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BASEMENT IMPACT ASSESSMENT ADDENDUM

20A FERNCROFT AVENUE

LONDON

NW3 7PH

CLIENT: Mr Elliot Graff
20A Ferncroft Avenue
London, NW3 7PH

JOB NO: P19-461

DATE: 12th March 2020 – Rev 0

Revision History

Revision	Date	Author	Checked	Notes

CONTENTS

APPENDICIES

APPENDIX A:	Geotechnical Consulting Group Ground Movement Impact Assessment Revision 1 dated February 2020
APPENDIX B:	Geotechnical Consulting Group Hydrogeological Impact Assessment Revision 1 dated March 2020.
APPENDIX C:	SIMPSON Retaining Wall Calculations Summary dated March 2020.
APPENDIX D	Campbell Reith's Basement Impact Assessment updated reply to Audit Query Tracker.

EXECUTIVE SUMMARY

This Addendum to Taylor Whalley Spyra Basement Impact Assessment (BIA) dated 12th December 2019 has been prepared in response to comments received from Campbell Reith's Basement Impact Assessment Audit ref 13398_02 revision D1 dated February 2020.

The Addendum has been provided to cover the request for further information as noted in the Audit and detailed in the Audit Query tracker to cover the following, Clarification with respect to excavation depth and nature of basement, Retaining wall calculations to reflect recommendations in Hydrogeological Assessment, Building Damage Assessment to ensure consistent with anticipated ground movement, Consideration to be given to impact of tree removal and Impact of infiltration tank to be considered.

The BIA Addendum concludes that after review of all information requested and the Geotechnical Consulting Group updated Ground Movement Assessment, confirmation of damage to adjoining properties is still Category 1 (Very Slight) in accordance with the Burland Scale and the updated Hydrogeological Impact Assessment confirming the proposed works are unlikely to have any significant impact on the local hydrogeology and on the surrounding properties.

1.0 INTRODUCTION

1.1 This Addendum to Basement Impact Assessment has been prepared by Simpson Associates as part of the Planning process (ref 2019/6220/P) and in response to Campbell Reith's Basement Impact Assessment Audit ref 13398_02 revision D1 dated February 2020.

1.2 The information contained within this Basement Impact Assessment Addendum is prepared in accordance with London Borough of Camden's Local Plan 2017, Camden Local Planning Policy A5 Basements, Camden Planning Guidance Basements March 2018, London Borough of Camden SFRA URS July 2014 and London Borough of Camden, Camden Geological, Hydrogeological and Hydrological Study.

1.3 The BIA report is authored by Chris Martin who is qualified as MEng, CEng, MIStructE. The attached GCG Hydrogeological Assessment is reviewed by J. A. Davis who is qualified as EuroGeol, CGeol, BSC, MSc, DIC, FGS The GCG Ground Movement Impact Assessment is authored by Dr Apollonia Gasparre who is qualified as Dott Ing, PhD, DIC, CEng, MICE.

1.4 The purpose of this Addendum is to provide clarification and further information as noted in Campbell Reith's Basement Impact Assessment Audit Query Tracker items 1 to 5.

1.5 Geotechnical Consulting Group (GCG) has reviewed Campbell Reith's Basement Impact Assessment Audit and has updated the Ground Movement Assessment and Hydrogeological Impact Assessment to provide additional information and clarify relevant points raised.

1.6 Simpson have reviewed the proposed foundation depths adjacent to the tree (T1) to be removed and referred to NHBC standards chapter 4.2 building near trees and also updated the calculation for increased ground water levels as noted in GCG Hydrogeological Impact Assessment.

2.0 CAMPBELL REITH BASEMENT IMPACT ASSESSMENT AUDIT QUERY TRACKER REF NUMBERS:

1. Stability Clarification required with respect to excavation depth and nature of basement retaining walls.
2. Stability Retaining wall calculations to be revised to reflect recommendations in hydrogeological assessment.
3. Stability Building damage assessment to be reviewed to ensure consistent with anticipated ground movements.
4. Stability Consideration to be given to impact of tree removal.
5. Hydrogeology / Hydrology Impact of infiltration tank to be considered.

3.0 RESPONSE TO ADDITIONAL INFORMATION REQUESTED BY CAMPBELLREITH BASEMENT IMPACT ASSESSMENT AUDIT

3.1 Item 1

The proposed foundations at the new extension and rear garden terrace are set 1.85m below existing ground level, reference to NHBC standards chapter 4.2 building near trees and a review of proposed foundations depths adjacent to the Cherry Tree T1 to be removed require a minimum foundation depth of 1.85m.

The proposed main basement retaining wall excavations are approximately 3m and 3.5m below the existing stepped ground floor levels.

3.2 Item 2

The retaining wall calculations are based on existing ground water levels, but as recommended within the Hydrological assessment the retaining walls will be designed for a higher water level with the ground water level set 1m below top of retained ground level.

3.3 Item 3

The building damage assessment accounts for the predicted ground movements, which include settlements of the perimeter walls due to the underpinning process (i.e. 5-10mm localised under the walls) and settlements and horizontal movements due to excavation (i.e. up to 5mm and reducing with distance from the retaining walls – see figures 9 and 10).

The trigger levels quoted in the report refer to measurements taken on the retaining structures and account for the movements described above. They are conservatively assessed in the interest of the neighbouring properties and assume that good workmanship is adopted throughout the construction process at 20a Ferncroft Avenue.

3.4 Item 4

A paragraph has been added to the GMA.

The new structures at 20a Ferncroft Avenue will be designed accounting for the removal of the Cherry tree and the presence of the other trees in the garden, in accordance with NHBC standards.

