26 West Hill Park

London, N6 6ND

Basement Impact Assessment Audit

For

London Borough of Camden

Project Number: 12985-55
Revision: F1

January 2020

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D2	September 2019	Comment	CBcb 12985-55-050919-26 West Hill Park-D2.doc	C Botsialas	E M Brown	E M Brown
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Author	C Botsialas, BSc MSc MBA CEng MIMMM CGeol FGS RoGEP Specialist
Project Partner	E M Brown, BSc MSc CGeol FGS
Project Number	12985-55
Project Name	26 West Hill Park
Planning Reference	2019/1426/P

Contents

Appendix

1.0 NON-TECHNICAL SUMMARY

1.1. CampbellReith was instructed by London Borough of Camden, (LBC) to carry out an audit on the Basement Impact Assessment submitted as part of the Planning Submission documentation for 26 West Hill Park, London, N6 6ND (planning reference 2019/1426/P). The basement is considered to fall within Category B as defined by the Terms of Reference.

1.2. The Audit reviewed the Basement Impact Assessment (BIA) for potential impact on land stability and local ground and surface water conditions arising from basement development in accordance with LBC's policies and technical procedures.

1.3. CampbellReith was able to access LBC's Planning Portal and gain access to the latest revision of submitted documentation and reviewed it against an agreed audit checklist. Additional information was provided by applicant's engineers.

1.4. The BIA has been carried out by individuals with suitable qualifications.

1.5. The site is rectangular in shape and is occupied by a three storey detached dwelling. The proposed development involves extending the lower ground floor to the front and the rear. Maximum excavation depths of c.5.50m are anticipated.

1.6. The site is located within the Hampstead Ponds catchment area but at a higher elevation and at a distance of c.130m from the ponds.

1.7. Monitoring indicated that groundwater may be encountered during construction and control of groundwater may be required, however, it is accepted that the proposed development is not anticipated to impact the wider hydrogeological environment.

1.8. The impact on land stability due to the proposed development has been demonstrated, with damage anticipated to be within acceptable levels for the nearby structures and infrastructure. Good control of workmanship shall be maintained throughout the construction stage.

1.9. An outline movement monitoring strategy was presented and should be finalised prior to construction.

1.10. It is accepted that there will be negligible impact to the hydrology of the site.

1.11. An outline construction programme was presented.

1.12. It can be confirmed that the proposal adheres to the requirements of the CPG Basements.

2.0 INTRODUCTION

2.1. CampbellReith was instructed by London Borough of Camden (LBC) on 10 May 2019 to carry out a Category B Audit on the Basement Impact Assessment (BIA) submitted as part of the Planning Submission documentation for 26 West Hill Park, London, N6 6ND (planning reference 2019/1426/P).

2.2. The Audit was carried out in accordance with the Terms of Reference set by LBC. It reviewed the Basement Impact Assessment for potential impact on land stability and local ground and surface water conditions arising from basement development.

2.3. A BIA is required for all planning applications with basements in Camden in general accordance with policies and technical procedures contained within:

- Guidance for Subterranean Development (GSD). Issue 01. November 2010. Ove Arup & Partners;
- Camden Planning Guidance Basements (CPG Basements). March 2018;
- Camden Development Policy (DP) 27: Basements and Lightwells;
- Camden Development Policy (DP) 23: Water;
- Local Plan Policy A5 Basements.

2.4. The BIA should demonstrate that schemes:

a) maintain the structural stability of the building and neighbouring properties;

b) avoid adversely affecting drainage and run off or causing other damage to the water environment;

c) avoid cumulative impacts upon structural stability or the water environment in the local area;

and evaluate the impacts of the proposed basement considering the issues of hydrology, hydrogeology and land stability via the process described by the GSD and to make recommendations for the detailed design.

2.5. LBC's Audit Instruction described the planning proposal as "*Lower ground floor rear / front extension and associated alterations to single family dwelling*".

The Audit Instruction confirmed that 26 West Hill Park neither involves nor is neighbour to any listed building.

2.6. CampbellReith accessed LBC's Planning Portal on 4 June 2019 and gained access to the following relevant documents for audit purposes:

- "Basement Impact Assessment, Surface Water BIA & Engineering Design and Construction Proposals" (Structural BIA report), dated 15/2/2019, issued by Croft Structural Engineers;
- "Ground Investigation and Basement Impact Assessment Report" (Geotechnical BIA report), dated February 2019, v1.01, issued by Ground & Water Ltd;
- "Geo-environmental Interpretative Report" (GI report), dated May 2017, issued by Chelmer Consultancy Services;
- "Design and Access Statement", issued by London Development & Construction;
- "Arboricultural Impact Assessment & Method Statement (to BS5837:2012)", dated 20/2/2019, issued by Trevor Heaps Arboricultural Consultancy Ltd;
- "Topographic Survey", dated October 2016, issued by CD Surveys Ltd;
- Planning application drawings dated 25/2/2019, issued by London Development & Construction, consisting of:
  - 001 Location Plan and Block Plan;
  - 02-B Existing Lower Ground Floor Plan;
  - 03-B Existing Ground Floor Plan;
  - 04-B Existing First Floor Plan and Roof Plan;
  - 05-B Existing Section and Elevation;
  - 06-B Existing Front and Rear Elevation;
  - 07-B Existing Landscape Plan;
  - 08-B Proposed Lower Ground Floor Plan;
  - 09-B Proposed Side Section and Proposed Elevation;
  - 10-B Proposed Front and Rear Elevations;
  - 11-B Proposed Landscape Plan;
  - 10-1-B Visualisation. Existing Condition. View 1;
  - 10-2-B Visualisation. Existing Condition. View 2.
- Planning Comments.

2.7. CampbellReith issued a BIA audit report (rev. D1) on 20/06/2019 raising a number of queries on the above relevant documents.

2.8. In response to the queries raised in the BIA audit report (rev. D1), the following report was received from Croft Structural Engineers, via LBC, on 2 and 5 August 2019:

- "Ground Investigation and Basement Impact Assessment Report", dated August 2019, v1.01, issued by Ground & Water Ltd.

2.9. CampbellReith issued a second BIA audit report (rev. D2) on 05/09/2019 raising a number of queries on the above revised BIA report.

2.10. In response to the queries raised in the BIA audit report (rev. D2), the following relevant documents (attached in Appendix 3 for ease of reference) were received from Croft Structural Engineers on 28 and 29 November 2019:

- "Ground Movement Assessment" report, dated November 2019, issued by Ground and Project Consultants Ltd (referred to as 'revised GMA' in this audit) – Included as Appendix G in the revised Geotechnical BIA report (see below);
- "Temporary Works Scheme Design" drawing, dated 28 November 2019, issued by Croft Structural Engineers;
- "Ground Investigation and Basement Impact Assessment Report" (referred to as 'revised Geotechnical BIA report' in this audit), dated November 2019, v4.01, issued by Ground & Water Ltd.

2.11. This report presents the findings of our audit on the above revised documents.

3.0 BASEMENT IMPACT ASSESSMENT AUDIT CHECK LIST

Item	Yes/No/NA	Comment
Are BIA Author(s) credentials satisfactory?	Yes	Refer to comment in audit paragraph 4.1.
Is data required by Cl.233 of the GSD presented?	Yes	
Does the description of the proposed development include all aspects of temporary and permanent works which might impact upon geology, hydrogeology and hydrology?	Yes	
Are suitable plan/maps included?	Yes	
Do the plans/maps show the whole of the relevant area of study and do they show it in sufficient detail?	Yes	
Land Stability Screening: Have appropriate data sources been consulted? Is justification provided for 'No' answers?	Yes	Refer to Section 3.1.2 of the revised Geotechnical BIA report.
Hydrogeology Screening: Have appropriate data sources been consulted? Is justification provided for 'No' answers?	Yes	Refer to Section 3.1.1 of the revised Geotechnical BIA report.
Hydrology Screening: Have appropriate data sources been consulted? Is justification provided for 'No' answers?	Yes	Refer to Section 4.3 of the Structural BIA report.
Is a conceptual model presented?	Yes	Refer to Section 5 of the revised Geotechnical BIA report.
Land Stability Scoping Provided? Is scoping consistent with screening outcome?	Yes	Refer to Section 3.2 of the revised Geotechnical BIA report.

Item	Yes/No/NA	Comment
Hydrogeology Scoping Provided? Is scoping consistent with screening outcome?	Yes	As above.
Hydrology Scoping Provided? Is scoping consistent with screening outcome?	Yes	Refer to Section 5.3 of the Structural BIA report.
Is factual ground investigation data provided?	Yes	Refer to the GI report.
Is monitoring data presented?	Yes	As above.
Is the ground investigation informed by a desk study?	Yes	Refer to Section 2 of the revised Geotechnical BIA report and Section 3 of the Structural BIA report.
Has a site walkover been undertaken?	Yes	Refer to Section 3.2 of the Structural BIA report.
Is the presence/absence of adjacent or nearby basements confirmed?	Yes	Refer to Section 3.2.3 of the Structural BIA report.
Is a geotechnical interpretation presented?	Yes	Refer to Section 7 of the revised Geotechnical BIA report.
Does the geotechnical interpretation include information on retaining wall design?	Yes	Refer to Sections 7.2 and 7.4 of the revised Geotechnical BIA report.
Are reports on other investigations required by screening and scoping presented?	Yes	An arboricultural report is presented.
Are the baseline conditions described, based on the GSD?	Yes	
Do the base line conditions consider adjacent or nearby basements?	Yes	The absence of adjacent basements was confirmed in Section 3.2.3 of the Structural BIA report.
Is an Impact Assessment provided?	Yes	Refer to Appendix G of the revised Geotechnical BIA report.

Item	Yes/No/NA	Comment
Are estimates of ground movement and structural impact presented?	Yes	A ground movement assessment (GMA) was presented in Appendix G of the revised Geotechnical BIA report.
Is the Impact Assessment appropriate to the matters identified by screening and scoping?	Yes	
Has the need for mitigation been considered and are appropriate mitigation methods incorporated in the scheme?	Yes	Refer to the revised Geotechnical BIA report.
Has the need for monitoring during construction been considered?	Yes	An outline monitoring strategy was presented in Section 7.4.3 of the Structural BIA report.
Have the residual (after mitigation) impacts been clearly identified?	Yes	
Has the scheme demonstrated that the structural stability of the building and neighbouring properties and infrastructure will be maintained?	Yes	
Has the scheme avoided adversely affecting drainage and run-off or causing other damage to the water environment?	Yes	Refer to Section 5.3.1 of the Structural BIA report.
Has the scheme avoided cumulative impacts upon structural stability or the water environment in the local area?	Yes	Refer to the revised GMA and the revised Geotechnical BIA report.
Does report state that damage to surrounding buildings will be no worse than Burland Category 1?	Yes	Refer to the revised GMA.
Are non-technical summaries provided?	Yes	

4.0 DISCUSSION

4.1. The Basement Impact Assessment (BIA) has been carried out by Ground & Water Ltd (Geotechnical BIA report), by Ground and Project Consultants (Ground Movement Assessment) and by Croft Structural Engineers (Structural BIA report), by individuals with suitable qualifications.

4.2. The site has an approximately rectangular shape and comprises a three storey detached dwelling with a front and a rear garden, a detached garage and an adjoining driveway towards West Hill Park. According to the Structural BIA report, the existing building is made of masonry and reinforced concrete. Due to the land within the property boundary sloping down from northeast to southwest, the lower ground floor is at street level at the front and extends below the garden level at the rear.

4.3. The proposed development involves extending the lower ground floor to the front and the rear, with the majority of the extension being below the existing building footprint and hardstanding areas. The front extension will be below a paved area beyond the front entrance of the building. The rear extension will include a swimming pool and will be partly located below a grassed area. Maximum excavation depths of c.5.50m are anticipated adopting a 'hit and miss' technique for the construction of the proposed lower ground floor reinforced concrete retaining walls and underpins as required. Outline construction sequence drawings and calculations are presented in the "Temporary Works Scheme Design" drawing and the Structural BIA report, respectively. The available data indicate one phase of underpin construction for the proposed shallower (c.3.50m) and deeper (c.5.50m) sections of the lower ground floor.

4.4. The BIA reports included screening and scoping sections for land stability, hydrogeology and hydrology, supported by a desk study and a site walkover as required by CPG Basements. The site is located within the Hampstead Ponds catchment area but at a higher elevation and at a distance of c.130m from the ponds. Based on GSD data, the site was indicated to be on the southwest edge of an area where a natural or manmade slope of between 7° and 10° is present. According to the revised Geotechnical BIA report, sectional drawings of 26 West Hill Park Road revealed that the adjacent street slopes between 5°-7° towards the southwest.

4.5. A site walkover survey undertaken in the past (February 2017, Chelmer) recorded minor cracks at the neighbouring boundary retaining walls to the southwest.

4.6. Existing British Geological Survey (BGS) information indicates that the site is located above a 'Secondary A' aquifer, the Claygate Member of London Clay Formation. A site-specific intrusive ground investigation was undertaken comprising two cable percussion boreholes (BH1 and BH2) to a depth of 10.10m and two hand excavated foundation inspection pits to a maximum depth of 0.66m. The ground investigation encountered Made Ground to depths between 0.45m and

0.90m overlying the Claygate Member; the latter consisted of firm to very stiff, brown grey, sandy silty clay to the termination depth of the boreholes, thus confirming the BGS data. Based on topographic survey data, the boreholes were formed from approximately 89m AOD, although the exact elevation was not confirmed.

4.7. Groundwater strikes were recorded during drilling of the boreholes BH1 and BH2 at depths of 7.00m (c.82m AOD) and 6.80m (c.82.20m AOD) respectively. During three monitoring visits undertaken in March and April 2017, groundwater was recorded in BH1 at depths of c.3.40m (c.85.60m AOD) and in BH2 at depths of c.1.70m bgl (c.87.30m AOD). The proposed lower ground floor slab is proposed to be at c.86m AOD, hence groundwater may be encountered during construction. Monitoring and measures to control groundwater during construction are recommended in the revised Geotechnical BIA report (page 43) including getting the advice of a specialist dewatering contractor prior to construction.

4.8. Considering the scale and depth of the proposed excavations, the current lower ground floor on site, the neighbouring structural levels and the monitored groundwater level, the proposed development is not anticipated to impact the wider hydrogeological environment. According to the revised Geotechnical BIA report the aquifer is expected to extend much deeper than the perched water which is present in the less cohesive bands of the Claygate Member. Also, in the same report it was stated (page 39) that any ground movements resulting from dewatering will be minimal and will not affect the integrity of the underpin trenches or beyond, and that no instability issues are expected due to running sand.

4.9. A geotechnical interpretation was provided. The methodology for deriving the values of bearing capacity was clarified in the revised Geotechnical BIA report (page 38).

4.10. A ground movement assessment (GMA) was undertaken and presented in the revised Geotechnical BIA report in Appendix G. The revised GMA assumed, in accordance with the structural drawings, that excavations will be required to c.3.50m and c.5.50m below ground level, with the latter being in the area of the proposed swimming pool towards the rear of the site.

4.11. According to the revised Geotechnical BIA report, unsupported excavations in the Made Ground and the Claygate Member are likely to be unstable. For this reason, the proposed construction sequence included temporary propping with the floor slabs subsequently installed to act as permanent props in the long term. Sacrificial trench sheeting to stop potential collapse prior to concreting could also be adopted, however, it is understood that temporary works will be finalised post-planning and prior to construction.

4.12. The revised GMA adopted CIRIA C760 methodology which is intended for embedded retaining walls, however, it is accepted that this approach can predict ground movements within the range typically anticipated for the proposed 'hit and miss' retaining wall techniques when carried out with good control of workmanship.

4.13. Anticipated vertical and horizontal ground movements up to 6mm were estimated due to basement excavation and retaining wall installation, using CIRIA C760. Additional undrained short term analysis undertaken using retaining wall specialist software confirmed similar values (up to 4mm) of anticipated ground movement. The additional analysis included proposed construction stages, props installation, existing and proposed loads. The global stability was also checked for critical sections.

4.14. The potential impact and damage to the neighbouring buildings situated at 25 West Hill Park, 23 and 25 Merton Lane was predicted as within Category 1 'very slight' damage or lower. In order to minimise the anticipated ground movement the recommendations presented in the revised GMA (Section 2.1) should be adopted, including good control of workmanship during construction. Although long term conditions were not included in the revised GMA, it is accepted that most of ground movement will occur during construction and given the anticipated low values (4 to 6mm) in the short term, the results and the damage assessment are considered acceptable.

4.15. In the context of the above, the impact on land stability due to the proposed development has been demonstrated, with Category 1 'very slight' damage or lower anticipated for nearby structures. Based on the low estimated values of ground movement and their distance from the proposed excavations, it is accepted that the surrounding infrastructure (highways, pavements, underground services) will not likely be affected assuming monitoring (as discussed below) and good control of workmanship are maintained throughout the construction stage.

4.16. According to the LBC website, there is in place planning permission for a side extension (2017/5176/P). It is understood that this side extension has been excluded from the current proposal and the revised Geotechnical BIA report has been updated accordingly.

4.17. An outline movement monitoring strategy was presented in the Structural BIA report (section 7.4.3) with reasonable movement trigger levels. This monitoring strategy should be further refined and finalised prior to construction.

4.18. The development is not within an area prone to flooding. According to the Structural BIA report there will be less than 2% increase in the hard surfaced areas across the site due to the proposed development. The requirement for any mitigation measures due to this minor increase in impermeable areas including the option of a green roof over the proposed extension suggested by the Structural BIA report, should be discussed with LBC. It is accepted that there will be negligible impact to the hydrology of the site due to the proposed development.

4.19. An outline construction programme was appended in the BIA documents.

4.20. Based on the above comments, all previous queries are closed and it can be confirmed that the proposal adheres to the requirements of the CPG Basements.

5.0 CONCLUSIONS

5.1. The Basement Impact Assessment (BIA) has been carried out by individuals with suitable qualifications.

5.2. The proposed development involves extending the lower ground floor to the front and the rear. Maximum excavation depths of c.5.50m are anticipated adopting a 'hit and miss' technique for the proposed retaining walls.

5.3. The site is located within the Hampstead Ponds catchment area but at a higher elevation and at a distance of c.130m from the ponds.

5.4. The BIA confirmed that the site is located above a 'Secondary A' aquifer, the Claygate Member of London Clay Formation. Monitoring indicated that groundwater may be encountered during construction and some control of groundwater may be required.

5.5. It is accepted that the proposed development is not anticipated to impact the wider hydrogeological environment.

5.6. The impact on land stability due to the proposed development has been demonstrated with Category 1 'very slight' damage or lower anticipated for the nearby structures. It is also accepted that the surrounding infrastructure (highways, pavements, underground services) will not likely be affected assuming monitoring and good control of workmanship is maintained throughout the construction stage.

5.7. The outline movement monitoring strategy should be finalised prior to construction.

5.8. It is accepted that there will be negligible impact to the hydrology of the site.

5.9. An outline construction programme was appended in the BIA documents.

5.10. It can be confirmed that the proposal adheres to the requirements of the CPG Basements.

Appendix 1: Residents' Consultation Comments

Pertinent to the BIA

Residents' Consultation Comments

Surname	Address	Date	Issue raised	Response
Rose (Chair Highgate CAAC)	-	16/4/2019	Uncertainty with ground and groundwater conditions	A site-specific ground investigation was undertaken thus reducing uncertainty. The groundwater conditions and the potential need for dewatering were discussed in the revised Geotechnical BIA report.
Newgas (West Hill Park Management Co Ltd)	-	21/4/2019	Recent planning permission has been granted to extend the house. Groundwater issues in the Claygate Member.	According to the available BIA documents the side extension does not form part of the current proposal. The groundwater conditions and the potential need for dewatering were discussed in the revised Geotechnical BIA report. The long term hydrogeology of the area is not anticipated to be affected.
Simon	-	21/4/2019	There is in place a planning permission for a side extension (2017/5176/P).	According to the available BIA documents the side extension does not form part of the current proposal.

Appendix 2: Audit Query Tracker

Audit Query Tracker

Query No	Subject	Query	Status	Date closed out
1	Stability	The methodologies for deriving bearing capacity and anticipated settlement and heave values should be clarified (Geotechnical BIA).	Closed	02/08/2019
2	Stability	The assumed distance to the closest structural element of 25 West Hill Park should be checked (GMA).	Closed	02/08/2019
3	Stability	The predicted ground movements in Appendix E (GMA) should be revised. They are not moderately conservative.	Closed	29/11/2019
4	Stability	The GMA should be revised to consider existing and proposed loads, underpin construction, long-term ground movements, and potential ground movement due to water ingress and dewatering. Detailed input and output of the specialist software used should be provided[1]. Additional information including calculations and drawings should be provided if a piled wall solution be utilised to deal with groundwater[2]. Additional clarifications and amendments are requested about the GMA as per the comments of Section 4[3] of this audit.	Closed	29/11/2019
5	Stability	The GMA should consider the potential impacts and mitigation measures for all potentially affected surrounding structures and infrastructure.	Closed	29/11/2019
6	Stability	The 'proposed side extension' shown in Figure 7 of the Geotechnical BIA report should be clarified.	Closed	02/08/2019
7	Stability	The outline movement monitoring strategy should be updated and finalised prior to construction in accordance with the revised GMA.	N/A	N/A
8	Stability	Consultation with utility owners should be undertaken should their utilities be affected by the proposed development.	N/A	N/A
9	Stability	Manmade cut slopes should be supported to avoid instability. This should be taken into account in the final design.	N/A	N/A

Notes to table

1. Added as a previous Geotechnical BIA report included specialist software.

2. Added as the engineering solution intended for dealing with groundwater and the impact this may have to neighbouring properties, should be clarified, as part of the Geotechnical BIA report and the GMA assessment.

3. Added due to additional calculations and graphs presented in the revised GMA.

Appendix 3: Supplementary Supporting Documents

1. 28 November 2019 email of Croft Structural Engineers
2. 29 November 2019 email of Croft Structural Engineers
3. "Ground Movement Assessment" report, November 2019, Ground and Project Consultants Ltd
4. "Temporary Works Scheme Design" drawing, 28 November 2019, Croft Structural Engineers
5. "Ground Investigation and Basement Impact Assessment Report", November 2019, Ground & Water Ltd

26 West Hill Park Camden BIA - [Camden ref. 2019/1426/P ; Audit ref. 12985-55]
Geoff Watson to: ChristosBotsialas@campbellreith.com 28/11/2019 19:24
Cc: "Nadezda Gobova", "Evgeniya Konopleva", "Diver, John", "Darina Jurovskaja", "Jon Smithson", "Chris Tomlin"
History:
This message has been forwarded.

1 Attachment

26 WHP GMA Report (Nov2019) Rev2.pdf

Dear Christos,

Thank you for your time on this audit since we last communicated. Please find a new ground movement analysis to close the remaining query for the BIA. As you are aware, Ground & Water appointed an external Chartered Geologist, Jon Smithson, for the assessment. I believe that you and Jon have been in direct contact and have come to an understanding of how best to close the query.

The report for the GMA is attached. Please note:

- The main body of the report is not more than 10 pages long (the remaining pages are appendices)
- The report contains a new and independent GMA using the CIRIA C760; the damage category ranges from 'Negligible' to 'Very Slight'
- The report contains an additional and independent analysis of the ground using GEO5 software. This is a supplementary analysis of the most critical areas and only the most relevant data is presented.

Thank you in advance for reviewing this.

Kind regards

Geoff Watson
Structural Engineer

CROFT STRUCTURAL ENGINEERS
Clock Shop Mews, Rear of 60 Saxon Rd, SE25 5EH
T: 020 8684 4744
D: 020 3763 2895
M: 07985 160448

RE: 26 West Hill Park Camden BIA - [Camden ref. 2019/1426/P ; Audit ref. 12985-55] - additional info
Geoff Watson to: ChristosBotsialas@campbellreith.com 29/11/2019 15:02
Cc: "Nadezda Gobova", "Evgeniya Konopleva", "Darina Jurovskaja", "Diver, John"
History:
This message has been forwarded.

3 Attachments

26 WHP (Temp works drawing) TW-10-Rev3 [A3].pdf

Appendices E+F (BIA, Pdisp input and output) - Nov 2019.pdf

Main (G.I. + BIA) report (Nov 2019) + figures.pdf

Dear Christos,

Please find additional information attached to support the GMA, issued yesterday.

The attachments include only the relevant extracts from previously submitted documents which have been revised to maintain consistency with the recent ground movement assessment. These are:

- Updated Ground Investigation and BIA report by Ground & Water (main body of report only). The alterations are appropriately highlighted. These include minor alterations to the ground depths to match those in the latest GMA and revised notes about groundwater.

- Updated PDisp Analysis (Appendices E&F). We understand that there were no remaining queries with this analysis; however, this has been updated with due to minor alterations with the input levels, described above.

- Alterations to the proposed temporary works by Croft (drawings TW-10), to simplify the construction by having the walls cast in one drive

Please note that the main, and only real significant, change is with the GMA. This is formally referred to as being part of Appendix G in Ground & Water's report. This was attached to the message that we sent yesterday.

Kind regards

Geoff Watson
Structural Engineer

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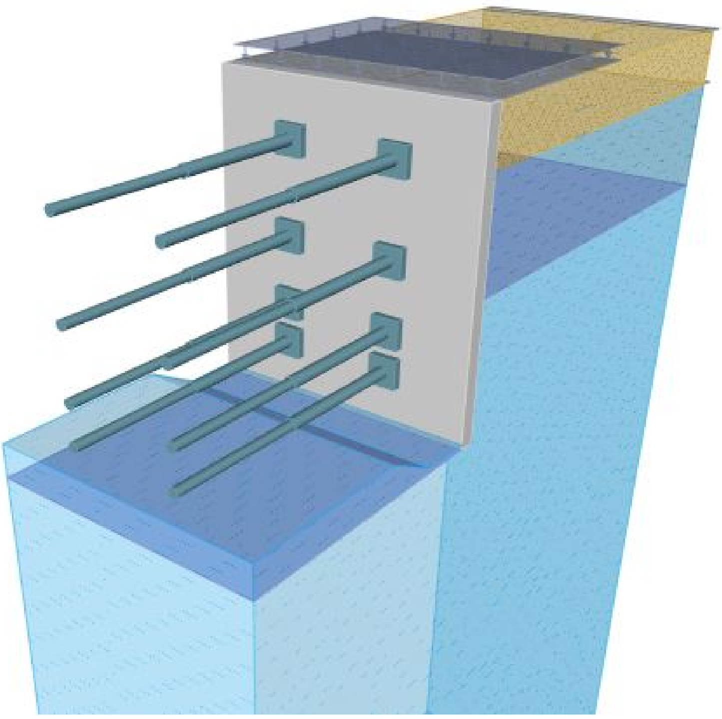
Report No. 60431-1

26 WEST HILL PARK

Ground Movement Assessment

November 2019



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Document Verification

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Signatures and Approvals				
Author	J Smithson		Date	28/11/18
Checker and Approver	O Hawes/ G Manning		Date	28/11/18

Disclaimer

This report has been prepared by Ground and Project Consultants Ltd, with reasonable skill, care and diligence within the agreed scope and terms of contract and taking account of the manpower and resources devoted to it by agreement with its client, and is provided by Ground and Project Consultants Ltd solely for the use of its client, Ground and Water Ltd.

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Contents

1 Introduction

Ground and Project Consultants Ltd (GPCL) have been instructed by Ground and Water Ltd to carry out a Ground Movement Assessment for the proposed basement at 26 West Hill Park, Hampstead, London N6 6ND.

The objectives of this report are to ascertain the expected ground movements and degree of any building damage at the structures adjacent to the site. A basement Impact has previously been carried out by Ground and Water and a previous assessment by Chelmer.

The scope of this report and approach are as follows:

- Establish expected ground conditions at the site from previous reports
- Develop an understanding of the proposals and their relationship to adjacent structures.
- Carry out an assessment of ground movement following the principles and procedures set out in C760 “Guidance on Embedded Retaining Wall Design”.
- Carry out stability and ground movement assessment using GeO5 Sheeting Check software.

This report has been prepared and approved by Jon Smithson, BSc, MSC, FGS, CGeol who is ca chartered geologist with over 30 years’ experience.

2 Assessment of Ground Movement

2.1 Movement due to wall installation and excavation

An assessment of ground movements has been carried out as follows:

Movements have been assessed for the adjoining properties and 'sub-properties as follows:

23 Merton Lane: Garage*

23 Merton Lane: House

25 Merton Lane: Garage

25 Merton Lane: House

25 West Hill Park: House

*Note: This building may have been converted to habitable use. Within this report any reference to '23 Merton Lane: Garage' will be for this building despite the suspected change of use. The relevant data for this building contained in this report is still valid.

The garages have been assessed separately as they are closer to the proposed basement then their respective properties. Other properties are considered too distant to be influenced by a well-executed basement construction.

The magnitude of ground movements has been assessed for the excavation in front of the retaining structure, i.e. the basement wall.

Movement due to wall installation may be discounted at this stage as it is understood that the property will be underpinned, and as such a wall will not be installed into the ground. Rather the 'wall' will be installed in 'hit and miss' panels as the excavation proceeds.

It is important to note that CIRIA report C760 is written for embedded retaining walls. Therefore, movement calculations for the excavation of soil and installation of underpins does not strictly apply to C760. There is no recognised method for calculating ground movements due to underpinned basements so C760 is used as a convenient and recognised approach.

To provide a comparison and to assess the stability of the basement during the construction phase analyses have been carried out using "Sheeting Check" retaining wall software. The software allows input of progressive construction stages and the installation of props. Horizontal and Vertical Ground Movements are calculated.

It is recognised that settlements are generally small where care and appropriate measures are taken in this type of basement construction.

Design drawings developed by the engineer (Croft Structural Engineers) have been reviewed and used to inform this assessment.

The following key assumptions have been made:

- The detailed design of the basement (and associated temporary works) has been carried out by an appropriately qualified and experienced structural engineer, to current professional standards and best practice.
- The maximum excavation depth is approximately 5.5m below ground floor level.
- The method of basement construction will be via underpinning.
- A high wall stiffness has been assumed.
- The wall will be propped using stiff closely spaced (2m) props in the temporary case at four levels for the deeper area (pool) as per the "Temporary Works Scheme Design" drawing.
- In the permanent case the wall will always be propped at floor level.

For the purposes of the calculations, the width and height of the subject properties have been estimated to be as follows:

Building reference	Distance from basement	Basement depth	Ground level	Roof level	Building Height	Building length/width
23 Merton Lane: Garage	2.50	5.40	87.86	90.88	3.02	4.25
23 Merton Lane: House	8.10	5.40	87.86	93.39	5.53	23.00
25 Merton Lane: Garage	7.25	5.40	88.76	91.76	3.00	4.25
25 Merton Lane: House	8.50	5.40	88.76	92.97	4.21	20.00
25 West Hill Park: House	3.00	3.50	91.30	99.26	7.96	12.40
25 West Hill Park: House (w.r.t. pool)	10.00	5.40	91.30	99.26	7.96	12.40

It is assumed that the soils are competent soils from the ground investigation. The Claygate Beds encountered beneath thin Made Ground were firm to stiff becoming stiff clay soils.

For each subject property the "effective" basement depth has been calculated, as the ground levels vary around the site and for example at the Merton Lane side (southwest), the effective basement depth is just 2.16 to 3.0m. These depths are tabulated as follows:

Building reference	Distance from basement	Basement depth	Ground level	Basement Floor Level	Effective Basement Depth
23 Merton Lane: Garage	2.50	5.40	87.86	85.70	2.16
23 Merton Lane: House	8.10	5.40	87.86	85.70	2.16
25 Merton Lane: Garage	7.25	5.40	88.76	85.70	3.06
25 Merton Lane: House	8.50	5.40	88.76	85.70	3.06
25 West Hill Park: House	3.00	3.50	91.30	87.30	4.00
25 West Hill Park: House (w.r.t. pool)	10.00	5.40	91.30	85.70	5.60

The following ground movements have been calculated in relation to ground movements, using figure 6.15 in C760. An allowance for wall installation movements has also been developed for comparison.

Structure	No Wall Installation Included (movements from excavation only)		
	Maximum Vertical Deflection Δ (mm)	Maximum Horizontal Movement dh (mm)	Building Damage Assessment
23 Merton Lane: Garage	0.4	0.6	0 Negligible
23 Merton Lane: House	1.3	2.1	0 Negligible
25 Merton Lane: Garage	0.9	0.9	0 Negligible
25 Merton Lane: House	1.5	2.8	0 Negligible
25 West Hill Park: House	2.8	4.8	1 Very Slight
25 West Hill Park: House (pool)	1.4	3.4	0 Negligible
Structure	**Wall Installation Included**		
	Maximum Vertical Deflection Δ (mm)	**Maximum Horizontal Movement dh (mm)**	**Building Damage Assessment**
23 Merton Lane: Garage	0.8	1.3	1 Very Slight
23 Merton Lane: House	1.6	2.1	0 Negligible
25 Merton Lane: Garage	1.5	1.2	1 Very Slight
25 Merton Lane: House	1.9	2.8	0 Negligible
25 West Hill Park: House	4.0	6.0	1 Very Slight
25 West Hill Park: House (pool)	1.8	3.4	0 Negligible

Note that the figures above do not represent the total ground movement but the differential movements which are predicted to be experienced by the building. The ground movement and building damage calculations are appended.

The calculations assumes that the wall is propped high and progressively as the excavation proceeds at lower levels during construction and therefore a high stiffness can be assumed. Note the drawing the drawing 'Temporary Works Scheme Design' by Croft indicates such that this is the intended process. It is understood that there will be adequate propping in the temporary case to justify this assumption and in the permanent case the structure will provide adequate support to the retaining walls and act as a high-level prop.

There are a number of key points to note in using this assessment:

- Most ground movement will occur during excavation of the basement and construction so the adequacy of temporary support will be critical in limiting ground movements.
- The speed of propping and support is key to limiting ground movements and limiting unpropped wall heights.
- Good workmanship will contribute to minimising ground movements.

- The calculation assumes the wall is in competent soil as per the findings of the ground investigation.

Ground movement can be minimised by adopting a number of measures, including:

- Ensuring that adequate propping is in place at all times during construction.
- Installation of the first (stiff) support quickly and early in the construction sequence.
- Avoid leaving ground unsupported.
- Minimise deterioration of the unexcavated soil mass by the use of blinding/ covering with a waterproof membrane.
- Avoid overbreak.
- If dewatering is required the control and appropriate design of the process must ensure that fines removal and drawdown are minimised.

It must be noted that the movements are calculated values based on the findings and methods of CIRIA C760. Larger movements may be generated if anyone or any combination of the above recommendations and/or assumptions are not heeded or if ground conditions are different from those anticipated by the investigation. Computer analysis suggests that ground movements are highly sensitive to prop and wall stiffness, so the use of stiff props both in the temporary and permanent cases is essential.

The actual magnitude of these movements will depend upon a number of factors described above and the nature of the ground expected may give rise to larger movements.

2.2 GEO5 Sheeting Check Analysis

GEO5 sheeting check software has analysed the ground movement independently of the CIRIA calculations. The results are of a similar scale and corroborate the movements evaluated by the CIRIA analysis. The software has also been used to evaluate the effectiveness of propping and influence of prop and wall stiffness.

Undrained analysis has been performed, with a series of construction stages, involved excavation propping further excavation, propping and so forth. The “Temporary Works Scheme Design” drawing has been used to inform this sequence.

Two sets of analyses have been conducted, one at the area adjacent to 23 Merton Lane and its garage and the other adjacent to 25 West Hill Park. Each assessment includes for line loads from the houses and garage as appropriate.

The analysis suggests the following ground movements, which are within those developed using C760. The figures developed in GEO5 have not been used to assess building damage.

Structure	Sheeting Check Calculations		
	Maximum Vertical Movement	Distance from the wall	Maximum Horizontal Movement
23 Merton Lane: House and Garage	4mm	2.2m	3mm
25 West Hill Park: House	4mm	1.7m	2mm

Global stability checks have also been carried out. These are satisfactory for both scenarios in the undrained condition.

Selected relevant output sheets are appended.

Deviation in the model from early propping demonstrates its importance in that if props are installed late then more ground movement occurs.

