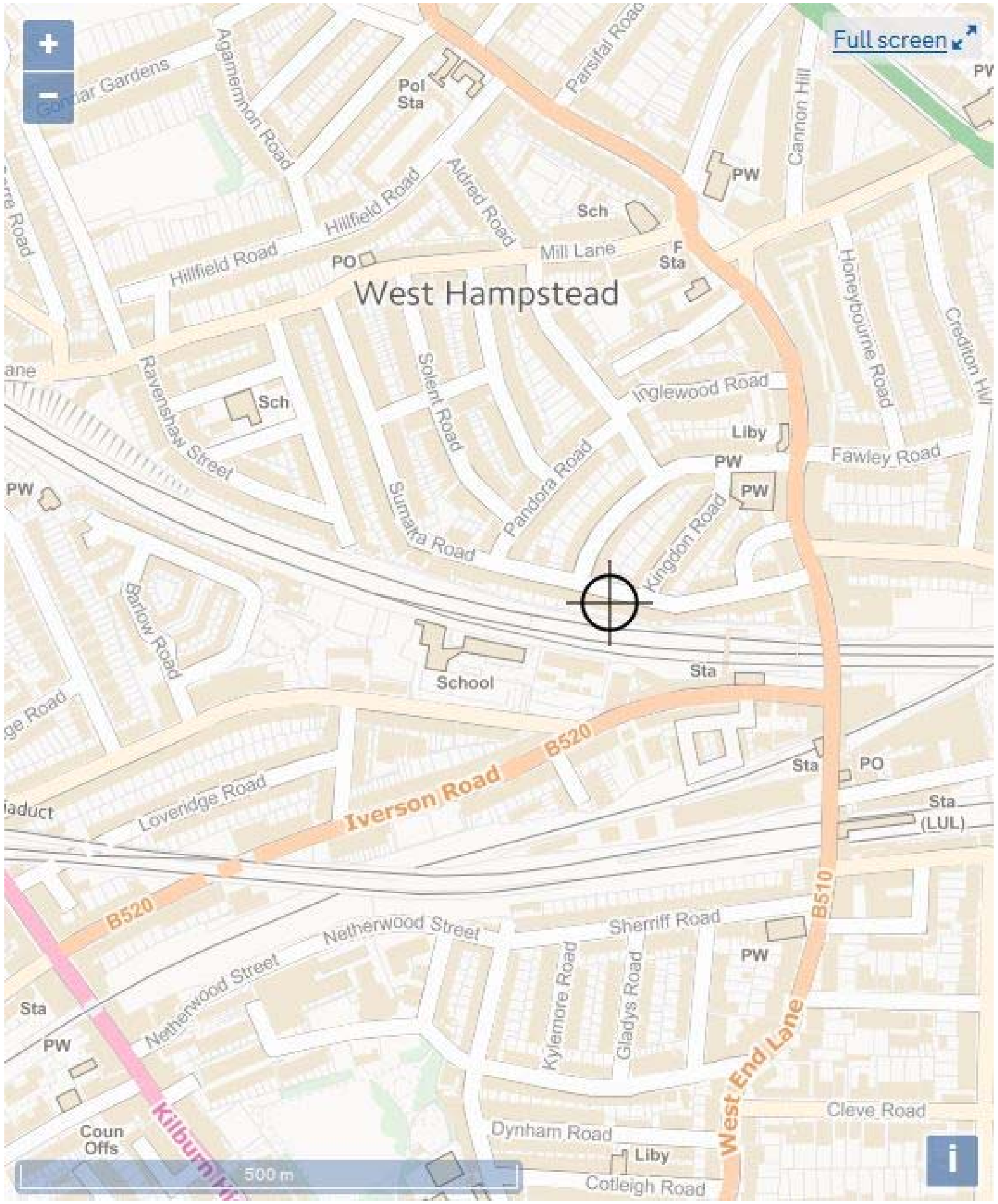

Figure 5 – NFIS, Flooding from Rivers and Sea

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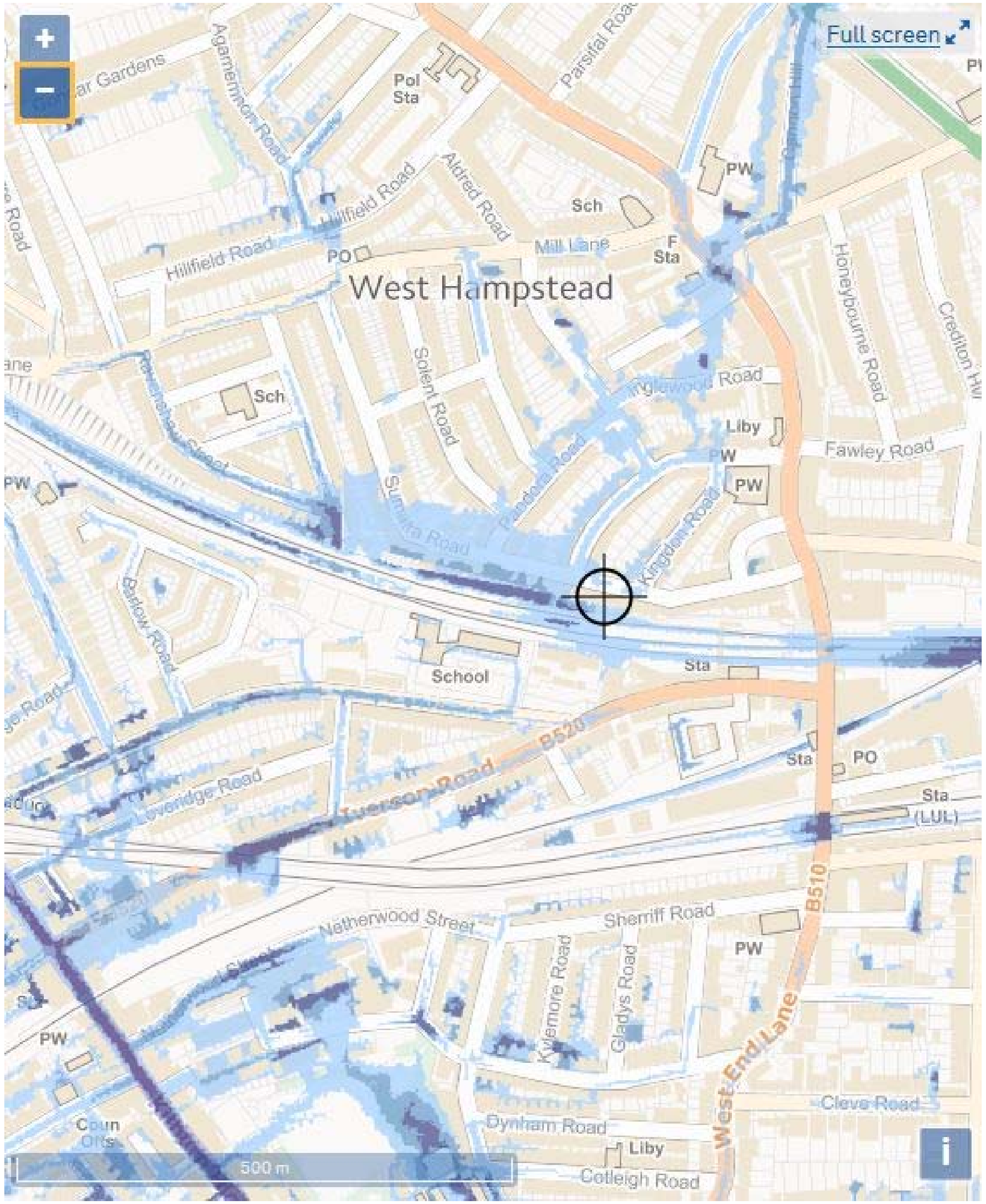