The removal of the Cherry tree has the potential to cause ground heave that could affect 20 Ferncroft Avenue. However, the tree to be removed is currently in poor condition, with restrained root grow. It is therefore likely that its current water demand is very limited and its removal would have small effects on the ground and nearby structures.

3.5 Item 5

The hydrogeological report refers to a SUDS system, which coincides with the attenuation tank mentioned in the main text of the BIA. This has been clarified in the main text of the BIA.

The impact of the SUDS/attenuation tank is discussed in the hydrogeological report (sections 4.3 and 6).

4.0 CONCLUSIONS

4.1 Geotechnical Consulting Group (GCG) have reviewed the additional information within the Addendum Basement Impact Assessment and provided updated Ground Movement Assessment (GMA) confirming that damage to adjoining neighbours will not exceed Category 1 (Very Slight) (refer to Appendix A).

4.2 Geotechnical Consulting Group (GCG) have reviewed the additional information within the Addendum Basement Impact Assessment and provided updated Hydrogeological Impact Assessment confirming the proposed works are unlikely to have any significant impact on the local hydrogeology and on the surrounding properties (refer to Appendix B).

4.3 The location and removal of the existing Cherry Tree (T1) has been reviewed against NHBC guidelines for proposed foundation depth. We confirm the removal of the tree will not have any undue effect on adjacent building foundations in the short or long term.

4.4 Updated retaining wall calculation are attached for increased ground water level as noted in GCG Hydrogeological Impact Assessment (refer to Appendix C).

4.5 Analysis of the various aspects of construction has been undertaken to demonstrate how the level of sequencing will enable the development to be constructed safely with ground movements within acceptable levels.

4.6 The project as currently envisaged is feasible in terms of the general construction process, structural stability, long term integrity of adjacent buildings and the existing site and surrounding infrastructure.

For and on behalf of

SIMPSON ASSOCIATES

CHRIS MARTIN
MEng, CEng, MIStruct

For and on behalf of

SIMPSON ASSOCIATES

GRAHAM BOSTON

Appendix A

Geotechnical Consulting Group Ground Movement Impact Assessment Revision 1
Dated February 2020

MR AND MRS GRAFF

20A FERNCROFT AVENUE

GROUND MOVEMENT IMPACT ASSESSMENT

REVISION 1

February 2020

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MR AND MRS GRAFF

20A FERNCROFT AVENUE

GROUND MOVEMENT IMPACT ASSESSMENT

REVISION 1

February 2020

DISCLAIMER

REVISION HISTORY

Revision	Date	Description	Produced by	Reviewed by
Rev 0	4/12/19	First Issue	A.Gasparre Dott Ing. PhD DIC CEng MICE	Dr. P. Smith BEng MSc DIC PhD
Rev 1	5/3/20	Comments on trees added	A.Gasparre Dott Ing. PhD DIC CEng MICE	Dr. P. Smith BEng MSc DIC PhD

MR AND MRS GRAFF

20A FERNCROFT AVENUE

GROUND MOVEMENT IMPACT ASSESSMENT

REVISION 1

FEBRUARY 2020

EXECUTIVE SUMMARY

A ground movement impact assessment has been undertaken for the site at 20a Ferncroft Avenue, where the ground floor of the existing house is to be extended and a basement with a front light well is to be created underneath the footprint of the original house and its rear patio.

The proposed basement will be constructed by underpinning the perimeter walls.

Ground movements associated with the proposal have been estimated using linear elastic analyses and an empirical method based on records of basement excavations. It is concluded that movements of the ground around the surrounding structures are small and as a result, predicted building damage will not exceed Category 1: very slight.

No impact on the highway and any utilities running along this is expected.

MR AND MRS GRAFF

20A FERNCROFT AVENUE

GROUND MOVEMENT IMPACT ASSESSMENT

REVISION 1

February 2020

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1 Introduction

It is proposed to extend the ground floor of the existing house at 20a Ferncroft Avenue and construct a basement underneath the original footprint of the house and its rear patio.

The Geotechnical Consulting Group LLP (GCG) have been commissioned to assess the impact of the proposed basement construction on the surrounding structures.

The expected movements around the site have been estimated using linear elastic analyses and an empirical approach that is based on field measurements of movements from a number of basement constructions across London (CIRIA C760).

Information on the project has been provided by Simpson Associates, the structural engineers for the project.

2 The site and the proposed redevelopment

The site lies within the Frognal & Fitzjohn Ward of the Camden Administrative Boundary and is located on the north side of Ferncroft Avenue, at approximately 200m to the south of West Heath (Figure 1a).

It stretches approximately 45m along a north-east to south-west direction and it is approximately 6m wide.

It includes an 8m long paved driveway at the front, a semidetached mansion house with a patio and a rear garden. Figure 1b shows a layout of the site.

The house is approximately 15m long and 6m wide. The ground level at its front is approximately 1m above the street level and the rear section of the house steps further up by approximately 0.5m. The survey drawings show that the ground level (to a Relative Datum RD) across the front section of the house is approximately +51mRD and it is approximately +51.5mRD across the rear section. It should be noted that these relative levels, also reported in the structural drawings, are approximately 50m below the Ordnance Datum (OD).

Figure 2 shows a plan of the existing ground floor and a north-east to south-west structural section through the existing house.

It is proposed to extend the existing ground floor to the rear of the house and create a new basement underneath its original footprint and the rear patio, with a lightwell at the front. Figure 3 shows a plan of the proposed basement and a section through the site.

The finished floor level of the new basement will be approximately +48mRD (i.e. +98mOD) and will require approximately a 3m deep excavation underneath the front of the house (considering the depth to the foundations) and 3.5m deep excavation underneath the rear part of the house and the patio.

The basement will be formed by underpinning the perimeter walls.