Appendix A: CIRIA C760 Data

Ground and Project CONSULTANTS LTD

Project	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 23 Merton Lane without Wall installation		
Date	28 November 2019	**Rev**	0

House Details, Background Data and Assumptions

Calculations based on C760 Pg155, assuming 10mm in 18m depth and zero movement at 1 x wall depth.

Adjacent Building 1	**23 Merton Lane - Garage**
Ground Level (m AOD)	87.86
Basement Depth (m)	5.4
Basement Floor level (m)	85.7
Effective Basement Depth (m)	2.16
Wall Depth (m)	0
Length (m)	4.25
Height (m)	3.02
Distance (m)	2.5
Far Side (m)	6.75
Adjacent Building 2	**23 Merton Lane - House**
Ground Level (m AOD)	87.86
Basement Depth (m)	5.4
Basement Floor level (m)	85.7
Effective Basement Depth (m)	2.16
Wall Depth (m)	0
Length (m)	23
Height (m)	5.53
Distance (m)	8.1
Far Side (m)	31.1

Movement Calculations for Wall Installation

Horizontal				
Distance from wall/wall depth (m)	**Distance (m)**	**Movement/ Wall Depth (%)**	**Horizontal Movement (mm)**	**Relevance to Adjacent Properties**
0.0	0.0	0.080	0.00	
0.1	0.0	0.070	0.00	
0.2	0.0	0.075	0.00	
0.3	0.0	0.060	0.00	
0.4	0.0	0.050	0.00	
0.5	0.0	0.044	0.00	
0.6	0.0	0.040	0.00	NS - Garage
0.7	0.0	0.035	0.00	
0.8	0.0	0.030	0.00	
0.9	0.0	0.020	0.00	
1.0	0.0	0.018	0.00	
1.1	0.0	0.015	0.00	
1.2	0.0	0.012	0.00	FS - Garage
1.3	0.0	0.010	0.00	
1.4	0.0	0.005	0.00	
1.5	0.0	0.000	0.00	NS - House
Vertical				
Distance from wall/wall depth (m)	**Distance (m)**	**Movement/ Wall Depth (%)**	**Vertical Movement (mm)**	**Relevance to Adjacent Properties**
0.0	0.0	0.050	0.0	
0.1	0.0	0.048	0.0	
0.2	0.0	0.046	0.0	
0.3	0.0	0.042	0.0	
0.4	0.0	0.040	0.0	
0.5	0.0	0.037	0.0	NS - Garage
0.6	0.0	0.035	0.0	
0.7	0.0	0.032	0.0	
0.8	0.0	0.029	0.0	
0.9	0.0	0.027	0.0	
1.0	0.0	0.025	0.0	
1.1	0.0	0.023	0.0	
1.2	0.0	0.020	0.0	FS - Garage
1.3	0.0	0.018	0.0	
1.4	0.0	0.016	0.0	
1.5	0.0	0.014	0.0	NS - House
1.6	0.0	0.011	0.0	
1.7	0.0	0.009	0.0	
1.8	0.0	0.007	0.0	
1.9	0.0	0.004	0.0	
2.0	0.0	0.000	0.0	FS - House

Deflection Ratio

	23 Merton Lane - Garage	23 Merton Lane - House
Delta	0.0	0.0
dh	0.0	0.0

Ground and Project CONSULTANTS LTD

Project:	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 23 Merton Lane without Wall installation		
Date:	28 November 2019	**Rev**	0

Assumptions

Calculations based on C760 Fig. 6.15 assume system stiffness =1000, FOS against base heave >3. zero at 3 x excavation depth as Fig. 6.11 a) and b)

High Stiffness	0.0075

Movement Calculations for Excavation

Horizontal Distance from wall/excavation depth (m)	Distance (m)	Horizontal Movement/ Wall Depth (%)	Horizontal Movement (mm)	Relevance to adjacent properties
0.0	0.0	0.15	3.2	
0.2	0.0	0.1425	3.1	
0.4	0.0	0.135	2.9	NS - Garage
0.6	0.0	0.1275	2.8	
0.8	0.0	0.12	2.6	
1.0	0.0	0.1125	2.4	
1.2	0.0	0.105	2.3	FS - Garage
1.4	0.0	0.0975	2.1	NS - House
1.6	0.0	0.09	1.9	
1.8	0.0	0.0825	1.8	
2.0	0.0	0.075	1.6	
2.2	0.0	0.0675	1.5	
2.4	0.0	0.06	1.3	
2.6	0.0	0.0525	1.1	
2.8	0.0	0.045	1.0	
3.0	0.0	0.0375	0.8	
3.2	0.0	0.03	0.6	
3.4	0.0	0.0225	0.5	
3.6	0.0	0.015	0.3	
3.8	0.0	0.0075	0.2	
4.0	0.0	0	0.0	FS - House

Vertical Distance from wall/excavation (m) depth	Distance (m)	Settlement/ Excavation Depth (%)	Settlement (mm)	Relevance to adjacent properties
0.0	0	0.040	0.86	
0.2	0	0.050	1.08	
0.4	0	0.070	1.51	
0.6	0	0.080	1.73	NS - Garage
0.8	0	0.070	1.51	
1.0	0	0.070	1.51	
1.2	0	0.060	1.30	FS - Garage
1.4	0	0.060	1.30	NS - House
1.6	0	0.050	1.08	
1.8	0	0.040	0.86	
2.0	0	0.035	0.76	
2.2	0	0.030	0.65	
2.4	0	0.025	0.54	
2.6	0	0.020	0.43	
2.8	0	0.015	0.32	
3.0	0	0.010	0.22	
3.2	0	0.005	0.11	
3.4	0	0.000	0.00	FS - House

Deflection Ratio

	23 Merton Lane - Garage	23 Merton Lane - House
Delta	0.43	1.3
dh	0.65	2.1

Ground and Project CONSULTANTS LTD

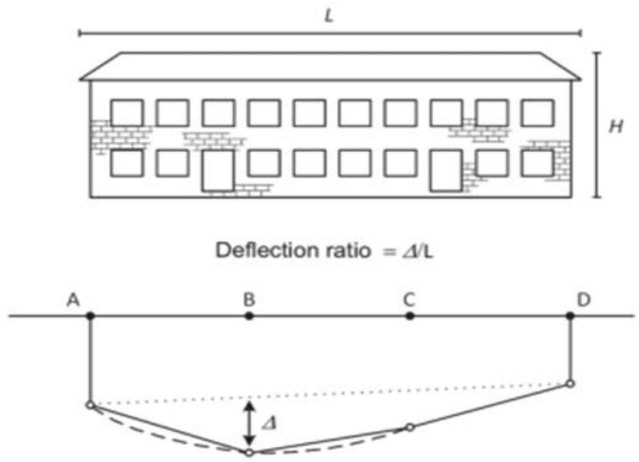
Project:	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 23 Merton Lane without Wall installation		
Date:	28 November 2019	**Rev**	0

Combined for Wall Installation and Excavation

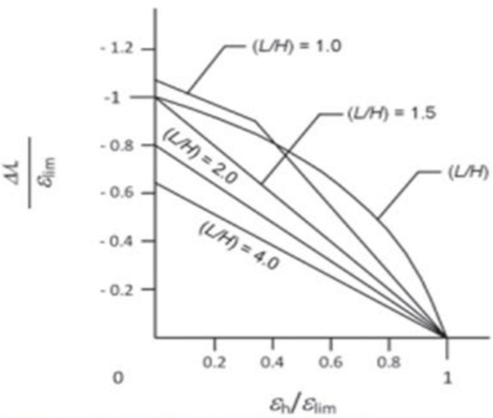
	23 Merton Lane - Garage	23 Merton Lane - House
Delta	0.4	1.3
dh	0.6	2.1

Movement Assessment

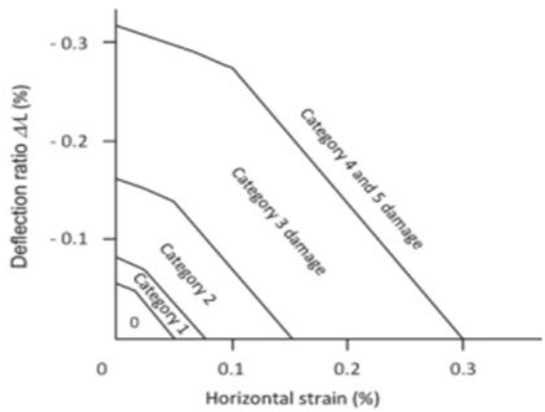
23 Merton Lane - Garage			
Horiz Strain (%)	dh/L	0.02	
Deflection Ratio (%)	Delta/L	0.01	
From Graph Fig 6.27©	Damage Category	1 V Slight	
From Graph Fig 6.27 (b)	Try elim	0.075	upper limit of damage category Table 6.4
L/H	1.4	Therefore eh/elim	0.2
Reading off Fig 6.27 (b) for closest L/H curve this gives Delta/L/ elim		0.7	
L	4250		
Therefore Delta = L x Reading x elim			
Delta (mm)	2.2		
Delta for combined wall installation and excavation is less :Damage category is confirmed as	0.3		
23 Merton Lane - House			
Horiz Strain (%)	dh/L	0.01	
Deflection Ratio (%)	Delta/L	0.01	
From Graph Fig 6.27©	Damage Category	0 Negligible	
From Graph Fig 6.27 (b)	Try elim	0.05	upper limit of damage category Table 6.4
L/H	4.2	Therefore eh/elim	0.2
Reading off Fig 6.27 (b) for closest L/H curve this gives Delta/L/ elim		0.5	
L	23000		
Therefore Delta = L x Reading x elim			
Delta (mm)	5.75		
Delta for combined wall installation and excavation is less : Damage category is confirmed as	0 Negligible		



a   Definition of deflection ratio



b   Influence of horizontal strain on $\Delta/L/\varepsilon_{lim}$



c   Relationship between damage category and deflection ratio and horizontal tensile strain for hogging for (*L*/*H*) = 1.0

Note

By adopting values of ε_{lim} associated with various damage categories given in Table 6.4, figure (b) can be developed into an interaction diagram showing the relationship between Δ/L and ε_h for a particular value of *L*/*H* figure (c) shows such a diagram for (*L*/*H*) = 1.0.

Figure 6.27   Relationship between damage category, deflection ratio and horizontal tensile strain (after Burland, 2001)

Ground and Project CONSULTANTS LTD

Project	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 23 Merton Lane with Wall		
Date	28 November 2019	**Rev**	0

House Details, Background Data and Assumptions

Calculations based on C760 Pg155, assuming 10mm in 18m depth and zero movement at 1 x wall depth.

Adjacent Building 1	**23 Merton Lane - Garage**
Ground Level (m AOD)	87.86
Basement Depth (m)	5.4
Basement Floor level (m)	85.7
Effective Basement Depth (m)	2.16
Wall Depth (m)	5.4
Length (m)	4.25
Height (m)	3.02
Distance (m)	2.5
Far Side (m)	6.75
Adjacent Building 2	**23 Merton Lane - House**
Ground Level (m AOD)	87.86
Basement Depth (m)	5.4
Basement Floor level (m)	85.7
Effective Basement Depth (m)	2.16
Wall Depth (m)	5.4
Length (m)	23
Height (m)	5.53
Distance (m)	8.1
Far Side (m)	31.1

Movement Calculations for Wall Installation

Horizontal

Distance from wall/wall depth (m)	Distance (m)	Movement/ Wall Depth (%)	Horizontal Movement (mm)	Relevance to Adjacent Properties
0.0	0.0	0.080	1.73	
0.1	0.5	0.070	1.51	
0.2	1.1	0.075	1.62	
0.3	1.6	0.060	1.30	
0.4	2.2	0.050	1.08	
0.5	2.7	0.044	0.95	
0.6	3.2	0.040	0.86	NS - Garage
0.7	3.8	0.035	0.76	
0.8	4.3	0.030	0.65	
0.9	4.9	0.020	0.43	
1.0	5.4	0.018	0.39	
1.1	5.9	0.015	0.32	
1.2	6.5	0.012	0.26	FS - Garage
1.3	7.0	0.010	0.22	
1.4	7.6	0.005	0.11	
1.5	8.1	0.000	0.00	NS - House

Vertical

Distance from wall/wall depth (m)	Distance (m)	Movement/ Wall Depth (%)	Vertical Movement (mm)	Relevance to Adjacent Properties
0.0	0.0	0.050	1.1	
0.1	0.5	0.048	1.0	
0.2	1.1	0.046	1.0	
0.3	1.6	0.042	0.9	
0.4	2.2	0.040	0.9	
0.5	2.7	0.037	0.8	NS - Garage
0.6	3.2	0.035	0.8	
0.7	3.8	0.032	0.7	
0.8	4.3	0.029	0.6	
0.9	4.9	0.027	0.6	
1.0	5.4	0.025	0.5	
1.1	5.9	0.023	0.5	
1.2	6.5	0.020	0.4	FS - Garage
1.3	7.0	0.018	0.4	
1.4	7.6	0.016	0.3	
1.5	8.1	0.014	0.3	NS - House
1.6	8.6	0.011	0.2	
1.7	9.2	0.009	0.2	
1.8	9.7	0.007	0.2	
1.9	10.3	0.004	0.1	
2.0	10.8	0.000	0.0	FS - House

Deflection Ratio

	23 Merton Lane - Garage	**23 Merton Lane - House**
Delta	0.4	0.3
dh	0.6	0.0

Project:	26 West Hill Park		
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Assumptions

Calculations based on C760 Fig. 6.15 assume system stiffness =1000, FOS against base heave >3. zero at 3 x excavation depth as Fig. 6.11 a) and b)

High Stiffness	0.0075

Movement Calculations for Excavation

Horizontal **Distance from wall/excavation depth (m)**	**Distance (m)**	**Horizontal Movement/ Wall Depth (%)**	**Horizontal Movement (mm)**	**Relevance to adjacent properties**
0.0	0.0	0.15	3.2	
0.2	1.1	0.1425	3.1	
0.4	2.2	0.135	2.9	NS - Garage
0.6	3.2	0.1275	2.8	
0.8	4.3	0.12	2.6	
1.0	5.4	0.1125	2.4	
1.2	6.5	0.105	2.3	FS - Garage
1.4	7.6	0.0975	2.1	NS - House
1.6	8.6	0.09	1.9	
1.8	9.7	0.0825	1.8	
2.0	10.8	0.075	1.6	
2.2	11.9	0.0675	1.5	
2.4	13.0	0.06	1.3	
2.6	14.0	0.0525	1.1	
2.8	15.1	0.045	1.0	
3.0	16.2	0.0375	0.8	
3.2	17.3	0.03	0.6	
3.4	18.4	0.0225	0.5	
3.6	19.4	0.015	0.3	
3.8	20.5	0.0075	0.2	
4.0	21.6	0	0.0	FS - House

Vertical **Distance from wall/excavation (m) depth**	**Distance (m)**	**Settlement/ Excavation Depth (%)**	**Settlement (mm)**	**Relevance to adjacent properties**
0.0	0	0.040	0.86	
0.2	1.08	0.050	1.08	
0.4	2.16	0.070	1.51	
0.6	3.24	0.080	1.73	NS - Garage
0.8	4.32	0.070	1.51	
1.0	5.4	0.070	1.51	
1.2	6.48	0.060	1.30	FS - Garage
1.4	7.56	0.060	1.30	NS - House
1.6	8.64	0.050	1.08	
1.8	9.72	0.040	0.86	
2.0	10.8	0.035	0.76	
2.2	11.88	0.030	0.65	
2.4	12.96	0.025	0.54	
2.6	14.04	0.020	0.43	
2.8	15.12	0.015	0.32	
3.0	16.2	0.010	0.22	
3.2	17.28	0.005	0.11	
3.4	18.36	0.000	0.00	FS - House

Deflection Ratio

	23 Merton Lane - Garage	**23 Merton Lane - House**
Delta	0.43	1.3
dh	0.65	2.1

Ground and Project
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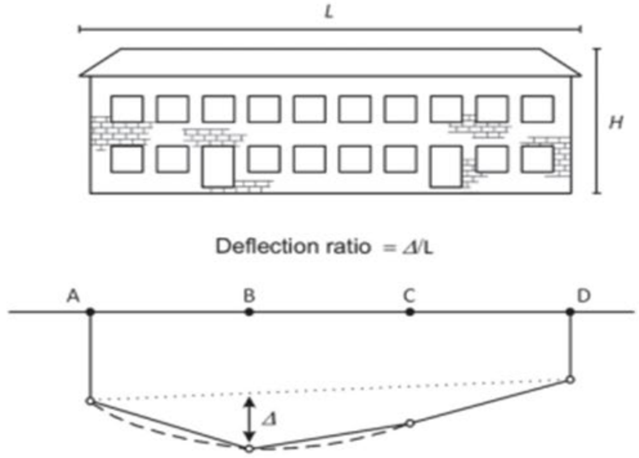
Project:	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 23 Merton Lane with Wall		
Date:	28 November 2019	**Rev**	0

Combined for Wall Installation and Excavation

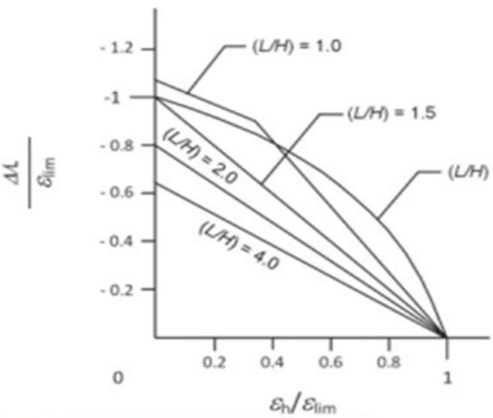
	23 Merton Lane - Garage	23 Merton Lane - House
Delta	0.8	1.6
dh	1.3	2.1

Movement Assessment

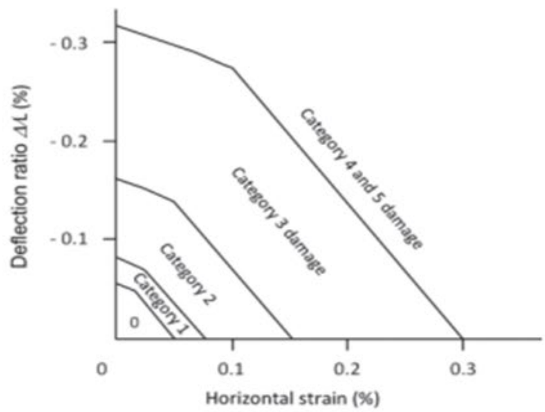
23 Merton Lane - Garage			
Horiz Strain (%)	dh/L	0.03	
Deflection Ratio (%)	Delta/L	0.02	
From Graph Fig 6.27©	Damage Category	1 V Slight	
From Graph Fig 6.27 (b)	Try elim	0.075	upper limit of damage category Table 6.4
L/H	1.4	Therefore eh/elim	0.4
Reading off Fig 6.27 (b) for closest L/H curve this gives Delta/L/ elim		0.6	
L	4250		
Therefore Delta = L x Reading x elim			
Delta (mm)	1.9125		
Delta for combined wall installation and excavation is less :Damage category is confirmed as	1 V Slight		
23 Merton Lane - House			
Horiz Strain (%)	dh/L	0.01	
Deflection Ratio (%)	Delta/L	0.01	
From Graph Fig 6.27©	Damage Category	0 Negligible	
From Graph Fig 6.27 (b)	Try elim	0.05	upper limit of damage category Table 6.4
L/H	4.2	Therefore eh/elim	0.2
Reading off Fig 6.27 (b) for closest L/H curve this gives Delta/L/ elim		0.5	
L	23000		
Therefore Delta = L x Reading x elim			
Delta (mm)	5.75		
Delta for combined wall installation and excavation is less : Damage category is confirmed as	0 Negligible		



a Definition of deflection ratio



b Influence of horizontal strain on $\Delta/L/\varepsilon_{lim}$



c Relationship between damage category and deflection ratio and horizontal tensile strain for hogging for (L/H) = 1.0

Note

By adopting values of ε_{lim} associated with various damage categories given in Table 6.4, figure (b) can be developed into an interaction diagram showing the relationship between Δ/L and ε_h for a particular value of L/H figure (c) shows such a diagram for $(L/H) = 1.0$.

Figure 6.27 Relationship between damage category, deflection ratio and horizontal tensile strain (after Burland, 2001)

Ground and Project CONSULTANTS LTD

Project	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 25 Merton Lane without Wall Installation		
Date	28 November 2019	**Rev**	0

House Details, Background Data and Assumptions

Calculations based on C760 Pg155, assuming 10mm in 18m depth and zero movement at 1 x wall depth.

Adjacent Building 1	**25 Merton Lane - Garage**
Ground Level (m AOD)	88.76
Basement Depth (m)	5.4
Basement Floor level (m)	85.7
Effective Basement Depth (m)	3.06
Wall Depth (m)	0
Length (m)	4.25
Height (m)	3
Distance (m)	7.25
Far Side (m)	11.5
Adjacent Building 2	**25 Merton Lane - House**
Ground Level (m AOD)	88.76
Basement Depth (m)	5.4
Basement Floor level (m)	85.7
Effective Basement Depth (m)	3.06
Wall Depth (m)	0
Length (m)	20
Height (m)	4.21
Distance (m)	8.5
Far Side (m)	28.5

Movement Calculations for Wall Installation

Horizontal

Distance from wall/wall depth (m)	Distance (m)	Movement/ Wall Depth (%)	Horizontal Movement (mm)	Relevance to Adjacent Properties
0.0	0.0	0.080	0.00	
0.1	0.0	0.070	0.00	
0.2	0.0	0.075	0.00	
0.3	0.0	0.060	0.00	
0.4	0.0	0.050	0.00	
0.5	0.0	0.044	0.00	
0.6	0.0	0.040	0.00	
0.7	0.0	0.035	0.00	
0.8	0.0	0.030	0.00	
0.9	0.0	0.020	0.00	
1.0	0.0	0.018	0.00	
1.1	0.0	0.015	0.00	
1.2	0.0	0.012	0.00	
1.3	0.0	0.010	0.00	NS - Garage
1.4	0.0	0.005	0.00	
1.5	0.0	0.000	0.00	NS - House

Vertical

Distance from wall/wall depth (m)	Distance (m)	Movement/ Wall Depth (%)	Vertical Movement (mm)	Relevance to Adjacent Properties
0.0	0.0	0.050	0.0	
0.1	0.0	0.048	0.0	
0.2	0.0	0.046	0.0	
0.3	0.0	0.042	0.0	
0.4	0.0	0.040	0.0	
0.5	0.0	0.037	0.0	
0.6	0.0	0.035	0.0	
0.7	0.0	0.032	0.0	
0.8	0.0	0.029	0.0	
0.9	0.0	0.027	0.0	
1.0	0.0	0.025	0.0	
1.1	0.0	0.023	0.0	
1.2	0.0	0.020	0.0	
1.3	0.0	0.018	0.0	NS - Garage
1.4	0.0	0.016	0.0	
1.5	0.0	0.014	0.0	
1.6	0.0	0.011	0.0	NS - House
1.7	0.0	0.009	0.0	
1.8	0.0	0.007	0.0	
1.9	0.0	0.004	0.0	
2.0	0.0	0.000	0.0	FS - Garage

Deflection Ratio

	25 Merton Lane - Garage	**25 Merton Lane - House**
Delta	0.0	0.0
dh	0.0	0.0

Ground and Project CONSULTANTS LTD

Project:	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 25 Merton Lane without Wall Installation		
Date:	28 November 2019	Rev	0

Assumptions

Calculations based on C760 Fig. 6.15 assume system stiffness =1000, FOS against base heave >3. zero at 3 x excavation depth as Fig. 6.11 a) and b)

High Stiffness	0.0075

Movement Calculations for Excavation

Horizontal Distance from wall/excavation depth (m)	Distance (m)	Horizontal Movement/ Wall Depth (%)	Horizontal Movement (mm)	Relevance to adjacent properties
0.0	0.0	0.15	4.6	
0.2	0.0	0.1425	4.4	
0.4	0.0	0.135	4.1	
0.6	0.0	0.1275	3.9	
0.8	0.0	0.12	3.7	
1.0	0.0	0.1125	3.4	
1.2	0.0	0.105	3.2	
1.4	0.0	0.0975	3.0	NS - Garage
1.6	0.0	0.09	2.8	NS - House
1.8	0.0	0.0825	2.5	
2.0	0.0	0.075	2.3	
2.2	0.0	0.0675	2.1	FS - Garage
2.4	0.0	0.06	1.8	
2.6	0.0	0.0525	1.6	
2.8	0.0	0.045	1.4	
3.0	0.0	0.0375	1.1	
3.2	0.0	0.03	0.9	
3.4	0.0	0.0225	0.7	
3.6	0.0	0.015	0.5	
3.8	0.0	0.0075	0.2	
4.0	0.0	0	0.0	FS - House

Vertical Distance from wall/excavation (m) depth	Distance (m)	Settlement/ Excavation Depth (%)	Settlement (mm)	Relevance to adjacent properties
0.0	0	0.040	1.22	
0.2	0	0.050	1.53	
0.4	0	0.070	2.14	
0.6	0	0.080	2.45	
0.8	0	0.070	2.14	
1.0	0	0.070	2.14	
1.2	0	0.060	1.84	
1.4	0	0.060	1.84	NS - Garage
1.6	0	0.050	1.53	NS - House
1.8	0	0.040	1.22	
2.0	0	0.035	1.07	
2.2	0	0.030	0.92	FS - Garage
2.4	0	0.025	0.77	
2.6	0	0.020	0.61	
2.8	0	0.015	0.46	
3.0	0	0.010	0.31	
3.2	0	0.005	0.15	
3.4	0	0.000	0.00	FS - House

Deflection Ratio

	25 Merton Lane - Garage	25 Merton Lane - House
Delta	0.92	1.5
dh	0.92	2.8

Ground and Project
CONSULTANTS LTD

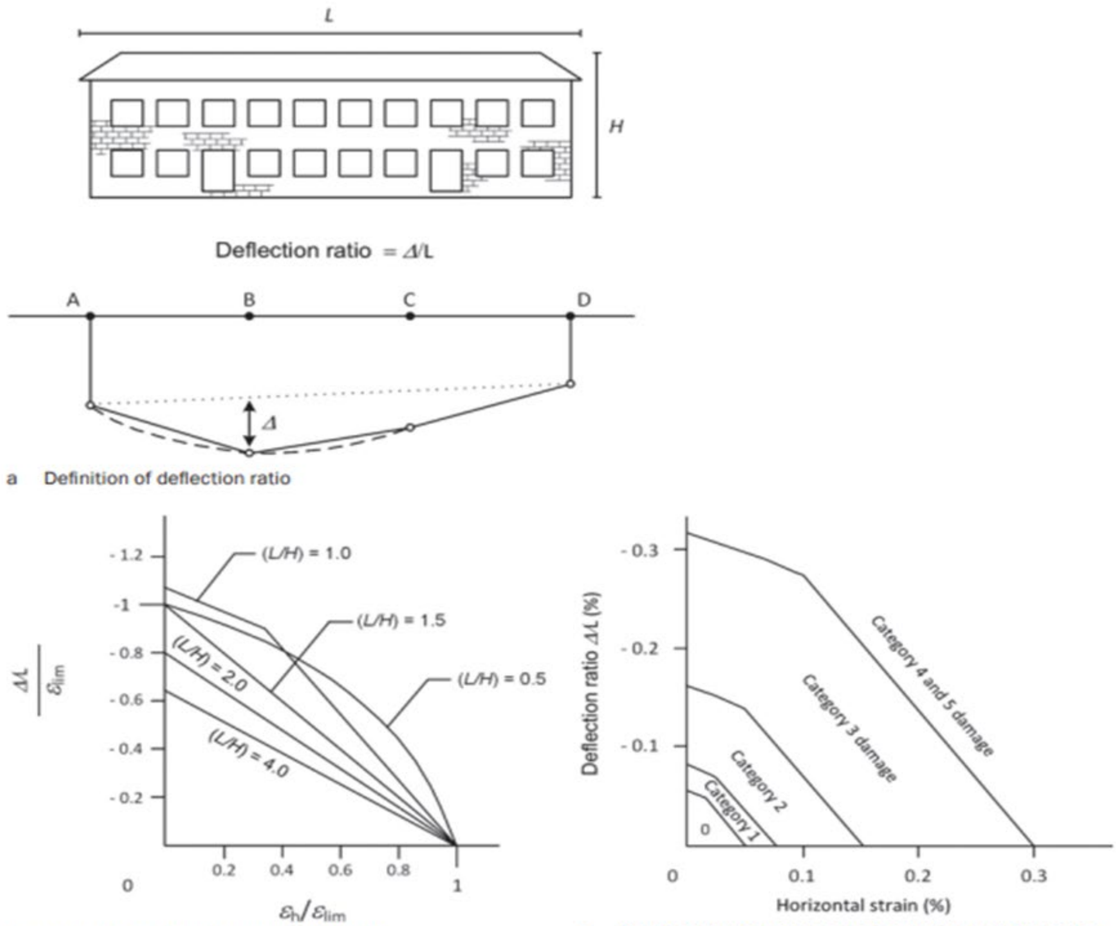
Project:	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 25 Merton Lane without Wall Installation		
Date:	28 November 2019	**Rev**	0

Combined for Wall Installation and Excavation

	25 Merton Lane - Garage	25 Merton Lane - House
Delta	0.9	1.5
dh	0.9	2.8

Movement Assessment

25 Merton Lane - Garage			
Horiz Strain (%)	dh/L	0.02	
Deflection Ratio (%)	Delta/L	0.02	
From Graph Fig 6.27©	Damage Category	1 V Slight	
From Graph Fig 6.27 (b)	Try elim	0.075	upper limit of damage category Table 6.4
L/H	1.4	Therefore eh/elim	0.3
Reading off Fig 6.27 (b) for closest L/H curve this gives Delta/L/ elim		0.6	
L	4250		
Therefore Delta = L x Reading x elim			
Delta (mm)	1.9125		
Delta for combined wall installation and excavation is less :Damage category is confirmed as	1 V Slight		
25 Merton Lane - House			
Horiz Strain (%)	dh/L	0.01	
Deflection Ratio (%)	Delta/L	0.01	
From Graph Fig 6.27©	Damage Category	0 Negligible	
From Graph Fig 6.27 (b)	Try elim	0.05	upper limit of damage category Table 6.4
L/H	4.8	Therefore eh/elim	0.3
Reading off Fig 6.27 (b) for closest L/H curve this gives Delta/L/ elim		0.4	
L	20000		
Therefore Delta = L x Reading x elim			
Delta (mm)	4		
Delta for combined wall installation and excavation is less : Damage category is confirmed as	0 Negligible		



a Definition of deflection ratio

b Influence of horizontal strain on $\Delta/L/\varepsilon_{lim}$

c Relationship between damage category and deflection ratio and horizontal tensile strain for hogging for $(L/H) = 1.0$

Note

By adopting values of ε_{lim} associated with various damage categories given in Table 6.4, figure (b) can be developed into an interaction diagram showing the relationship between Δ/L and ε_h for a particular value of L/H figure (c) shows such a diagram for $(L/H) = 1.0$.

Figure 6.27 Relationship between damage category, deflection ratio and horizontal tensile strain (after Burland, 2001)

Ground and Project CONSULTANTS LTD

Project	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 25 Merton Lane with Wall		
Date	28 November 2019	**Rev**	0

House Details, Background Data and Assumptions

Calculations based on C760 Pg155, assuming 10mm in 18m depth and zero movement at 1 x wall depth.