Figure 6 – NFIS, Flooding from Surface Water

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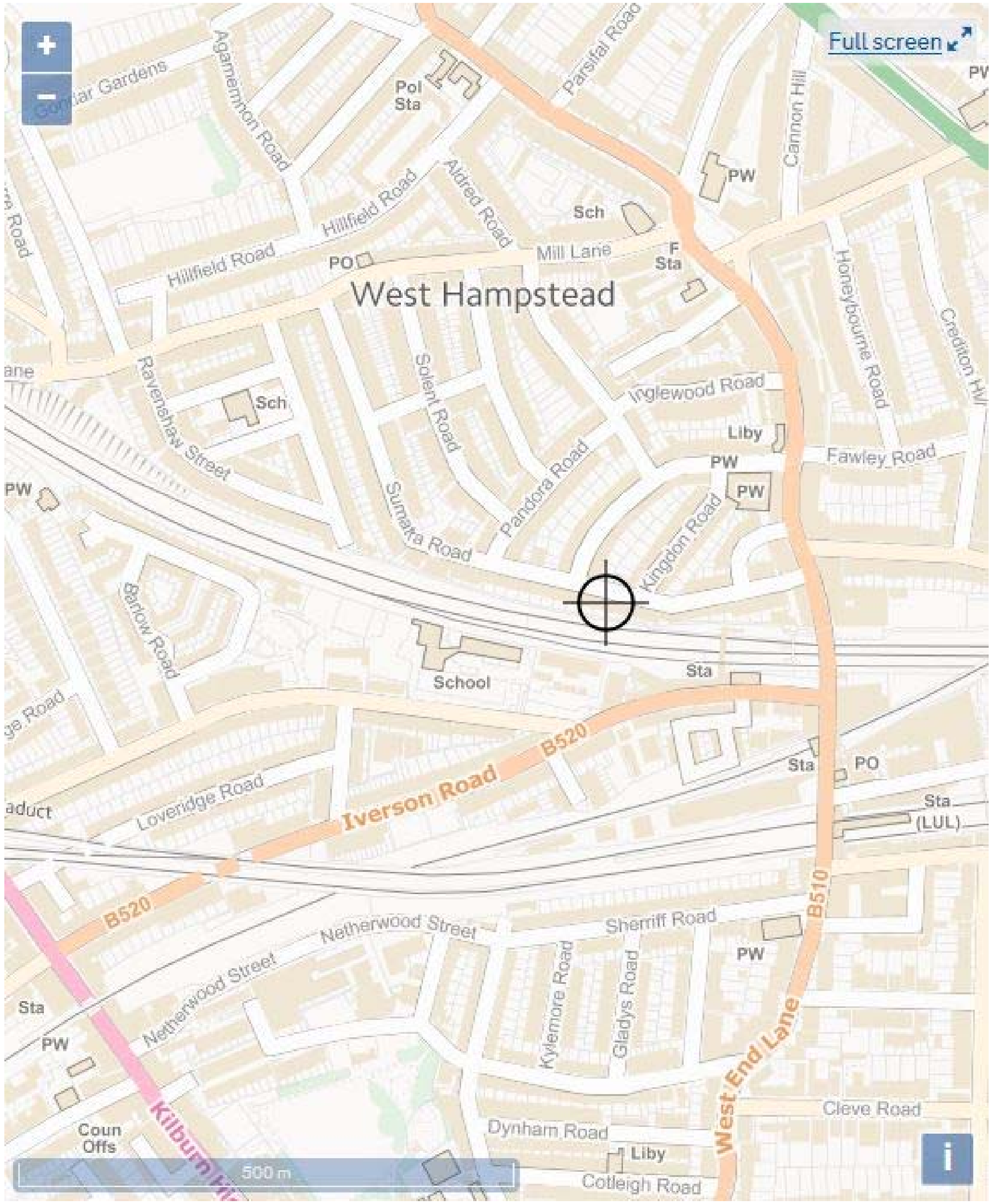

Figure 7 – NFIS, Flooding from Reservoirs

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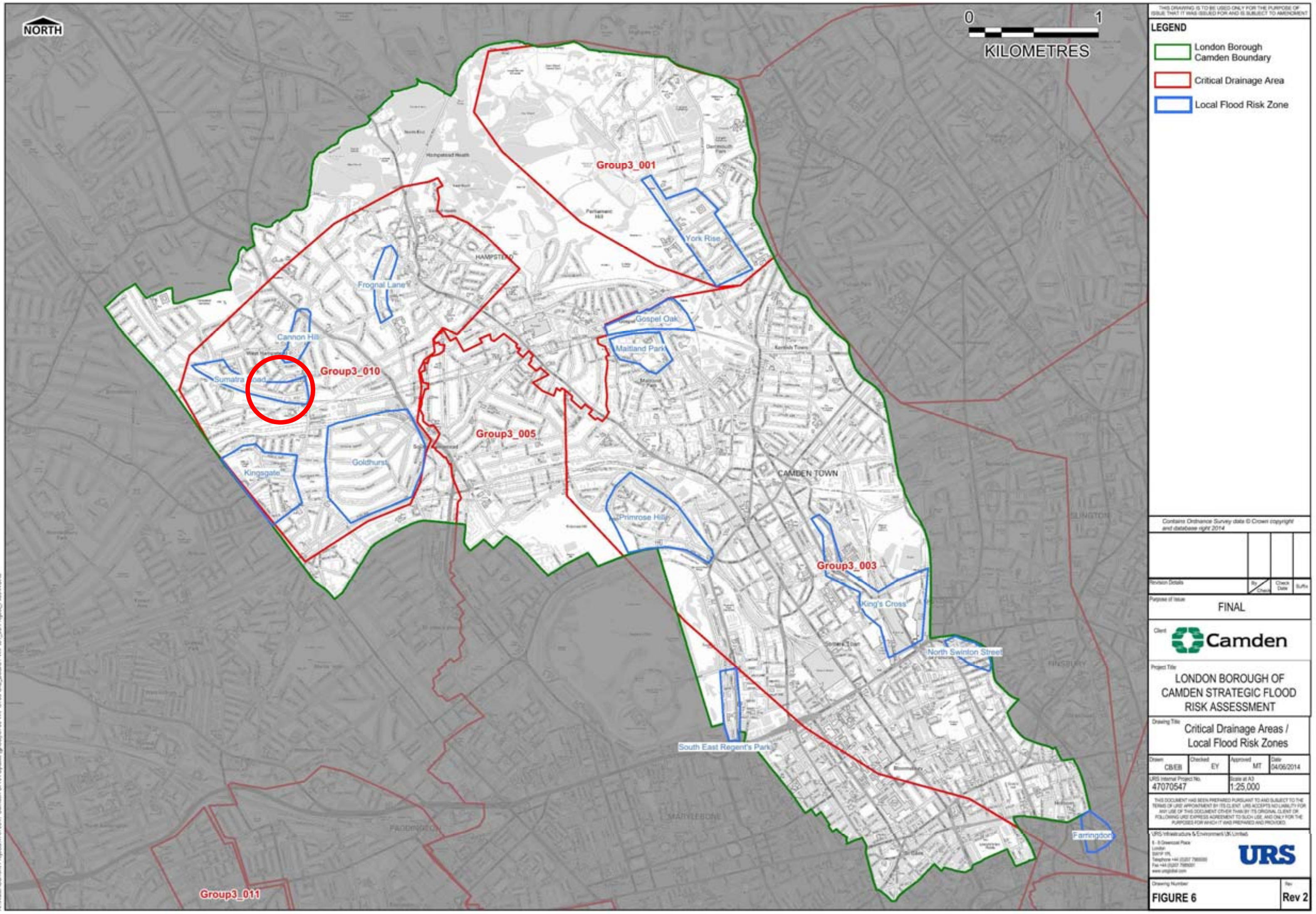