It is understood that a tree within the footprint of the new ground floor extension is to be removed and other trees, currently in the garden, will be retained. The new foundations of the ground floor extension will be designed accounting for the trees in accordance with the NHBS standards.

3 The surrounding structures

The proposed basement construction could cause ground movements that extend to the surrounding structures. Those that could be most affected are the adjacent properties 20 and 22 Ferncroft Avenue.

3.1 20 Ferncroft Avenue

This property is to the west of the site. It includes a three storey semidetached masonry house with mansard roof, a front driveway and a rear garden.

The house is approximately 12m x 17m in plan and is set approximately 8m back from Ferncroft Avenue. It shares its eastern wall with 20a Ferncroft Avenue and the ground level at its front is approximately 1m above street level (Figure 4a).

The property does not appear to have a basement. For the purposes of this assessment its walls will be conservatively assumed to be founded at 0.5m below ground level (bgl) and the property will be assumed to be in good structural condition.

3.2 22 Ferncroft Avenue

This property is to the east of 20a Ferncroft Avenue. It includes a detached masonry house with front driveway and rear garden.

The house is approximately 20m x 12m in plan and it is set about 8m back from Ferncroft Avenue and 3m away from the boundary with 20a Ferncroft Avenue (Figure 1b). A garage, approximately 3m x 7m in plan, is to the west of the house and extends to the boundary with 20a Ferncroft Avenue.

The house has recently undergone refurbishment, which included the deepening and extension of a previous cellar to form a basement that extends underneath the footprint of the house except the front hallway. The basement was formed by underpinning the walls of the house to a depth of approximately 3m bgl. The garage was also constructed as part of these refurbishment.

A section through the house is included in Figure 4b.

4 Ground Conditions

A detailed assessment of the ground and groundwater conditions across the site is provided in the GCG's Hydrogeological Impact Assessment (Geotechnical Consulting Group, 2019).

In summary, the site is on the edge of a ridge on ground sloping to the south-west and south-east. The ground stratigraphy under the site includes Claygate Member, expected to extend to approximately 10m depth and London Clay, expected to extend to depths in excess of 60m (Figure 5).

The Claygate Member at the site includes mainly clayey deposits with a layer of sand, found from 4.5m depth at the front of the site. At the site this is approximately 1.5m thick and appears to dip and reduce in thickness westwards and northwards.

Groundwater was found in this sandy layer at approximately 5.3m depth. However, perched water might also be present at shallow depths within the clayey deposits of the Claygate Members. This is expected to flow in a discontinuous manner across the site.

5 Ground movement analyses

5.1 Background

The construction method for the redevelopment envisages that, having removed the ground slab, the party walls will be underpinned and the shallower sections of the front lightwell wall and the rear basement wall will be formed using underpinning techniques. The internal walls of the house will then be picked up by Pynford beams, supported either on the underpinnings or on the new ground floor slab, which will be formed in bays as the excavation of the new basement proceeds. It is understood that excavation will commence from the front and rear sections of the new basement after completion of the deeper sections of the front lightwell wall and the rear basement wall.

Inside and outside the basement area ground movements during and after the works would be due mainly to:

- Underpinning of the perimeter walls
- Excavation for the extension of the basement, which would induce a reduction of vertical and lateral stresses in the ground along the excavation boundaries.

The magnitude and distribution of the ground movements caused by these operations are a function of changes of load in the ground and workmanship. The way that the existing buildings around the site respond to these movements is dependent on their current conditions and the precautions that are taken to reduce the risk of building movements.

Ground movements inside the basement area should be accounted for in the design of the new basement structure.

5.2 Estimated ground movements

5.2.1 Underpinning along party walls

The underpinning will be approximately 3m deep under the front section of the house and 3.5m deep at the rear.

The construction of the underpinning would induce ground movements due to the transfer of vertical loads from the current to deeper foundation levels and to the sequential excavation of underpinning slots.

The loads transferred from the existing to the new foundation levels during the underpinning process are in the order of 110kN/m to 135kN/m along the party walls and 25-29kN/m along the front and rear walls. The ground movements due to this load transfer have been estimated using PDisp.

The program assumes a linear elastic behaviour of the soil and determines the changes in the vertical stresses and settlement/heave using a Boussinesq approach. Elastic vertical strains are calculated on the basis of the calculated stress changes and then integrated to obtain vertical movements. The calculations represent free field movements unaffected by the stiffness of structures and therefore are likely to be conservative. The soil parameters used for the analyses are summarised in Appendix 1.

The results of the analyses show that the movements due to the transfer of loads during underpinning are settlements of the party walls in the order of 1-2mm (Figure 6).

Based on experience, the construction of shallow underpinning under relatively lightly loaded walls carried out with good workmanship and in the dry can induce negligible horizontal movements and localised settlements of the underpinned wall only in the order of 5-10mm.

Considering the depth of the proposed underpinning at 20a Ferncroft Avenue and assuming that the works will be carried out with good workmanship and in the dry, the expected settlements could be limited to 5mm. Horizontal movements would be expected to be negligible provided that the back face of the pits is shored.

However, perched water could be present at shallow depths and should dewatering be required, additional ground movements could occur as a consequence of the loss of fines during this operation. The magnitude of these movements is difficult to predict as it depends on the occurrence of water ingress and its amount. Care should be taken during the dewatering process to limit ground movements. It is recommended that an observational method based on the monitoring of the underpinned walls is implemented to control ground movements.

If necessary, ground treatment could be employed to enable the works to be carried out in the dry.

For the purposes of this assessment it is assumed that the ground movements due to underpinning can be controlled to be less than 10mm. Any damage caused by these movements will be localised to the underpinned walls and should be capable of being repaired afterwards.