Adjacent Building 1	**25 Merton Lane - Garage**
Ground Level (m AOD)	88.76
Basement Depth (m)	5.4
Basement Floor level (m)	85.7
Effective Basement Depth (m)	3.06
Wall Depth (m)	5.4
Length (m)	4.25
Height (m)	3
Distance (m)	7.25
Far Side (m)	11.5
Adjacent Building 2	**25 Merton Lane - House**
Ground Level (m AOD)	88.76
Basement Depth (m)	5.4
Basement Floor level (m)	85.7
Effective Basement Depth (m)	3.06
Wall Depth (m)	5.4
Length (m)	20
Height (m)	4.21
Distance (m)	8.5
Far Side (m)	28.5

Movement Calculations for Wall Installation

Horizontal

Distance from wall/wall depth (m)	**Distance (m)**	**Movement/ Wall Depth (%)**	**Horizontal Movement (mm)**	**Relevance to Adjacent Properties**
0.0	0.0	0.080	2.45	
0.1	0.5	0.070	2.14	
0.2	1.1	0.075	2.30	
0.3	1.6	0.060	1.84	
0.4	2.2	0.050	1.53	
0.5	2.7	0.044	1.35	
0.6	3.2	0.040	1.22	
0.7	3.8	0.035	1.07	
0.8	4.3	0.030	0.92	
0.9	4.9	0.020	0.61	
1.0	5.4	0.018	0.55	
1.1	5.9	0.015	0.46	
1.2	6.5	0.012	0.37	
1.3	7.0	0.010	0.31	NS - Garage
1.4	7.6	0.005	0.15	
1.5	8.1	0.000	0.00	NS - House

Vertical

Distance from wall/wall depth (m)	**Distance (m)**	**Movement/ Wall Depth (%)**	**Vertical Movement (mm)**	**Relevance to Adjacent Properties**
0.0	0.0	0.050	1.5	
0.1	0.5	0.048	1.5	
0.2	1.1	0.046	1.4	
0.3	1.6	0.042	1.3	
0.4	2.2	0.040	1.2	
0.5	2.7	0.037	1.1	
0.6	3.2	0.035	1.1	
0.7	3.8	0.032	1.0	
0.8	4.3	0.029	0.9	
0.9	4.9	0.027	0.8	
1.0	5.4	0.025	0.8	
1.1	5.9	0.023	0.7	
1.2	6.5	0.020	0.6	
1.3	7.0	0.018	0.6	NS - Garage
1.4	7.6	0.016	0.5	
1.5	8.1	0.014	0.4	
1.6	8.6	0.011	0.3	NS - House
1.7	9.2	0.009	0.3	
1.8	9.7	0.007	0.2	
1.9	10.3	0.004	0.1	
2.0	10.8	0.000	0.0	FS - Garage

Deflection Ratio

	23 Merton Lane - Garage	**23 Merton Lane - House**
Delta	0.6	0.3
dh	0.3	0.0

Ground and Project
CONSULTANTS LTD

Project:	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 25 Merton Lane with Wall		
Date:	28 November 2019	**Rev**	0

Assumptions

Calculations based on C760 Fig. 6.15 assume system stiffness =1000, FOS against base heave >3. zero at 3 x excavation depth as Fig. 6.11 a) and b)

High Stiffness	0.0075

Movement Calculations for Excavation

Horizontal **Distance from wall/excavation depth (m)**	**Distance (m)**	**Horizontal Movement/ Wall Depth (%)**	**Horizontal Movement (mm)**	**Relevance to adjacent properties**
0.0	0.0	0.15	4.6	
0.2	1.1	0.1425	4.4	
0.4	2.2	0.135	4.1	
0.6	3.2	0.1275	3.9	
0.8	4.3	0.12	3.7	
1.0	5.4	0.1125	3.4	
1.2	6.5	0.105	3.2	
1.4	7.6	0.0975	3.0	NS - Garage
1.6	8.6	0.09	2.8	NS - House
1.8	9.7	0.0825	2.5	
2.0	10.8	0.075	2.3	
2.2	11.9	0.0675	2.1	FS - Garage
2.4	13.0	0.06	1.8	
2.6	14.0	0.0525	1.6	
2.8	15.1	0.045	1.4	
3.0	16.2	0.0375	1.1	
3.2	17.3	0.03	0.9	
3.4	18.4	0.0225	0.7	
3.6	19.4	0.015	0.5	
3.8	20.5	0.0075	0.2	
4.0	21.6	0	0.0	FS - House

Vertical **Distance from wall/excavation (m) depth**	**Distance (m)**	**Settlement/ Excavation Depth (%)**	**Settlement (mm)**	**Relevance to adjacent properties**
0.0	0	0.040	1.22	
0.2	1.08	0.050	1.53	
0.4	2.16	0.070	2.14	
0.6	3.24	0.080	2.45	
0.8	4.32	0.070	2.14	
1.0	5.4	0.070	2.14	
1.2	6.48	0.060	1.84	
1.4	7.56	0.060	1.84	NS - Garage
1.6	8.64	0.050	1.53	NS - House
1.8	9.72	0.040	1.22	
2.0	10.8	0.035	1.07	
2.2	11.88	0.030	0.92	FS - Garage
2.4	12.96	0.025	0.77	
2.6	14.04	0.020	0.61	
2.8	15.12	0.015	0.46	
3.0	16.2	0.010	0.31	
3.2	17.28	0.005	0.15	
3.4	18.36	0.000	0.00	FS - House

Deflection Ratio

	25 Merton Lane - Garage	**25 Merton Lane - House**
Delta	0.92	1.5
dh	0.92	2.8

Ground and Project CONSULTANTS LTD

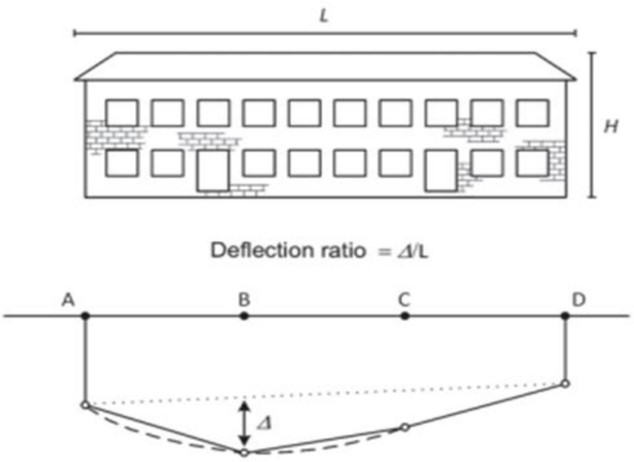
Project:	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 25 Merton Lane with Wall		
Date:	28 November 2019	**Rev**	0

Combined for Wall Installation and Excavation

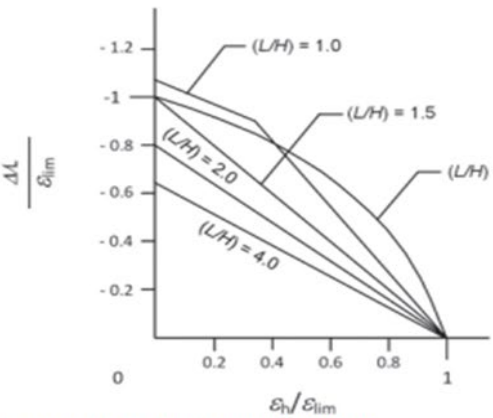
	25 Merton Lane - Garage	**25 Merton Lane - House**
Delta	1.5	1.9
dh	1.2	2.8

Movement Assessment

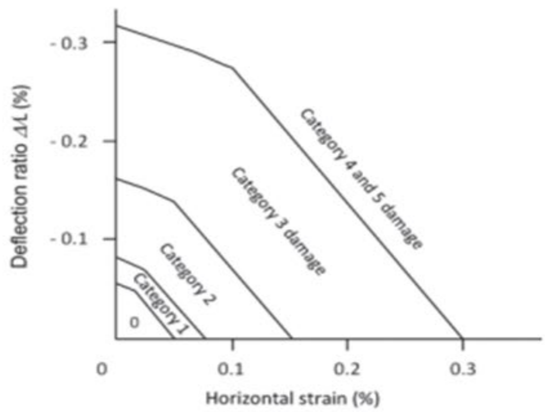
25 Merton Lane - Garage			
Horiz Strain (%)	dh/L	0.03	
Deflection Ratio (%)	Delta/L	0.03	
From Graph Fig 6.27©	Damage Category	1 V Slight	
From Graph Fig 6.27 (b)	Try elim	0.075	upper limit of damage category Table 6.4
L/H	1.4	Therefore eh/elim	0.4
Reading off Fig 6.27 (b) for closest L/H curve this gives Delta/L/ elim		0.6	
L	4250		
Therefore Delta = L x Reading x elim			
Delta (mm)	1.9125		
Delta for combined wall installation and excavation is less :Damage category is confirmed as	1 V Slight		
25 Merton Lane - House			
Horiz Strain (%)	dh/L	0.01	
Deflection Ratio (%)	Delta/L	0.01	
From Graph Fig 6.27©	Damage Category	0 Negligible	
From Graph Fig 6.27 (b)	Try elim	0.05	upper limit of damage category Table 6.4
L/H	4.8	Therefore eh/elim	0.3
Reading off Fig 6.27 (b) for closest L/H curve this gives Delta/L/ elim		0.5	
L	20000		
Therefore Delta = L x Reading x elim			
Delta (mm)	5		
Delta for combined wall installation and excavation is less : Damage category is confirmed as	0 Negligible		



a   Definition of deflection ratio



b   Influence of horizontal strain on $\Delta/L/\varepsilon_{lim}$



c   Relationship between damage category and deflection ratio and horizontal tensile strain for hogging for $(L/H) = 1.0$

Note

By adopting values of ε_{lim} associated with various damage categories given in Table 6.4, figure (b) can be developed into an interaction diagram showing the relationship between Δ/L and ε_h for a particular value of L/H figure (c) shows such a diagram for $(L/H) = 1.0$.

Figure 6.27   Relationship between damage category, deflection ratio and horizontal tensile strain (after Burland, 2001)

Ground and Project CONSULTANTS LTD

Project	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 25 West Hill Park No wall installation		
Date	28 November 2019	**Rev**	0

House Details, Background Data and Assumptions

Calculations based on C760 Pg155, assuming 10mm in 18m depth and zero movement at 1 x wall depth.

Building	**25 West Hill Park**
Ground Level (m AOD)	91.3
Basement Depth (m)	4.0
Basement Floor level (m)	87.3
Effective Basement Depth (m)	4.0
Wall Depth (m)	0.0
Length (m)	12.4
Height (m)	8.0
Distance (m)	3.0
Far Side (m)	15.4

Movement Calculations for Wall Installation

Horizontal

Distance from wall/wall depth (m)	**Distance (m)**	**Movement/ Wall Depth (%)**	**Horizontal Movement (mm)**	**Relevance to Adjacent Properties**
0.0	0.0	0.080	0.00	
0.1	0.0	0.070	0.00	
0.2	0.0	0.075	0.00	
0.3	0.0	0.060	0.00	
0.4	0.0	0.050	0.00	
0.5	0.0	0.044	0.00	
0.6	0.0	0.040	0.00	
0.7	0.0	0.035	0.00	
0.8	0.0	0.030	0.00	NS
0.9	0.0	0.020	0.00	
1.0	0.0	0.018	0.00	
1.1	0.0	0.015	0.00	
1.2	0.0	0.012	0.00	
1.3	0.0	0.010	0.00	
1.4	0.0	0.005	0.00	
1.5	0.0	0.000	0.00	FS

Vertical

Distance from wall/wall depth (m)	**Distance (m)**	**Movement/ Wall Depth (%)**	**Vertical Movement (mm)**	**Relevance to Adjacent Properties**
0.0	0.0	0.050	0.0	
0.1	0.0	0.048	0.0	
0.2	0.0	0.046	0.0	
0.3	0.0	0.042	0.0	
0.4	0.0	0.040	0.0	
0.5	0.0	0.037	0.0	
0.6	0.0	0.035	0.0	
0.7	0.0	0.032	0.0	
0.8	0.0	0.029	0.0	NS
0.9	0.0	0.027	0.0	
1.0	0.0	0.025	0.0	
1.1	0.0	0.023	0.0	
1.2	0.0	0.020	0.0	
1.3	0.0	0.018	0.0	
1.4	0.0	0.016	0.0	
1.5	0.0	0.014	0.0	
1.6	0.0	0.011	0.0	
1.7	0.0	0.009	0.0	
1.8	0.0	0.007	0.0	
1.9	0.0	0.004	0.0	
2.0	0.0	0.000	0.0	FS

Deflection Ratio

	25 West Hill Park
Delta	0.0
dh	0.0

Ground and Project CONSULTANTS LTD

Project:	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 25 West Hill Park No wall installation		
Date:	28 November 2019	**Rev**	0

Assumptions

Calculations based on C760 Fig. 6.15 assume system stiffness =1000, FOS against base heave >3. zero at 3 x excavation depth as Fig. 6.11 a) and b)

High Stiffness	0.0075

Movement Calculations for Excavation

Horizontal **Distance from wall/excavation depth (m)**	**Distance (m)**	**Horizontal Movement/ Wall Depth (%)**	**Horizontal Movement (mm)**	**Relevance to adjacent properties**
0.0	0.0	0.15	6.0	
0.2	0.0	0.1425	5.7	
0.4	0.0	0.135	5.4	
0.6	0.0	0.1275	5.1	
0.8	0.0	0.12	4.8	NS
1.0	0.0	0.1125	4.5	
1.2	0.0	0.105	4.2	
1.4	0.0	0.0975	3.9	
1.6	0.0	0.09	3.6	
1.8	0.0	0.0825	3.3	
2.0	0.0	0.075	3.0	
2.2	0.0	0.0675	2.7	
2.4	0.0	0.06	2.4	
2.6	0.0	0.0525	2.1	
2.8	0.0	0.045	1.8	
3.0	0.0	0.0375	1.5	
3.2	0.0	0.03	1.2	
3.4	0.0	0.0225	0.9	
3.6	0.0	0.015	0.6	
3.8	0.0	0.0075	0.3	
4.0	0.0	0	0.0	FS

Vertical **Distance from wall/excavation (m) depth**	**Distance (m)**	**Settlement/ Excavation Depth (%)**	**Settlement (mm)**	**Relevance to adjacent properties**
0.0	0	0.040	1.60	
0.2	0	0.050	2.00	
0.4	0	0.070	2.80	NS
0.6	0	0.080	3.20	
0.8	0	0.070	2.80	
1.0	0	0.070	2.80	
1.2	0	0.060	2.40	
1.4	0	0.060	2.40	
1.6	0	0.050	2.00	
1.8	0	0.040	1.60	
2.0	0	0.035	1.40	
2.2	0	0.030	1.20	
2.4	0	0.025	1.00	
2.6	0	0.020	0.80	
2.8	0	0.015	0.60	
3.0	0	0.010	0.40	
3.2	0	0.005	0.20	
3.4	0	0.000	0.00	FS

Deflection Ratio

	25 West Hill Park
Delta	2.80
dh	4.80

Ground and Project CONSULTANTS LTD

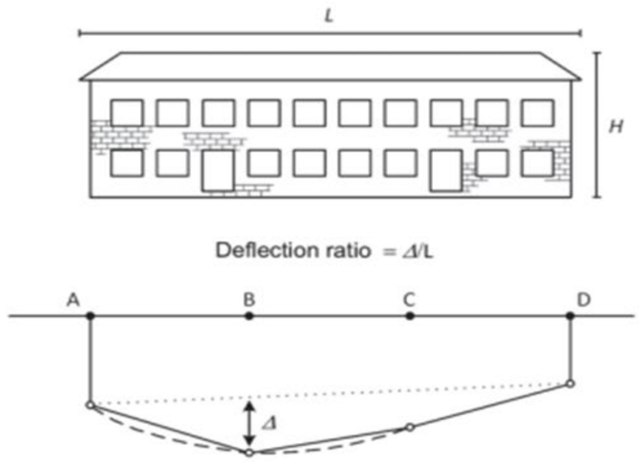
Project:	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 25 West Hill Park No wall installation		
Date:	28 November 2019	**Rev**	0

Combined for Wall Installation and Excavation

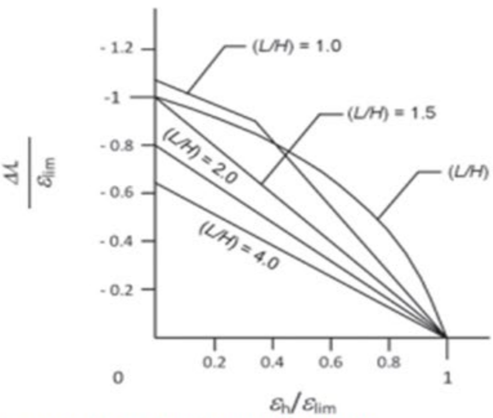
	25 West Hill Park
Delta	2.8
dh	4.8

Movement Assessment

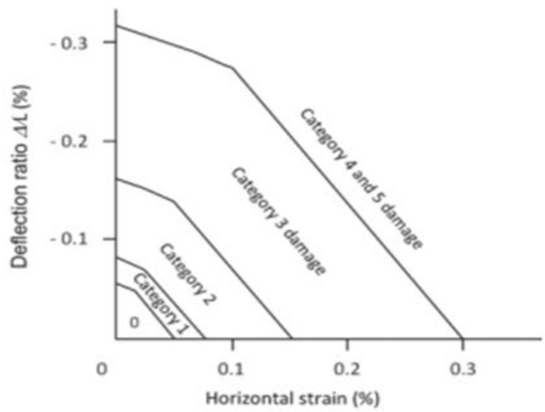
25 West Hill Park			
Horiz Strain (%)	dh/L	0.04	
Deflection Ratio (%)	Delta/L	0.02	
From Graph Fig 6.27©	Damage Category	1 V Slight	
From Graph Fig 6.27 (b)	Try elim	0.075	upper limit of damage category Table 6.4
L/H	1.6	Therefore eh/elim	0.5
Reading off Fig 6.27 (b) for closest L/H curve this gives Delta/L/ elim		0.5	
L	12400		
Therefore Delta = L x Reading x elim			
Delta (mm)	4.65		
Delta for combined wall installation and excavation is less :Damage category is confirmed as	1 V Slight		



a Definition of deflection ratio



b Influence of horizontal strain on $\Delta/L/\varepsilon_{lim}$



c Relationship between damage category and deflection ratio and horizontal tensile strain for hogging for (L/H) = 1.0

Note

By adopting values of ε_{lim} associated with various damage categories given in Table 6.4, figure (b) can be developed into an interaction diagram showing the relationship between Δ/L and ε_h for a particular value of L/H figure (c) shows such a diagram for $(L/H) = 1.0$.

Figure 6.27 Relationship between damage category, deflection ratio and horizontal tensile strain (after Burland, 2001)

Project	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 25 West Hill Park with respect to the pool, no wall		
Date	28 November 2019	Rev	0

House Details, Background Data and Assumptions

Calculations based on C760 Pg155, assuming 10mm in 18m depth and zero movement at 1 x wall depth.

Building	25 West Hill Park - w.r.t. pool
Ground Level (m AOD)	91.3
Basement Depth (m)	5.6
Basement Floor level (m)	85.7
Effective Basement Depth (m)	5.6
Wall Depth (m)	0.0
Length (m)	12.4
Height (m)	8.0
Distance (m)	10.0
Far Side (m)	15.4

Movement Calculations for Wall Installation

Horizontal

Distance from wall/wall depth (m)	Distance (m)	Movement/ Wall Depth (%)	Horizontal Movement (mm)	Relevance to Adjacent Properties
0.0	0.0	0.080	0.00	
0.1	0.0	0.070	0.00	
0.2	0.0	0.075	0.00	
0.3	0.0	0.060	0.00	
0.4	0.0	0.050	0.00	
0.5	0.0	0.044	0.00	
0.6	0.0	0.040	0.00	
0.7	0.0	0.035	0.00	
0.8	0.0	0.030	0.00	
0.9	0.0	0.020	0.00	
1.0	0.0	0.018	0.00	
1.1	0.0	0.015	0.00	
1.2	0.0	0.012	0.00	
1.3	0.0	0.010	0.00	
1.4	0.0	0.005	0.00	
1.5	0.0	0.000	0.00	NS

Vertical

Distance from wall/wall depth (m)	Distance (m)	Movement/ Wall Depth (%)	Vertical Movement (mm)	Relevance to Adjacent Properties
0.0	0.0	0.050	0.0	
0.1	0.0	0.048	0.0	
0.2	0.0	0.046	0.0	
0.3	0.0	0.042	0.0	
0.4	0.0	0.040	0.0	
0.5	0.0	0.037	0.0	
0.6	0.0	0.035	0.0	
0.7	0.0	0.032	0.0	
0.8	0.0	0.029	0.0	
0.9	0.0	0.027	0.0	
1.0	0.0	0.025	0.0	
1.1	0.0	0.023	0.0	
1.2	0.0	0.020	0.0	
1.3	0.0	0.018	0.0	
1.4	0.0	0.016	0.0	
1.5	0.0	0.014	0.0	
1.6	0.0	0.011	0.0	
1.7	0.0	0.009	0.0	
1.8	0.0	0.007	0.0	NS
1.9	0.0	0.004	0.0	
2.0	0.0	0.000	0.0	FS

Deflection Ratio

	25 West Hill Park - w.r.t. pool
Delta	0.0
dh	0.0

Ground and Project CONSULTANTS LTD

Project:	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 25 West Hill Park with respect to the pool, no wall		
Date:	28 November 2019	**Rev**	0

Assumptions

Calculations based on C760 Fig. 6.15 assume system stiffness =1000, FOS against base heave >3. zero at 3 x excavation depth as Fig. 6.11 a) and b)

High Stiffness	0.0075

Movement Calculations for Excavation

Horizontal **Distance from wall/excavation depth (m)**	**Distance (m)**	**Horizontal Movement/ Wall Depth (%)**	**Horizontal Movement (mm)**	**Relevance to adjacent properties**
0.0	0.0	0.15	8.4	
0.2	0.0	0.1425	8.0	
0.4	0.0	0.135	7.6	
0.6	0.0	0.1275	7.1	
0.8	0.0	0.12	6.7	
1.0	0.0	0.1125	6.3	
1.2	0.0	0.105	5.9	
1.4	0.0	0.0975	5.5	
1.6	0.0	0.09	5.0	
1.8	0.0	0.0825	4.6	
2.0	0.0	0.075	4.2	
2.2	0.0	0.0675	3.8	
2.4	0.0	0.06	3.4	NS
2.6	0.0	0.0525	2.9	
2.8	0.0	0.045	2.5	
3.0	0.0	0.0375	2.1	
3.2	0.0	0.03	1.7	
3.4	0.0	0.0225	1.3	
3.6	0.0	0.015	0.8	
3.8	0.0	0.0075	0.4	
4.0	0.0	0	0.0	FS

Vertical **Distance from wall/excavation (m) depth**	**Distance (m)**	**Settlement/ Excavation Depth (%)**	**Settlement (mm)**	**Relevance to adjacent properties**
0.0	0	0.040	2.24	
0.2	0	0.050	2.80	
0.4	0	0.070	3.92	
0.6	0	0.080	4.48	
0.8	0	0.070	3.92	
1.0	0	0.070	3.92	
1.2	0	0.060	3.36	
1.4	0	0.060	3.36	
1.6	0	0.050	2.80	
1.8	0	0.040	2.24	
2.0	0	0.035	1.96	
2.2	0	0.030	1.68	
2.4	0	0.025	1.40	NS
2.6	0	0.020	1.12	
2.8	0	0.015	0.84	
3.0	0	0.010	0.56	
3.2	0	0.005	0.28	
3.4	0	0.000	0.00	FS

Deflection Ratio

	25 West Hill Park - w.r.t. pool
Delta	1.40
dh	3.36

Ground and Project CONSULTANTS LTD

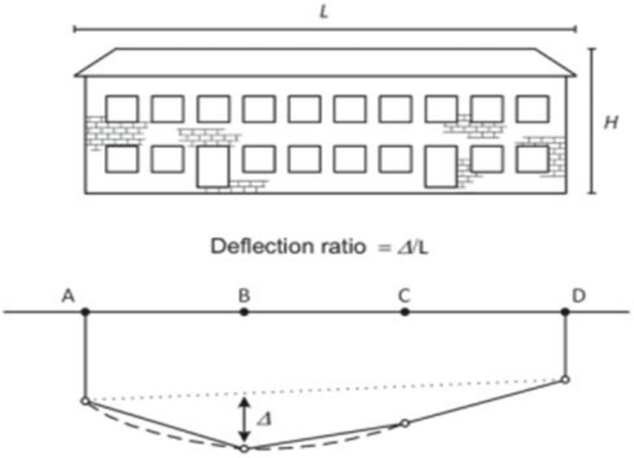
Project:	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 25 West Hill Park with respect to the pool, no wall		
Date:	28 November 2019	**Rev**	0

Combined for Wall Installation and Excavation

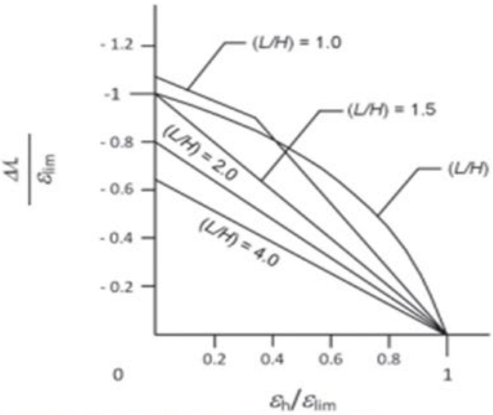
	25 West Hill Park - w.r.t. pool
Delta	1.4
dh	3.4

Movement Assessment

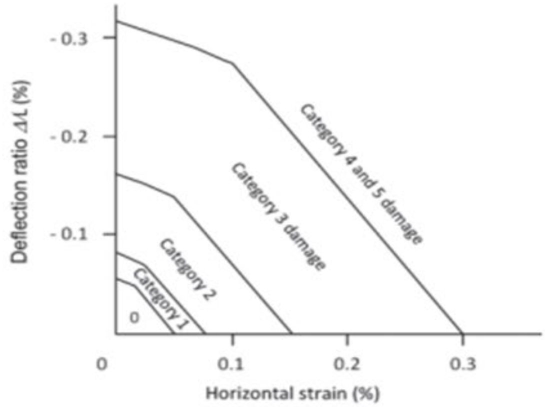
25 West Hill Park - w.r.t. pool			
Horiz Strain (%)	dh/L	0.03	
Deflection Ratio (%)	Delta/L	0.01	
From Graph Fig 6.27©	Damage Category	0 Negligible	
From Graph Fig 6.27 (b)	Try elim	0.05	upper limit of damage category Table 6.4
L/H	1.6	Therefore eh/elim	0.5
Reading off Fig 6.27 (b) for closest L/H curve this gives Delta/L/ elim		0.5	
L	12400		
Therefore Delta = L x Reading x elim			
Delta (mm)	3.1		
Delta for combined wall installation and excavation is less :Damage category is confirmed as	0 Negligible		



a Definition of deflection ratio



b Influence of horizontal strain on $\Delta/L/\varepsilon_{lim}$



c Relationship between damage category and deflection ratio and horizontal tensile strain for hogging for $(L/H) = 1.0$

Note

By adopting values of ε_{lim} associated with various damage categories given in Table 6.4, figure (b) can be developed into an interaction diagram showing the relationship between Δ/L and ε_h for a particular value of L/H figure (c) shows such a diagram for $(L/H) = 1.0$.

Figure 6.27 ***Relationship between damage category, deflection ratio and horizontal tensile strain (after Burland, 2001)***

Ground and Project CONSULTANTS LTD

Project	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 25 West Hill Park with respect to the pool		
Date	28 November 2019	**Rev**	0

House Details, Background Data and Assumptions

Calculations based on C760 Pg155, assuming 10mm in 18m depth and zero movement at 1 x wall depth.

Building	**25 West Hill Park - w.r.t. pool**
Ground Level (m AOD)	91.3
Basement Depth (m)	5.6
Basement Floor level (m)	85.7
Effective Basement Depth (m)	5.6
Wall Depth (m)	5.6
Length (m)	12.4
Height (m)	8.0
Distance (m)	10.0
Far Side (m)	15.4

Movement Calculations for Wall Installation

Horizontal

Distance from wall/wall depth (m)	**Distance (m)**	**Movement/ Wall Depth (%)**	**Horizontal Movement (mm)**	**Relevance to Adjacent Properties**
0.0	0.0	0.080	4.48	
0.1	0.6	0.070	3.92	
0.2	1.1	0.075	4.20	
0.3	1.7	0.060	3.36	
0.4	2.2	0.050	2.80	
0.5	2.8	0.044	2.46	
0.6	3.4	0.040	2.24	
0.7	3.9	0.035	1.96	
0.8	4.5	0.030	1.68	
0.9	5.0	0.020	1.12	
1.0	5.6	0.018	1.01	
1.1	6.2	0.015	0.84	
1.2	6.7	0.012	0.67	
1.3	7.3	0.010	0.56	
1.4	7.8	0.005	0.28	
1.5	8.4	0.000	0.00	NS

Vertical

Distance from wall/wall depth (m)	**Distance (m)**	**Movement/ Wall Depth (%)**	**Vertical Movement (mm)**	**Relevance to Adjacent Properties**
0.0	0.0	0.050	2.8	
0.1	0.6	0.048	2.7	
0.2	1.1	0.046	2.6	
0.3	1.7	0.042	2.4	
0.4	2.2	0.040	2.2	
0.5	2.8	0.037	2.1	
0.6	3.4	0.035	2.0	
0.7	3.9	0.032	1.8	
0.8	4.5	0.029	1.6	
0.9	5.0	0.027	1.5	
1.0	5.6	0.025	1.4	
1.1	6.2	0.023	1.3	
1.2	6.7	0.020	1.1	
1.3	7.3	0.018	1.0	
1.4	7.8	0.016	0.9	
1.5	8.4	0.014	0.8	
1.6	9.0	0.011	0.6	
1.7	9.5	0.009	0.5	
1.8	10.1	0.007	0.4	NS
1.9	10.6	0.004	0.2	
2.0	11.2	0.000	0.0	FS

Deflection Ratio

	25 West Hill Park - w.r.t. pool
Delta	0.4
dh	0.0

Ground and Project CONSULTANTS LTD

Project:	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 25 West Hill Park with respect to the pool		
Date:	28 November 2019	**Rev**	0

Assumptions

Calculations based on C760 Fig. 6.15 assume system stiffness =1000, FOS against base heave >3. zero at 3 x excavation depth as Fig. 6.11 a) and b)

High Stiffness	0.0075

Movement Calculations for Excavation

Horizontal **Distance from wall/excavation depth (m)**	**Distance (m)**	**Horizontal Movement/ Wall Depth (%)**	**Horizontal Movement (mm)**	**Relevance to adjacent properties**
0.0	0.0	0.15	8.4	
0.2	1.1	0.1425	8.0	
0.4	2.2	0.135	7.6	
0.6	3.4	0.1275	7.1	
0.8	4.5	0.12	6.7	
1.0	5.6	0.1125	6.3	
1.2	6.7	0.105	5.9	
1.4	7.8	0.0975	5.5	
1.6	9.0	0.09	5.0	
1.8	10.1	0.0825	4.6	
2.0	11.2	0.075	4.2	
2.2	12.3	0.0675	3.8	
2.4	13.4	0.06	3.4	NS
2.6	14.6	0.0525	2.9	
2.8	15.7	0.045	2.5	
3.0	16.8	0.0375	2.1	
3.2	17.9	0.03	1.7	
3.4	19.0	0.0225	1.3	
3.6	20.2	0.015	0.8	
3.8	21.3	0.0075	0.4	
4.0	22.4	0	0.0	FS

Vertical **Distance from wall/excavation (m) depth**	**Distance (m)**	**Settlement/ Excavation Depth (%)**	**Settlement (mm)**	**Relevance to adjacent properties**
0.0	0	0.040	2.24	
0.2	1.12	0.050	2.80	
0.4	2.24	0.070	3.92	
0.6	3.36	0.080	4.48	
0.8	4.48	0.070	3.92	
1.0	5.6	0.070	3.92	
1.2	6.72	0.060	3.36	
1.4	7.84	0.060	3.36	
1.6	8.96	0.050	2.80	
1.8	10.08	0.040	2.24	
2.0	11.2	0.035	1.96	
2.2	12.32	0.030	1.68	
2.4	13.44	0.025	1.40	NS
2.6	14.56	0.020	1.12	
2.8	15.68	0.015	0.84	
3.0	16.8	0.010	0.56	
3.2	17.92	0.005	0.28	
3.4	19.04	0.000	0.00	FS

Deflection Ratio

	25 West Hill Park - w.r.t. pool
Delta	1.40
dh	3.36

Ground and Project CONSULTANTS LTD

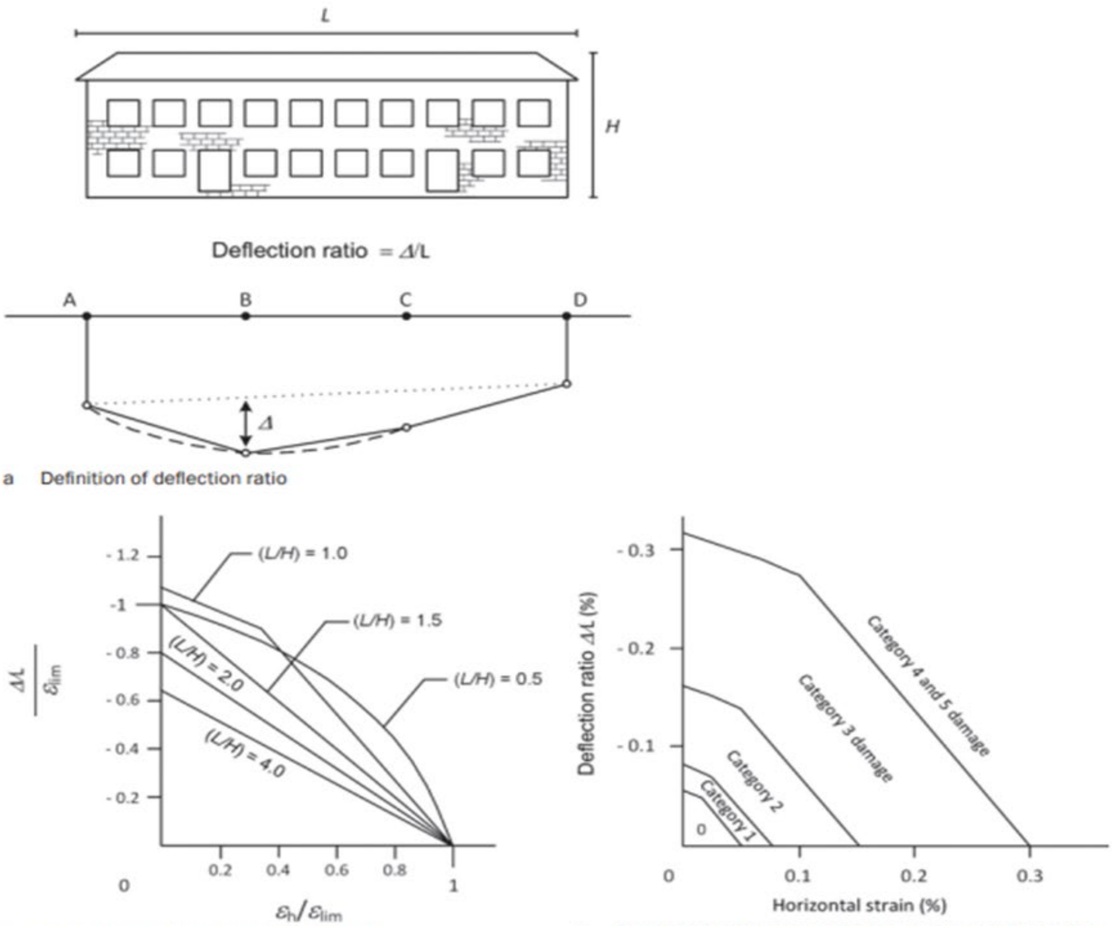
Project:	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 25 West Hill Park with respect to the pool		
Date:	28 November 2019	**Rev**	0

Combined for Wall Installation and Excavation

	25 West Hill Park - w.r.t. pool
Delta	1.8
dh	3.4

Movement Assessment

25 West Hill Park - w.r.t. pool			
Horiz Strain (%)	dh/L	0.03	
Deflection Ratio (%)	Delta/L	0.01	
From Graph Fig 6.27©	Damage Category	0 Negligible	
From Graph Fig 6.27 (b)	Try elim	0.05	upper limit of damage category Table 6.4
L/H	1.6	Therefore eh/elim	0.5
Reading off Fig 6.27 (b) for closest L/H curve this gives Delta/L/ elim		0.5	
L	12400		
Therefore Delta = L x Reading x elim			
Delta (mm)	3.1		
Delta for combined wall installation and excavation is less :Damage category is confirmed as	0 Negligible		



a Definition of deflection ratio

b Influence of horizontal strain on $\Delta/L/\varepsilon_{lim}$

c Relationship between damage category and deflection ratio and horizontal tensile strain for hogging for $(L/H) = 1.0$

Note

By adopting values of ε_{lim} associated with various damage categories given in Table 6.4, figure (b) can be developed into an interaction diagram showing the relationship between Δ/L and ε_h for a particular value of L/H figure (c) shows such a diagram for $(L/H) = 1.0$.

Figure 6.27 Relationship between damage category, deflection ratio and horizontal tensile strain (after Burland, 2001)

Project	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 25 West Hill Park		
Date	28 November 2019	Rev	0

House Details, Background Data and Assumptions

Calculations based on C760 Pg155, assuming 10mm in 18m depth and zero movement at 1 x wall depth.