Figure 8 – SFRA, Critical Drainage Areas

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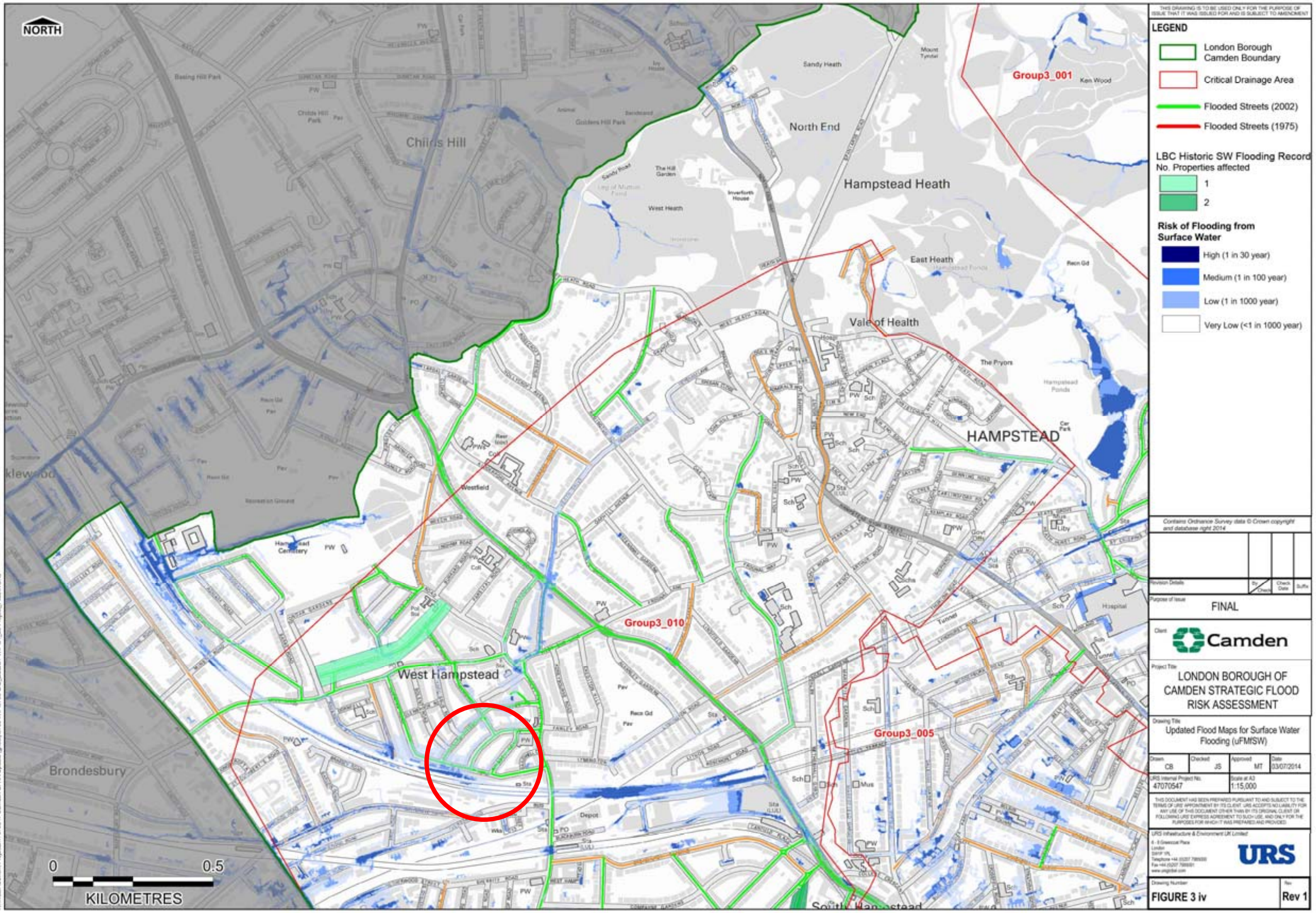

Figure 9 – SFRA, Flooded Streets

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Figure 10 – Critical Scenarios

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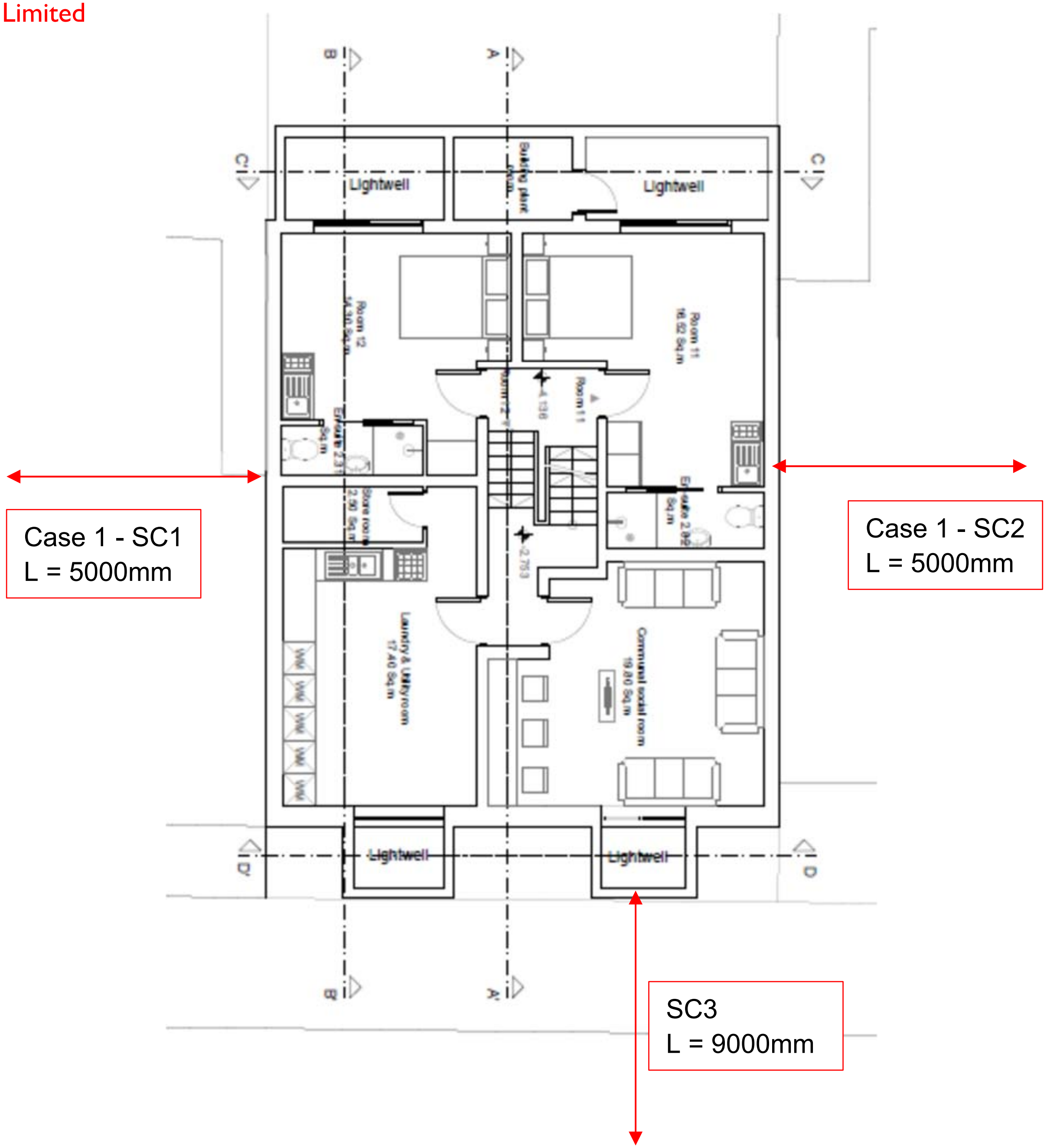