The new underpinned walls will have to be designed to retain the ground accounting for the surcharge of the structures behind.

5.2.2 Movements due to excavation

The excavation would cause upward ground movements inside the excavated area as a result of the vertical change (reduction) of loads on the excavated surface and downwards movements outside the excavated area as a result of the deflections of the retaining walls due to the loss of horizontal support in front of them.

The ground movements inside the excavated area have been estimated using PDisp.

The pressures removed as a result of excavation are approximately 60kPa across the front of the house and 70kPa across the rear. The results of the analyses show that at the end of the excavation the ground would move upwards by 4-5mm in the central part of the site and 1-2mm along the edges (Figure 7). It should also be noted that the loads of the internal walls will be transferred onto the underpinnings as excavation proceeds. This will increase the loads on the underpinning causing some additional settlements of the walls and reducing the global swelling of the ground. Conservatively, these movements are not considered in this stage.

Behind the retaining walls the ground would settle and move towards the excavation as the walls bend due to the reduction of lateral support in front of them. Empirical data based on the movements of ground behind retaining walls as a result of excavations in typical London ground conditions (CIRIA C760) show that the ground movements behind the excavation depend on the propping sequence and on the depth of the excavation (Figure 8). It should also be noted that the CIRIA's database refers to embedded retaining walls, but there is a lack of reliable data on ground movements behind underpinning so the same plots are typically used also for underpinning.

At the site Claygate Members are present, which are not typically found in Central London. However, the nature of the soil is such that the CIRIA's database is applicable.

The new ground floor slab will be formed prior to the excavation across the site, which will provide support to the retaining walls. The data in Figure 8 suggest that for a 3m deep excavation the maximum settlements are in the order of 2mm and the maximum horizontal movements are approximately 4mm. Across the rear of the house, where the excavation is slightly deeper, the maximum settlements due to the 3.5m excavation would be expected to be approximately 3mm and the maximum horizontal movements would be expected to be less than 5mm.

These movements would occur behind long sections, at the corners they would be restricted to about half of the predicted values.

The ground behind the walls would tend to sag and therefore the maximum settlements would occur at approximately 1.5-2m behind the walls.

The ground movements due to excavation would add to those due to the construction of the underpinning.

Contour plots of the total predicted ground movements due to excavation only around the new basement area have been constructed and are shown in Figures 9 and 10.

Settlements of the underpinned walls due to the underpinning process are to be added to those shown.

5.2.3 Long term movements

At the end of construction the new structural loads will be carried by the new basement raft connected to the underpinnings. This will cause some small settlements that would tend to reduce the swelling under the central of the ground caused by the excavation (Figure 11a).

In the long term the ground will continue to move as an effect of the net change of pressures caused by the redevelopment. Figure 11b shows an approximate distribution of the expected long term movements across the new basement. In the central part of the site the ground could swell up to 15mm, while along the walls it would settle up to 5-6mm.

It should be noted that PDisp does not account for the stiffness of the structures and therefore in reality the distribution of movements would be smoother than predicted in Figure 11. Also, the connection between the underpinning and the new basement raft will tend to distribute the new structural loads more uniformly.

The basement raft should be designed for the predicted upwards movements occurring from its construction, which can be estimated from the difference between the movements in Figure 11b and those in Figure 7 (end of excavation).

6 Discussion of results

6.1 Effects of ground movements on adjacent structures

The predicted ground movements due to the redevelopment of the site will cause distortions of the ground that could affect the surrounding structures. The potential damage to these structures can be estimated as suggested in CIRIA C760 by looking at the combined effects of the horizontal strains and the deflection ratio, which is the ratio between the maximum distortion of a structure and its length.

These effects are discussed below:

20 Ferncroft Avenue

The results of the assessment show that the party wall of this house would tend to settle as the wall is underpinned. These movements could cause cracks to develop at the junctions of this wall. Assuming that the movements during the underpinning operations are well controlled and the settlements of this wall do not exceed 5mm, their impact is unlikely to be of concern for the structure.

During excavation the expected ground movements would tend to cause distortions in the order of 0.0125% across the front of the house and 0.02% across its rear. The tensile strains will be just under 0.04%. These would induce a potential damage that can be classified as no worse than Category 1 in the Damage Category Table shown in Figure 12.

In the long term no significant ground movements are expected outside the boundary of the site that can be of concern for the existing structure on 20 Ferncroft Avenue.

The removal of the Cherry tree in rear garden of the property has the potential to cause ground heave that could affect 20 Ferncroft Avenue. Cherry trees have moderate water demand and the tree to be removed is currently in poor condition, with restrained root grow. Its current water demand is therefore likely to be limited and its removal would therefore be expected to have small effects on the ground and the nearby structures.

22 Ferncroft Avenue

The underpinning of the party walls at 20a Ferncroft Avenue is unlikely to affect the main house on 22 Ferncroft Avenue, which, at its closest approach is 3m away from the site boundary.

The garage immediately adjacent to the rear section of the proposed basement could be affected by the underpinning activities. However, the garage has an independent wall and will not be directly underpinned. Considering that the movements due to underpinning will be controlled, the movements associated to this activity are not likely to have significant effects on the garage.

During excavation the expected ground movements across 22 Ferncroft Avenue are such that the garage would tend to tilt away from the excavation experiencing distortions that would give rise to deflection ratio of approximately 0.03% and horizontal strains of 0.05%. These strains could cause damage that could be classified well within Category 1 (very slight) in the Damage Category Table shown in Figure 12.

The main house is founded at a level similar to the proposed basement level on 20a Ferncroft Avenue. It is therefore unlikely to experience any significant movement from the works at the site. The potential damage on the main house could therefore be classified as Category 0.