Building	25 West Hill Park
Ground Level (m AOD)	91.3
Basement Depth (m)	4.0
Basement Floor level (m)	87.3
Effective Basement Depth (m)	4.0
Wall Depth (m)	4.0
Length (m)	12.4
Height (m)	8.0
Distance (m)	3.0
Far Side (m)	15.4

Movement Calculations for Wall Installation

Horizontal

Distance from wall/wall depth (m)	Distance (m)	Movement/ Wall Depth (%)	Horizontal Movement (mm)	Relevance to Adjacent Properties
0.0	0.0	0.080	3.20	
0.1	0.4	0.070	2.80	
0.2	0.8	0.075	3.00	
0.3	1.2	0.060	2.40	
0.4	1.6	0.050	2.00	
0.5	2.0	0.044	1.76	
0.6	2.4	0.040	1.60	
0.7	2.8	0.035	1.40	
0.8	3.2	0.030	1.20	NS
0.9	3.6	0.020	0.80	
1.0	4.0	0.018	0.72	
1.1	4.4	0.015	0.60	
1.2	4.8	0.012	0.48	
1.3	5.2	0.010	0.40	
1.4	5.6	0.005	0.20	
1.5	6.0	0.000	0.00	FS

Vertical

Distance from wall/wall depth (m)	Distance (m)	Movement/ Wall Depth (%)	Vertical Movement (mm)	Relevance to Adjacent Properties
0.0	0.0	0.050	2.0	
0.1	0.4	0.048	1.9	
0.2	0.8	0.046	1.8	
0.3	1.2	0.042	1.7	
0.4	1.6	0.040	1.6	
0.5	2.0	0.037	1.5	
0.6	2.4	0.035	1.4	
0.7	2.8	0.032	1.3	
0.8	3.2	0.029	1.2	NS
0.9	3.6	0.027	1.1	
1.0	4.0	0.025	1.0	
1.1	4.4	0.023	0.9	
1.2	4.8	0.020	0.8	
1.3	5.2	0.018	0.7	
1.4	5.6	0.016	0.6	
1.5	6.0	0.014	0.6	
1.6	6.4	0.011	0.4	
1.7	6.8	0.009	0.4	
1.8	7.2	0.007	0.3	
1.9	7.6	0.004	0.2	
2.0	8.0	0.000	0.0	FS

Deflection Ratio

	25 West Hill Park
Delta	1.2
dh	1.2

Ground and Project CONSULTANTS LTD

Project:	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 25 West Hill Park		
Date:	28 November 2019	**Rev**	0

Assumptions

Calculations based on C760 Fig. 6.15 assume system stiffness =1000, FOS against base heave >3. zero at 3 x excavation depth as Fig. 6.11 a) and b)

High Stiffness	0.0075

Movement Calculations for Excavation

Horizontal **Distance from wall/excavation depth (m)**	**Distance (m)**	**Horizontal Movement/ Wall Depth (%)**	**Horizontal Movement (mm)**	**Relevance to adjacent properties**
0.0	0.0	0.15	6.0	
0.2	0.8	0.1425	5.7	
0.4	1.6	0.135	5.4	
0.6	2.4	0.1275	5.1	
0.8	3.2	0.12	4.8	NS
1.0	4.0	0.1125	4.5	
1.2	4.8	0.105	4.2	
1.4	5.6	0.0975	3.9	
1.6	6.4	0.09	3.6	
1.8	7.2	0.0825	3.3	
2.0	8.0	0.075	3.0	
2.2	8.8	0.0675	2.7	
2.4	9.6	0.06	2.4	
2.6	10.4	0.0525	2.1	
2.8	11.2	0.045	1.8	
3.0	12.0	0.0375	1.5	
3.2	12.8	0.03	1.2	
3.4	13.6	0.0225	0.9	
3.6	14.4	0.015	0.6	
3.8	15.2	0.0075	0.3	
4.0	16.0	0	0.0	FS
Vertical **Distance from wall/excavation (m) depth**	**Distance (m)**	**Settlement/ Excavation Depth (%)**	**Settlement (mm)**	**Relevance to adjacent properties**
0.0	0	0.040	1.60	
0.2	0.8	0.050	2.00	
0.4	1.6	0.070	2.80	NS
0.6	2.4	0.080	3.20	
0.8	3.2	0.070	2.80	
1.0	4	0.070	2.80	
1.2	4.8	0.060	2.40	
1.4	5.6	0.060	2.40	
1.6	6.4	0.050	2.00	
1.8	7.2	0.040	1.60	
2.0	8	0.035	1.40	
2.2	8.8	0.030	1.20	
2.4	9.6	0.025	1.00	
2.6	10.4	0.020	0.80	
2.8	11.2	0.015	0.60	
3.0	12	0.010	0.40	
3.2	12.8	0.005	0.20	
3.4	13.6	0.000	0.00	FS

Deflection Ratio

	25 West Hill Park
Delta	2.80
dh	4.80

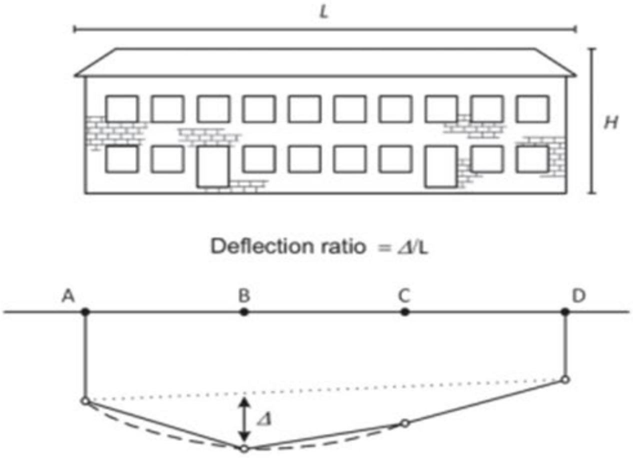
Project:	26 West Hill Park		
Project No.	60433		
Calc Title	Ground Movement Assessment at 25 West Hill Park		
Date:	28 November 2019	**Rev**	0

Combined for Wall Installation and Excavation

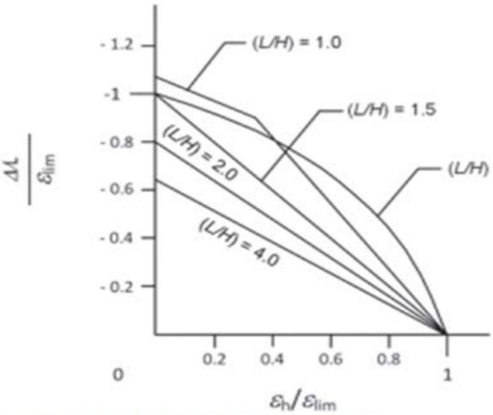
	25 West Hill Park
Delta	4.0
dh	6.0

Movement Assessment

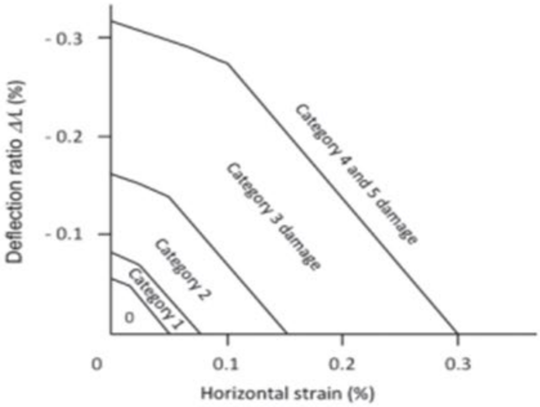
25 West Hill Park			
Horiz Strain (%)	dh/L	0.05	
Deflection Ratio (%)	Delta/L	0.03	
From Graph Fig 6.27©	Damage Category	1 V Slight	
From Graph Fig 6.27 (b)	Try elim	0.075	upper limit of damage category Table 6.4
L/H	1.6	Therefore eh/elim	0.6
Reading off Fig 6.27 (b) for closest L/H curve this gives Delta/L/ elim		0.45	
L	12400		
Therefore Delta = L x Reading x elim			
Delta (mm)	4.185		
Delta for combined wall installation and excavation is less :Damage category is confirmed as	1 V Slight		



a Definition of deflection ratio



b Influence of horizontal strain on $\Delta/L/\varepsilon_{lim}$



c Relationship between damage category and deflection ratio and horizontal tensile strain for hogging for $(L/H) = 1.0$

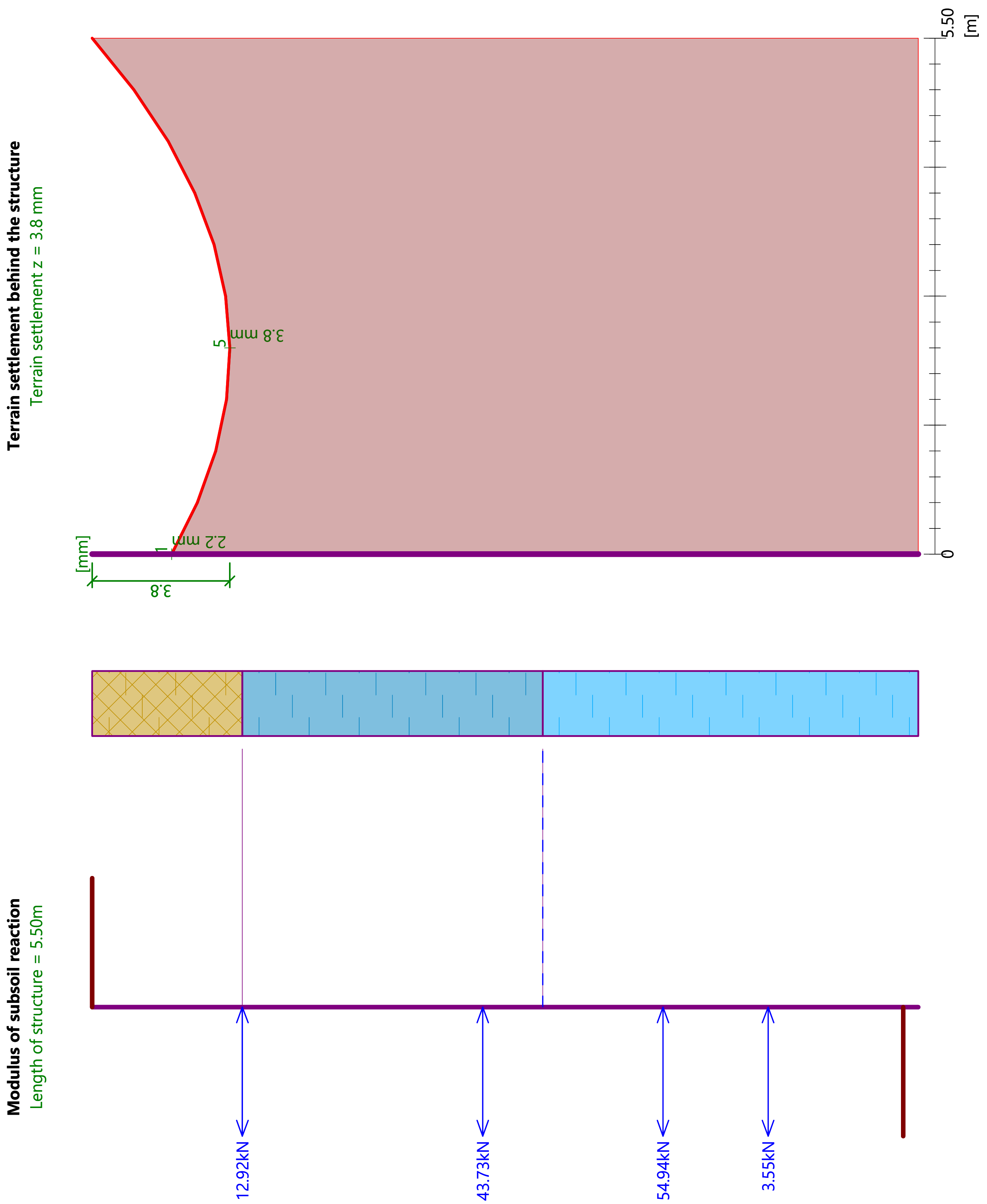
Note

By adopting values of ε_{lim} associated with various damage categories given in Table 6.4, figure (b) can be developed into an interaction diagram showing the relationship between Δ/L and ε_h for a particular value of L/H figure (c) shows such a diagram for $(L/H) = 1.0$.

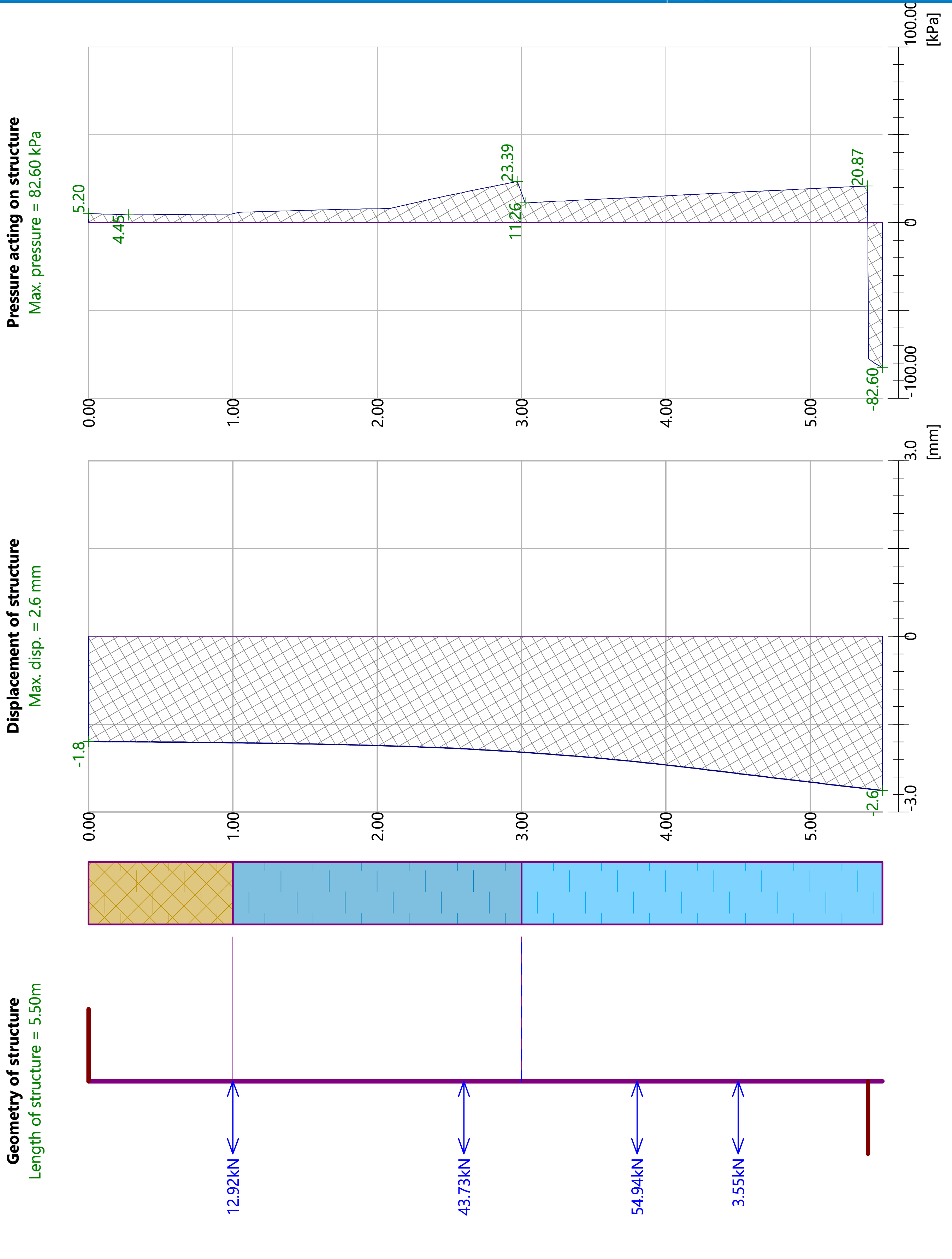
Figure 6.27 Relationship between damage category, deflection ratio and horizontal tensile strain (after Burland, 2001)

Appendix B: GEO5 Output

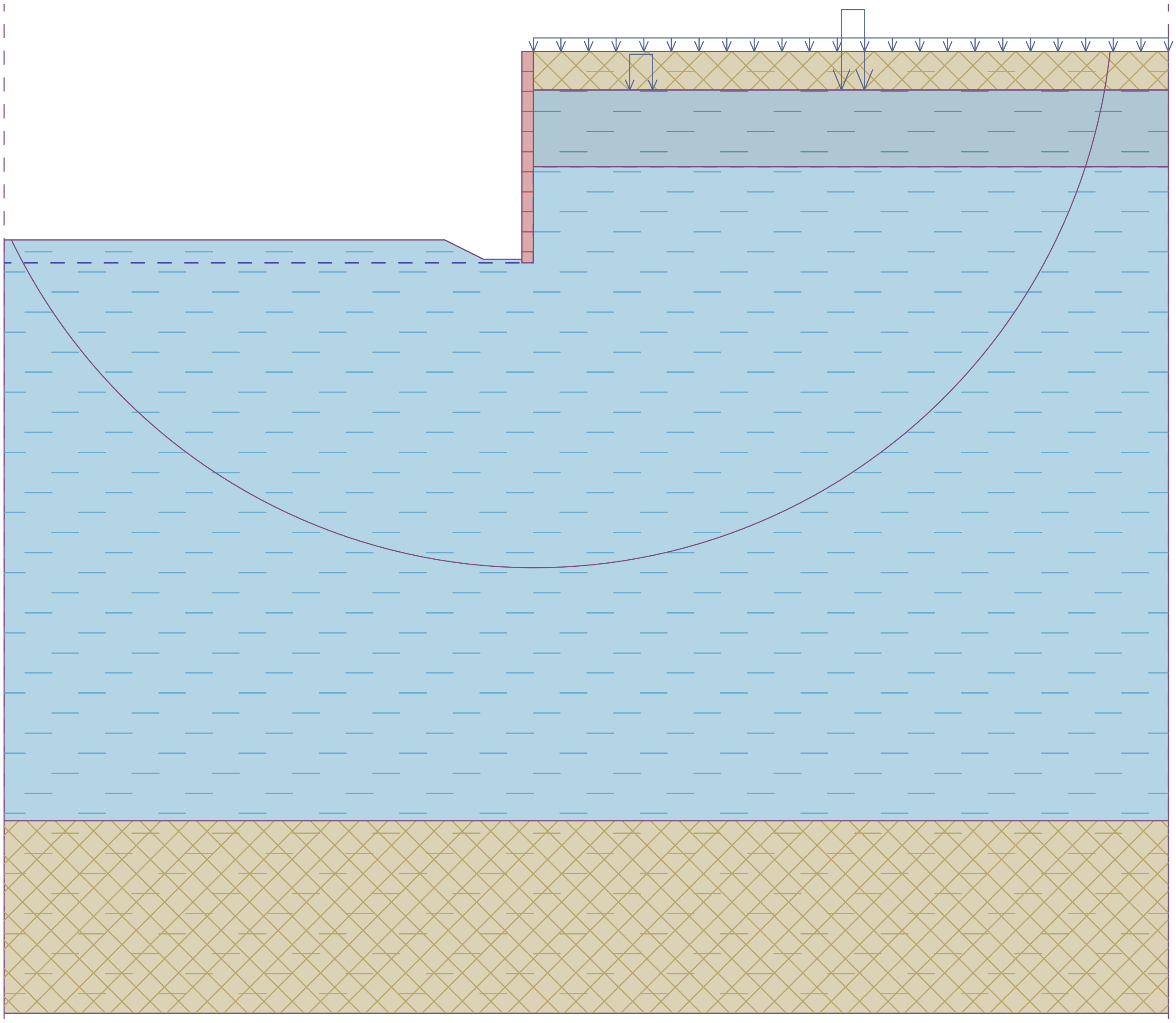
Name : | **Stage - analysis : 8 - 1**



Name :	Stage - analysis : 8 - 1



Name : | **Stage - analysis : 1 - 1**



The slip surface after optimization.

Slope stability verification (Bishop)

Combination 1

Sum of active forces : F_a = 1104.90 kN/m

Sum of passive forces : F_p = 1554.22 kN/m

Sliding moment : M_a = 16639.78 kNm/m

Resisting moment : M_p = 23406.52 kNm/m

Utilization : 71.1 %

Slope stability ACCEPTABLE

Combination 2

Sum of active forces : F_a = 341.19 kN/m

Sum of passive forces : F_p = 516.31 kN/m

Sliding moment : M_a = 3050.22 kNm/m

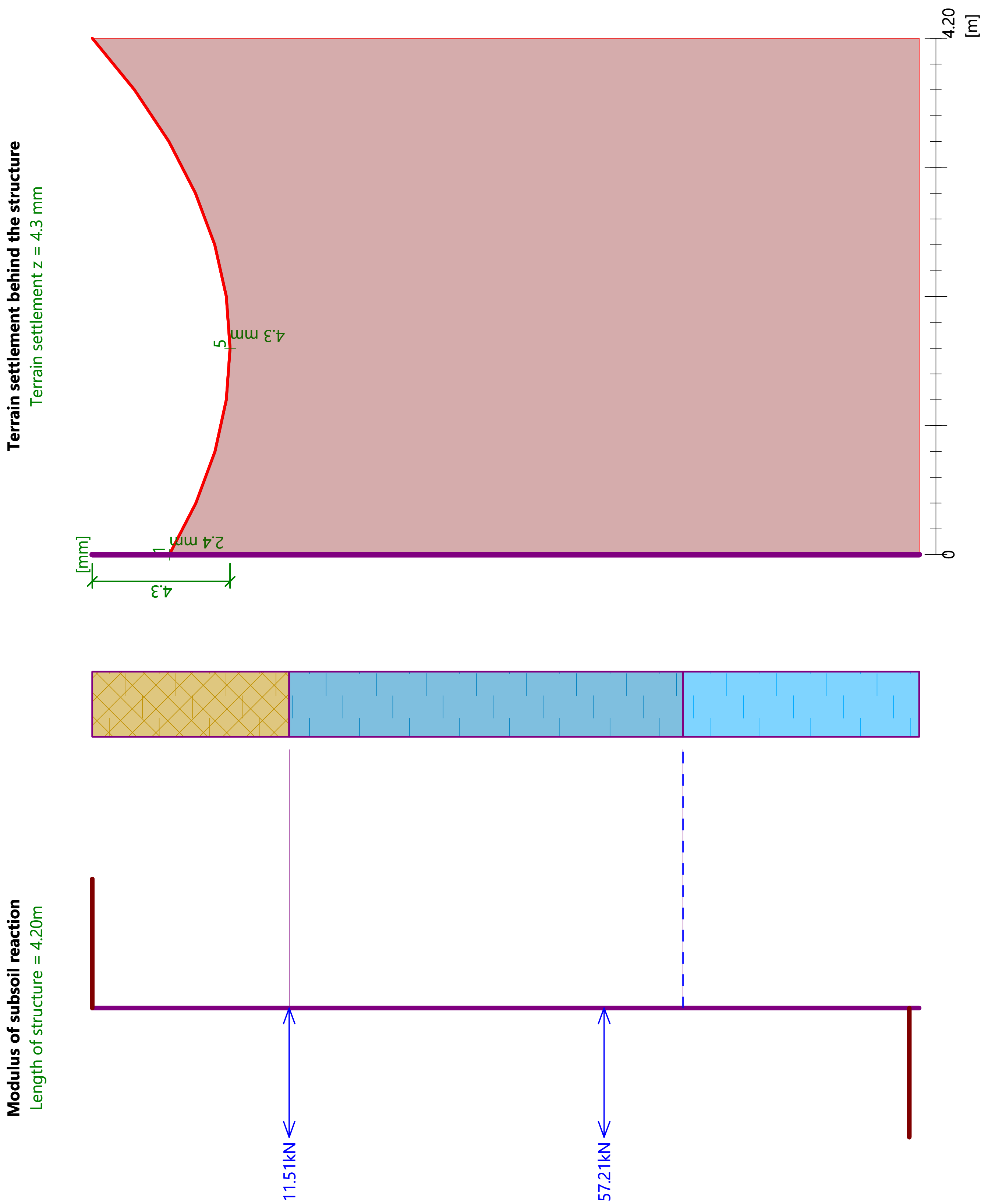
Resisting moment : M_p = 4615.77 kNm/m

Utilization : 66.1 %

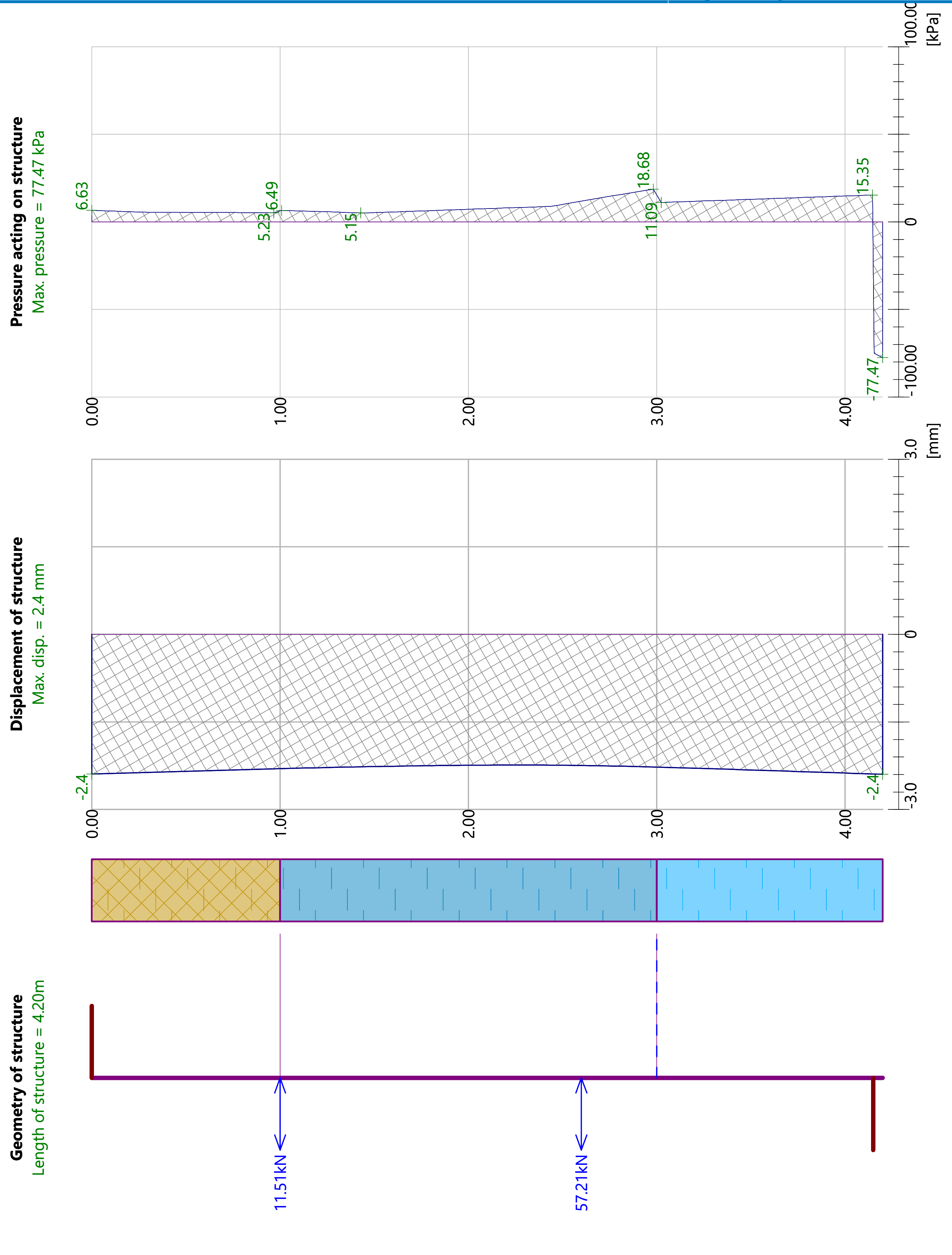
Slope stability ACCEPTABLE

Optimized slip surface for : Combination 1

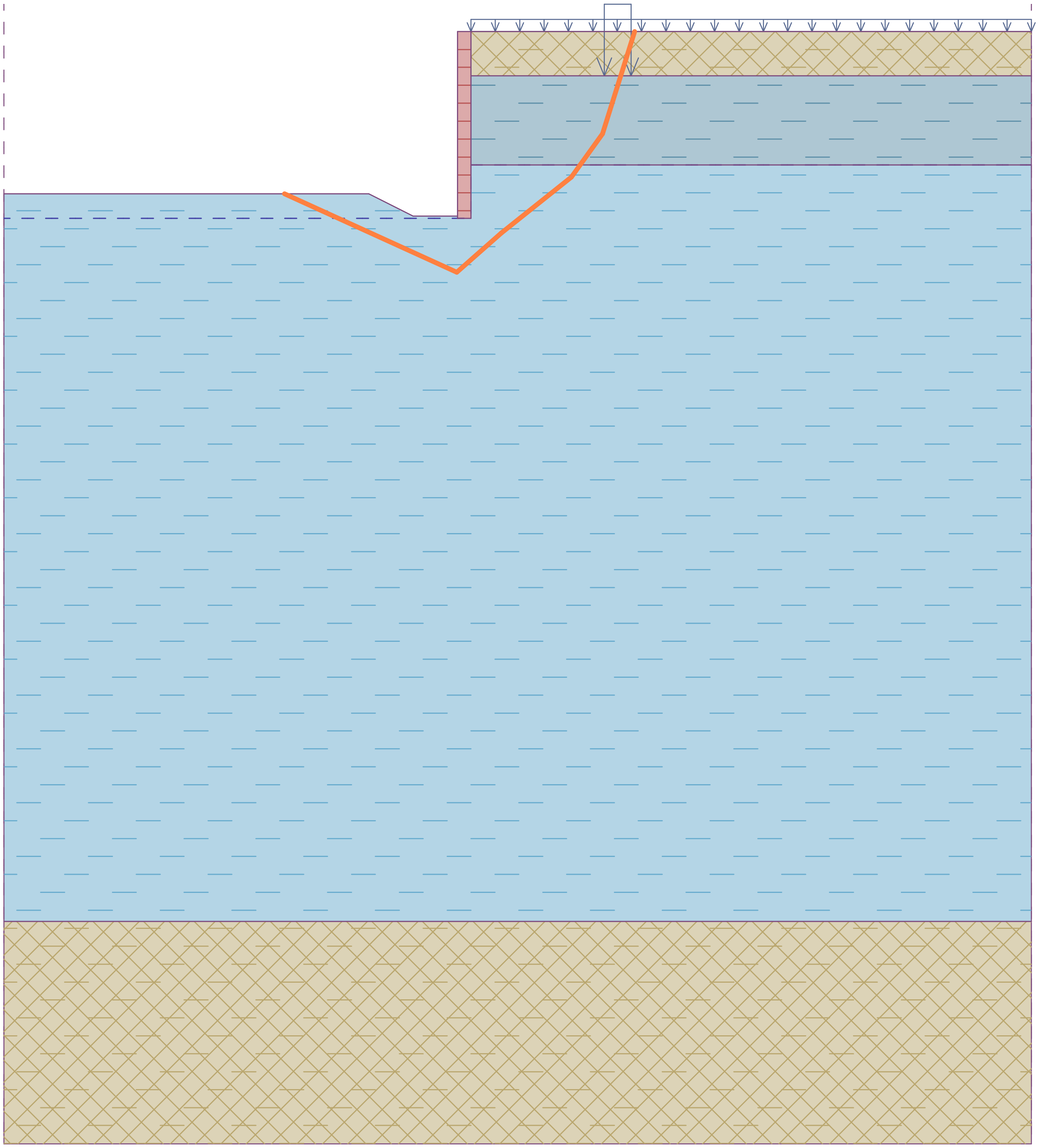
Name : **Stage - analysis : 5 - 1**



Name : | **Stage - analysis : 5 - 1**



Name : **Stage - analysis : 1 - 1**



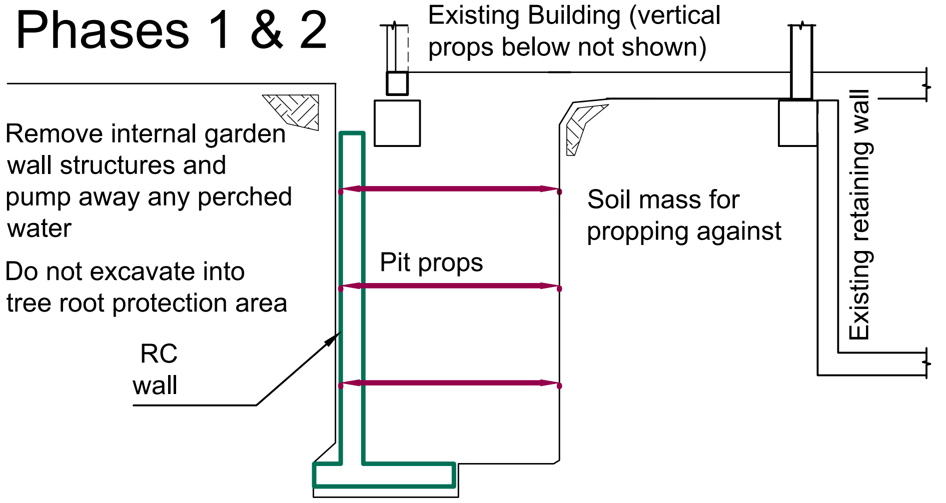
The slip surface after optimization.

Slope stability verification (Janbu)
Combination 1
Utilization : 71.2 %

Slope stability ACCEPTABLE
Combination 2
Utilization : 63.0 %

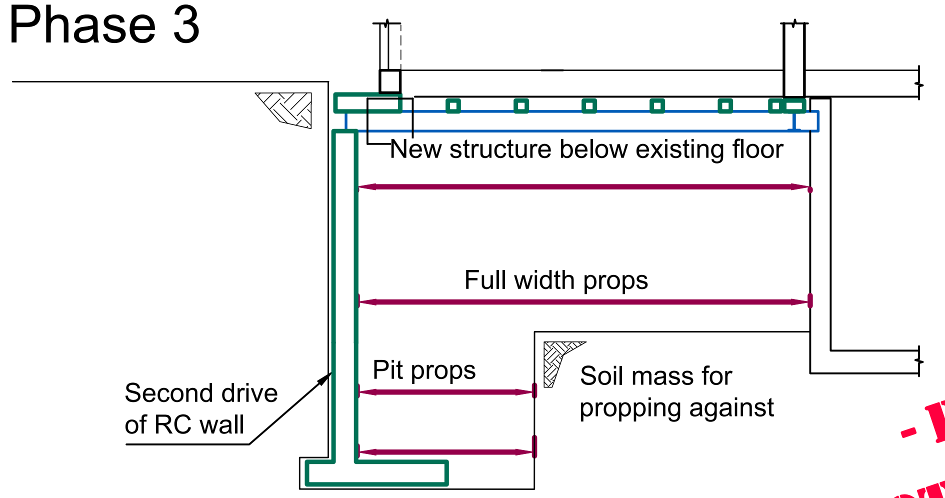
Slope stability ACCEPTABLE
Optimized slip surface for : Combination 1

Phases 1 & 2

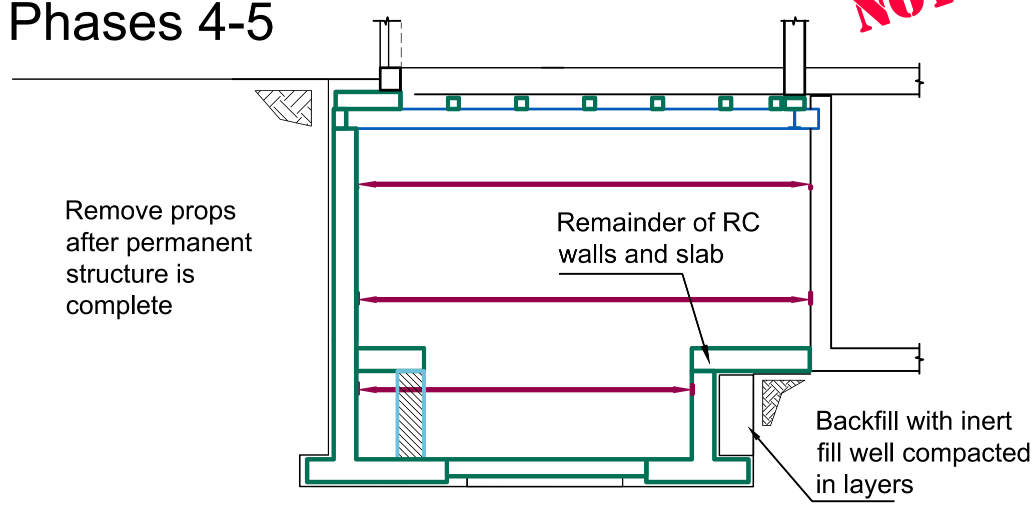


Detailed temporary works and related health and safety proceedures TBA at Detailed Design Stage

Phase 3



Phases 4-5



Typical section through rear basement showing construction sequence

(1:100)

PHASES 1-5

1. Excavate and construct retaining wall, closest to existing retaining wall (near 23 Merton Lane). Width of retaining wall not to exceed 1200mm. Install lateral and vertical props as excavation progresses.

2. Repeat above for remaining segments of retaining wall, excavating progressively in the direction towards No 25 West Hiill Park. Do not proceed to excavate for a wall segement until at least 48 hours after the previous adjacent wall has been cast

3. Remove remainder of soil mass to basement floor level. Install full width cross props as the excavation progresses then install roof structure. Repeat for adjacent segments.

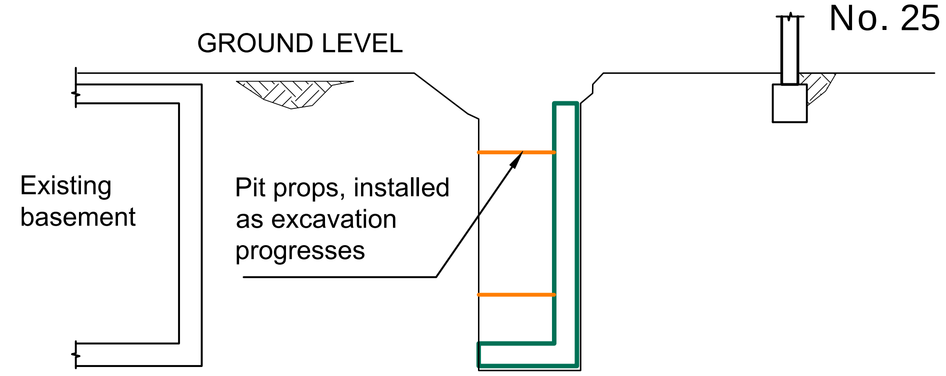
4. Complete the remainder of the retaining wall around the pool (closest to original building) in hit and miss segments. Prop as the excavation progresses.