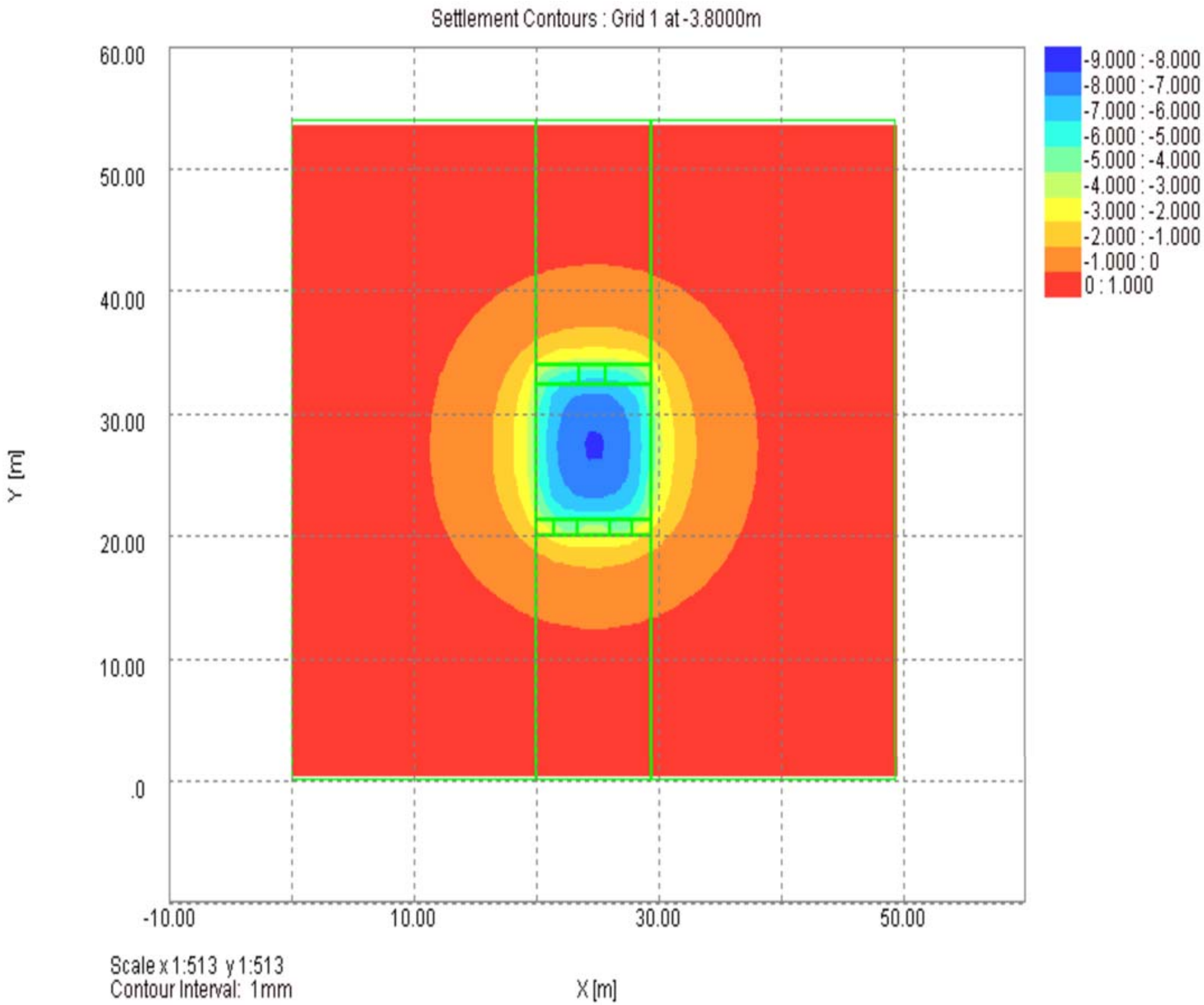

Figure 11 – GMA, Short Term Heave

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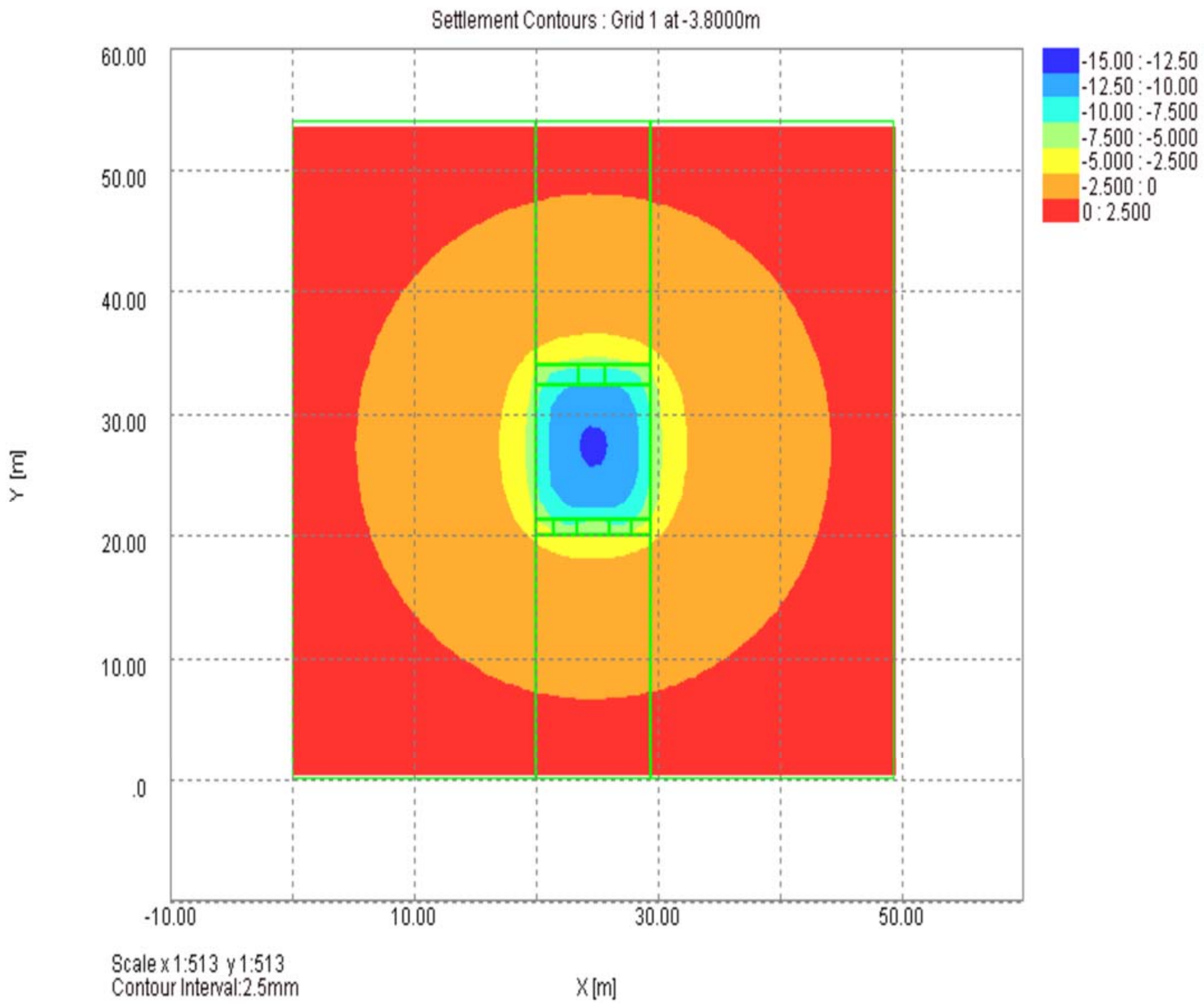

Figure 12 – GMA, Long Term Heave

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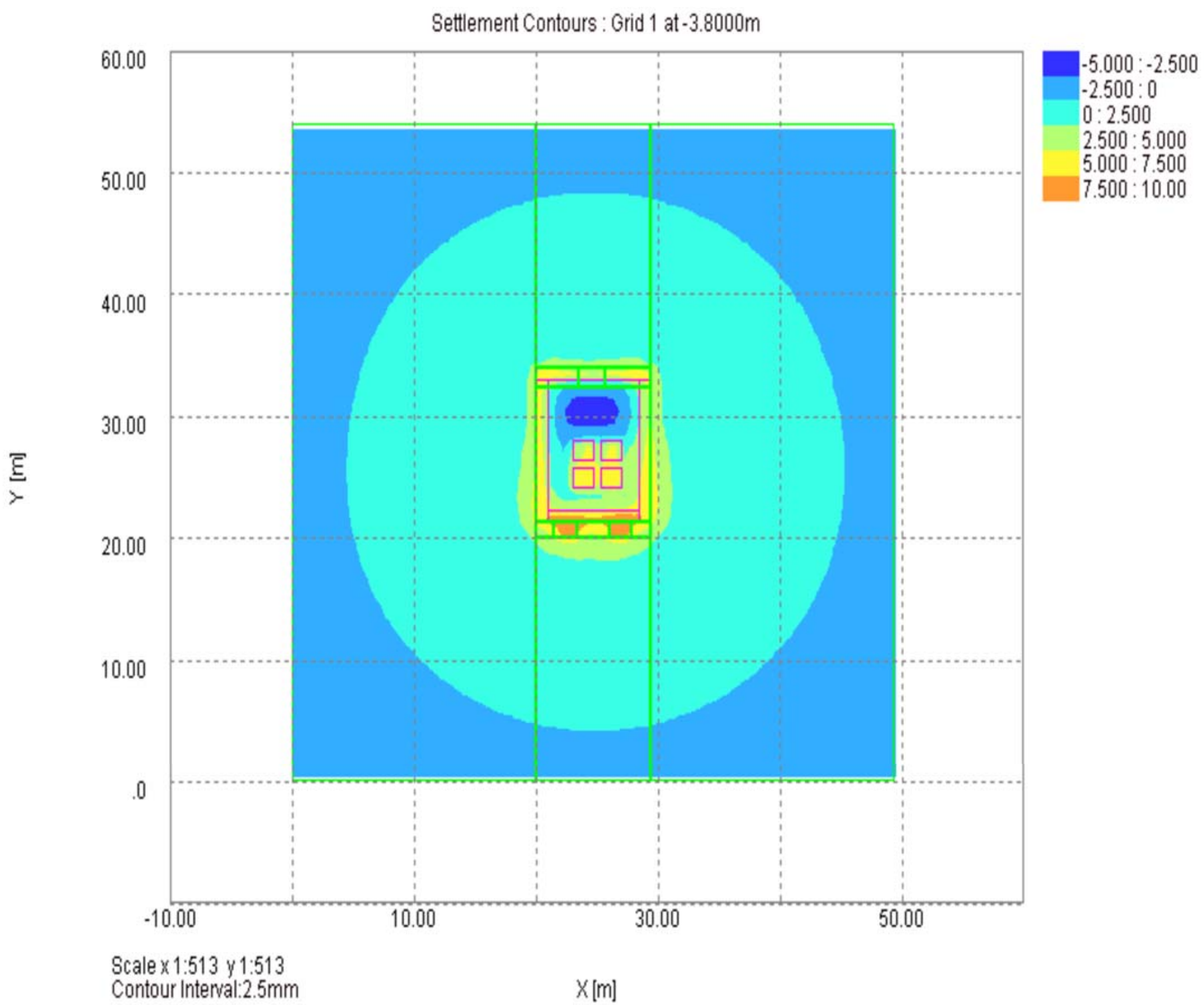