Other surrounding structures and infrastructures

The contour plots in Figure 9 and 10 show that limited ground movements are expected across Ferncroft Avenue (i.e less than 2mm horizontally and <1mm vertically).

The impact of the predicted movements across Ferncroft Avenue and any utility running underneath is expected to be negligible.

6.2 Monitoring

It would be prudent to monitor movements during construction. Monitoring targets could be installed on the walls of the existing house and on the adjacent properties and on the retained structures. Base readings should be taken before work commences.

In the different stages of the construction movements could be small and maybe within the limits of the measurement accuracy. Therefore it is suggested that only overall trigger levels are applied to movements of the walls.

Based on the predictions discussed above, the following trigger levels on the horizontal and vertical movements of the retaining structure are suggested:

Trigger Level	Movements [mm]
green	<7
amber	7-10
red	>10

It should be noted that the movements above account for the total expected movements, which include the settlements due to underpinning and the movements due to excavation.

7 Slope stability issues

The Hampstead area and the surroundings are considered to be vulnerable to slope instability due to the ground conditions and the sloping gradient of the ground.

Potential land instability has generally been associated to slopes of 8° or greater both in the London Clay and in the Claygate Member (Denness et al., 1976; Ellison et al. 2004) although the mechanisms that could drive the potential instability are different in the two types of soils.

Figure 13 shows the areas that are prone to slope stability issues as mapped by the British Geological Survey (BGS) (Arup, 2010). The BGS mapping is based on factors such as geology and groundwater conditions, in addition to the slope angle.

The specific site conditions at 20a Ferncroft Avenue do not suggest that issues with general land stability exist.

The maximum slope of the ground across the site is approximately 2^o and will not be altered.

The retaining walls of the new basement will be designed for the surcharge of the existing structures and the ground behind.

During construction the walls will be propped and "out of balance forces" will be partly resisted by the ground in direct bearing and sliding ("passive" resistance) of the opposing wall or transmitted through the side walls to the soil in shear. In the permanent condition there will be no additional global "out of balance forces" over and above those present in the temporary condition.

Given the hydrological conditions of the site, it is unlikely that pore water pressure increase in the clayey units of the Claygate Member could cause instability of the ground.

8 Conclusions

The impact of the proposed basement construction on the surrounding structures has been assessed using empirical methods and linear elastic analyses.

The excavated area will be subjected to upward movements due to the net load changes following the basement excavation. The design of the basement foundation should be carried out considering these load changes and the associated movements.

Providing that good workmanship and a robust construction sequence are used and that full support from high level is provided to the retaining walls during excavations, the basement construction is unlikely to cause settlements and horizontal strains that would induce other than limited damage to the surrounding structures.

Monitoring of movement during construction is recommended.

9 REFERENCES

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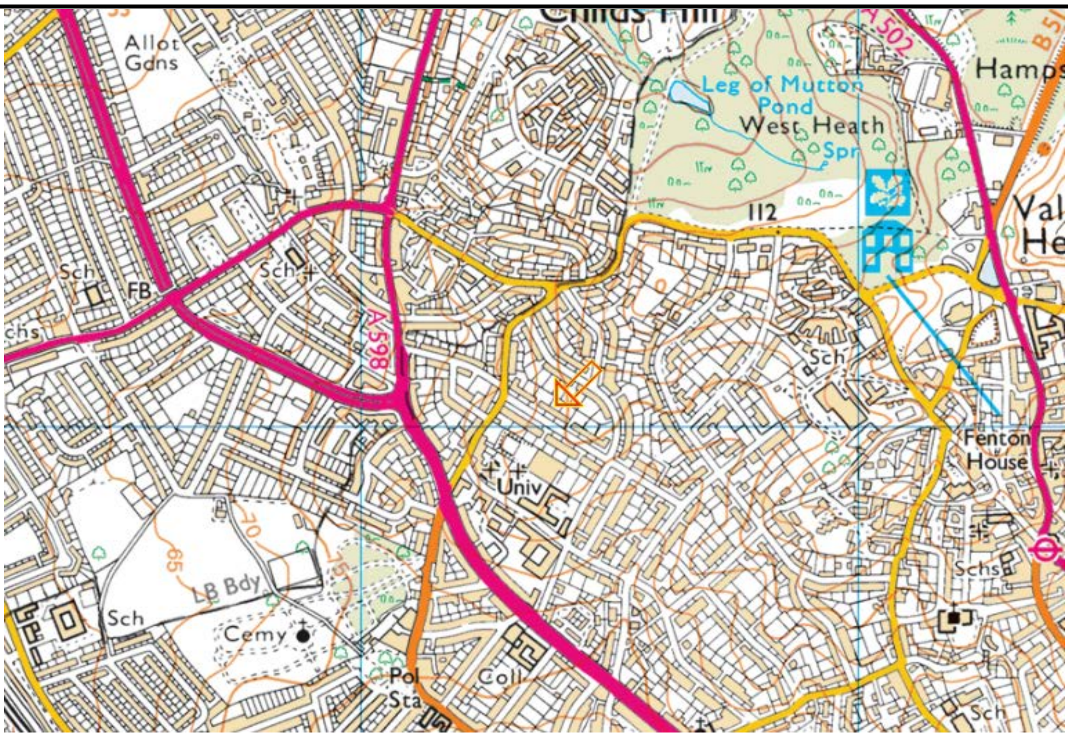
FIGURES