5. Cast remainder of slab for pool and surrounding floor

- PLANNING ISSUE -
NOT FOR CONSTRUCTION

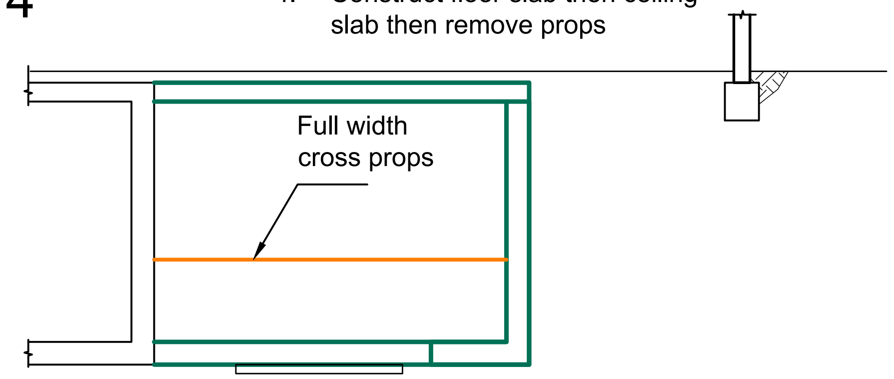
Phases 1 - 3

1. Remove existing paving slabs and pump away perched water, if present
2. Carefully excavate retaining walls in segments shown in plan
3. Install RC retaining wall and base. Excavate remaining soil in basement after retaining walls are cast. Install full width cross props.



Phase 4

4. Construct floor slab then ceiling slab then remove props



Typical section through front basement showing construction sequence

(1:100)

Rev	Date	Amendments
3	28.11.2019	Sequence for rear basement simplified
2	01.02.2019	For New Planning Submission
1	04.06.2017	For Planning submission
-	21.04.2017	First issue for comment

Job No.	Drawn	Scale
161206	GW	As shown @ A3
	Chk'd: -	
Dwg No.	**Rev.**	**Date**
TW-10	3	April 2017

Client: E. A. Konopleva

Project: 26 West Hill Park

Title : Temporary Works Scheme Design

Croft Structural Engineers
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020 8684 4744
www.croftse.co.uk

GROUND INVESTIGATION AND BASEMENT IMPACT ASSESSMENT REPORT

for the site at

26 WEST HILL PARK, LONDON BOROUGH OF CAMDEN, LONDON N6 6ND

on behalf of

CROFT STRUCTURAL ENGINEERS

Report Reference: GWPR2950/GIR/November 2019		Status: FINAL
Issue:	Prepared By:	Verified By:
V4.01 November 2019		
	Darina Jurovskaja BSc (Hons) MSc Geotechnical and Geo-Environmental Engineer	Francis Williams M.Geol. (Hons) C.Geol FGS CEnv AGS Director
File Reference: Ground and Water/Project Files/ GWPR2950 26 West Hill Park, Camden N6 6ND		

CONTENTS

FIGURES

APPENDICES

1.0 INTRODUCTION

1.1 General

Ground and Water Limited were instructed by Croft Structural Engineers, on the 7th January 2019, to review a Ground Investigation and undertake Basement Impact Assessment on a site at 26 West Hill Park, Camden, London N6 6ND. The scope of the investigation was detailed within the Ground and Water Limited fee proposal ref: GWQ4290.

1.2 Aims of the Investigation

The aim of the investigation was understood to be to supply the client and their designers with information regarding the ground conditions underlying the site to assist them in preparing an appropriate scheme for development.

The investigation was to be undertaken to provide parameters for the design of foundations by means of in-situ and laboratory geotechnical testing undertaken on soil samples recovered from trial holes.

The requirements of the Camden Planning Guidance: Basements (2018), and London Borough of Camden, Camden Geological, Hydrogeological and Hydrological Study, Guidance for Subterranean Development (November 2010) was reviewed with respect to this report.

A Desk Study and full-scale contamination assessment were not part of the remit of this report.

The techniques adopted for the investigation were chosen considering the anticipated ground conditions and development proposals on-site, and bearing in mind the nature of the site, limitations to site access and other logistical limitations.

1.3 Conditions and Limitations

This report has been prepared based on the terms, conditions and limitations outlined within Appendix A.

Total reliance has been placed on the trial hole logs and geotechnical testing (ref. Chelmer Consultancy Services, dated: May 2017). The report can be seen in Appendix B. No liability can be taken for these results or their shortcomings.

Revision of this report was based on correspondence from Campbell Reith to provide an independent Ground Movement Analysis (GMA). Updates to the report have been highlighted in yellow

2.0 SITE SETTING/GEOTECHNICAL DESK STUDY

2.1 Site Location

The site comprised a 783m^2 rectangular shaped plot of land, orientated in a north-west to south-east direction, located to the north-west of West Hill Park, north-eastern portion of the London Borough of Camden, north London.

The national grid reference for the centre of the site was approximately TQ 27903 86844. A site location plan is given within Figure 1. A plan showing the site area is given within Figure 2.

2.2 Site Description

The site was occupied a detached two-storey brick-built residential dwelling in the south-eastern portion of the site. The existing building on-site was understood to have a lower ground floor at a depth of 3.50m bgl with an existing swimming pool at an approximate depth of 5.00m bgl. The front comprised a detached garage in the southern corner and paved parking area. The rear of the property comprised soft landscaping with a mature tree. The site was noted to be slightly sloping towards north-west, whilst the residential property was noted to be constructed on the slope in the south-eastern portion of the site. An aerial view of the site is given within Figure 3.

An inspection of an onsite topographical survey, which can be seen in Figure 4, indicated that the northern corner of the rear garden had a maximum elevation of 91.75mAODm whilst the western corner had the lowest elevated of 88.70mAOD. The maximum elevation of the front of the property was noted to be 93.84mAOD and the lowest elevation was measured to be 87.68mAOD.

The residential property was constructed at an elevation of 91.20 – 91.23mAOD at the rear and at 91.28 – 91.33mAOD at the front. An existing sectional drawing and plan view of the site with elevations can be seen in Figures 5 and 6, respectively.

Merton Lane located to the north-west of the site was noted to be moderately sloping towards north-east at a 7^o angle.

The neighbouring properties did not appear to have an existing basement. A sectional plan showing the current differential ground floor depths can be seen in Figure 7.

2.3 Proposed Development

At the time of reporting, November 2019, the proposed development was understood to comprise the extension of an existing lower ground floor to the front and rear of the property, extending beyond the existing footprint of the property. The rear extension was proposed to accommodate a swimming pool at an approximate depth of 5.40m bgl, whilst the remainder of the lower ground floor was proposed to be constructed at a depth of 3.50m bgl. It was understood that the proposed development also involved the backfilling of the existing pool and re-location of it to the rear of the property.

A plan view and sectional view of the proposed development can be seen in Figure 8 and 9, respectively.

The proposed development fell within Geotechnical Design Category 2 in accordance with Eurocode 7.

Based on data supplied by the structural engineer, it was understood that the basement will be constructed based on loading bearing retaining wall underpins and a lower ground floor slab. The loads implied by one of the most heavily loaded walls were 64 – 98.6kN/m^2. Segmental wall base width varied between 1.50 and 2.20m. The remainder of the construction will comprise a semi-ground bearing concrete slab with self-weight of ~10kN/m^2.

The proposed development was understood not to involve any re-profiling of the site and its immediate environs. It is understood that no trees will be removed to facilitate the construction of the basement. However, a single mature tree was noted at the rear of the property along the north-western boundary and 2No. smaller trees were noted at the front of the property along the south-eastern boundary.

2.4 Site History

The object of this search was to report on the history of the site and its environs from available County Series, Ordnance Survey and Aerial Photography Maps dating from the late 19th Century to the present day and downloaded from Groundsure Environmental Insight. In the following sections dealing with individual maps, only features considered to have a potential impact on the site and usually within a notional 250 metre radius of the site boundaries are discussed. Any distances quoted for features remote from the site have been scaled from the maps and are only approximate. The north point and approximate extent of the site are indicated on each figure. The historical maps referred to are given within Appendix C. The implications of the map search are discussed later within this report. The historic map review can be seen tabulated below.

Table 1: Environmental Significance of Data From Historical Maps

Date	Scale	Site	Environs
1870	1:2,500	The area along the north-western boundary of the site comprised a terraced embanked land, whilst the central and south-eastern portions of the site comprised a wooden area.	The site environs comprised open densely vegetated area with roads and detached residential properties (Merton Lodge, Highgate Lodge, Holly Terrace and Holly Lodge) to the north, north-east and east of the site. The Highgate Ponds were noted ~190 – 300m west and north-east. No data for the southern portion of the map.
1896	1:1,056 1:2,500	The central and south-eastern portions of the site comprised an open area. The north-western portion of the site comprised a line of bushes and hedges. Remainder as previous map.	A residential property marked as Lodge with associated greenhouse buildings was noted ~70m south of the site. More residential housing was noted to the south-east of the site. Remainder as previous map.
1914 – 1915	1:2,500	As previous map.	Highgate Nurseries were noted between 210 – 270m north of the site. Remainder as previous map.
1936	1:2,500	As previous map.	A major residential development was noted to the north-west, north, north-east, east and south-east of the site. The Highgate Nurseries were no longer noted. Remainder as previous map.
1950 – 1951	1:1,250	The southern portion of the site was occupied by a wooden area with a part of a small outbuilding along the central part of the north-eastern boundary. Remainder as previous map.	A series of embanked land areas were noted ~70m north-east, and between ~95 – 250m south and south-west. Remainder as previous map.
1950 – 1952	1:1,250 1:2,500	As previous map.	As previous map.
1962	1:2,500	As previous map.	An embanked land ~90m south was replaced by a residential property. A Holly Court School with associated land was noted to replace an open air school ~10 – 100m north of the site. Remainder as previous map.
1967 – 1968	1:2,500	No data.	No data for western and central portions of the map. Remainder as previous map.
1970	1:2,500	No data.	No data for western and central portions of the map. Poor quality for the eastern portion of the map. Remainder as previous map.
1975 – 1979	1:1,250	The site was occupied by a detached residential property in the central portion of the site and a detached garage in the southern corner of the site. The Site Layout was similar to the current one.	A major residential development was noted to the north-east, east, south-east. An electricity substation was noted ~20m north-west of the site. Previously noted embanked land areas to the south were no longer noted. No data for the northern and eastern portions of the map. Remainder as previous map.
1979	1:1,250	As previous map.	No data for northern and eastern portions of the map. Remainder as previous map.
1991 (x2)	1:1,250	As previous map.	An electricity substation was noted ~210m south-east of the site.

2.5 Geology

The geology map of the British Geological Survey of Great Britain of the Royal Borough of Camden area (Sheet No. 256 North London) revealed the site to be situated on the Claygate Member of the London Clay Formation. Made Ground was noted ~185m north-east of the site.

Figure 3 of the Camden Geological, Hydrogeological and Hydrological Study (see Figure 10 of this report) and the BGS geology maps indicated Made Ground was noted ~185m north-east of the site.

Made Ground

Made Ground is shown in areas where material was deposited by man upon the natural ground surface. There are two main categories:

- Natural materials produced either as spoil from mineral extractions, or dug for the construction of various embankments and raised areas, including bunds for flood defenses.
- Waste in landfill sites; recycling of waste construction materials is leading to their increased usage in urban and industrial landscaping.

Claygate Member of the London Clay Formation

The Claygate Member of the London Clay Formation comprises alternating layers of clayey sand and sandy clays. The sands usually overlie the clays. The clays are typically brown to mauve mottled and are overconsolidated. The bed is transitional and overlays the undivided London Clay Formation. It has been used extensively for brick making.

A BGS borehole located ~40m north-west of the site revealed 1.40m of Topsoil over a stiff brown mottled sandy clay to a depth of 5.67m bgl and a stiff blue London Clay for the remainder of the borehole, a depth of 6.10m bgl.

2.6 Slope Stability and Subterranean Developments

The majority of the site was situated within an area where a natural or man-made slope of between 7^{o} and 10^{o} was present (Figure 16 Camden Geological, Hydrogeological and Hydrological Study, Figure 11 of this report). Sectional drawings of 26 West Hill Park Road revealed the street sloped between 5 – 7^{o} (Figure 4).

Figure 17 of the Camden Geological, Hydrogeological and Hydrological Study indicated the site was situated to the northern edge of the area prone to landslides (see Figure 12 of this report).

Figure 18 of the Camden Geological, Hydrogeological and Hydrological Study indicated that the site was located ~1.8km south-west of the Northern Underground and ~1.05km south of the London Overground (see Figure 13 of this report).

2.7 Hydrogeology and Hydrology

A study of the aquifer maps on the DEFRA website, and Figure 8 of the Camden Geological, Hydrogeological and Hydrological Study (see Figure 14 of this report), revealed the site was located on the **Secondary (A) Aquifer** comprising the Claygate member of the London Clay Formation. No designation was given for any superficial deposits due to their likely absence.

Superficial (Drift) deposits are permeable unconsolidated (loose) deposits, for example, sands and gravels. The bedrock is described as solid permeable formations e.g. sandstone, chalk and limestone.

Secondary (A) Aquifers consist of deposits with permeable layers capable of supporting water supplies at a local rather than strategic scale, and in some cases forming an important source of base flow to rivers. These are generally aquifers formerly classified as Minor Aquifers.

Examination of the DEFRA records and Figure 8 of the Camden Geological, Hydrogeological and Hydrological Study (see Figure 14 of this report) showed that the site did **not** fall within a Groundwater Source Protection Zone as classified in the Policy and Practice for the Protection of Groundwater.

In accordance with Figure 12 of the Camden Geological, Hydrogeological and Hydrological Study (see Figure 15 of this report), the nearest surface water feature was the Model Boating Pond associated with Hampstead Heath located ~130m south-west.

In accordance with Figure 11 of the Camden Geological, Hydrogeological and Hydrological Study (see Figure 16 of this report) a historical watercourse, associated with Highgate Ponds, was noted ~130m west of site. It should be noted that no significant surface water feature in historic maps were present within a close proximity of the site.

Figure 14 of the Camden Geological, Hydrogeological and Hydrological Study revealed the site was located within the Highgate Chain Catchment Ponds (see Figure 17 of this report).

From analysis of hydrogeological and topographical maps, groundwater was anticipated to be encountered at moderate to deep depth (5 – 8m below existing ground level (bgl)) and it was considered that the groundwater was flowing in a south-westerly direction in accordance with the local topography.

Examination of the Environment Agency records showed that the site was not situated within a floodplain or flood warning area. Figure 15 the Camden Geological, Hydrogeological and Hydrological Study revealed Spencer Rise did not experience any flooding in 1975 and 2002. The closest area that experienced flooding in 1975 was Swain's Lane located ~615m south-east of the site (see Figure 18 of this report).

A plan showing the location of the site with respect to Environment Agency Flood Maps can be seen in Figure 19.

Data from the Environment Agency website indicated West Hill Park was not at a risk of surface water flooding. However, Merton Lane, located immediately to the north-west of the site, was noted to have a low risk of surface water flooding with flood depth of <300mm and a velocity of over 0.25m/s. A plan showing the location of the site with respect to Environment Agency Surface Water Flooding Maps can be seen in Figure 20.

2.8 Radon

BRE 211 (2015) Map 5 of London, Sussex and West Kent revealed the site **was not** located within an area where mandatory protection measures against the ingress of Radon were required. The site **was not** located within an area where a risk assessment was required.

3.0 BASEMENT IMPACT ASSESSMENT

This stage should identify any areas of concern and therefore focus efforts on further investigation.

3.1 Stage 1: Screening

3.1.1 Subterranean (Groundwater) Screening Flowchart

Question 1a. Is the site located directly above an aquifer?

Yes. A study of the aquifer maps on the Environment Agency website, and Figure 8 of the Camden Geological, Hydrogeological and Hydrological Study, revealed the site to be located on a Secondary (A) Aquifer comprising the bedrock of the Claygate Member of the London Clay Formation (see Figure 11 of this report). **Take forward to scoping.**

Question 1b. Will the proposed basement extend beneath the water table surface?

Unlikely. From analysis of hydrogeological and topographical maps, groundwater was anticipated to be encountered at moderate depth (5 – 8m below existing ground level (bgl)). A maximum dig depth of 5.40m bgl is being considered. **However, Ground Investigation could be considered. Take forward to scoping.**

Question 2. Is the site within 100m of a watercourse, well (used/disused) or potential spring line?

No. No current watercourse, wells or potential spring lines are noted within a 100m radius of the site. In accordance with Figure 11 of the Camden Geological, Hydrogeological and Hydrological Study a historical watercourse, associated with Highgate Ponds, was noted ~130m west of site. It should be noted that no watercourse/wells or potential spring lines besides the ponds to the south-west were noted in historic map. **No further action.**

Question 3. Is the site within the catchment of the pond chains on Hampstead Heath?

Yes. Figure 14 of the Camden Geological, Hydrogeological and Hydrological Study revealed the site was located within the Highgate Chain Catchment Ponds (see Figure 17 of this report). **Take forward to scoping.**

Question 4. Will the proposed development result in a change in the proportion of hard surface/paved areas?

No. The basement will be excavated below the entire footprint of the existing building and underneath the existing patio at the rear of the property. **No further action.**

Question 5. As part of the 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)?

Marginally. At the time of reporting, November 2019, no significant change in the amount of surface water discharged into the ground was anticipated. The approximate change of hardstanding increase was measured to be under 10m^2. **Take forward to scoping.**

Question 6. Is the lowest point of the proposed excavation (allowing for any drainage and foundation space under the basement floor) close to or lower than the mean water level in any local pond or spring line?

No. As the basement floor was proposed to be founded at 5.40m bgl the lowest point of the proposed excavation will not be in close proximity or lower than the mean water level. The topographical web-map indicated that the site was located at an approximate depth of 90.00mAOD, whilst the area in the proximity of Hampstead Heath ponds was located at an approximate depth of 73mAOD. **No further action.**

3.1.2 Land Stability Screening Flowchart

Question 1. Does the existing site include slopes, natural or manmade, greater than 7 degrees (approximately 1 in 8)?

No. The site was marked to be on the south-west edge of an area where a natural or man-made slope of between 7^o and 10^o was present (Figure 16 Camden Geological, Hydrogeological and Hydrological Study, Figure 11 of this report). Sectional drawings of 26 West Hill Park Road revealed the street sloped between 5 – 7^o (Figure 4). **No further action.**

Question 2. Will the proposed re-profiling of landscaping at site change slopes at the property boundary to more than 7deg (approximately 1 in 8)?

No. No re-profiling of landscaping is anticipated to occur. **No further action.**

Question 3. Does the development neighbour land, including railway cuttings and the like, with a slope greater than 7deg (approximately 1 in 8)?

No. Sectional drawings of West Hill Park revealed the street sloped between 5 – 7^o (Figure 4). There were no railway cuttings in the immediate vicinity. **No further action.**

Question 4. Is the site within a wider hillside setting in which the general slope is greater than 7 degrees (approximately 1 in 8)?

No. Sectional drawings of 26 West Hill Park and Figure 16 of the Camden Geological, Hydrogeological and Hydrological Study revealed the street sloped between 5 – 7^o (Figure 4). It also should be noted that the site environs were sloping towards south-west towards the Highgate Ponds. **No further action.**

Question 5: Is the London Clay the shallowest strata at the site?

No. The geological map (sheet 256) indicates that the site is underlain the Claygate Member of the London Clay Formation. **No further action.**

Question 6: Will any tree/s 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? (Note that consent is required from LB Camden to undertake work to any tree/s protected by a Tree Protection Order or to tree/s in a Conservation Area if the tree is over certain dimensions).

No. No trees are to be removed in the excavation of the basement. **However, an arboricultural assessment on impact of the tree should be undertaken. No further action.**

Question 7: Is there a history of seasonal shrink-swell subsidence in the local area, and/or evidence of such effects at the site?

None known. However, the Claygate Member of the London Clay Formation is indicated as being present at the property, which has the potential for volume change. **Take forward to scoping.**

Question 8: Is the site within 100m of a watercourse or a potential spring line?

No. Figure 11 of the Camden Geological, Hydrogeological and Hydrological Study indicates no watercourses or potential spring lines are present in the vicinity of the site (see Figure 13 of this report). A historical watercourse, associated with Highgate Ponds, was noted ~130m west of the site. **No further action.**

Question 9: Is the site within an area of previously worked ground?

No. Examination of the geology maps revealed the site was not within an area of previously worked ground. However, Figure 3 of the Camden Geological, Hydrogeological and Hydrological Study (see Figure 10 of this report) and the BGS geology maps indicated Made Ground was noted ~185m north-east of the site. Also, it should be noted that there may be Made Ground associated with past construction activities (see Geotechnical Desk Study). **Take forward to scoping.**

Question 10: Is the site within an aquifer? If so, will the proposed basement extend beneath the water table such that dewatering may be required during construction?
Yes. The Claygate Member of the London Clay Formation is classified by the Environment Agency as a **Secondary A Aquifer** (permeable layers capable of supporting water supplies at a local rather than strategic scale, and in some cases forming an important source of base flow to rivers). **Take forward to scoping.**

Question 11: Is the site within 50m of the Hampstead Heath ponds?
No. The Highgate Ponds were located ~130m south-west of the site. **No further action.**

Question 12: Is the site within 5m of a highway or pedestrian right of way?
Yes. The nearest highway and pedestrian right of way was noted along the Merton Lane, located ~5m north-west of the site boundary, but ~18m north-west of the proposed basement. **No further action.**

Question 13: Will the proposed basement significantly increase the differential depth of foundations relative to neighbouring properties? Yes. It was understood that No. 25 West Hill Park was located up the slope to the north-east of the site and did not have a lower ground floor/semi basement. However, No. 23 and No. 25 Merton Lane, located to the west and south-west/south of the site, were noted to be constructed at the bottom of the slope with a ground floor at an approximate depth of 87mAOD. As the basement was proposed to be constructed at an approximate depth of 86mAOD, indicating a slight differential depth between the neighbouring properties to the south-west and south of the site. **Take forward to scoping.**

Question 14: Is the site over (or within the exclusion zone of) any tunnels, e.g. railway lines?
No. The site was located ~1.8km south-west of the Northern Underground and ~1.05km south of the London Overground. **No further action.**

3.2 Stage 2: Scoping

3.2.1 Conceptual Site Model & Matters of Concern

There are nine areas of concerns that the Screening process have highlighted.

1. **Perched water within the Made Ground or the Claygate Member of the London Clay Formation** – the basement may encounter perched water within any Made Ground or groundwater relating to the Claygate Member of the London Clay Formation, during construction. This is to be taken forward for further assessment to confirm depth of the saturated Aquifer;

2. **Soil Moisture** – There was potential for soil moisture content to affect the development. This is to be taken forward for further assessment;

3. **Claygate Member of the London Clay Formation/Shrink and Swell** – The basement is anticipated to be founded in the Claygate Member of the London Clay Formation. The soils are likely to have medium to high plasticity and volume change potential. The concrete mix design should take appropriate account of sulphate levels (testing to BRE Special Digest). Heave on removal of overburden pressure may be a risk;

4. **Potential for Made Ground;** Examination of the geology maps revealed the area was not in the vicinity of worked ground. However, there will be some Made Ground associated with past construction activities. Also, Figure 3 of the Camden Geological, Hydrogeological and Hydrological Study (see Figure 10 of this report) and the BGS geology maps indicated Made Ground was noted ~185m north-east of the site.

5. **Differential Foundation Depths** – It was understood that No. 25 West Hill Park was located up the slope to the north-east of the site and did not have a lower ground floor/semi basement. However, No. 23 and No. 25 Merton Lane, located to the west and south-west/south of the site, were noted to be constructed at the bottom of the slope with a ground floor at an approximate depth of 87mAOD. As the basement was proposed to be constructed at an approximate depth of 86mAOD, indicating a one-metre differential depth between the neighbouring properties to the south-west and south of the site. Therefore, it was likely a Ground Movement Analysis (GMA) will be required.

6. **Retaining Walls** should be appropriately designed**;**

7. **Tree and Bushes.** A mature tree was noted at the rear of the property along the north-western boundary also 2No. smaller mature trees were noted at the front of the property. Care should be taken to minimise root damage during construction works. Should bushes be removed there is potential for the soils to swell as a result which may affect this and neighbouring properties and this should be accounted for in design and further assessed. **An arboricultural assessment on impact of the tree should be undertaken. The basement is to be constructed outside of the tree canopy so thought to be low risk**;

8. **Surface Water/Drainage.** The lower ground floor was proposed to be constructed extending beyond the existing footprint of the property. The rear of the lower ground level was proposed to be constructed below the existing patio and also underneath the soft landscaping area in the western corner of the property. Therefore, there will be a slight increase in amounts of hardstanding onsite, changing the surface water drainage area. **To be carried forward into structural design;**

9. **Highgate Chain Catchment Zone.** The site was located within the Highgate Chain Catchment Zone, indicating that the basement development may have the potential to divert or displace groundwater. **To be carried forward into structural design.**

10. **Backfilling of the existing swimming pool.** Backfilling of the existing swimming pool may cause changes to the land stability as well as groundwater flow. **To be carried forward into structural design.**

A site-specific ground investigation should be undertaken to inform design. The scope of the investigation can be seen within Section 4 of this report. The results of the investigation are given within Sections 5 and 6 with the conclusions and recommendations provided within Section 7 of this report.

A ground movement assessment should be undertaken. The results of ground movement assessments undertaken on the neighbouring properties to the site can be seen within Section 7.7 of this report.

4.0 FIELDWORK

4.1 Scope of Works

Site works were undertaken on the 17th February and 2nd March 2017 by Chelmer Site Investigation Laboratories Limited (CSI). The site investigation comprised the drilling of 2No. Continuous Flight Auger Boreholes (BH1 – BH2) to a depth of 10.10m bgl and blgl (BH1 was excavated below a ground floor level and BH2 was excavated below a lower ground floor level). The site investigation also comprised the excavation of 2No. Trial Pit/Foundation Exposures (TP1 – TP2). TP1 was excavated to a depth of 0.65m blgl (below lower ground level) and TP2 was excavated to a depth of 0.66m blgl. Shear Vane Testing was undertaken on the samples undertaken from both boreholes, BH1 – BH2.

2No. combined ground-gas and groundwater wells were installed in BH1 and BH2 to 10.00m bgl. A detailed description of the constructed wells was not provided within the Ground Investigation Report by Chelmer Site Investigation Laboratories Limited (CSI) (ref. GENV/8522; dated May 2017).

The approximate locations of the trial holes can be seen within Appendix B.

Prior to commencing the ground investigation, a walkover survey was carried out to identify the presence of underground services and drainage. Where underground services/drainage were suspected and/or positively identified, exploratory positions were relocated away from these areas.

4.2 Sampling Procedures

Small disturbed samples were recovered from the trial holes at the depths shown on the trial hole records. Soil samples were generally retrieved from each change of strata and/or at specific areas of concern. Samples were also taken at approximately 0.5m intervals during broad homogenous soil horizons.

A selection of samples were despatched for geotechnical testing purposes.

5.0 ENCOUNTERED GROUND CONDITIONS

5.1 Soil Conditions

All exploratory holes were logged by Chelmer Site Investigation Laboratories Limited (CSI), generally in accordance with BS EN 14688 'Geotechnical Investigation and Testing – Identification and Classification of Soil'.

The ground conditions encountered within the trial holes drilled on the site generally conformed to that anticipated from examination of the geology map. A capping of Made Ground was noted to overlie the soils of the Claygate Member of the London Clay Formation.

The ground conditions encountered during the investigation are described in this section. For more complete information about the Made Ground and the Claygate Member of the London Clay Formation at particular points, reference must be made to the individual trial hole logs within Appendix B.

The trial hole location plan can be viewed in Figure 21.

For the purposes of discussion, the succession of conditions encountered in the trial holes in descending order can be summarised as follows:

Made Ground
Claygate Member of the London Clay Formation

Made Ground
A capping of the block paving and concrete was noted to a depth of 0.40m blgl in BH2 and a capping of paved ground to 0.04m blgl in TP1. Made Ground was encountered from ground level in BH1 and TP2 and underlying this capping in BH2 and TP1 to proved depths of between 0.45 – 0.90m bgl in BH1 – BH2 and TP1, and unproved depths of 0.66m blgl in TP2.

A 0.10m capping of concrete was noted in TP1 underlying the Made Ground, from a depth of 0.35m blgl.

Claygate Member of the London Clay Formation
Soils of the Claygate Member of the London Clay Formation were encountered underlying the Made Ground, from depths of 0.45 – 0.90m bgl in BH1 – BH2 and TP1 for the remainder of the trial holes, depths of between 0.65 – 10.00m bgl. The soils were generally described as an orange to brown sandy silty clay becoming a dark grey sandy silty clay with depth. The sand was fine grained.

Rare shell fragments were noted at 8.00m bgl in BH1.

Onsite engineer appraisals of the soils recovered from the Claygate of the London Clay Formation indicated the soils varied between firm to very stiff.

5.2 Foundation Exposures

A description of the foundation layout and ground conditions encountered within the hand dug trial pit/foundation exposures are given within this section of the report.

TP/FE1
Trial pit foundation exposure, TP1, was excavated adjacent to the south-western side of the single-

storey lower ground floor level. The exact location of the trial hole can be seen in Figure 21 and a section drawing of the foundation encountered can be seen in Figure 22.

The foundation exposure was measured from ground level.

The foundation layout encountered consisted of a brick wall to ground level. From ground level to a depth of 0.35m blgl a brick wall was noted resting upon the Claygate Member of the London Clay Formation. The soils were described as a firm thinly laminated grey silty clay and orange fine sand with rare sub-angular fine flint gravel. The ground conditions encountered directly surrounding the foundation are shown in Figure 22.

TP/FE2

Trial pit foundation exposure, TP2, was excavated adjacent to a front garden retaining wall in a raised flower bed in the area of the driveway. The exact location of the trial hole can be seen in Figure 21 and a section drawing of the foundation encountered during TP/FE2 can be seen in Figure 23.

The foundation exposure was measured from ground level.

The foundation layout encountered consisted of a brick wall to ground level. From ground level to a depth of 0.05m blgl a brick wall was noted which rested upon a brick foundation that stepped out by 0.10m and was 0.60m thick. The base of the brick foundation was noted at 0.65m blgl constructed on Made Ground, comprising a dark brown/black slightly sandy silt with rare brick fragments and rare sub-angular fine flint gravel. The sand was fine grained. The ground conditions encountered directly surrounding the foundation are shown in Figure 23.

5.3 Roots Encountered

Roots were noted to 2.00m bgl in BH1, to 0.50m blgl in BH2 and to a maximum depth of a trial pit of 0.66m blgl in TP2. No roots were noted in TP1.

It must be noted that the chance of determining actual depth of root penetration through a narrow diameter borehole is low. Roots may be found to greater depths 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.

5.4 Groundwater Conditions

Groundwater strikes were noted during the site investigation at a depth of 7.00m bgl in BH1 and at 6.80m bgl in BH2. Groundwater observations made during subsequent groundwater monitoring visits can be seen tabulated below.

Depth of Groundwater Strikes/Standing Groundwater Within Trial Holes		
Trial Hole	**Date**	**Depth of Groundwater (m bgl)**
BH1	15/03/2017	3.40
	22/03/2017	3.44
	12/04/2017	Not taken
BH2	15/03/2017	1.74
	22/03/2017	1.72
	12/04/2017	1.80

Changes in groundwater level occur for a number of reasons including seasonal effects and variations in drainage. Exact groundwater levels may only be determined through long term measurements from monitoring wells installed on-site. The investigation was undertaken in March and April 2018 when groundwater levels are likely to be falling from their annual maximum (highest elevation).

Isolated pockets of groundwater may be perched within any Made Ground found at other locations around the site.

5.5 Obstructions

No artificial or natural sub-surface obstructions were noted during excavation/drilling of the trial holes.

6.0 IN-SITU AND LABORATORY GEOTECHNICAL TESTING

6.1 In-situ Testing

Share Vane Testing was conducted by Chelmer Site Investigation Laboratories Limited (CSI) (ref. GENV/8522; dated May 2017). The testing was undertaken on disturbed samples taken during the drilling of 2No. Continuous Flight Auger Boreholes (BH1 – BH2) at 1.0m intervals for the entire depths of boreholes.

The cohesive soils of the Claygate Member of the London Clay Formation were tested to have medium to high undrained shear strength (70 – >120kPa).

6.2 Laboratory Geotechnical Testing

A programme of geotechnical laboratory testing, scheduled and carried out by Chelmer Site Investigation Laboratories Limited (CSI) and Nicholls Colton Group, was undertaken on samples recovered from the Claygate Member of the London Clay Formation. The results of the tests are presented in Appendix B.

The test procedures used were generally in accordance with the methods described in BS1377:1990.

Details of the specific tests used in each case are given below:

Standard Methodology for Laboratory Geotechnical Testing		
Test	**Standard**	**Number of Tests**
Atterberg Limit Tests	BS1377:1990:Part 2:Clauses 3.2, 4.3 & 5	12
Particle Size Distribution	BS1377:1990:Part 2:Clause 9	3
Water Soluble Sulphate & pH	BS1377:1990:Part 3:Clause 5	1
BRE Suite	BRE Special Digest 1 “Concrete in Aggressive Ground (BRE, 2005).	2

6.2.1 Atterberg Limit Tests

A précis of Atterberg Limit Tests undertaken on eleven samples of the Claygate Member of the London Clay Formation can be seen tabulated below.

Atterberg Limit Tests Results Summary							
Stratum/Depth	**Moisture Content (%)**	**Passing 425 µm sieve (%)**	**Modified PI (%)**	**Soil Class**	**Consistency Index (Ic)**	**Volume Change Potential**	
						BRE	**NHBC**
Claygate Member of the London Clay Formation	19 – 34	<95	33 – 43	CH	Stiff to Very Stiff	Medium	Medium

NB: NP – Non-plastic
BRE Volume Change Potential refers to BRE Digest 240 (based on Atterberg results)
Soil Classification based on British Soil Classification System.
Consistency Index (Ic) based on BS EN IS0 14688-2:2004.

6.2.2 Comparison of Soil’s Moisture Content with Index Properties

6.2.2.1 Liquidity Index Analyses

The results of the Atterberg Limit tests undertaken on eleven samples of the Claygate Member of the London Clay Formation were analysed to determine the

Liquidity Index of the samples. This gives an indication as to whether the samples recovered showed a moisture deficit and their degree of consolidation. The results are tabulated overpage.

The test results are presented within Appendix B.

Liquidity Index Calculations Summary					
Stratum/Trial Hole/Depth	**Moisture Content (%)**	**Plastic Limit (%)**	**Modified Plasticity Index (%)**	**Liquidity Index**	**Result**
Claygate of the London Clay Formation BH1/1.50m bgl	23	18	39.00	0.13	Heavily Overconsolidated
Claygate of the London Clay Formation BH1/3.00m bgl	20	18	34.00	0.06	Heavily Overconsolidated
Claygate of the London Clay Formation BH1/3.50m bgl	21	17	35.00	0.11	Heavily Overconsolidated
Claygate of the London Clay Formation BH1/4.50m bgl	26	17	33.00	0.27	Overconsolidated
Claygate of the London Clay Formation BH1/8.00m bgl	31	19	35.00	0.34	Overconsolidated
Claygate of the London Clay Formation BH2/1.50m bgl	19	17	33.00	0.06	Heavily Overconsolidated
Claygate of the London Clay Formation BH2/2.50m bgl	27	16	33.00	0.33	Overconsolidated
Claygate of the London Clay Formation BH2/3.50m bgl	28	17	34.00	0.32	Overconsolidated
Claygate of the London Clay Formation BH2/4.50m bgl	27	18	34.00	0.26	Overconsolidated
Claygate of the London Clay Formation BH2/5.50m bgl	28	17	36.00	0.31	Overconsolidated
Claygate of the London Clay Formation BH2/10.00m bgl	29	20	41.00	0.22	Overconsolidated
Claygate of the London Clay Formation TP1/0.40m bgl	34	20	43.00	0.33	Overconsolidated

Liquidity Index testing revealed no evidence for moisture deficit within the overconsolidated to heavily overconsolidated samples of the Claygate Member of the London Clay Formation tested.