Figure 13 – GMA, Long Term Movements Case 1

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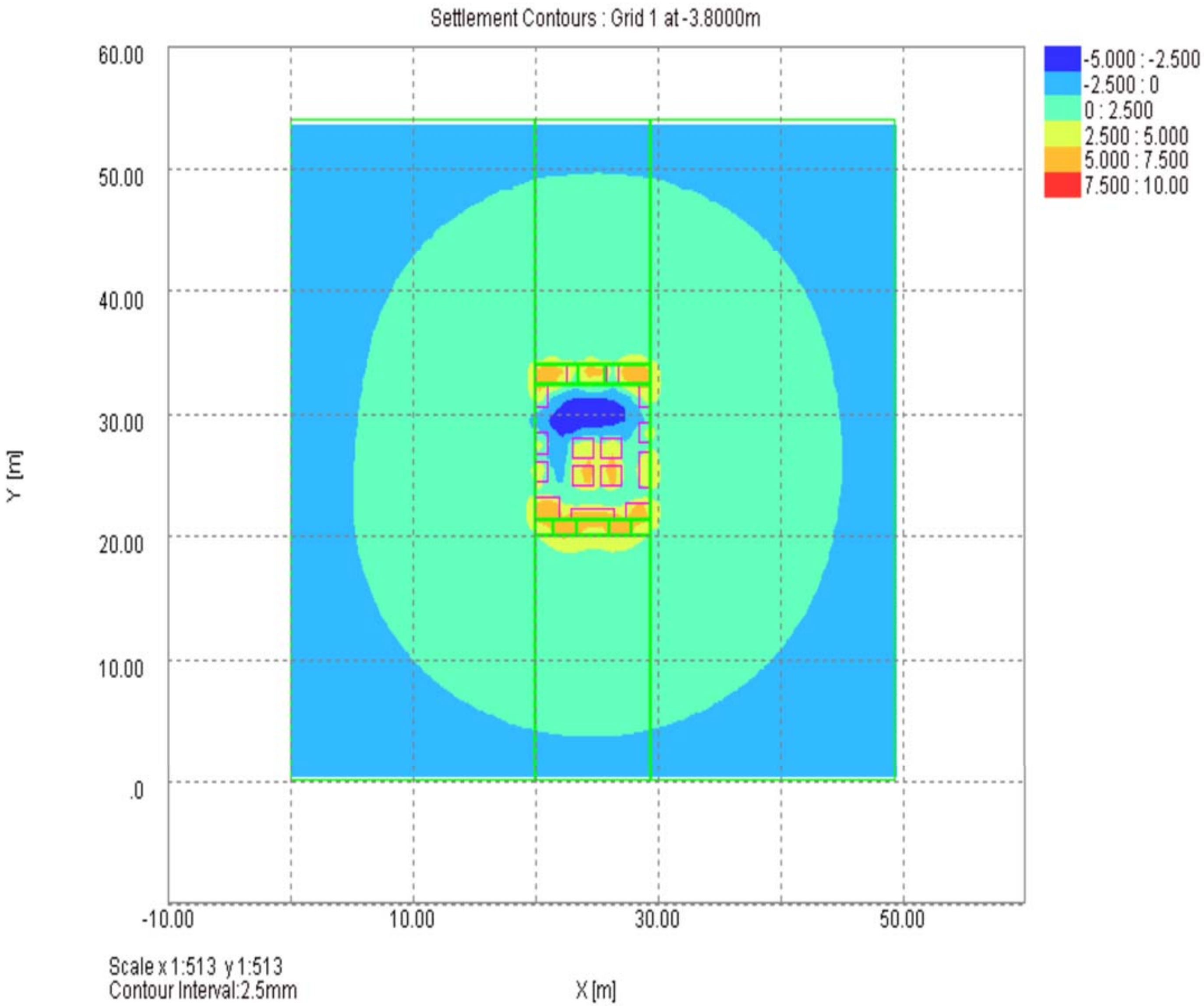

Figure 14 – GMA, Long Term Movements Case 2

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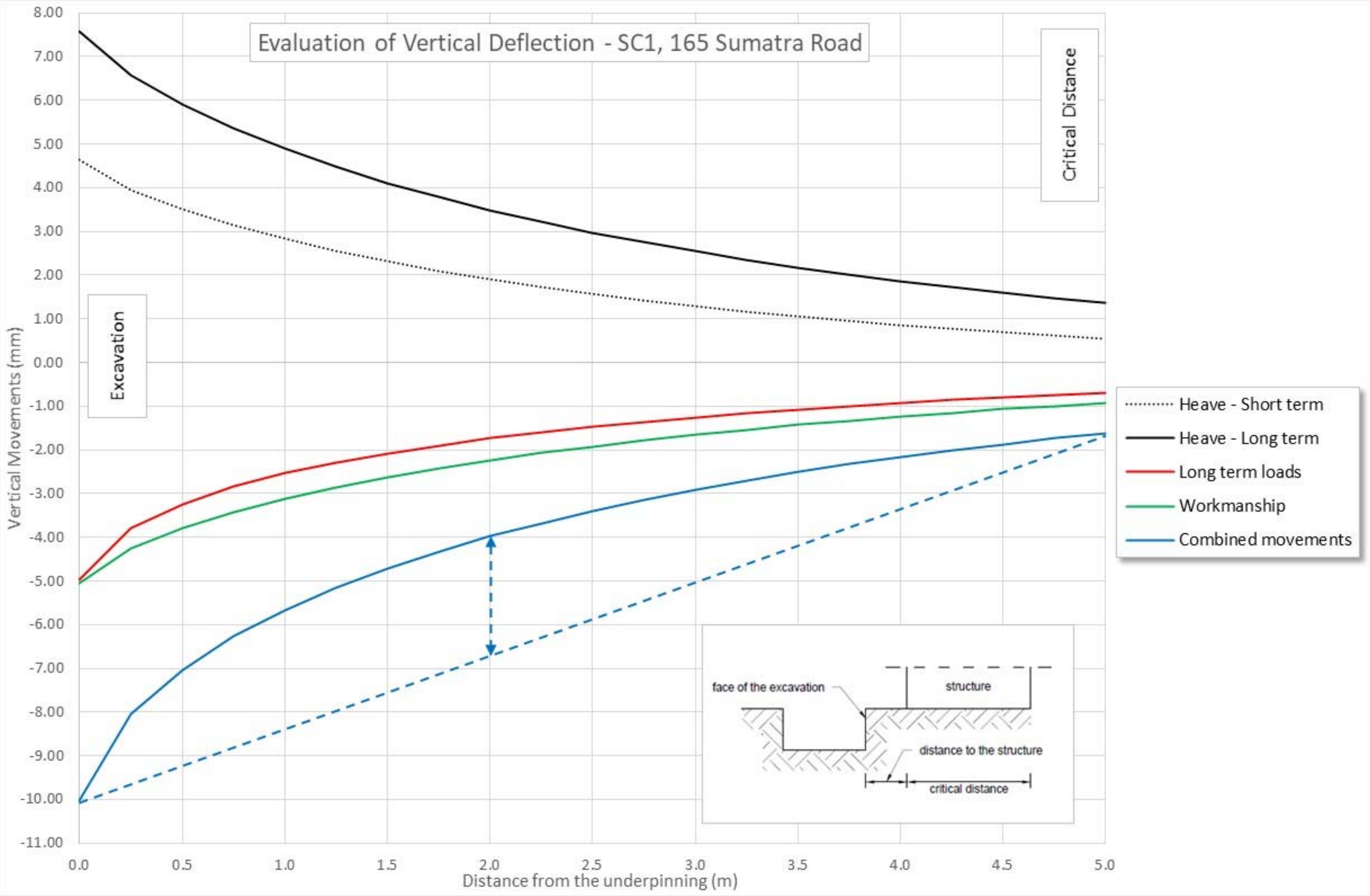