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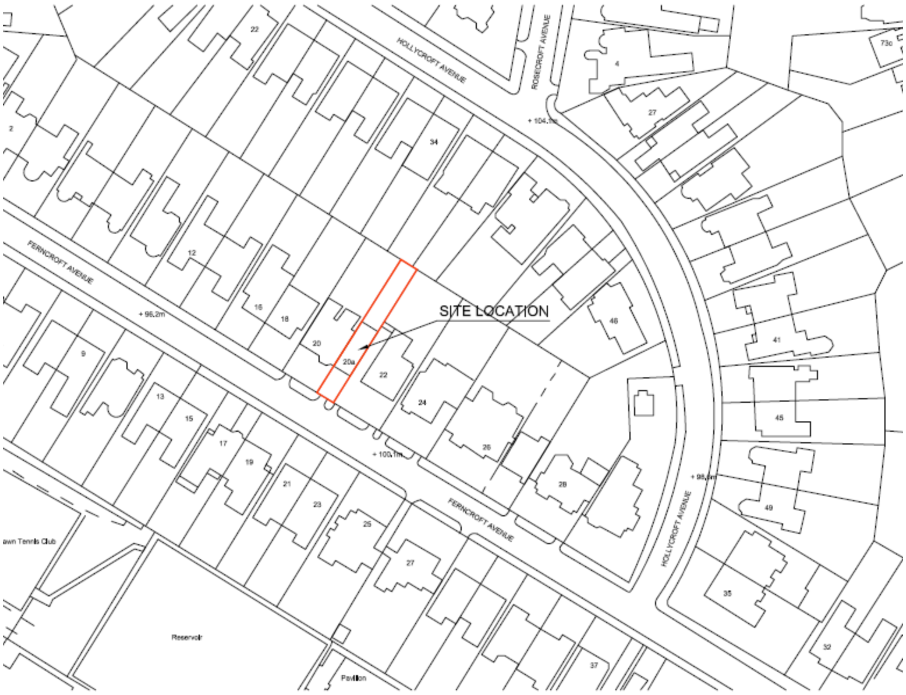
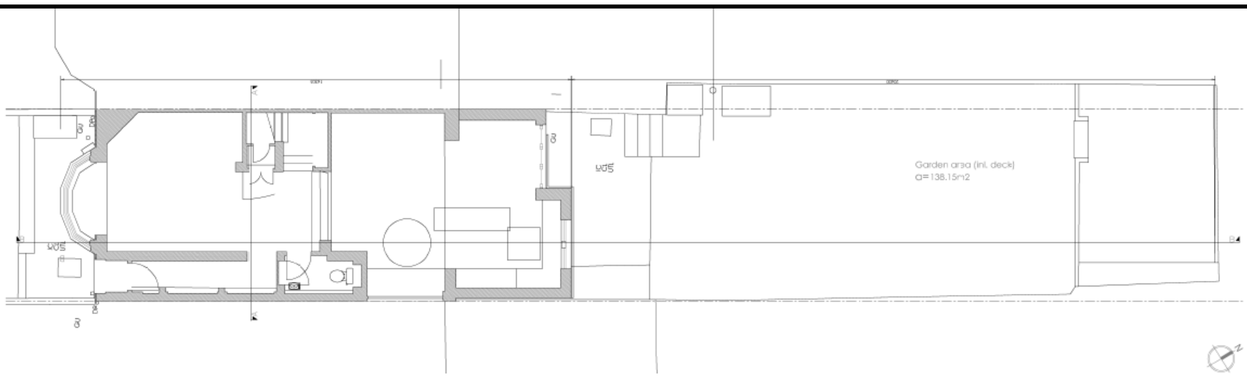


	Mr and Mrs Graff 20a Ferncroft Avenue	**Figure**
	The Site a) Location b) site layout	**1**