6.2.2.2 Liquid Limit

A comparison of the soil moisture content and the liquid limit can be seen tabulated overpage.

Moisture Content vs. Liquid Limit				
Strata/Trial Hole/Depth/Soil Description	**Moisture Content (MC) (%)**	**Liquid Limit (LL) (%)**	**40% Liquid Limit (LL)**	**Result**
Claygate of the London Clay Formation BH1/1.50m bgl	23	59	23.6	**MC < 0.4 x LL (Potential Significant Moisture Deficit)**
Claygate of the London Clay Formation BH1/3.00m bgl	20	54	21.6	**MC < 0.4 x LL (Potential Significant Moisture Deficit)**
Claygate of the London Clay Formation BH1/3.50m bgl	21	54	21.6	**MC < 0.4 x LL (Potential Significant Moisture Deficit)**
Claygate of the London Clay Formation BH1/4.50m bgl	26	52	20.8	MC > 0.4 x LL (No Significant Moisture Deficit)
Claygate of the London Clay Formation BH1/8.00m bgl	31	56	22.4	MC > 0.4 x LL (No Significant Moisture Deficit)
Claygate of the London Clay Formation BH2/1.50m bgl	19	52	20.8	**MC < 0.4 x LL (Potential Significant Moisture Deficit)**
Claygate of the London Clay Formation BH2/2.50m bgl	27	51	20.4	MC > 0.4 x LL (No Significant Moisture Deficit)
Claygate of the London Clay Formation BH2/3.50m bgl	28	53	21.2	MC > 0.4 x LL (No Significant Moisture Deficit)
Claygate of the London Clay Formation BH2/4.50m bgl	27	53	21.2	MC > 0.4 x LL (No Significant Moisture Deficit)
Claygate of the London Clay Formation BH2/5.50m bgl	28	55	22	MC > 0.4 x LL (No Significant Moisture Deficit)
Claygate of the London Clay Formation BH2/10.00m bgl	29	63	25.2	MC > 0.4 x LL (No Significant Moisture Deficit)
Claygate of the London Clay Formation TP1/0.40m bgl	34	65	26	MC > 0.4 x LL (No Significant Moisture Deficit)

The results in the table above indicated that a potential significant moisture deficit was present within four sample of the Claygate Member of the London Clay Formation tested (BH1/1.50m, BH1/3.00m, BH1/3.50m and BH2/1.50m bgl). The samples were generally described as a firm to stiff orange brown to brown sandy silty clay. The sand was fine grained. Roots were noted to a depth of 2.00m bgl in BH1 and to 0.50m bgl in BH2. Therefore, the apparent potential significant moisture deficit within BH1 was most likely due to the water demand from trees within the top 2.00m and presence of silt and sand below to root penetration zone. The apparent potential significant moisture deficit within BH2 was most likely due to the presence of sand and silt as well as lithology of soils.

The results in the table above indicate that the remaining eight samples of the overconsolidated Claygate Member of the London Clay Formation tested showed no evidence of a significant moisture deficit.

6.2.3 Particle Size Distribution (PSD) Tests

The results of PSD testing undertaken on three granular samples of the Claygate Member of the London Clay Formation tabulated below.

PSD Test Results Summary			
Trial Hole/Depth/Soil Description	Volume Change Potential Range		Passing 63µm sieve Range (%)
	BRE	NHBC	
Claygate Member of the London Clay Formation BH1/2.50m bgl (Sandy silty CLAY with rare coarse gravel)	**Yes**	**Yes**	58
Claygate Member of the London Clay Formation BH1/5.00m bgl (Sandy silty CLAY).	**Yes**	**Yes**	64
Claygate Member of the London Clay Formation BH2/4.00m bgl (Sandy silty CLAY).	**Yes**	**Yes**	65

NB Volume Change Potential refers to BRE Digest 240 (based on Grading test results).
Shrinkability refers to NHBC Standards Chapter 4.2 (based on Grading test results).

Volume Change Potential – BRE 240 states that a soil has a volume change potential when the clay fraction exceeds 15%. Only the silt and clay combined fraction are determined by sieving therefore the volume change potential is estimated from the percentage passing the 63µm sieve.

NHBC Standards Chapter 4.2 states that a soil is shrinkable if the percentage of silt and clay passing the 63µm sieve is greater than 35% and the Plasticity Index is greater than 10%.

6.2.4 BRE Special Digest 1

In accordance with BRE Special Digest 1 'Concrete in Aggressive Ground' (BRE, 2005) two samples of the Made Ground (BH1/0.50m and BH2/0.50m bgl) and five samples of the Claygate Member of the London Clay Formation (BH1/2.00, BH1/4.50m, BH1/10.00m, BH2/3.00m and BH2/8.00m bgl) were scheduled for laboratory analysis to determine parameters for concrete specification.

The results are given within Appendix B and a summary is tabulated below.

Summary of Results of BRE Special Digest Testing			
Determinand	Unit	Minimum	Maximum
pH	-	7.6	10.8
Water-soluble Sulphate	mg/kg	83	990
Acid-Soluble Sulphate	%	0.04	0.33
Total Sulphur	%	0.01	0.78

6.3 Chemical Laboratory Testing – Human Health Risk Assessment

A programme of chemical laboratory testing, scheduled by Chelmer Site Investigation Laboratories Limited (CSI), and carried out by Nicholls Colton Group, was undertaken on 2No. samples of Made Ground (BH1/0.25m and BH2/0.50m bgl).

A Desk Study and full-scale contamination assessment were not part of the report (ref. GENV/8522; dated May 2017). However, two soil sample was sent off for analysis for a broad range of contaminants in accordance with DEFRA/CLEA methodologies.

The site comprised a rectangular shaped plot of land, 805m^2 in area with two sampling locations, given an unknown hotspot shape, the sampling density means that a hotspot with an area of approximately 301.88m^2 and a radius of approximately 9.80m would be encountered (CLR 4).

Soil sampling depths were chosen to reflect the receptors of concern, human health, and typically comprised a surface or near surface sample and then at approximately 0.50m depth increments thereafter, extending into the underlying natural soils. The receptors relevant to the sampling depths can be seen overpage:

Near surface samples	Direct ingestion, dermal contact and dust inhalation. Protection of end-users and maintenance workers e.g. Landscape Gardeners. Protection of shallow rooted plants. Perched Water/Surface Water Run-off Protection of groundwater/controlled waters
>0.5m below ground level	Protection of deep rooted plants. Perched Water/Surface Water Run-off Protection of groundwater/controlled waters

The depth of soil sampling can be seen within the trial hole logs presented in Appendix B.

The analysis suite is presented below and comprised:

- Semi-metals and heavy metals incl. Arsenic, Cadmium, Chromium (incl. Hexavalent Chromium), Copper, Lead, Mercury, Nickel, Selenium, Vanadium, Zinc (BH1/0.25m and BH2/0.50m bgl);
- Asbestos screen (BH1/0.25m and BH2/0.50m bgl);
- Polycyclic Aromatic Hydrocarbons (PAH's) incl. Naphthalene, Acenaphthylene, Acenaphthene, Fluorene, Phenanthrene, Anthracene, Fluoranthene, Pyrene, Benzo(a)anthracene, Chrysene, Benzo(b)fluoranthene, Benzo(k)fluoranthene, Benzo(a)pyrene, Indeno(1,2,3-cd)pyrene, Dibenz(a,h)anthracene, Benzo(ghi)perylene: (BH1/0.25m and BH2/0.50m bgl);
- Fuel Oils – Speciated TPH including full aliphatic/aromatic split (BH1/0.25m and BH2/0.50m bgl).

The chemical laboratory results are presented in Appendix B.

6.3.1 Soil Assessment Criteria

The derivation of Soil Assessment Criteria used within this report can be seen within Appendix D.

6.3.2 Determination of Representative Contamination Concentration

At the time of reporting, November 2019, the proposed development was understood to comprise the extension of an existing lower ground floor to the front and rear of the property, extending beyond the existing footprint of the property. The rear extension was proposed to accommodate a swimming pool at an approximate depth of 5.40m bgl, whilst the remainder of the lower ground floor was proposed to be constructed at a depth of 3.500m bgl. It was understood that the proposed development also involved the backfilling of the existing pool and re-location of it to the rear of the property.

Therefore, the results of the chemical laboratory testing were compared to the LQM/CIEH Suitable 4 Use Levels (S4UL), for a **'Residential with homegrown produce'** land-use scenario, as this was considered the most appropriate land-use scenario.

Where no LQM/CIEH S4UL/C4SL LLTC was available for a particular determinant then preliminary reference was made to the laboratory detection limit of the determinant. If a positive concentration was noted then further risk assessment was undertaken.

Where a contaminant of concern's LQM/CIEH S4UL/C4SL LLTC varies according to the Soil's Organic Matter (SOM), it is generally recommended to identify the SOM value to derive the appropriate SGV/GAC. As the SOM value was not available, a worst-case scenario of 1.0% SOM value was used for this report.

The results of the comparison of the representative contaminant concentrations are presented in the table below.

Soil Guideline Values and General Acceptance Criteria Results	
Substance	Sample Location Where available LQM/CIEH S4UL/, CSL4 LLTC or GAC were exceeded for relevant land-use scenario "Residential with Homegrown Produce" Land-Use Scenario
Arsenic	None
Boron	None
Cadmium	None
Chromium (III)	None
Hexavalent Chromium (VI)	None
Lead	None
Mercury (Elemental)	None
Nickel	None
Selenium	None
Vanadium	None
Copper	None
Zinc	None
Boron	None
Cyanide (Total)	None
Phenol	None
TPH C5 – C6 (aliphatic)	None
TPH C6 – C8 (aliphatic)	None
TPH C8 - C10 (aliphatic)	None
TPH C10 - C12 (aliphatic)	None
TPH C12 - C16 (aliphatic)	None
TPH C16 - C21 (aliphatic)	None
TPH C21 - C34 (aliphatic)	None
TPH C5 – C7 (aromatic)	None
TPH C7 – C8 (aromatic)	None
TPH C8 - C10 (aromatic)	None
TPH C10 - C12 (aromatic)	None
TPH C12 - C16 (aromatic)	None
TPH C16 - C21 (aromatic)	None
TPH C21 - C35 (aromatic)	None
Naphthalene	None
Acenapthylene	None
Acenapthene	None
Fluorene	None
Phenanthrene	None
Anthracene	None
Fluoranthene	None
Pyrene	None
Benzo(a)anthracene	None
Chrysene	None
Benzo(b)fluoranthene	None
Benzo(k)fluoranthene	None
Indeno(1,2,3-cd)pyrene	None
Benzo(ghi)perylene	None
Benzo(a)pyrene	None
Dibenz(a,h)anthracene	None
Benzene	None
Toluene	None
Ethylbenzene	None
Xylene (o, m & p)	None
MTBE	None
Asbestos Screen	None

Chemical laboratory testing revealed no elevated levels of determinants in excess of the **'Residential with home-grown use (RwHP)'** land use scenario adopted.

7.0 ENGINEERING CONSIDERATIONS

7.1 Soil Characteristics and Geotechnical Parameters

Based on the results of the intrusive investigation and geotechnical laboratory testing the following interpretations have been made with respect to engineering considerations.

- Made Ground was noted to proved depths of between 0.45 – 0.90m bgl in BH1 – BH2 and TP1 and unproved depth of 0.66m blgl in TP2.

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

  Made Ground may be found to deeper depth at other locations on the site, especially close to former structures/foundations and service runs.

- Soils of the Claygate Member of the London Clay Formation were encountered underlying the Made Ground, from depths of 0.45 – 0.90m bgl in BH1 – BH2 and TP1 for the remainder of the trial holes, depths of between 0.65 – 10.00m bgl. The soils were generally described as an orange to brown sandy silty clay becoming a dark grey sandy silty clay with depth. The sand was fine grained.

  Rare shell fragments were noted at 8.00m bgl in BH1.

  Onsite engineer appraisals of the soils recovered from the Claygate of the London Clay Formation indicated the soils varied between firm to very stiff.

  The cohesive soils of the Claygate Member of the London Clay Formation were shown to have a **medium** potential for volume change in accordance both BRE240 and NHBC Standards Chapter 4.2.

  Shear Vane Testing was carried out on disturbed samples recovered from BH1 and BH2. The cohesive soils of the Claygate Member of the London Clay Formation were tested to have medium to high undrained shear strength (70 – >120kPa).

  The results of geotechnical testing revealed that potential significant moisture deficit was present within four sample of the Claygate Member of the London Clay Formation tested (BH1/1.50m, BH1/3.00m, BH1/3.50m and BH2/1.50m bgl). The samples were generally described as a firm to stiff orange brown to brown sandy silty clay. The sand was fine grained. Roots were noted to a depth of 2.00m bgl in BH1 and to 0.50m bgl in BH2. Therefore, the apparent potential significant moisture deficit within BH1 was most likely due to the water demand from trees within the top 2.00m and presence of silt and sand below to root penetration zone. The apparent potential significant moisture deficit within BH2 was most likely due to the presence of sand and silt as well as lithology of soils.

  The cohesive soils of the Claygate Member of the London Clay Formation were considered to be a suitable stratum for the proposed traditional strip or mat foundations associated with the basement. The settlements induced on loading were likely to be moderate.

The final design of foundations will need to take into account the volume change potential of the soil, the depth of root penetration and/or moisture deficit and the likely serviceability and settlement requirements of the proposed structure. These parameters for design are discussed in the next section of this report.

- Groundwater strikes were noted during the site investigation at a depth of 7.00m bgl in BH1 and at 6.80m bgl in BH2. Groundwater observations made during subsequent groundwater monitoring visits can be seen tabulated below.

Depth of Groundwater Strikes/Standing Groundwater Within Trial Holes		
Trial Hole	**Date**	**Depth of Groundwater (m bgl)**
BH1	15/03/2017	3.40
	22/03/2017	3.44
	12/04/2017	Not taken
BH2	15/03/2017	1.74
	22/03/2017	1.72
	12/04/2017	1.80

It should be noted that the water noted during the subsequent visits was most likely perched water within silty/sandy bands of the Made Ground and Claygate Member of the London Clay Formation. The levels do not represent the actual saturated aquifer.

It was considered that the return monitoring visits encountered perched water within Made Ground and silt and sand pockets of the Claygate Member of the London Clay Formation.

- Roots were noted to 2.00m bgl in BH1, to 0.50m bgl in BH2 and to a maximum depth of a trial pit of 0.66m bgl in TP2. No roots were noted in TP1.

7.2 Basement Foundations

At the time of reporting, November 2019, the proposed development was understood to comprise the extension of an existing lower ground floor to the front and rear of the property, extending beyond the existing footprint of the property. The rear extension was proposed to accommodate a swimming pool at an approximate depth of 5.40m bgl, whilst the remainder of the lower ground floor was proposed to be constructed at a depth of 3.500m bgl. It was understood that the proposed development also involved the backfilling of the existing pool and re-location of it to the rear of the property.

The proposed development fell within Geotechnical Design Category 2 in accordance with Eurocode 7.

Based on data supplied by the structural engineer, it was understood that the basement will be constructed based on loading bearing retaining wall underpins and a lower ground floor slab. Segmental wall base width varied between 1.50m at 3.50m bgl and 2.20m at 5.40m bgl. The remainder of the construction will comprise a suspended concrete slab with self-weight of ~10kN/m^2.

Foundations should be designed in accordance with soils of **medium volume change potential** in accordance with BRE Digest 240 and NHBC Chapter 4.2.

Given the cohesive nature of the shallow deposits foundations must **not** be placed within cohesive root penetrated and/or desiccated soils and the influence of the trees surrounding the site must be taken into account (NHBC Standards Chapter 4.2). It is recommended that foundations are taken at least 300mm into non-root penetrated strata.

Where trees are mentioned in the text this means existing trees, recently removed trees (approximately 15 years to full recovery on cohesive soils) and those planned as part of the site landscaping. Should trees be removed from the footprint of the proposed building then an alternative foundation system, such as piles or isolated pads should be considered.

Roots were noted to 2.00m bgl in BH1, to 0.50m bgl in BH2 and to a maximum depth of a trial pit of 0.66m bgl in TP2. No roots were noted in TP1. Made Ground was noted to proved depths of between 0.45 – 0.90m bgl in BH1 – BH2 and TP1 and unproved depth of 0.66m blgl in TP2.

Given the above and the depth of roots noted in the boreholes, it was concluded that a founding depth of 3.50m and 5.40m bgl was considered suitable for the proposed basement.

The formation level for the extension must be carefully inspected for the presence of fresh/live roots. Should live roots be noted at formation level then the formation level should be extended at least 300mm into non-root penetrated soils.

The following bearing capacities could be adopted for 5.0m long by 0.75m, 1.00m and 2.20m wide retaining wall strip footings, 1.5m square pads at depths of 3.50m and 5.40m bgl.

The bearing capacities were determined based on the Shear Vane Testing carried out on disturbed samples of the Claygate Member of the London Clay Formation.

The geotechnical parameters, tabulated below, have been used when modelling bearing capacities for spread foundation design, and are deemed conservative.

The effective width of a basement wall (5.00m by 1.50m at 3.50m bgl and 5.00m by 2.20m at 5.40m bgl) has been provided by the Structural Engineer.

The following table was derived based on the results from in-situ testing, carried out on site and the use of geotechnical parameters / relationships based on literature, using Geostru Software 'Dynamic Probing'.

Layer (m bgl)	Unit Volume Weight (kN/m^3)	Saturated Unit Volume Weight (kN/m^3)	Undrained Cohesion (kN/m^2)
0.00 – 0.50	14.51	18.04	4.9
0.50 – 1.00	18.63	18.73	39.13
1.00 – 1.50	20.30	22.36	75.71
1.50 – 2.00	20.40	22.46	83.16
2.00 – 2.40	20.59	22.65	92.87
2.50 – 3.00	20.69	20.99	107.58
3.00 – 3.50	20.69	21.28	112.38
3.50 – 4.00	20.69	21.48	117.39
4.00 – 4.50	20.69	21.48	117.39
4.50 – 5.00	20.69	21.48	117.39
5.00 – 5.50	20.69	21.48	117.39
5.50 – 6.00	20.69	21.48	117.39
6.00 – 6.50	20.69	21.48	117.39
6.50 – 7.00	20.69	21.48	117.39
7.00 – 7.50	20.69	21.48	117.39
7.50 – 8.00	20.69	21.48	117.39

The Bearing Capacities were derived using the software Geostru 'LoadCap', based on a modelled borehole, based on BH1 and BH2. The Limit State refers to the ultimate bearing capacity of the soil based on bearing capacity calculations and a factor of safety (3) incorporated. The allowable bearing capacity should be taken as the combination of the tables with avoiding shear failure and not exceeding a settlement of 25mm.

The software uses commonly derived equations for bearing capacity to derive figures based on the ground model formed, using layer weighted averages. The bearing capacities are calculated via several methods (e.g. Terzaghi, etc) and the most conservative outputs are selected as proposed bearing capacities.

Limit State: Bearing Capacities Calculated		
Depth (m BGL)	**Foundation System**	**Limit Bearing Capacity (kN/m²)**
3.50	5.00m by 0.75m Strip	~220
	5.00m by 1.00m Strip	~225
	1.50m by 1.50m Pad	~245
5.40	5.00m by 0.75m Strip	~245
	5.00m by 1.00m Strip	~245
	1.50m by 1.50m Pad	~265
	5.00m by 2.20m Strip	~255

The bearing capacities calculated were cross-referenced with proposals included within BS 8004:2015 Code of Practice for Foundations and based on a 5m long by 1m wide foundation and a maximum settlement of 25mm, based on in-situ testing results and inspection of samples recovered.

Excavations must be kept dry and either concreted or blinded as soon after excavation as possible. If water were allowed to accumulate on the formation for even a short time not only would an increase in heave occur resulting from the soil increasing in volume by taking up water, but also the shear strength and hence the bearing capacity would also be reduced.

Groundwater strikes were noted during the site investigation at a depth of 7.00m bgl in BH1 and at 6.80m bgl in BH2. Groundwater observations made during subsequent groundwater monitoring visits can be seen tabulated below.

Depth of Groundwater Strikes/Standing Groundwater Within Trial Holes		
Trial Hole	**Date**	**Depth of Groundwater (m bgl)**
BH1	15/03/2017	3.40
	22/03/2017	3.44
	12/04/2017	Not taken
BH2	15/03/2017	1.74
	22/03/2017	1.72
	12/04/2017	1.80

These were considered to comprise perched water within the silty / sandy bands of the cohesive Claygate Member and were not considered to comprise the actual depth of the saturate aquifer. This is further supported by initial groundwater strikes at depths of ~7.00m bgl.

Therefore, perched water may be encountered within the Made Ground and in sand and silt pockets of the Claygate Member of the London Clay Formation, especially after period of prolonged rainfall. **This should be taken into account in final design.**

The basement must be suitably tanked to prevent ingress of surface water run-off.

If the construction works take place during the winter months, when the groundwater level is expected to be at its higher elevation, perched water could accumulate thus, pumping of perched water during the basement excavation would be required. **It should be noted that it was considered that dewatering is not required.**

To avoid any potential for land instability as well as eruptions in a groundwater floor, it is considered necessary to use engineering fill for backfilling of the existing swimming pool.

7.2.1 Settlement/Heave Analysis

Basement Geometry and Loads:

Analyses of vertical ground movements (heave or settlement) have been undertaken using PDISP software in order to assess the potential magnitudes of movements which may result from the changes of vertical stresses caused by excavation and construction of the basement. Due to limitations of the software, these analyses have not modelled the horizontal forces on the retaining walls, so have simplified the stress regime. It was considered, however, that the results obtained were suitable for a stress / movement situation of this project.

The structural engineers have provided the existing and proposed loads across the building and the existing / proposed basements.

The existing basement comprised a complex geometry and was reviewed as a series of rectangles. The geometry of the basement along with the neighbouring buildings can be seen within Appendix E.

The existing loads of the basement at 3.50m bgl in the northern portion of the building along the northern slab of the building were considered to be a uniformly distributed load of 70kN/m^2.

The existing loads of the retaining walls along the southern portion of the building were considered to be 15kN/m^2.

The existing lower ground floor was modelled at a depth of 3.50m bgl, whilst the existing pool was modelled at a depth of 5.40m bgl.

Proposed Loads:

The proposed structure will act as a load bearing retaining wall strip footings with thickened edges and a suspended slab. The proposed loads and dimensions of each retaining wall can be viewed within Figure 8A.

For calculation of the heave/settlement, the proposed basement was separated by 13No. rectangles, representing different retaining walls along the proposed basement. The geometry of the basement can be seen within Appendix F.

It was understood that a south-eastern corner of the existing basement was proposed to be excavated to accommodate for a proposed basement. Overall, due to the removal of water in the existing swimming pool and a subsequent backfilling of it, the overall proposed load for the area was calculated to be 3kN/m^2.

The overburden pressure release of excavations to 3.50m bgl and 5.40m bgl were 66.50kN/m^2 and 102.60kN/m^2 respectively, based on the specific weight of the soil being 19kN/m^2.

The proposed loads for basement retaining walls at 3.50m bgl varied between 60 – 80kN/m^2. The proposed loads for basement retaining walls at 5.40m bgl varied between 50 – 95kN/m^2.

The slabs were proposed to have a weight of between 10 – 15kN/m^2 and were modelled at depths of between 3.50 – 5.40m bgl.

A small section in the southern portion of the building included a demolition of the existing walls in this area. This load was applied reversed to account for this.

All excavation loads were also applied reversed, as the software does not take soil removal into account automatically.

Four stages of construction, in terms of the net change in vertical pressure, have been modelled:

- **Stage 1:** Existing loads, excavation of underpins, short term conditions;
- **Stage 2:** Above loads together with construction/installation of retaining walls at formation levels, short term conditions;
- **Stage 3:** Above loads together with bulk excavation of basement to formation levels, short term conditions;
- **Stage 4:** As Stage 3 for long term conditions.

Ground Conditions:

The ground profile was based on the site-specific ground investigation, as presented in Section 7.1 of this report. The input of the geotechnical parameters and layers used in PDisp 20.0 can be seen in Appendix E.

Undrained soil parameters have been used to estimate the potential short-term movements, which include the "immediate / short term" movements as a result of the basement excavation and construction. The model is based on the assumption that the soils behave elastically, which provides a reasonable approximation to soil behaviour at small strains. Drained / long term parameters have been used to provide an estimate of the total movement, which includes long term swelling that will continue for a number of years.

The elastic analysis requires values of soil stiffness at various levels to calculate displacements. Values of stiffness for the soils at this site are readily available from published data and we have used a well-established method to provide our estimates. This relates values of Eu and E', the undrained and drained Young's Modulus (Eu and E' respectively), to values of undrained shear strength (Cu), as described by published data indicating stiffness values of 700 x Cu for the Claygate Member of the London Clay Formation and a ratio of E' to Eu of 0.80, which is considered a sensible approach for this stage in the design. **The design lines for the Cu were based on trend lines on the resulting Cu values obtained from the Hand vane values recorded.**

The short-term and long-term geotechnical properties of the soil strata used for the PDisp analyses are summarised in the table below.

Summary of geotechnical parameters for PDisp analysis					
Strata	**Level (m bgl)**	**Undrained Shear Strength (Cu) – derived from Shear Vane Testing**	**Short Term, undrained Young's Modulus (Eu)****	**Long Term, drained Young's Modulus (E)**	**Poisson's Ratio (N)**
Made Ground	GL – 0.90	56.39**	10000	8000	0.40
Claygate Member of the London Clay Formation	0.90 – 3.00	94 – 116	39473 (top) 91070 (base)	31578 (top) 72856 (base)	0.40
Claygate Member of the London Clay Formation	3.00 – 5.50	116 – 141.44	91070 (top) 99008 (base)	72856 (top) 79206 (base)	0.40
Claygate Member of the London Clay Formation	5.50 – 50.00*	141.44 – 207.57	99008 (top) 145299 (base)	79206 (top) 116239 (base)	0.40
Where for Made Ground: Eu = 10000 E = 8000 Where for cohesive Claygate Member of the London Clay Formation: Eu = 700 * Cu E = Eu * 0.80 **Based on "Burland JB, Standing, JR, and Jardine, FM (2001) *Building response to tunnelling, case studies from construction of the Jubilee Line Extension* CIRIA Special Publication 200" and other literature, based on in-situ testing recorded on site.**					

* It should be noted that the London Clay Formation was to be expected at depth, however, was not modelled due to the negligible change in geotechnical values and was considered acceptable.

** Undrained Young's Modulus was calculated using 2No. design lines: (1) C_u=-18.90z+56.39 between GL – 3.90m bgl and (2) C_u=-2.70z + 126.57 between 3.90 – 50.00m bgl.

<u>Pdisp Analysis</u>

Three dimensional analyses of vertical displacements have been undertaken using Pdisp software and the basement geometry, loads/stresses and ground conditions outlined above in order to assess the potential magnitudes of ground movements (heave or settlement) which may result from the vertical stress changes caused by excavation of the basement. Four stages of construction, in terms of the net change in vertical pressure, have been modelled:

- **Stage 1:** Existing loads, excavation of underpins, short term conditions;
- **Stage 2:** Above loads together with construction/installation of retaining walls at formation levels, short term conditions;
- **Stage 3:** Above loads together with bulk excavation of basement to formation levels, short term conditions;
- **Stage 4:** As Stage 3 for long term conditions.

The results of the analyses for Stages 1 – 4 are presented as contour plots within Appendix F.

<u>Heave / Settlement Assessment</u>

Excavation of the basement will cause immediate elastic heave in response to the stress reduction, followed by long-term plastic swelling as the overconsolidated clays take up groundwater. The rate of plastic swelling in overconsolidated clays will be determined largely by the permeability of the clay and the availability of water. As a result, the rate of swelling may be relatively rapid where water-bearing laminations of silt/sand are present in the sandy silty clays of the Claygate Member of the London Clay Formation, however, in the areas where sand is absent and the soils comprise homogenous silty clays with low permeability, the swelling can take decades to reach full equilibrium. The structures of these basements will need to be designed to enable them to accommodate the swelling displacements/pressures developed underneath them.

The ranges of predicted short-term and long-term movements for different proposed sections at different levels are presented in the table below (in accordance with PDisp 20.0 results only).

Summary of predicted vertical displacements (derived from PDisp)				
Location	**Stage 1 (short term)**	**Stage 2 (short term)**	**Stage 3 (short term)**	**Stage 4 (long-term)**
Eastern portion of the basement (lower ground floor)	0 – 2mm Settlement and 1mm of Heave on the edges between GL – 3.50m bgl*	2 – 4mm Settlement between GL – 3.50m bgl	0 – 3mm Settlement between GL – 3.50m bgl	1 – 3mm Settlement between GL – 3.50m bgl
North-western portion of the basement	1 – 3mm Heave between GL – 3.50m bgl	0.00 – 2.00mm Settlement between GL – 3.50m bgl	0 – 2mm Heave between GL – 3.50m bgl	0 – 2mm Heave between GL – 3.50m bgl
South-western portion of the basement	1 – 3mm Heave between GL – 5.40m bgl	0.50 – 1.50mm Settlement at 5.40m bgl	1 – 3mm Heave between GL – 5.40m bgl	1 – 4mm Heave between GL – 5.40m bgl

*Although the settlement in the eastern portion of the building was calculated, it should be noted that most of the short-term and long-term settlements would have most likely happened prior to this investigation after the property was constructed.

The ranges of predicted short-term and long-term settlements for Stages 2 – 4 were amended in accordance with assumed additional 10mm settlement due to 2No. underpinning stages. It should be noted that the settlement was not added to Stage 1 as no underpin installation was included at that stage.

A Structural Engineer will need to review the anticipated ground movements and assess their potential impact on the existing structure and neighbouring properties. The use of a cellcore at the centre of the basement slab should reduce the heave outlined in the table above.

Groundwater Considerations & Movements

Groundwater strikes were noted during the site investigation at a depth of 7.00m bgl in BH1 and at 6.80m bgl in BH2. Groundwater observations made during subsequent groundwater monitoring visits were between 1.72 – 3.44m bgl (borehole position).

These were considered to comprise perched water within the silty / sandy bands of the cohesive Claygate Member of the London Clay Formation and were not considered to comprise the actual depth of the saturate aquifer. This is further supported by initial groundwater strikes at depths of ~7.00m bgl.

Therefore, perched water may be encountered within the Made Ground and in sand and silt pockets of the Claygate Member of the London Clay Formation, especially after period of prolonged rainfall.

If the construction works take place during the winter months, when the groundwater level is expected to be at its higher elevation, perched water could accumulate thus, pumping of perched water during the basement excavation would be required.

The advice of a reputable dewatering company should be sought, sump pumps may be required, depending on the rate of water ingress.

It was considered that sheet piling will not be required, as the saturated aquifer is expected to be much lower than the perched water measurements, trapped in less cohesive bands of the Claygate Member, which is primarily clay and stable. No instability issues including running sands were reported during the ground investigation and none are expected on these ground conditions and any pumping of perched water was considered to be limited. Any movements resulting from limited perched water pumping will be minimal and will not affect the integrity of the underpin trenches or beyond.

7.3 Piled Foundations

Given the results of the investigation a piled foundation scheme was considered unlikely to be required at this site.

7.4 Basement Excavations & Stability

Shallow excavations in the Made Ground and the Claygate Member of the London Clay Formation are likely to be marginally stable at best. Long, deep excavations, through both of these strata are likely to become unstable.

The excavations must not affect the integrity of the adjacent structures beyond the boundaries. The excavations must be supported by suitably designed retaining walls.

The retaining walls will need to be constructed based on soils encountered with an appropriate angle of shear resistance (*Φ'*) and effective cohesion (*C'*) for the ground conditions encountered, regarding long term considerations, as well using an appropriate undrained shear strength Cu for short term considerations. Characteristic undrained shear strengths Cu for the soils of the Claygate Member of the London Clay Formation should be based on the in situ testing, triaxial testing and an SPT conversion factor of 5 – 4.5 for clays.

The overlying retained Made Ground (to ~0.45 – 0.90m bgl) needs to be taken into account in the design of the basement. Conservative geotechnical parameters should be considered.

The Cu values for the natural soils described in Section 5.1 can be considered (a conversion factor of 5).

Based on the ground conditions encountered the following characteristic soil parameters could be used in the design of retaining walls for a long-term consideration. These have been designated based on the SPT profile recorded, results of geotechnical classification tests and reference to literature.

Retaining Wall/Basement Design Parameters – Long term design					
Strata	Unit Volume Weight (kN/m³)	Cohesion Intercept (c') (kPa)	Angle of Shearing Resistance (Ø)	Ka	Kp
Made Ground	~13 – 15	0	12	0.66	1.52
Claygate Member of the London Clay Formation	~15 – 20	0	24	0.42	2.37

As geotechnical testing defined the Claygate Member of the London Clay Formation as being heavily overconsolidated, a Ko value was unobtainable due to Overconsolidation Ratio (OCR) being unknown. A Ko range of 1.0 – 2.8 can be used for consolidated Claygate Member of the London Clay Formation.

The value of K adopted in design calculations should allow for the effects of wall installation. In general, it may be appropriate to adopt a K value of 1.0 from simple elastic (i.e. where the pre-failure deformation of the soil is assumed to be linear) soil-structure interaction analysis on overconsolidated fine-grained soils. **The structural engineer will be required to account for this in the final design.**

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 before excavations are entered by personnel.

Perched water maybe encountered within the Made Ground or/and silty pockets of the Claygate Member of the London Clay Formation, especially after period of prolonged rainfall. **This should be taken into account in final design.**

Should groundwater be encountered across the site, dewatering from sumps introduced into the floor of the excavation may be required, especially after a period of excessive rainfall. The advice of a reputable dewatering company should be sought.

7.5 Hydrogeological Effects

A study of the aquifer maps on the DEFRA website, and Figure 8 of the Camden Geological, Hydrogeological and Hydrological Study (see Figure 14 of this report), revealed the site was located on the **Secondary (A) Aquifer** comprising the Claygate member of the London Clay Formation. No designation was given for any superficial deposits due to their likely absence.

Figure 14 of the Camden Geological, Hydrogeological and Hydrological Study revealed the site was located within the Highgate Chain Catchment Ponds (see Figure 17 of this report).

The ground conditions encountered generally comprised a capping of Made Ground over cohesive soils with some granular bands of the Claygate Member of the London Clay Formation. Based on a visual appraisal of the soils encountered the permeability of the Claygate Member of the London Clay Formation Beds were likely to be low permeability.

Groundwater strikes were noted during the site investigation at a depth of 7.00m bgl in BH1 and at 6.80m bgl in BH2. Groundwater observations made during subsequent groundwater monitoring visits can be seen tabulated below.

Depth of Groundwater Strikes/Standing Groundwater Within Trial Holes		
Trial Hole	**Date**	**Depth of Groundwater (m bgl)**
BH1	15/03/2017	3.40
	22/03/2017	3.44
	12/04/2017	Not taken
BH2	15/03/2017	1.74
	22/03/2017	1.72
	12/04/2017	1.80

Based on the above, it was considered likely that the basement construction may encounter perched water within the Made Ground and the soils of the Claygate Member of the London Clay Formation. Therefore, very limited pumping of perched water during the basement excavation may be required. **It should be noted that actual dewatering was not considered to be required.**

Figure 14 of the Camden Geological, Hydrogeological and Hydrological Study revealed the site was located within the Highgate Chain Catchment Ponds (see Figure 17 of this report). It was considered that the proposed development may affect the groundwater flow (very slightly), however, it was recommended to construct additional drainage as a part of the proposed development. This must be taken into account by a Structural Engineer in final design.

The basement was therefore considered unlikely to affect the saturated aquifer underlying the site as the groundwater level was considered to be at depth (8m bgl (below ground level)).

In relation to the basement, once constructed, the Made Ground will act as a slightly porous medium for water to migrate however additional drainage should be considered as the Claygate Member of the London Clay Formation will act as a barrier for groundwater migration.

7.6 Ground Movement Analysis

The Ground Movement Analysis was undertaken by Ground and Project Consultants Limited (report ref.: 60431-1). The results can be seen in Appendix G.