Figure 15 – GMA, Case 1 Scenario SC1

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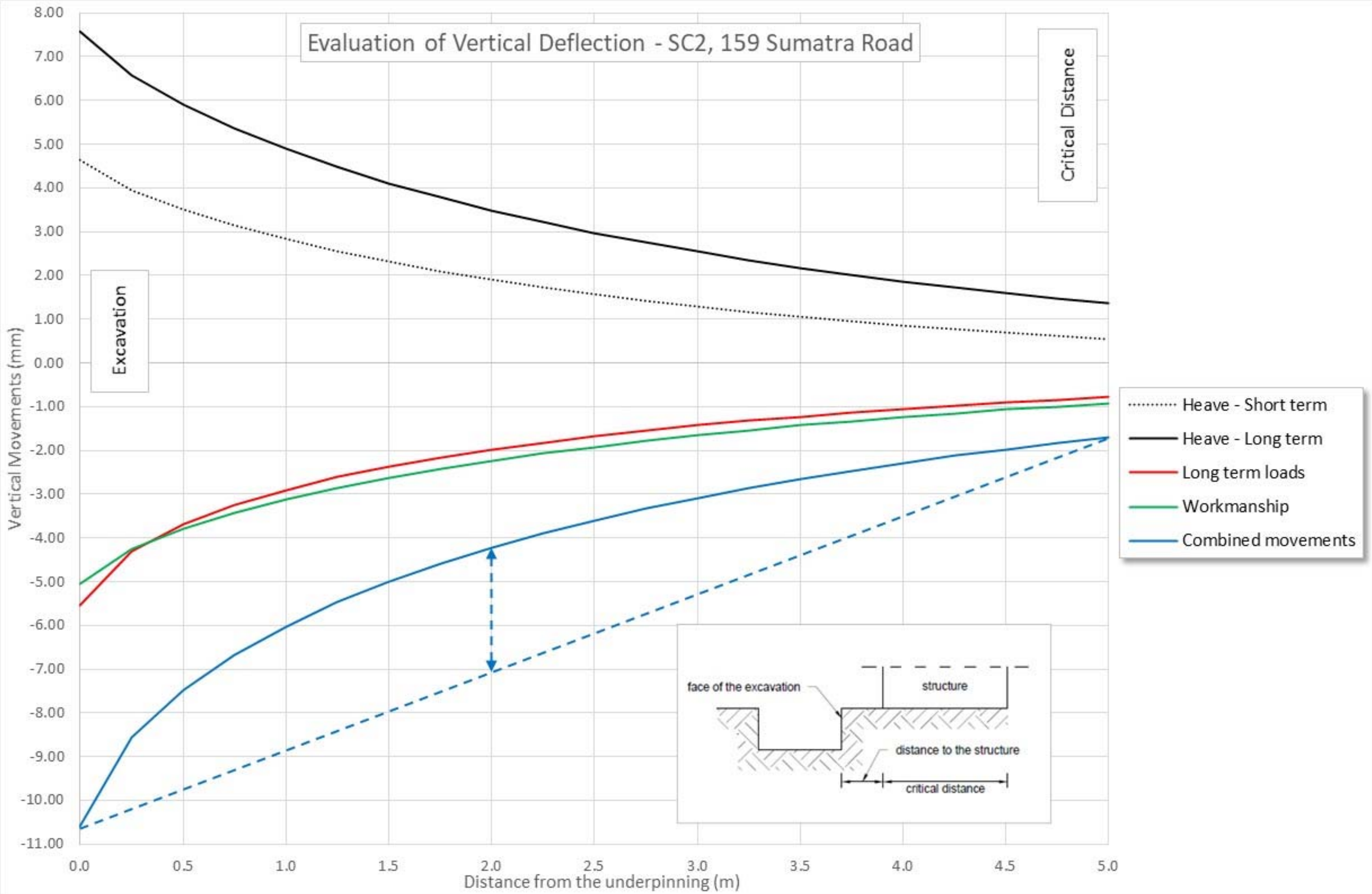

Figure 16 – GMA, Case 1 Scenario SC2

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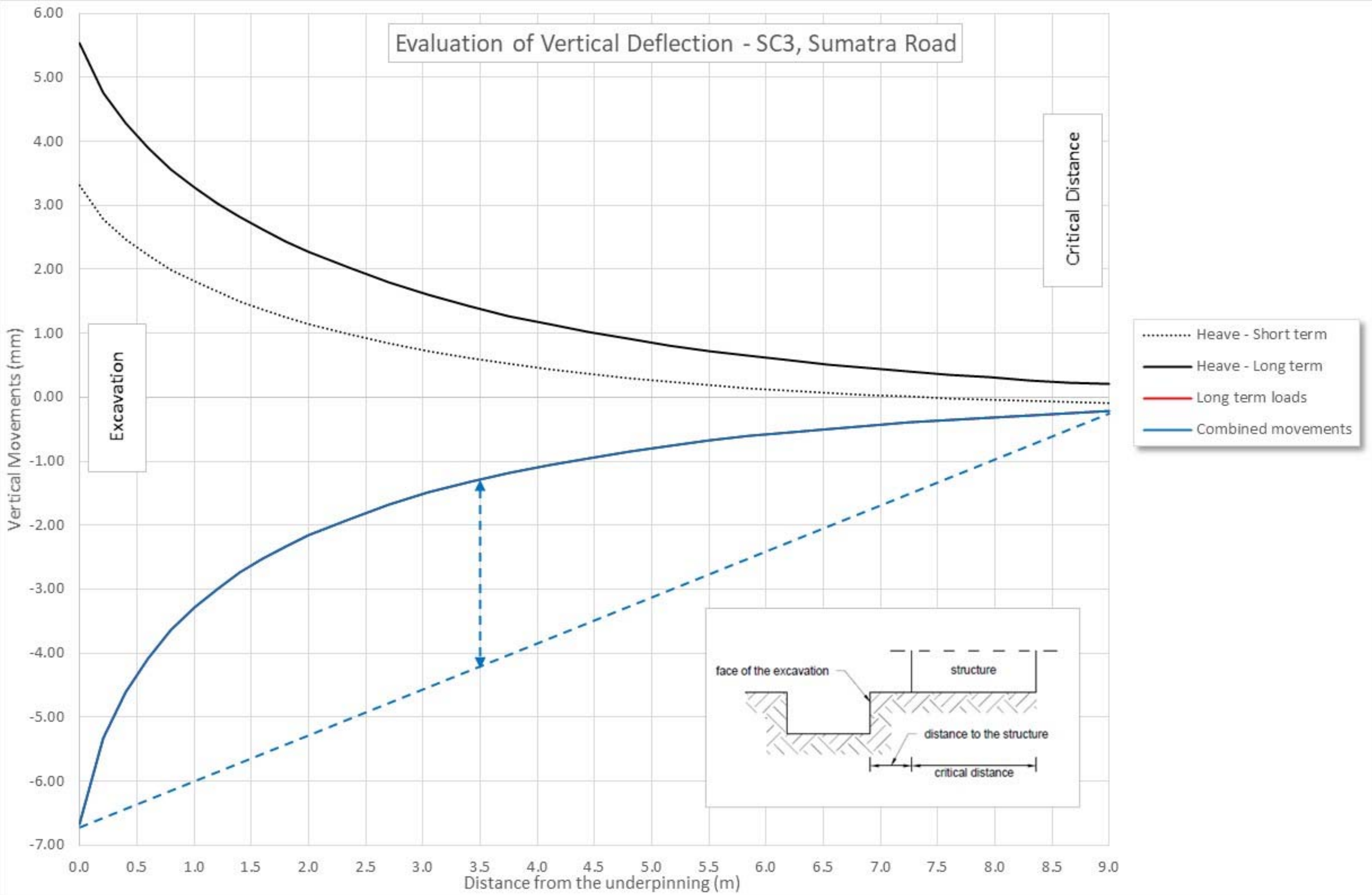