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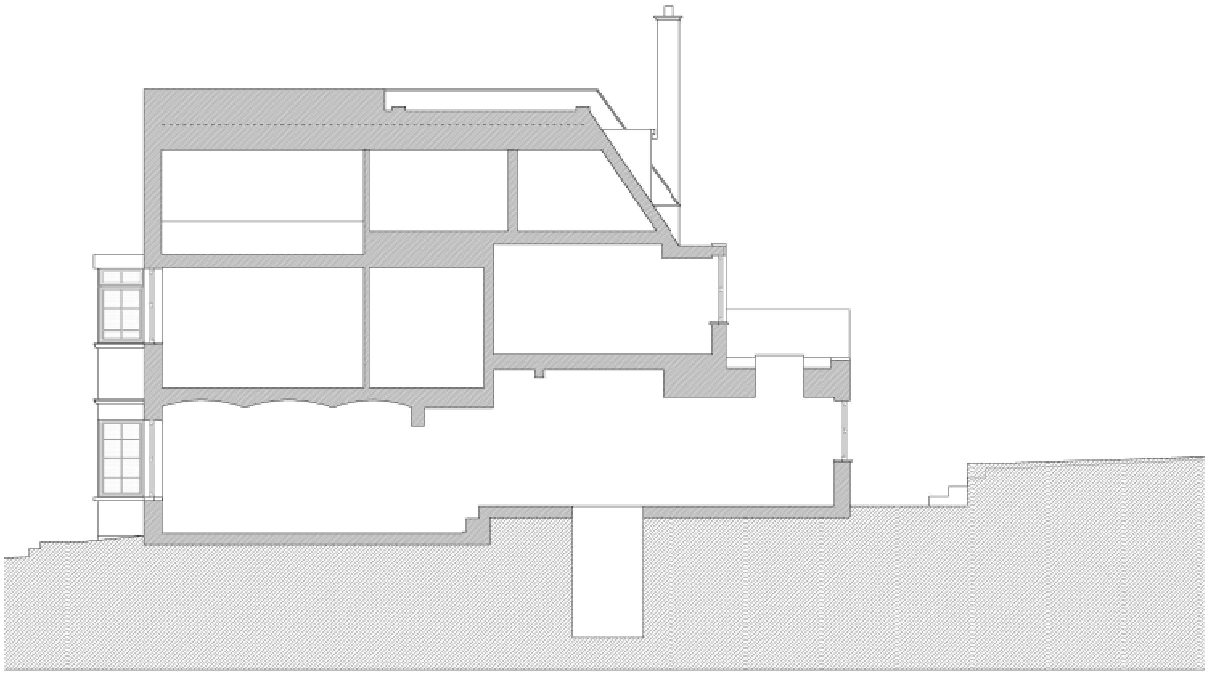
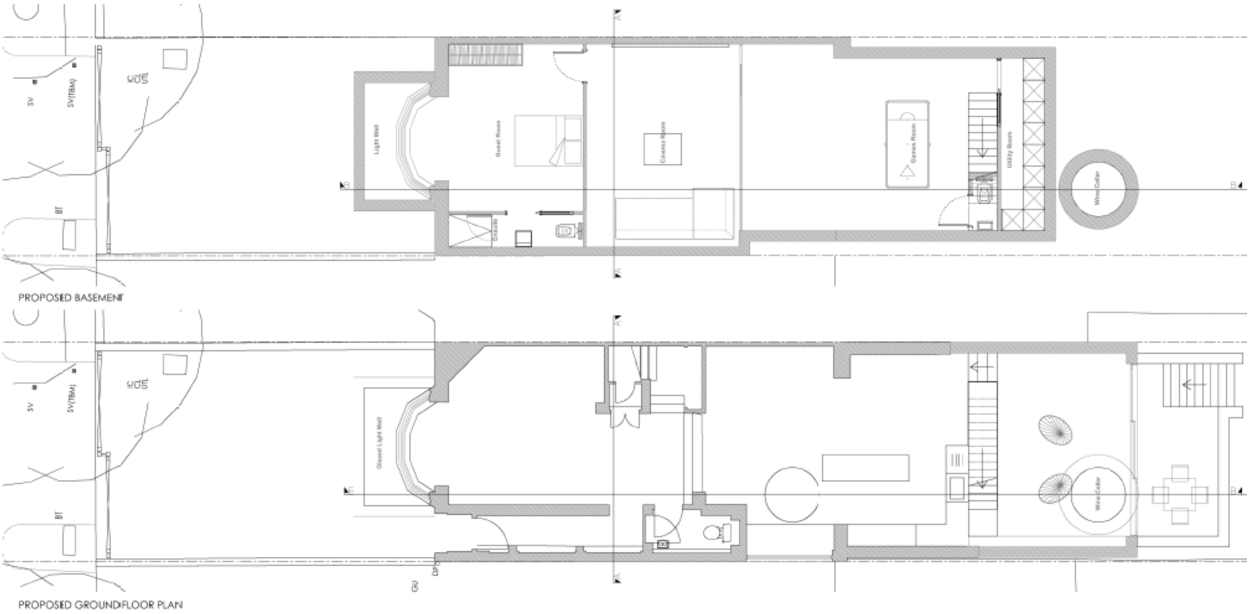


(b)

	Mr and Mrs Graff 20a Ferncroft Avenue	**Figure** **2**
	The existing property (a) Plan of lower ground floor (b) north-east to south-west structural section	



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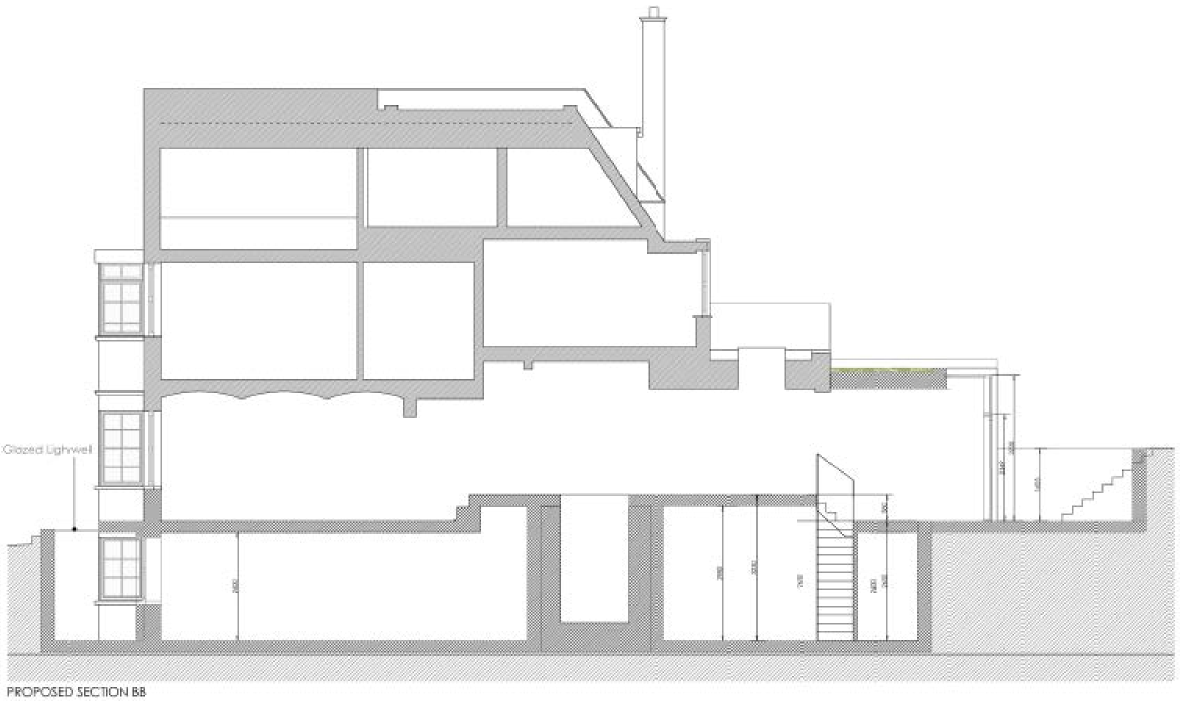
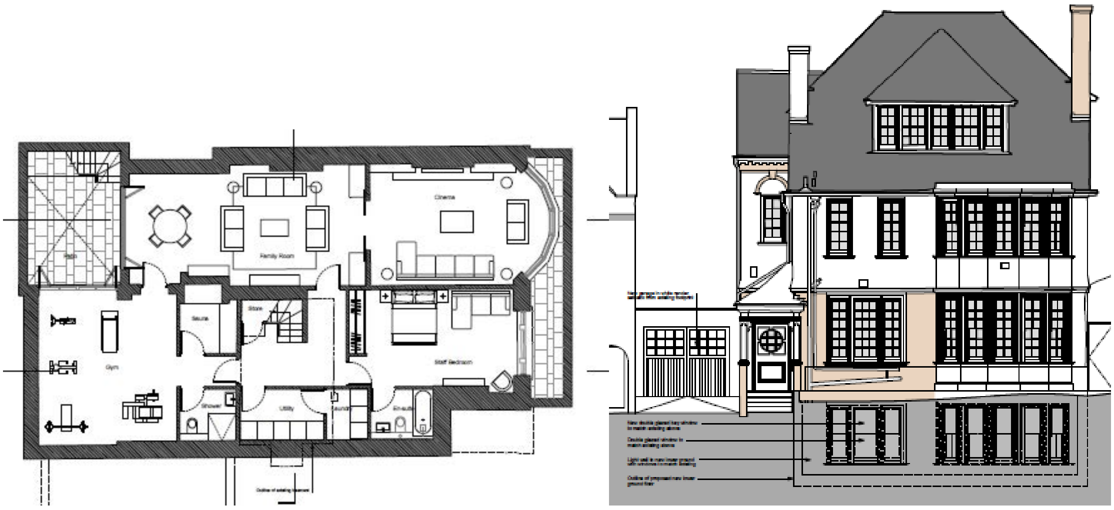