It can be seen that the GMA revealed Negligible to Very Slight movements in relation to the neighbouring properties, which was considered acceptable.

7.7 Sub-Surface Concrete

For the classification given below, the "mobile" and "natural" case was adopted given the geology encountered and the residential use of the site.

Made Ground

The water-soluble sulphates in the Made Ground tested (from the chemical laboratory testing) were found to range between 61 – 110mg/kg with a pH of 8.5 – 10.3.

Natural Ground – Claygate Member London Clay Formation

The water-soluble sulphate concentration in the samples (from the geotechnical laboratory testing) ranged from 83 – 990mg/l with a pH range of 7.6 – 10.8. The total potential sulphate (3x total sulphur) concentration ranged between 0.03 – 2.34%.

Therefore, sulphate concentrations measured in 2:1 water/soil extracts taken from the natural ground and total potential sulphate concentrations, fell into Class DS-4 of the BRE Special Digest 1, 2005, 'Concrete in Aggressive Ground'. Table C1 of the Digest indicated an ACEC (Aggressive Chemical Environment for Concrete) classification of AC-4.

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 the pH of the soils.

7.8 Surface Water Disposal

Soakaways constructed within the cohesive soils of the Claygate Formation of the London Clay Formation are unlikely to prove satisfactory due to negligible to low anticipated infiltration rates.

Consultation with the Environment Agency must be sought regarding any use that may have an impact on groundwater resources.

7.9 Stage 5 Review

The conceptual site model given within Section 3.2.1 identified five matters of concern for the property. These concerns have been assessed within the report and the conclusions can be seen tabulated overpage.

Stage 5 Review		
Highlighted Area	**Site Specific Concern**	**Assessment**
Perched water within the Made Ground or the Claygate Member of the London Clay Formation	The basement may encounter perched water within the Made Ground or silt bands of the London Clay Formation during construction.	Groundwater strikes were noted during the site investigation at a depth of 7.00m bgl in BH1 and at 6.80m bgl in BH2. Returned monitoring visits on 15th and 22nd March and 12th April 2017 revealed groundwater at depths of between 3.40 – 3.44m bgl in BH1 and 1.72 – 1.80m bgl in BH2. These were considered to comprise perched water within the silty / sandy bands of the cohesive Claygate Member and were not considered to comprise the actual depth of the saturate aquifer. This is further supported by initial groundwater strikes at depths of ~7.00m bgl. Based on the above it is considered likely the basement will encounter perched water. However, the basement will not affect the saturated aquifer underlying the site. It should be noted that actual dewatering was not considered to be required. The cumulative effects of basements in groundwater is not a consideration at this site. Limited pumping of perched water during the basement excavation would be required. It was considered that sheet piling will not be required, as the saturated aquifer is expected to be much lower than the perched water measurements, trapped in less cohesive bands of the Claygate Member, which is primarily clay and stable. No instability issues including running sands were reported during the ground investigation and none are expected on these ground conditions and any pumping of perched water is expected to be limited. Any movements resulting from limited perched water pumping will be minimal and will not affect the integrity of the underpin trenches or beyond. As the site was located within the Highgate Ponds Catchment Zone, it was considered that the proposed development may affect the groundwater flow, however, it was recommended to construct additional drainage as a part of the proposed development. This must be taken into account by a Structural Engineer in final design.
Soil Moisture/ Trees and Bushes	There is potential for soil moisture content to affect the development.	Geotechnical analysis revealed the soils to be heavily overconsolidated with potentially root exacerbated moisture deficit at BH1/1.50m bgl. A lithologically controlled moisture deficit was noted at BH1/3.00m, BH1/3.50 and BH2/1.50m bgl.

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Stage 5 Review (Cont'd)		
Highlighted Area	**Site Specific Concern**	**Assessment**
Claygate Member of the London Clay Formation/ Shrink and Swell	The basement is anticipated to be founded in the Claygate Member of the London Clay Formation. The soils are likely to have medium to high plasticity and volume change potential. The concrete mix design should take appropriate account of sulphate levels (testing to BRE Special Digest). Heave on removal of overburden pressure may be a risk.	Geotechnical testing revealed the Claygate Member of the London Clay Formation to have medium volume change potential in accordance with BRE240 and NHBC Standards Chapter 4.2. Sulphate concentrations measured in 2:1 water/soil extracts taken from the Claygate Member of the London Clay Formation from geotechnical analysis fell into Class DS-4 of the BRE Special Digest 1, 2005, 'Concrete in Aggressive Ground'. **Sub-surface concrete specification is discussed further in Section 7.9 of this report.** **Heave on removal of overburden pressure is discussed within Section 6.2 of this report.**
Differential Foundation Depths	It will be important to account for the shallow nature of existing footings at the property and its neighbours. Ground Movement Assessment is required.	Ground movement assessment was carried out on the neighbouring properties by Ground and Project Consultants Limited, which can be seen within Appendix G. In terms of building damage assessment and with reference to Table 2.5 of C580/C760 (after Burland et al, 1977), the 'Description of typical damage' given the calculated movements it is likely to fall within category of damage '0' Negligible to '1' Very Slight. No risk to services and roads was considered to be present. Mitigation measures relating to ground movement can be viewed within the Ground Movement Assessment Report (ref.: 60431-1).
Retaining Walls	Appropriate Design	Parameters for retaining wall design and ground movement provided in Section 7.4 of this report. **Structural Design will need to take this into account.**
Backfill of Swimming Pool	Land Stability and Groundwater Flow	To avoid any potential for land instability as well as eruptions in a groundwater floor, it is considered necessary to use engineering fill for backfilling of the existing swimming pool.

7.10 Discovery Strategy

There may be areas of contamination that have not been identified during the course of the intrusive investigation. For example, there may have been underground storage tanks (UST's) not identified during the Ground Investigation for which there is no historical or contemporary evidence.

Such occurrences may be discovered during the demolition and construction phases for the redevelopment of the site.

Groundworkers should be instructed to report to the Site Manager any evidence for such contamination; this may comprise visual indicators, such as fibrous materials within the soil, discolouration, or odours and emission. Upon discovery advice must be taken from a suitably qualified person before proceeding, such that appropriate remedial measures and health and safety protection may be applied.

Should a new source of contamination be suspected or identified then the Local Authority will need to be informed.

7.11 Waste Disposal

The excavation of foundations is likely to produce waste which will require classification and then recycling or removal from site.

Under the Landfill (England and Wales) Regulations 2002 (as amended), prior to disposal all waste must be classified as;

- Inert;
- Non-hazardous, or;
- Hazardous.

The Environment Agency's Hazardous Waste Technical Guidance (WM3) document outlines the methodology for classifying wastes.

Once classified the waste can be removed to the appropriately licensed facilities, with some waste requiring pre-treatments prior to disposal.

Based on a risk phrase analysis of the remaining chemical laboratory test results, in accordance with EC Hazardous Waste Directive and undertaken by Ground and Water Limited, the sample of Made Ground tested (BH1/0.25m and BH2/0.50m bgl) were classified as **NON-HAZARDOUS.** The results of the assessment are given within Appendix H.

A Full WAC Solid Suite Test with single batch leachate was undertaken on one sample of the Made Ground (BH1/0.50m bgl) to determine which landfill category the waste conformed to. The results of the WAC test can be seen in Appendix B. The sample fell into the **INERT Waste Landfill** category.

It is important to note that whilst we consider our in-house assessment tool to be an accurate interpretation of the requirements of WM3, therefore producing an initial classification in accordance with the guidance, landfill operators have their own assessment tools and can often come to different conclusions. As a result, some landfill operators could refuse to take apparently suitable waste. It is recommended that the receiving landfill views the results of this assessment and the chemical laboratory results to determine their own classification.

7.12 Imported Material

Any soil which is to be imported onto the site must undergo chemical analysis to prove that it is suitable for the purpose for which it is intended.

The Topsoil must be fit for purpose and must either be supplied with traceable chemical laboratory test certificates or be tested, either prior to placing (ideally) or after placing, to ensure that the human receptor cannot come into contact with compounds that could be detrimental to human health. The compounds that are to be tested for are those given in the LQM CIEH Generic Assessment Criteria, which can be viewed in Appendix D of this report.

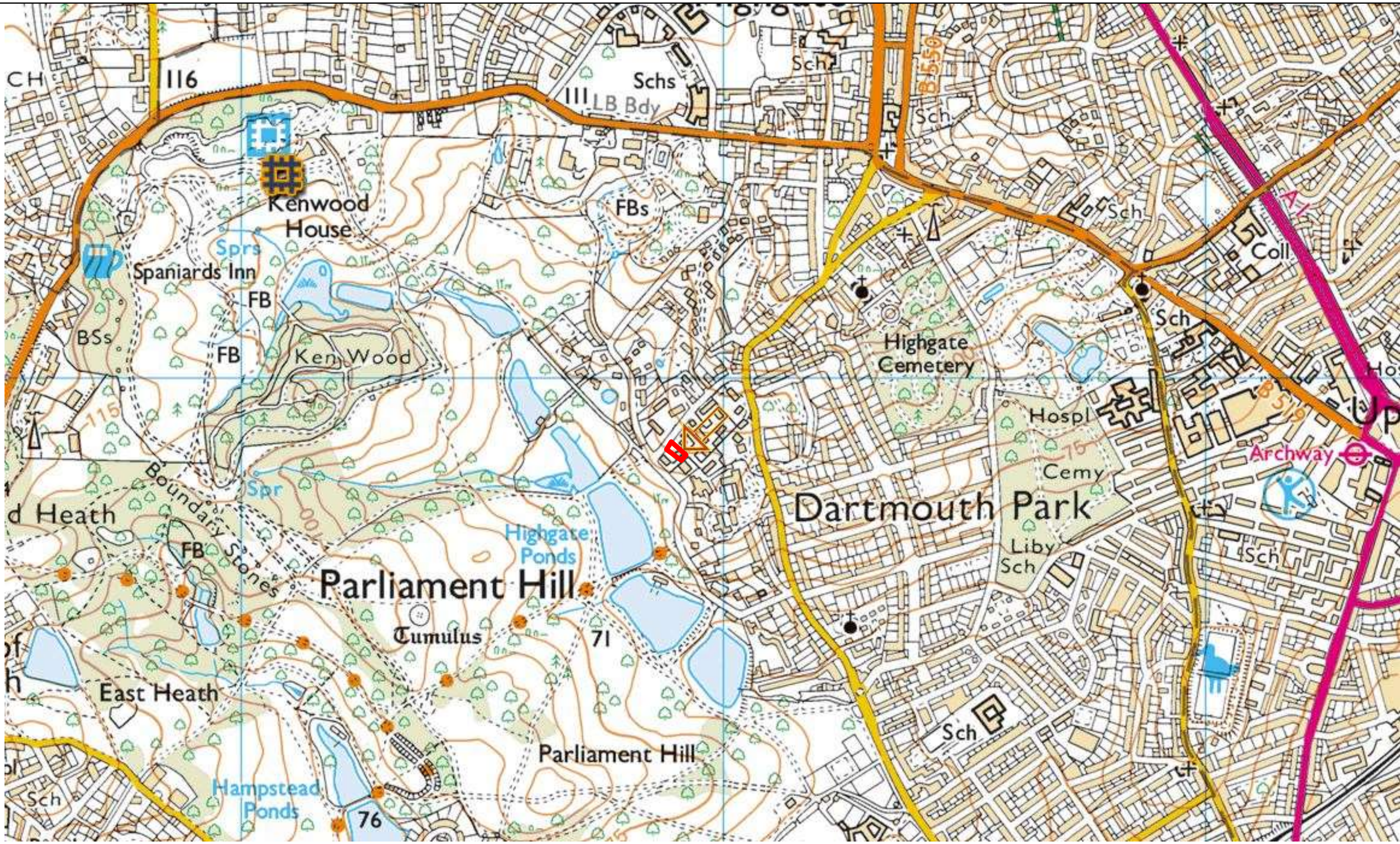
7.13 Duty of Care

Groundworkers must maintain a good standard of personal hygiene including the wearing of overalls, boots, gloves and eye protectors and the use of dust masks during periods of dry weather.

To prevent exposure to airborne dust by both the general public and construction personnel the site should be kept damp during dry weather and at other times when dust were generated as a result of construction activities.

The site should be securely fenced at all times to prevent unauthorised access. Washing facilities should be provided and eating restricted to mess huts.

N



APPROXIMATE SITE BOUNDARY

NOTE: NOT TO SCALE

Project: 26 West Hill Park, London Borough of Camden, London N6 6ND		**Figure 1**
Client: Croft Structural Engineers	**Date:** November 2019	ground&water
Site Location Plan	**Ref:** GWPR2950	

NOTE: NOT TO SCALE

Project: 26 West Hill Park, London Borough of Camden, London N6 6ND		**Figure 2**
Client: Croft Structural Engineers	**Date:** November 2019	
Site Development Area	**Ref:** GWPR2950	

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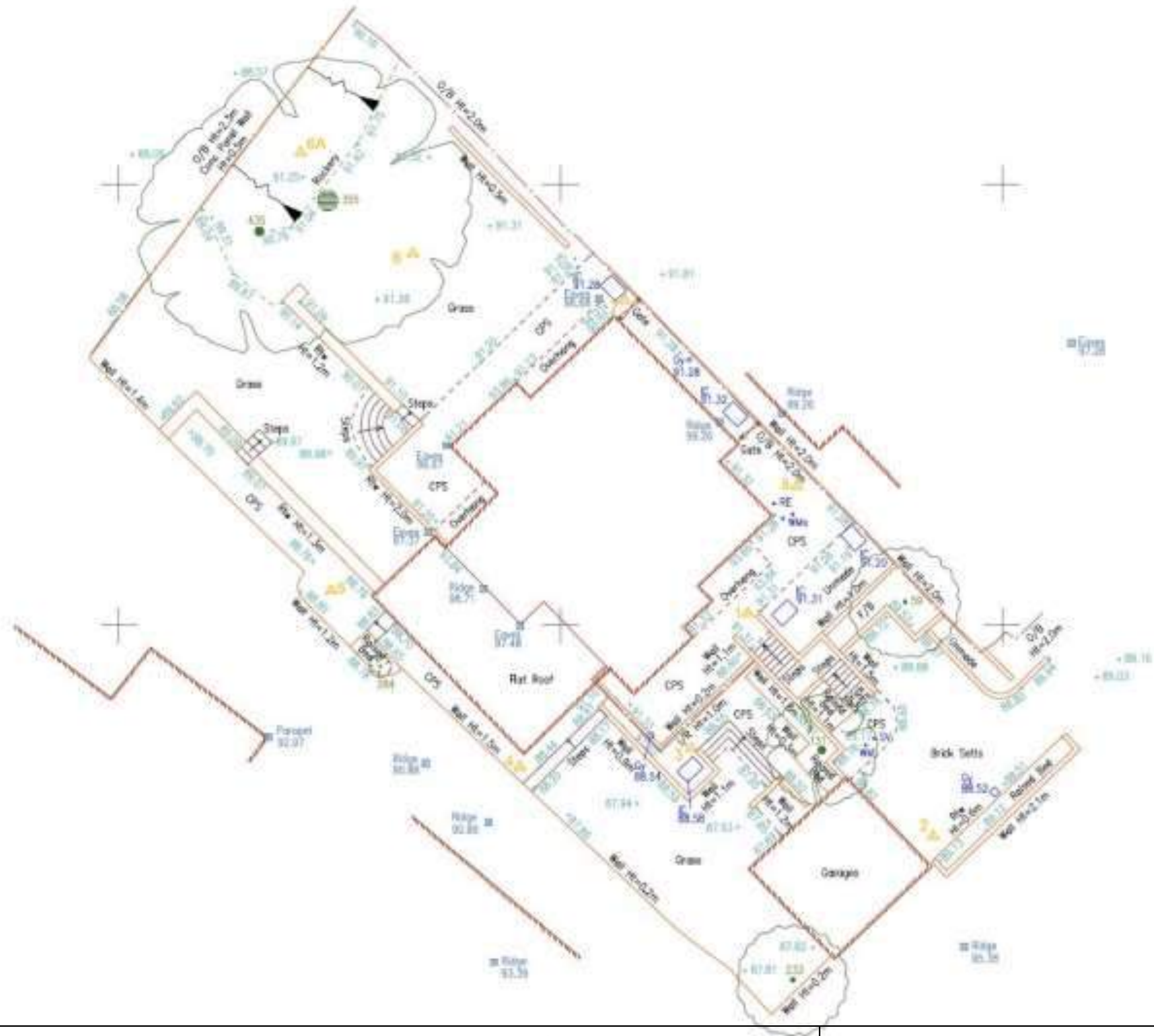
APPROXIMATE SITE BOUNDARY

NOTE: NOT TO SCALE

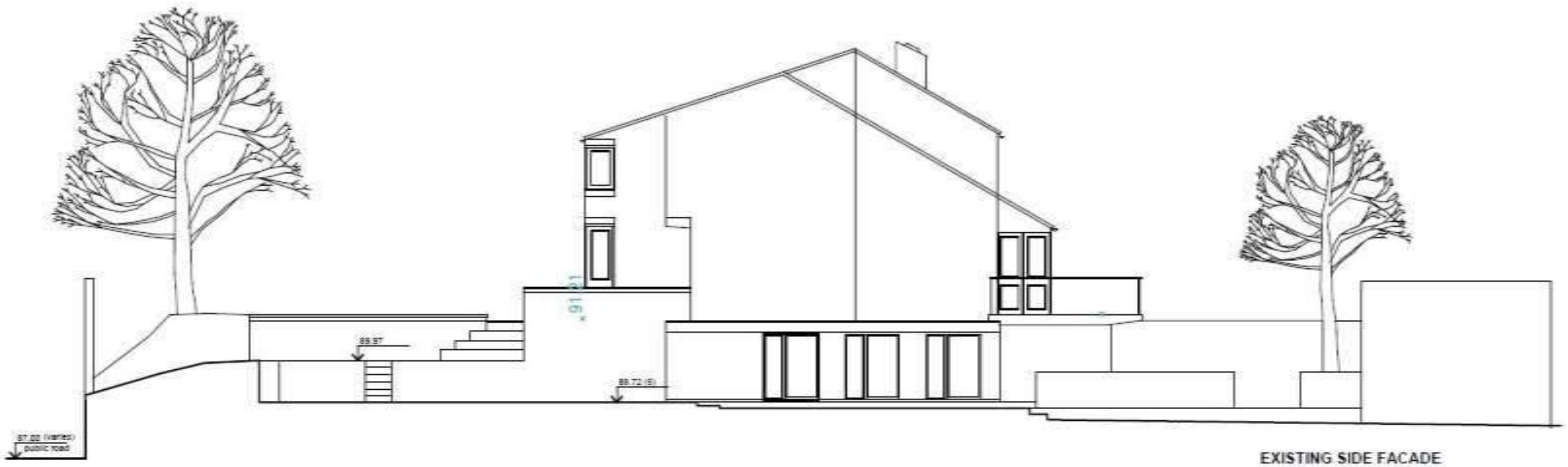
Project: 26 West Hill Park, London Borough of Camden, London N6 6ND		**Figure 3**
Client: Croft Structural Engineers	**Date:** November 2019	ground&water
Aerial View of Site	**Ref:** GWPR2950	



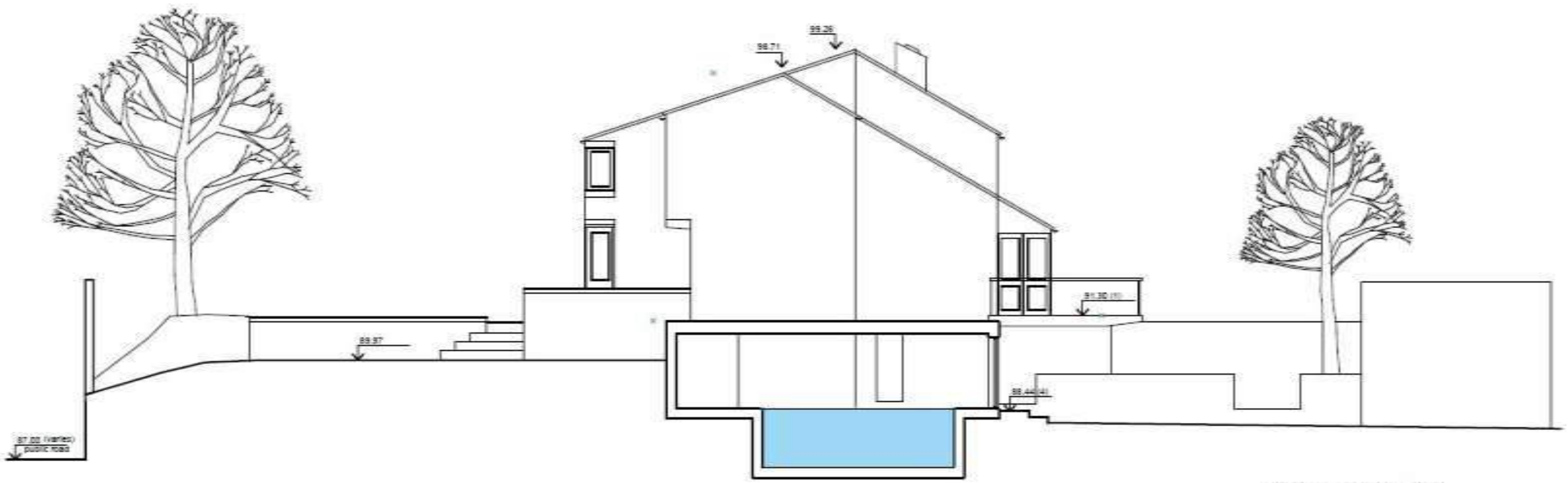
NOTE: NOT TO SCALE

Project: 26 West Hill Park, London Borough of Camden, London N6 6ND	**Figure 4**
Client: Croft Structural Engineers	**Date:** November 2019
Topographical Survey	**Ref:** GWPR2950

ground&water



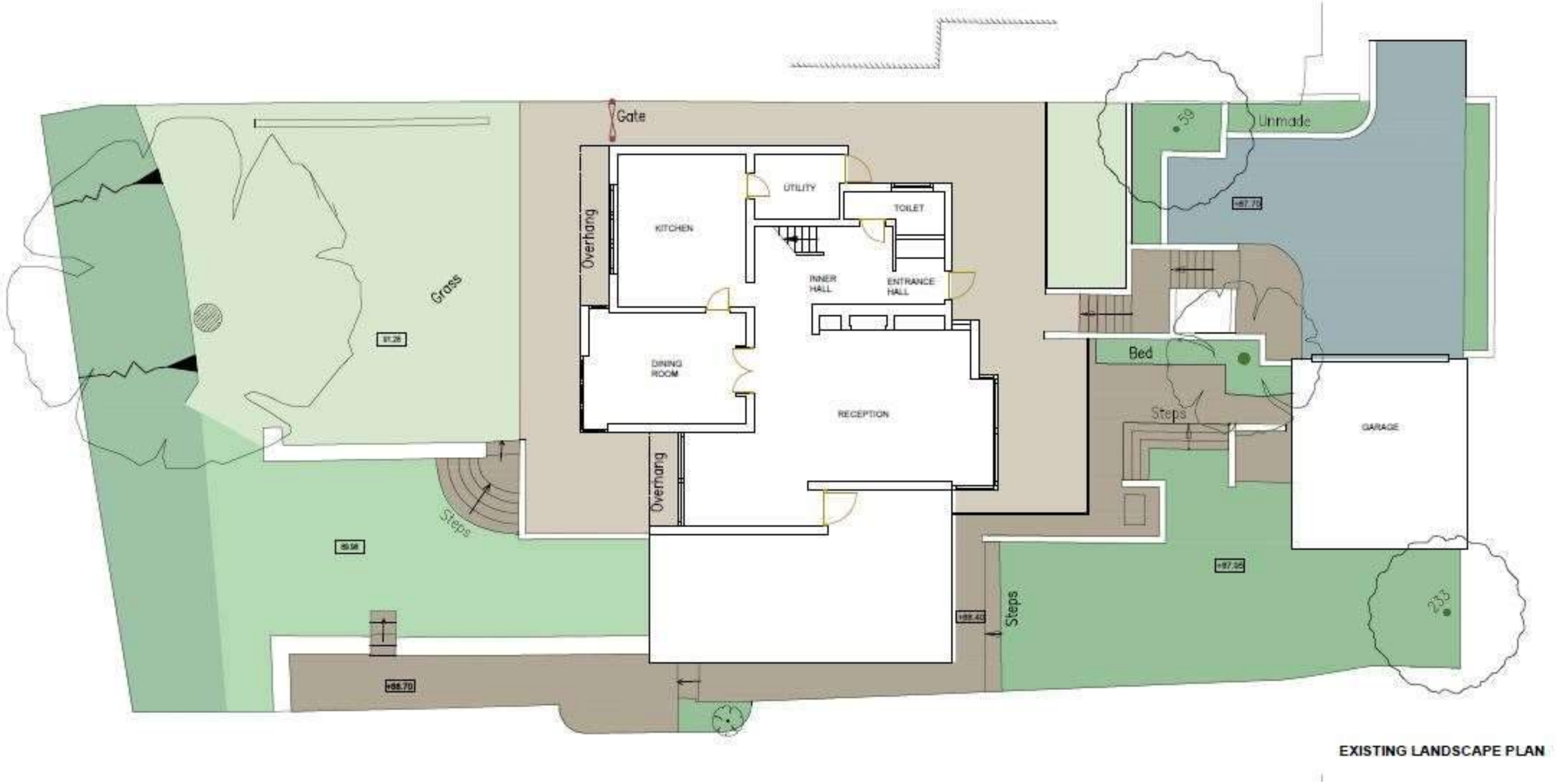
EXISTING SIDE FACADE



EXISTING SECTION A-A

NOTE: NOT TO SCALE

Project: 26 West Hill Park, London Borough of Camden, London N6 6ND	**Figure 5**
Client: Croft Structural Engineers / **Date:** November 2019	ground&water
Existing Sectional Plan / **Ref:** GWPR2950	



APPROXIMATE SITE BONDARY

NOT TO SCALE

Project: 26 West Hill Park, London Borough of Camden, London N6 6ND

Client: Croft Structural Engineer

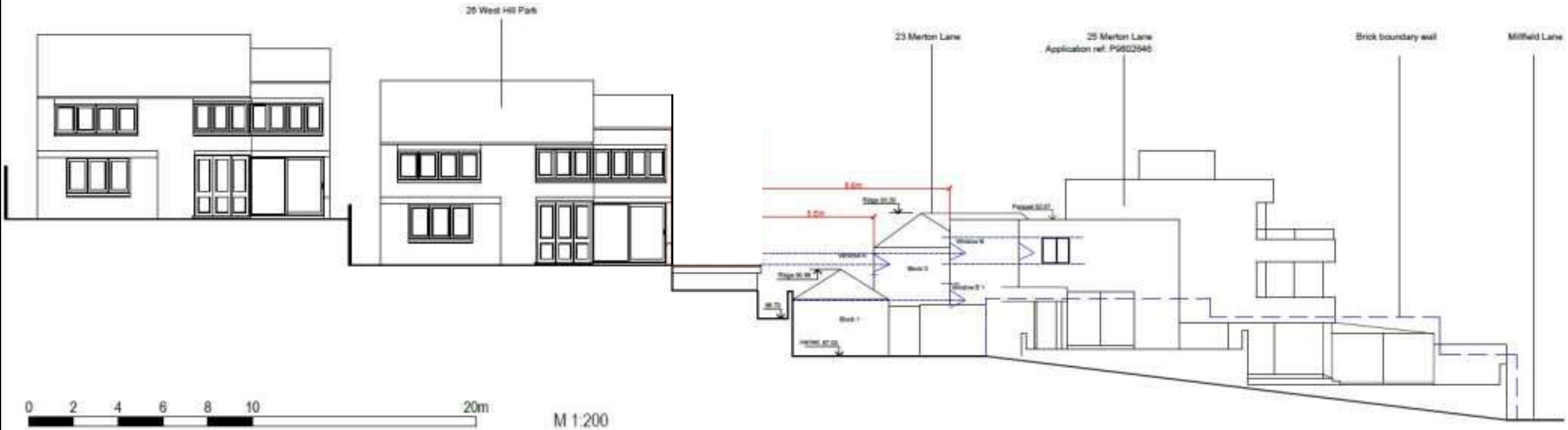
Date: November 2019

Existing Plan View

Ref: GWPR2950

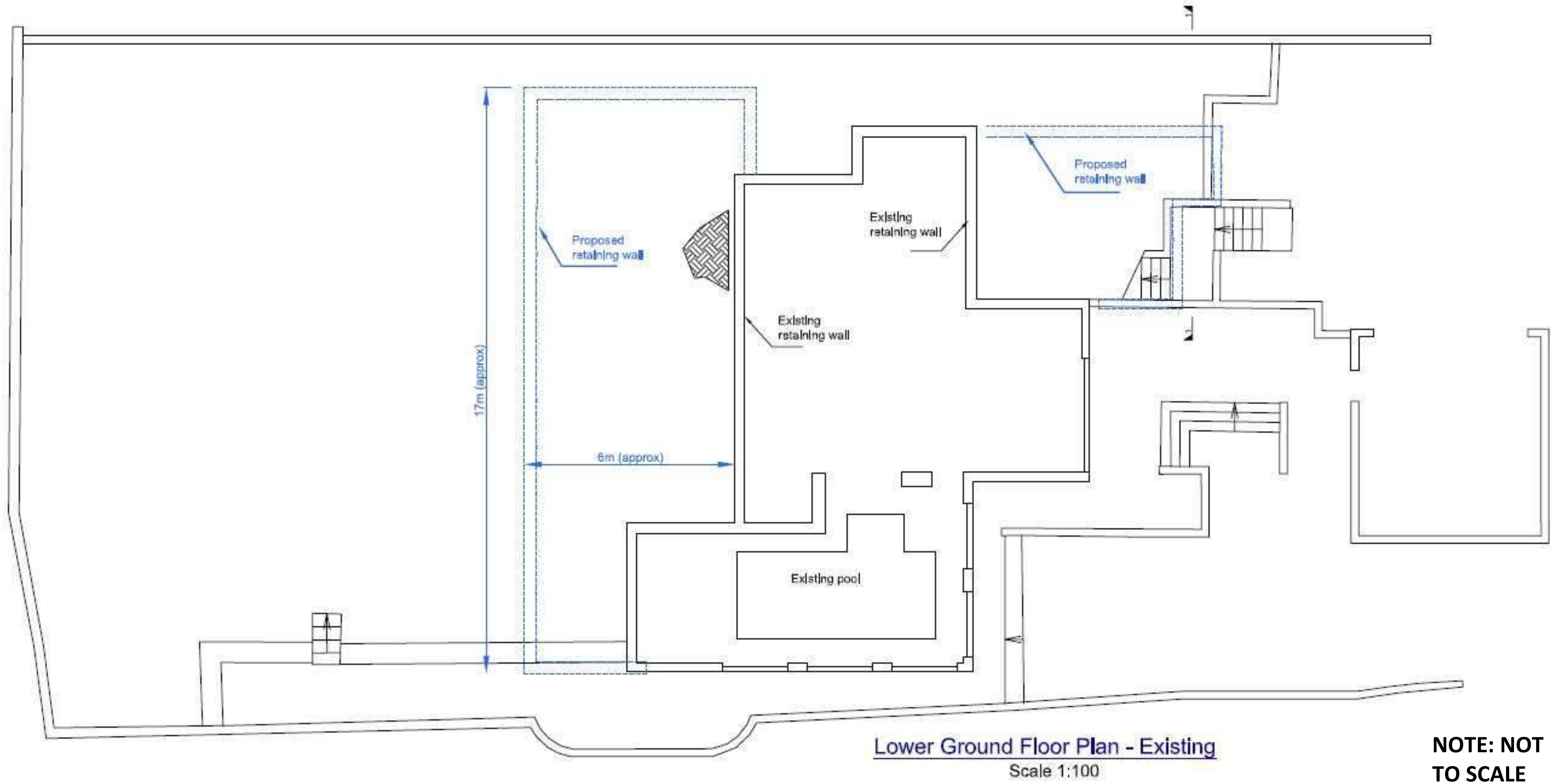
Figure 6

ground&water



NOTE: NOT TO SCALE

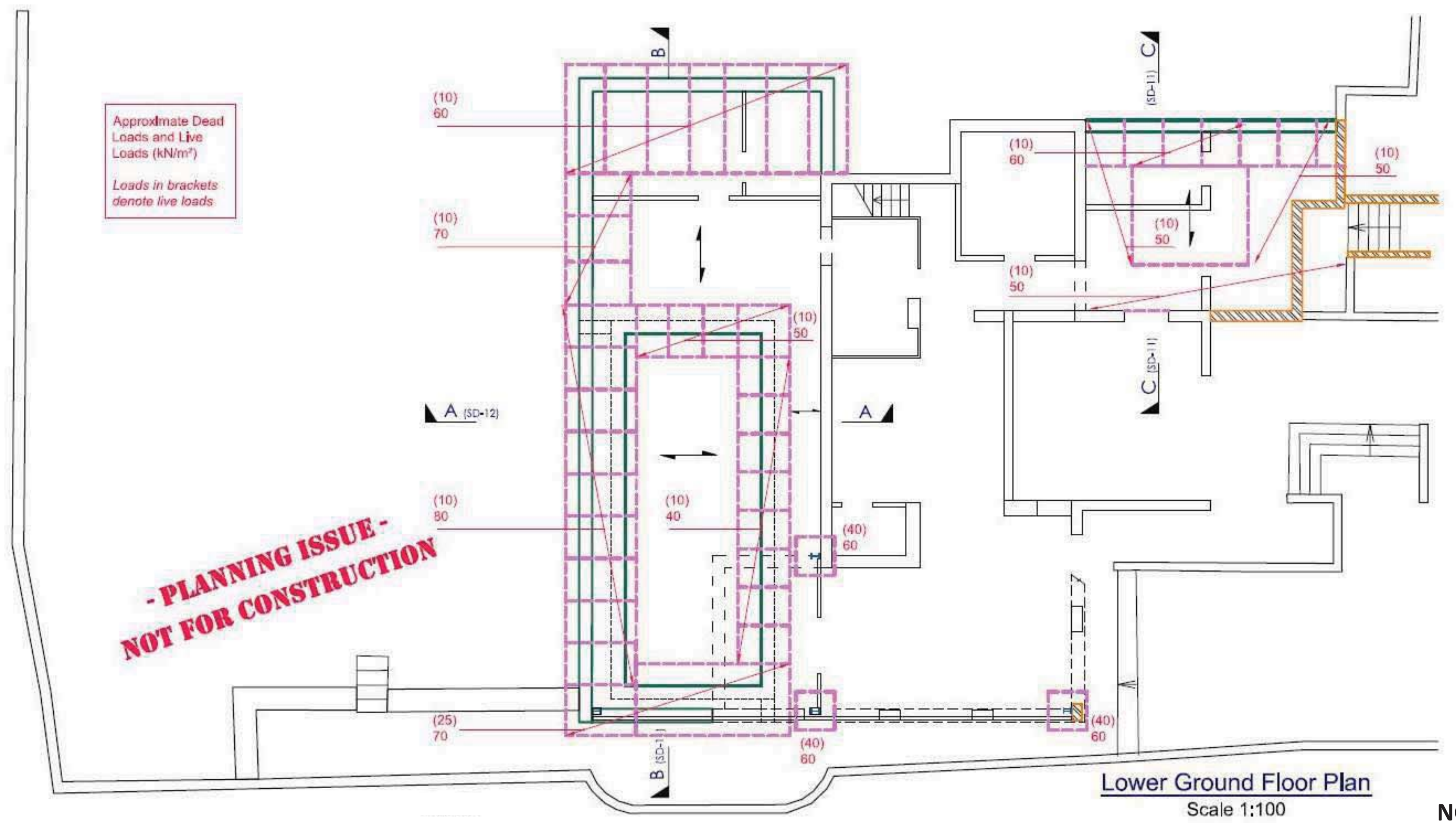
Project: 26 West Hill Park, London Borough of Camden, London N6 6ND	**Figure 7**
Client: Croft Structural Engineer — **Date:** November 2019	ground&water
Sectional View of Merton Lane — **Ref:** GWPR2950	