Figure 17 – GMA, Case 1 Scenario SC3

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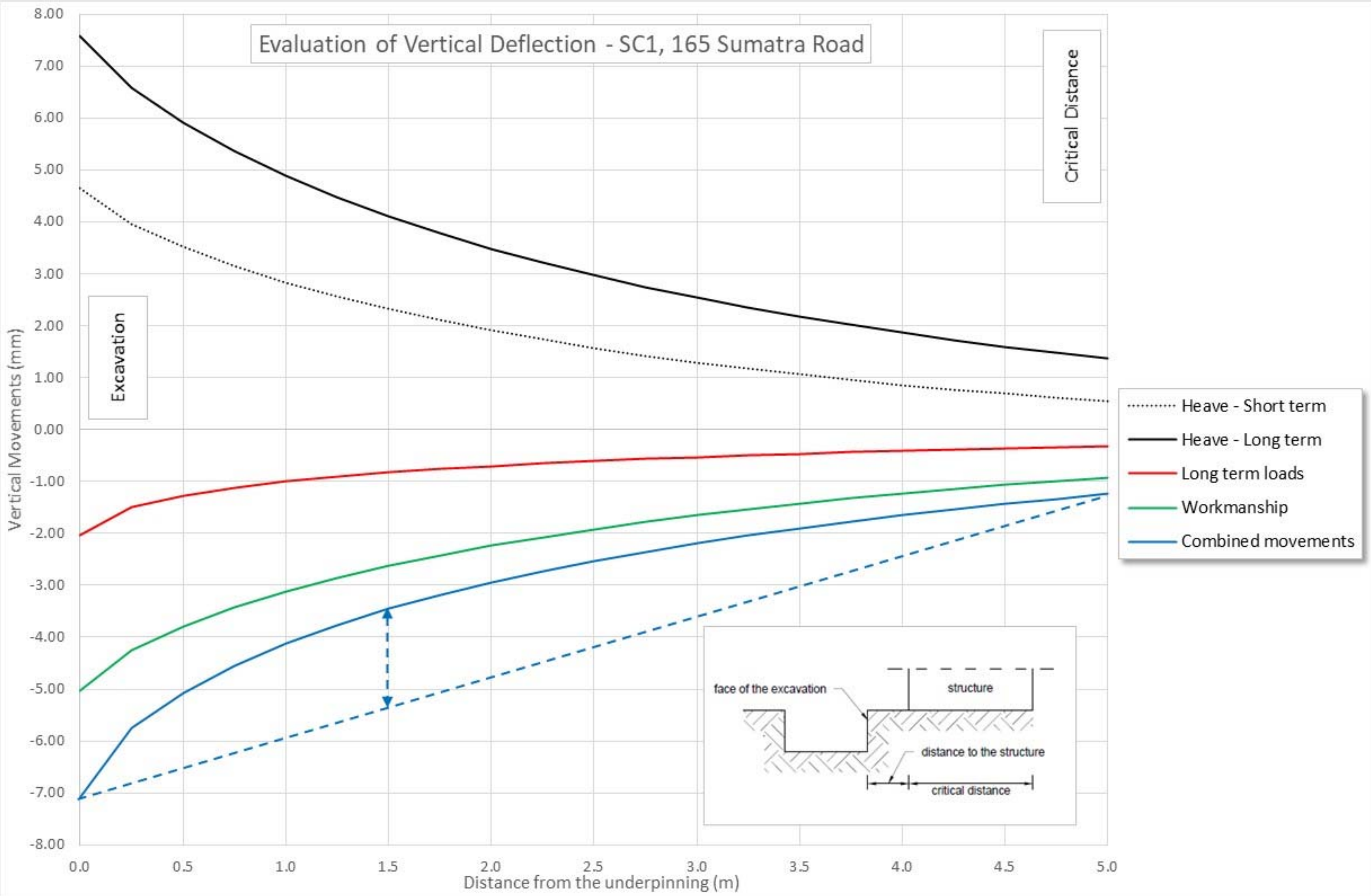

Figure 18 – GMA, Case 2 Scenario SC1

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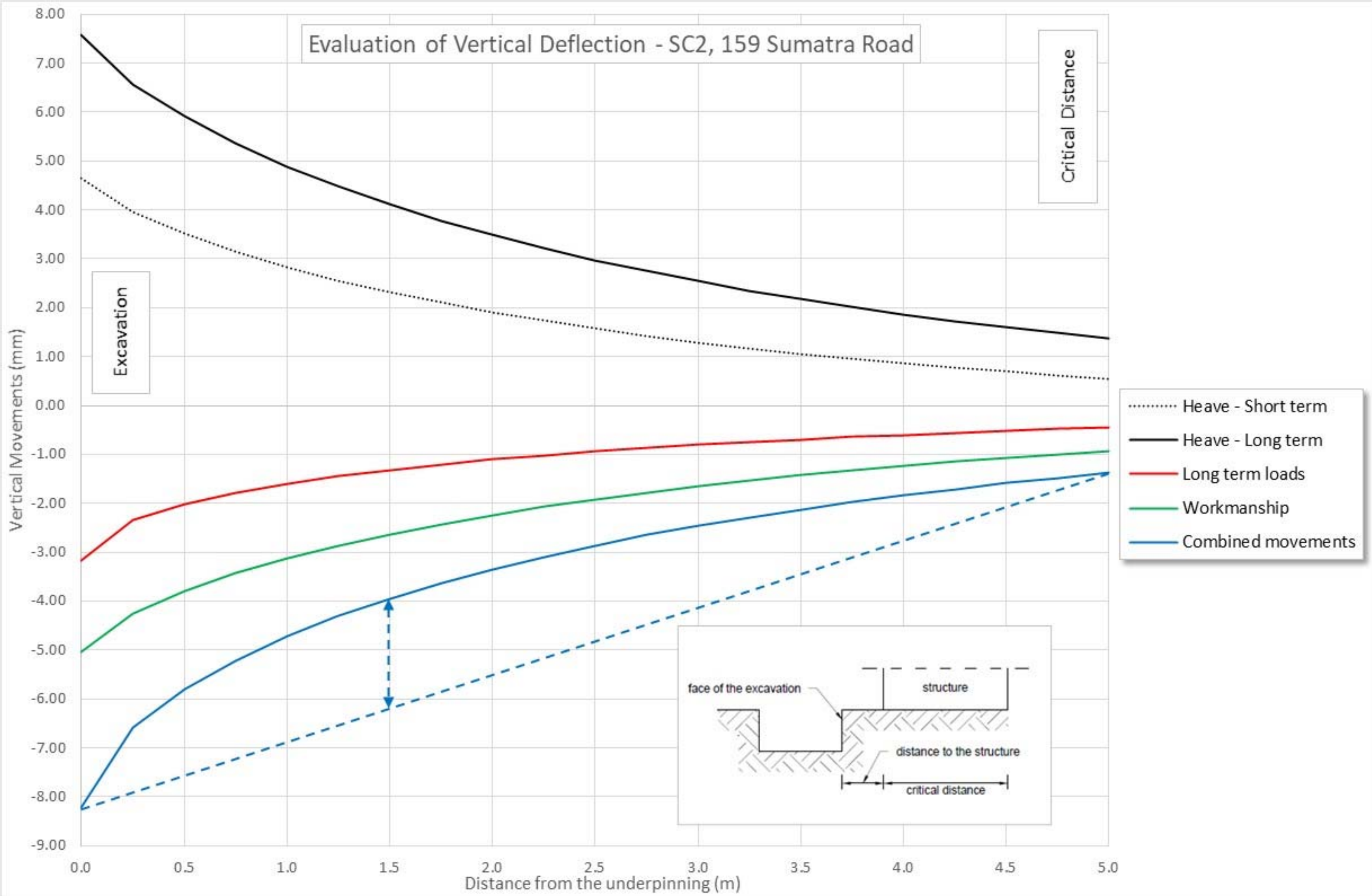