NOTE: NOT TO SCALE

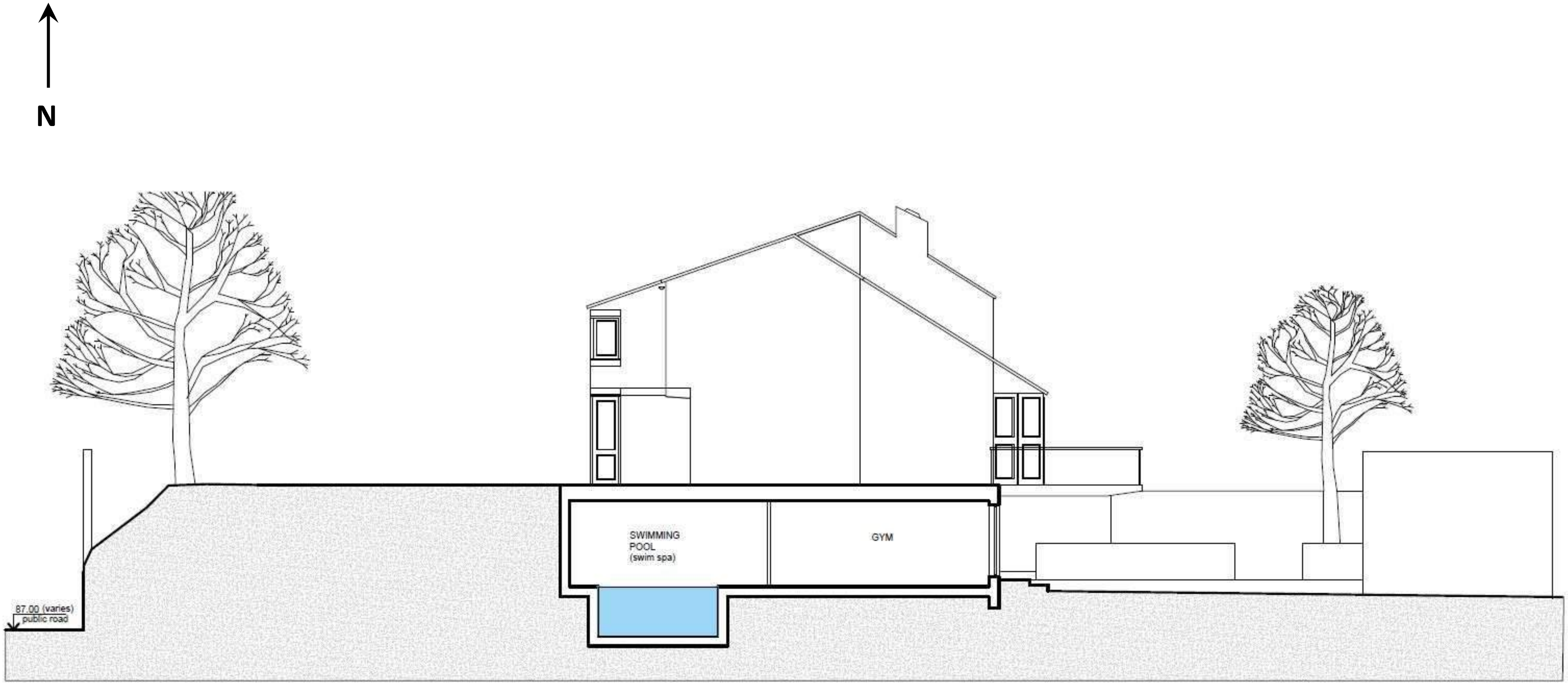
Project: 26 West Hill Park, London Borough of Camden, London N6 6ND	**Figure 8**
Client: Croft Structural Engineer	**Date:** November 2019
Proposed Development – Plan View	**Ref:** GWPR2950

ground&water



NOTE: NOT TO SCALE

Project: 26 West Hill Park, London Borough of Camden, London N6 6ND		**Figure 8A**
Client: Croft Structural Engineer	**Date:** November 2019	ground&water
Proposed Development – Plan View	**Ref:** GWPR2950	

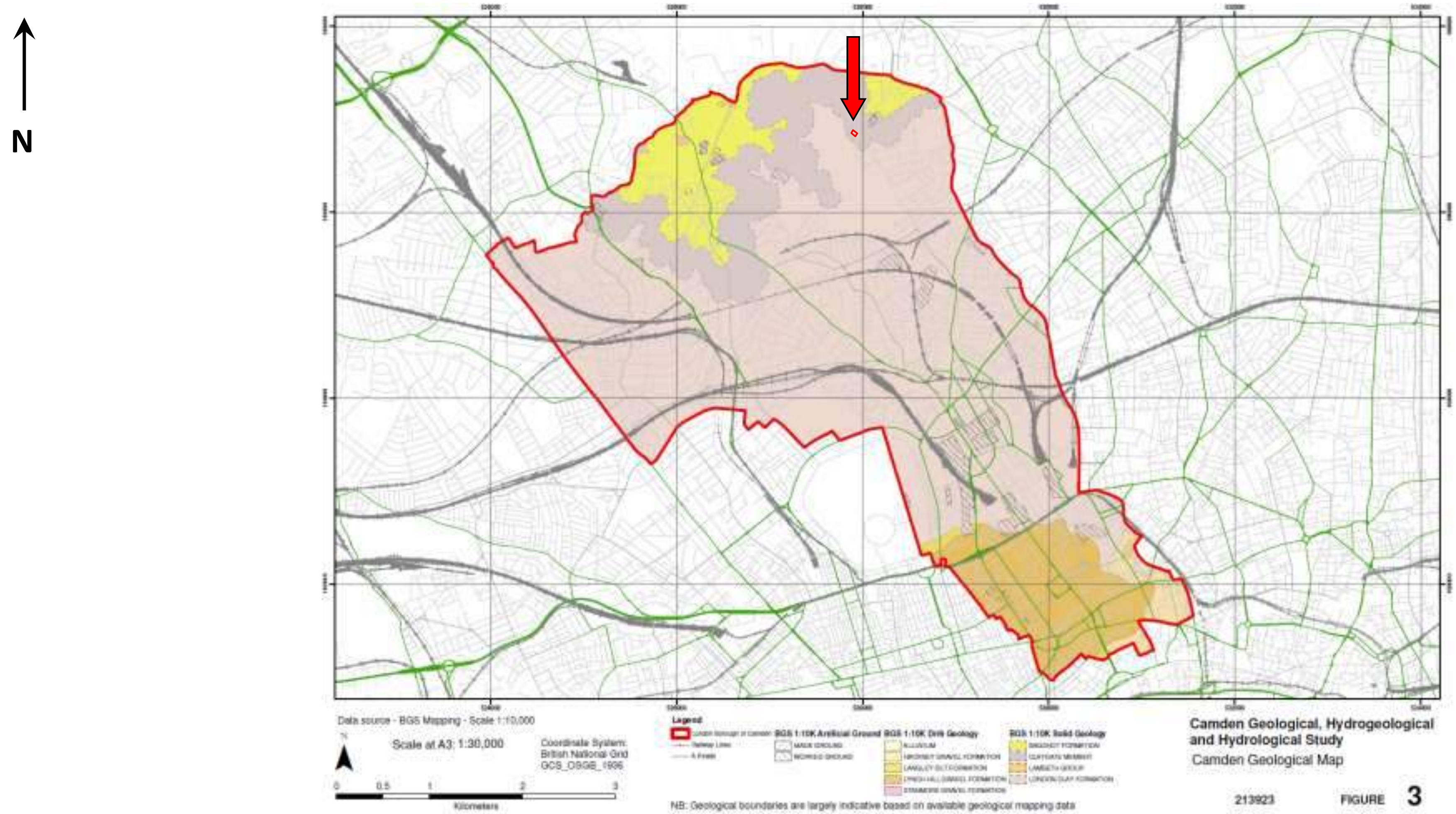


PROPOSED SECTION AA

APPROXIMATE SITE BOUNDARY

NOTE: NOT TO SCALE

Project: 26 West Hill Park, London Borough of Camden, London N6 6ND		Figure 9
Client: Croft Structural Engineer	Date: November 2019	ground&water
Proposed Development – Section View	Ref: GWPR2950	



APPROXIMATE SITE BOUNDARY

NOTE: NOT TO SCALE

Project: 26 West Hill Park, London Borough of Camden, London N6 6ND		**Figure 10**
Client: Croft Structural Engineer	**Date:** November 2019	ground&water
North Camden Geological, Hydrogeological and Hydrological Study - Figure 3	**Ref:** GWPR2950	

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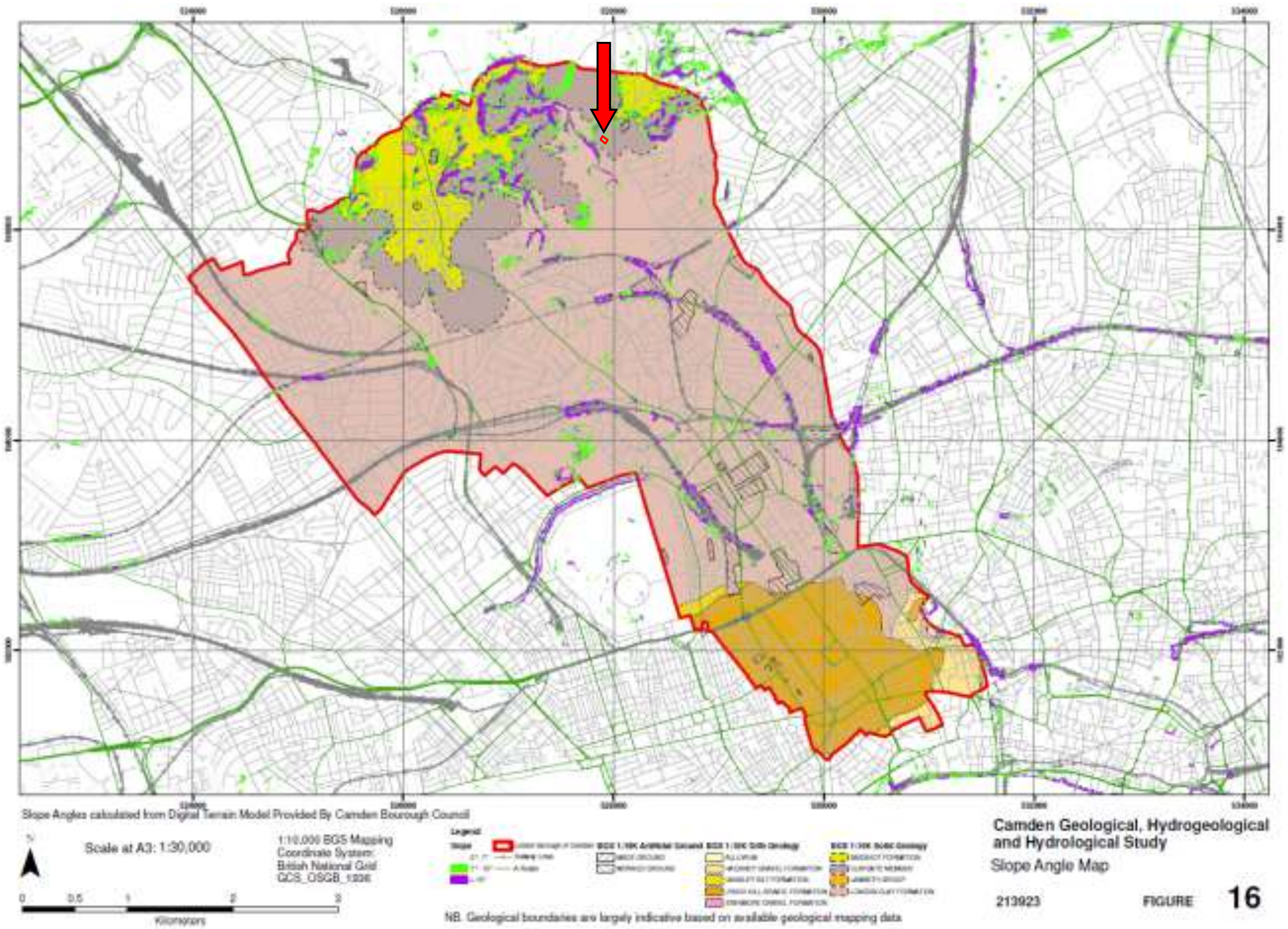

APPROXIMATE SITE BOUNDARY

NOTE: NOT TO SCALE

Project: 26 West Hill Park, London Borough of Camden, London N6 6ND		**Figure 11**
Client: Croft Structural Engineer	**Date:** November 2019	ground&water
Camden Geological, Hydrogeological and Hydrological Study - Figure 16	**Ref:** GWPR2950	

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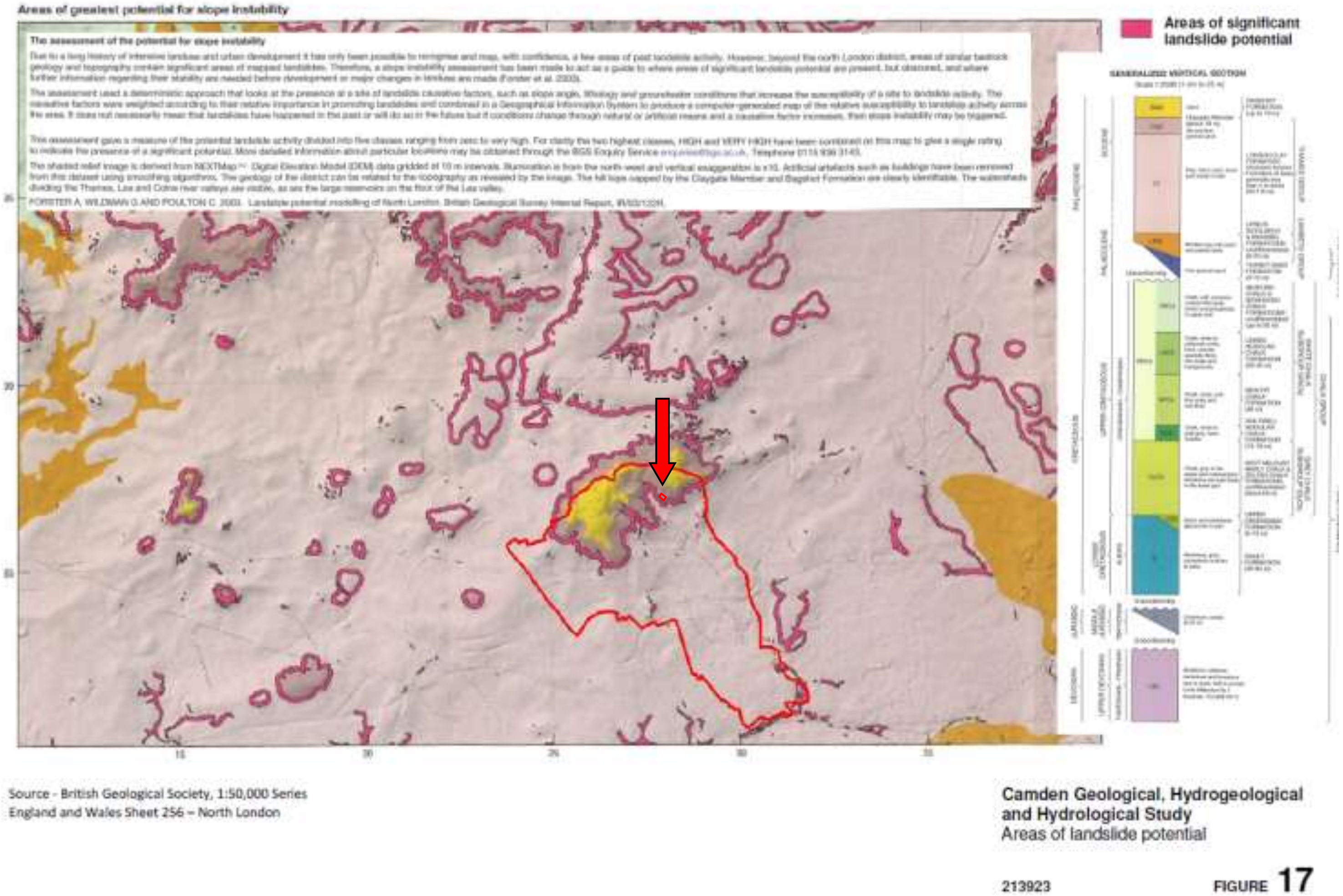

APPROXIMATE SITE BOUNDARY

NOTE: NOT TO SCALE

Project: 26 West Hill Park, London Borough of Camden, London N6 6ND	**Figure 12**
Client: Croft Structural Engineer — **Date:** November 2019	ground&water
Camden Geological, Hydrogeological and Hydrological Study - Figure 17 — **Ref:** GWPR2950	

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Source - London Borough of Camden, January 2010. *Camden Core Strategy Proposed Submission.*

Camden Geological, Hydrogeological and Hydrological Study
Transport Infrastructure

213923 FIGURE **18**

APPROXIMATE SITE BOUNDARY

NOTE: NOT TO SCALE

Project: 26 West Hill Park, London Borough of Camden, London N6 6ND	**Figure 13**
Client: Croft Structural Engineer	**Date:** November 2019
Camden Geological, Hydrogeological and Hydrological Study - Figure 18	**Ref:** GWPR2950

ground&water

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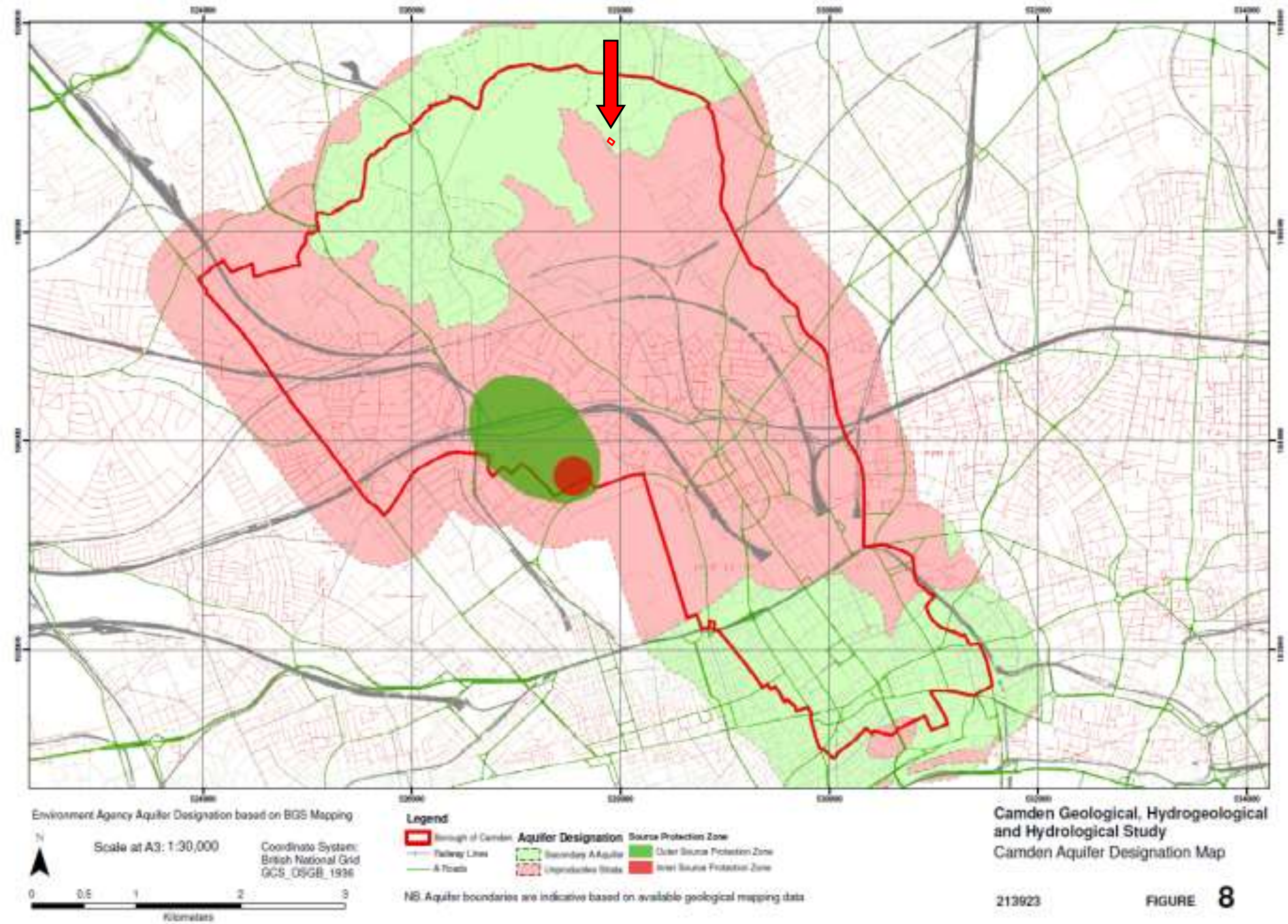

APPROXIMATE SITE BOUNDARY

NOTE: NOT TO SCALE

Project: 26 West Hill Park, London Borough of Camden, London N6 6ND		**Figure 14**
Client: Croft Structural Engineer	**Date:** November 2019	ground&water
Camden Geological, Hydrogeological and Hydrological Study - Figure 8	**Ref:** GWPR2950	

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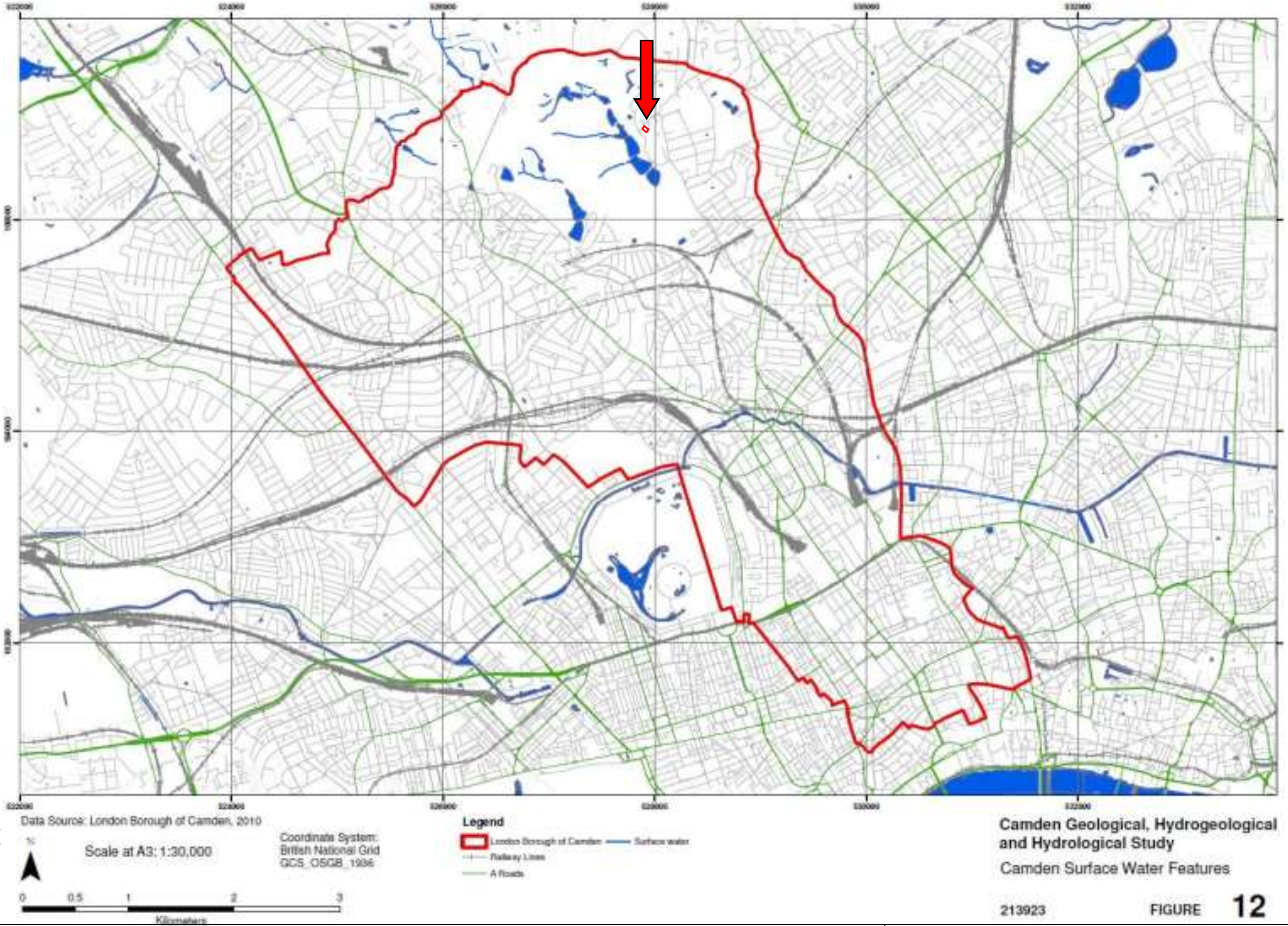

APPROXIMATE SITE BOUNDARY

NOTE: NOT TO SCALE

Project: 26 West Hill Park, London Borough of Camden, London N6 6ND	**Figure 15**
Client: Croft Structural Engineer	**Date:** November 2019
Camden Geological, Hydrogeological and Hydrological Study - Figure 12	**Ref:** GWPR2950

ground&water

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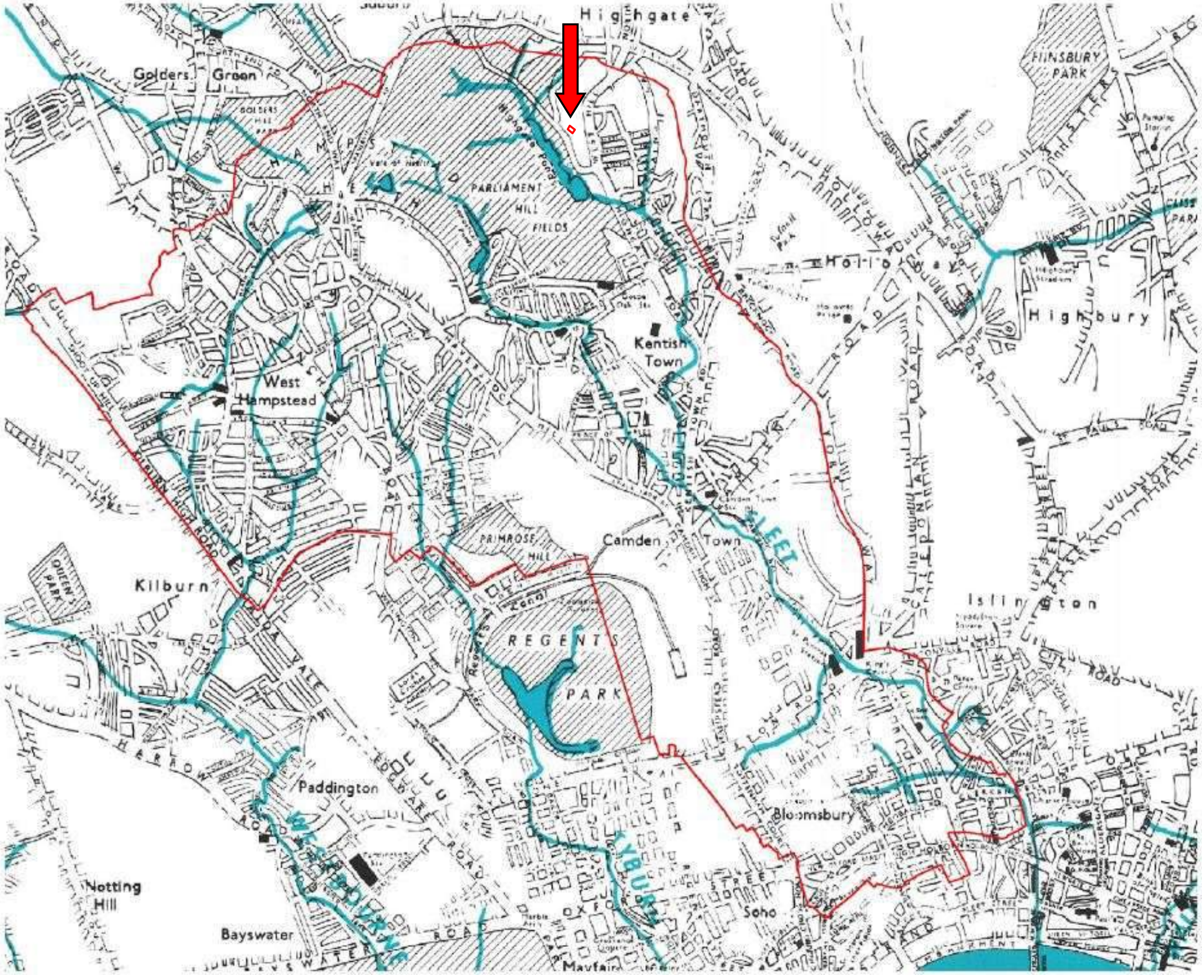

APPROXIMATE SITE BOUNDARY

Source – Barton, Lost Rivers of London

Camden Geological, Hydrogeological and Hydrological Study
Watercourses

213923 FIGURE 11

NOTE:
NOT TO SCALE

Project: 26 West Hill Park, London Borough of Camden, London N6 6ND		**Figure 16**
Client: Croft Structural Engineer	**Date:** November 2019	
Camden Geological, Hydrogeological and Hydrological Study - Figure 11	**Ref:** GWPR2950	ground&water

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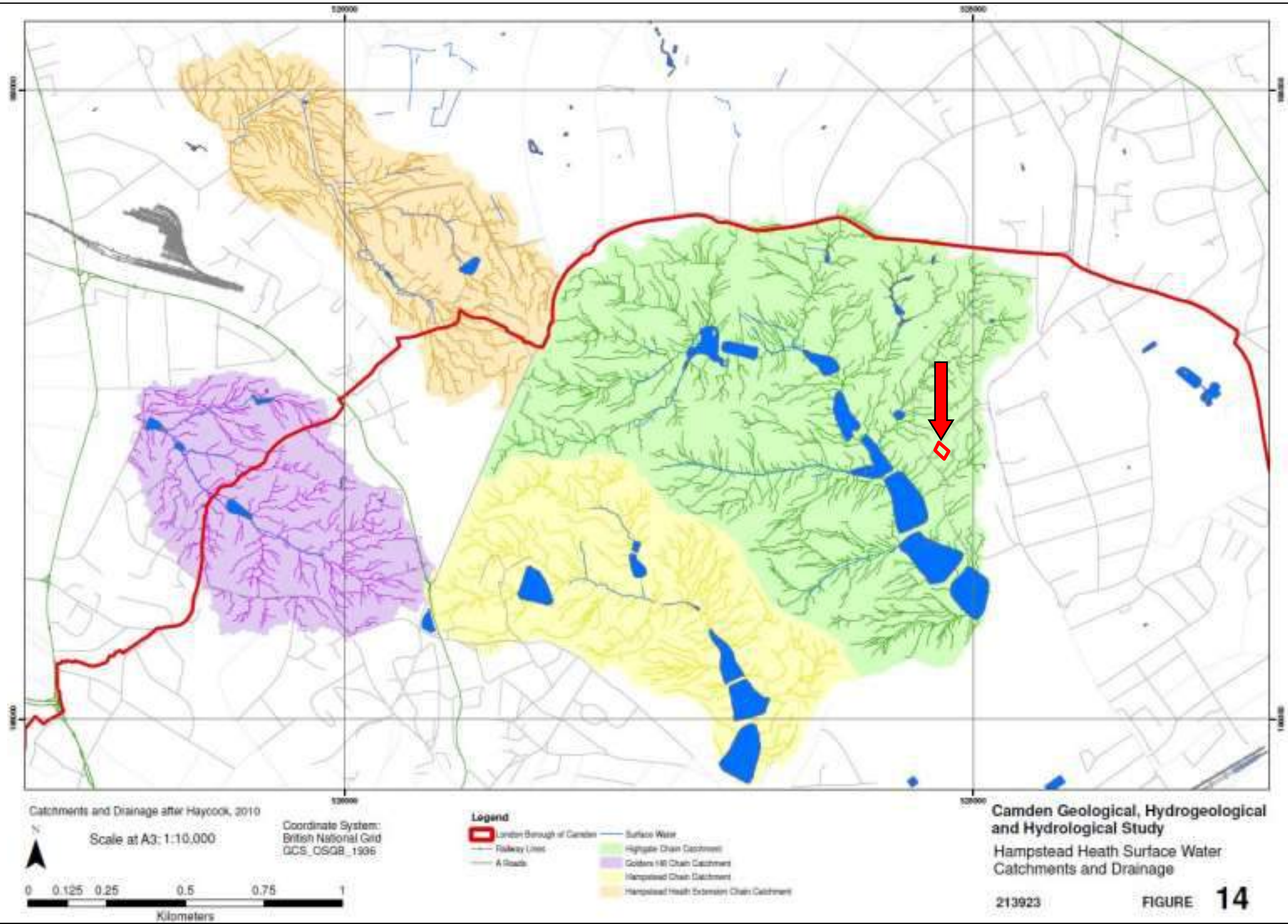

APPROXIMATE SITE BOUNDARY

NOTE:
NOT TO SCALE

Project: 26 West Hill Park, London Borough of Camden, London N6 6ND		**Figure 17**
Client: Croft Structural Engineer	**Date:** November 2019	ground&water
Camden Geological, Hydrogeological and Hydrological Study - Figure 14	**Ref:** GWPR2950	

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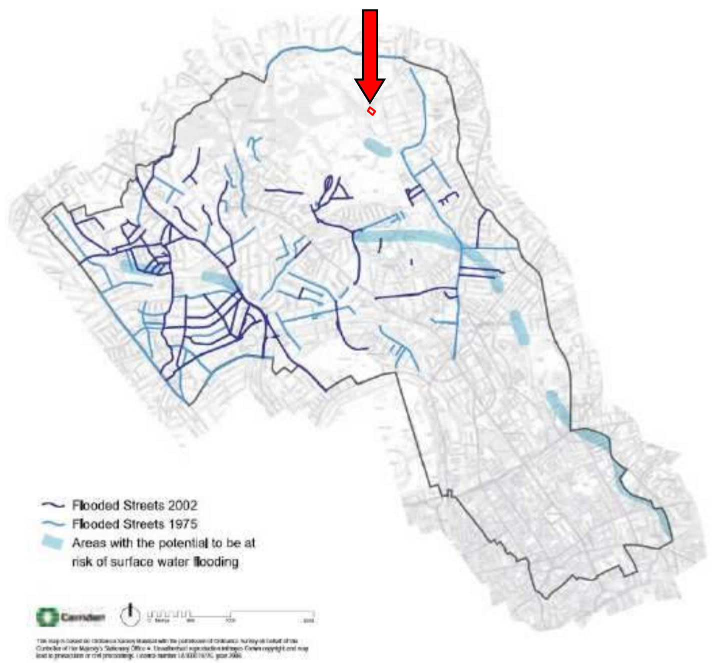

Figure 5 from Core Strategy, London Borough of Camden

Camden Geological, Hydrogeological and Hydrological Study
Flood Map

213923 FIGURE 15

APPROXIMATE SITE BOUNDARY

NOTE: NOT TO SCALE

Project: 26 West Hill Park, London Borough of Camden, London N6 6ND		Figure 18
Client: Croft Structural Engineer	Date: November 2019	ground&water
Camden Geological, Hydrogeological and Hydrological Study - Figure 15	Ref: GWPR2950	

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APPROXIMATE SITE BOUNDARY

NOTE: NOT TO SCALE

Project: 26 West Hill Park, London Borough of Camden, London N6 6ND		**Figure 19**
Client: Croft Structural Engineer	**Date:** November 2019	ground&water
Flood Map for Planning	**Ref:** GWPR2950	

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APPROXIMATE SITE BOUNDARY

NOTE: NOT TO SCALE

Project: 26 West Hill Park, London Borough of Camden, London N6 6ND		**Figure 20**
Client: Croft Structural Engineer	**Date:** November 2019	ground&water
Surface Water Flooding Map	**Ref:** GWPR2950	

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