Figure 19 – GMA, Case 2 Scenario SC2

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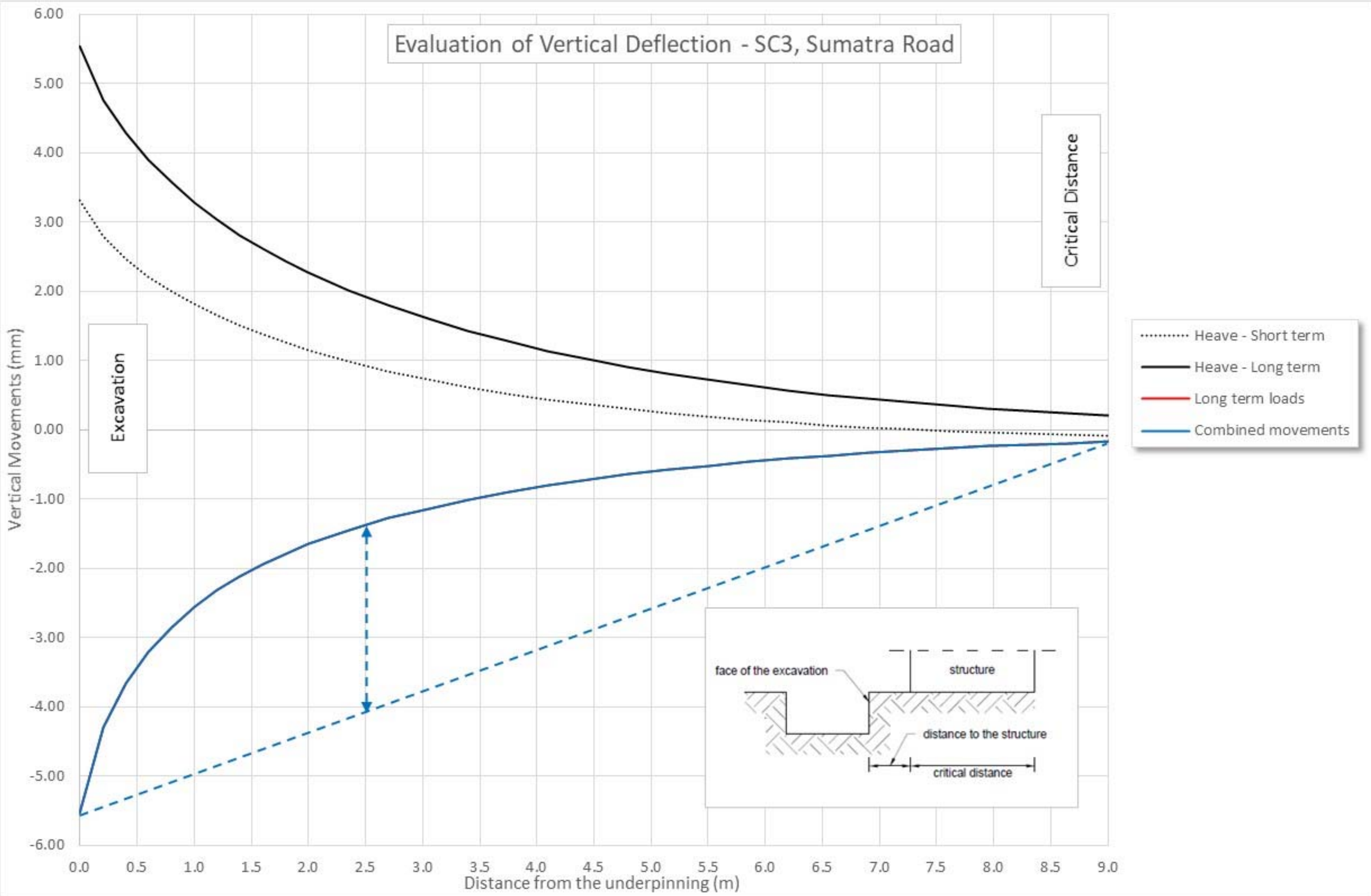

Figure 20 – GMA, Case 2 Scenario SC3

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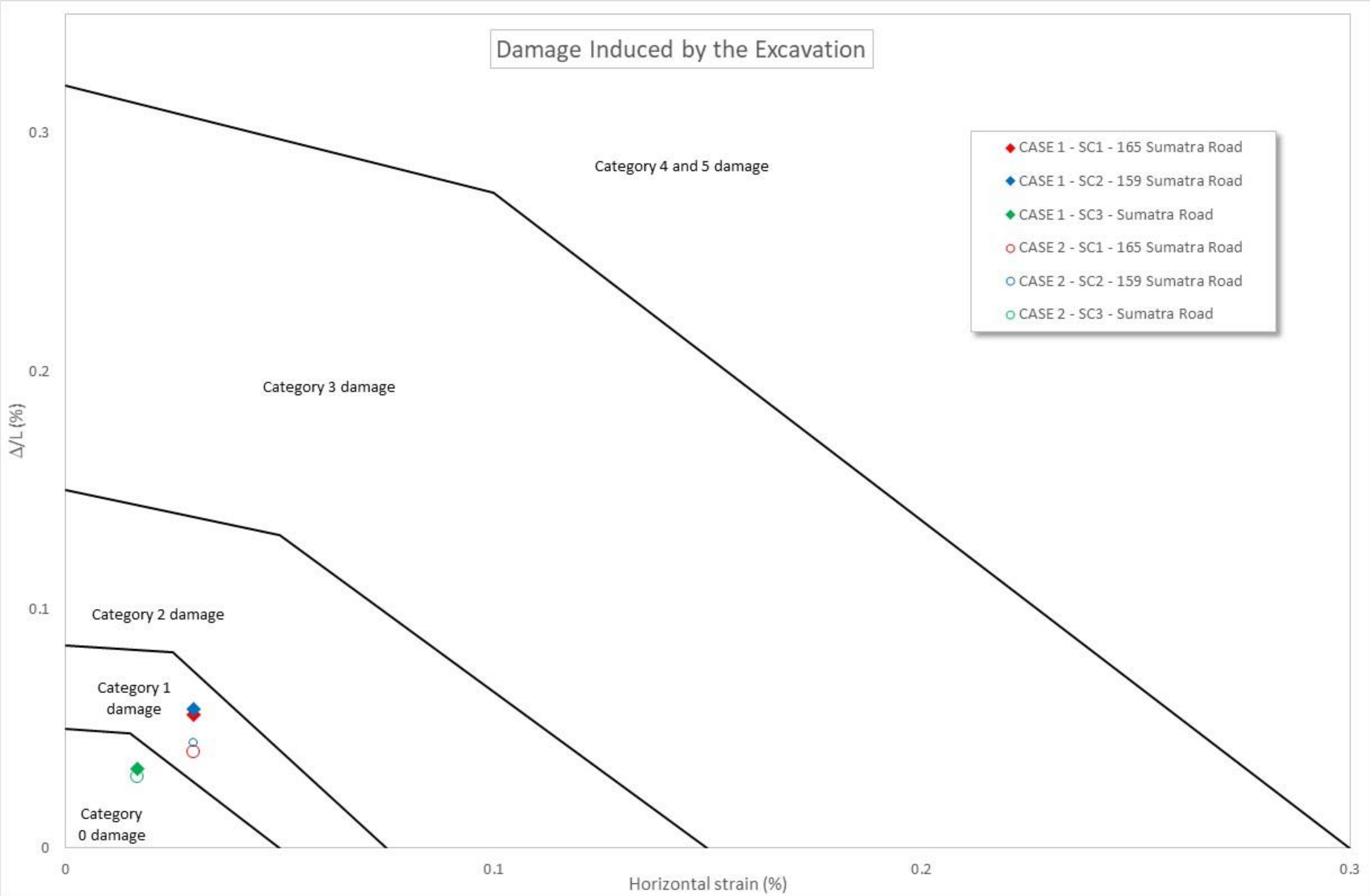

Figure 21 – GMA, Damage Category

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