(b)

GCG	Mr and Mrs Graff 20a Ferncroft Avenue	**Figure** **3**
	The proposed structure (a) Plan of basement (b) north-west to south-east section	

(a)



(b)

Mr and Mrs Graff

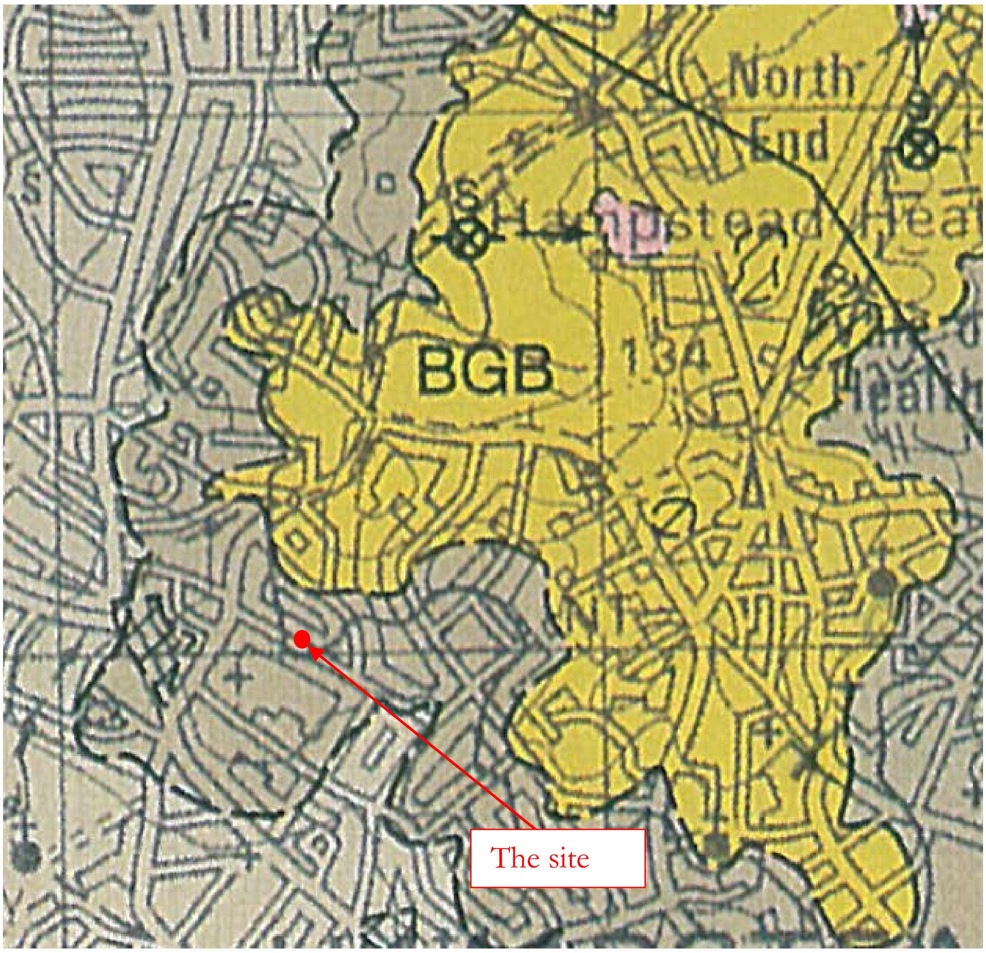
20a Ferncroft Avenue

The adjacent properties on Ferncroft Avenue

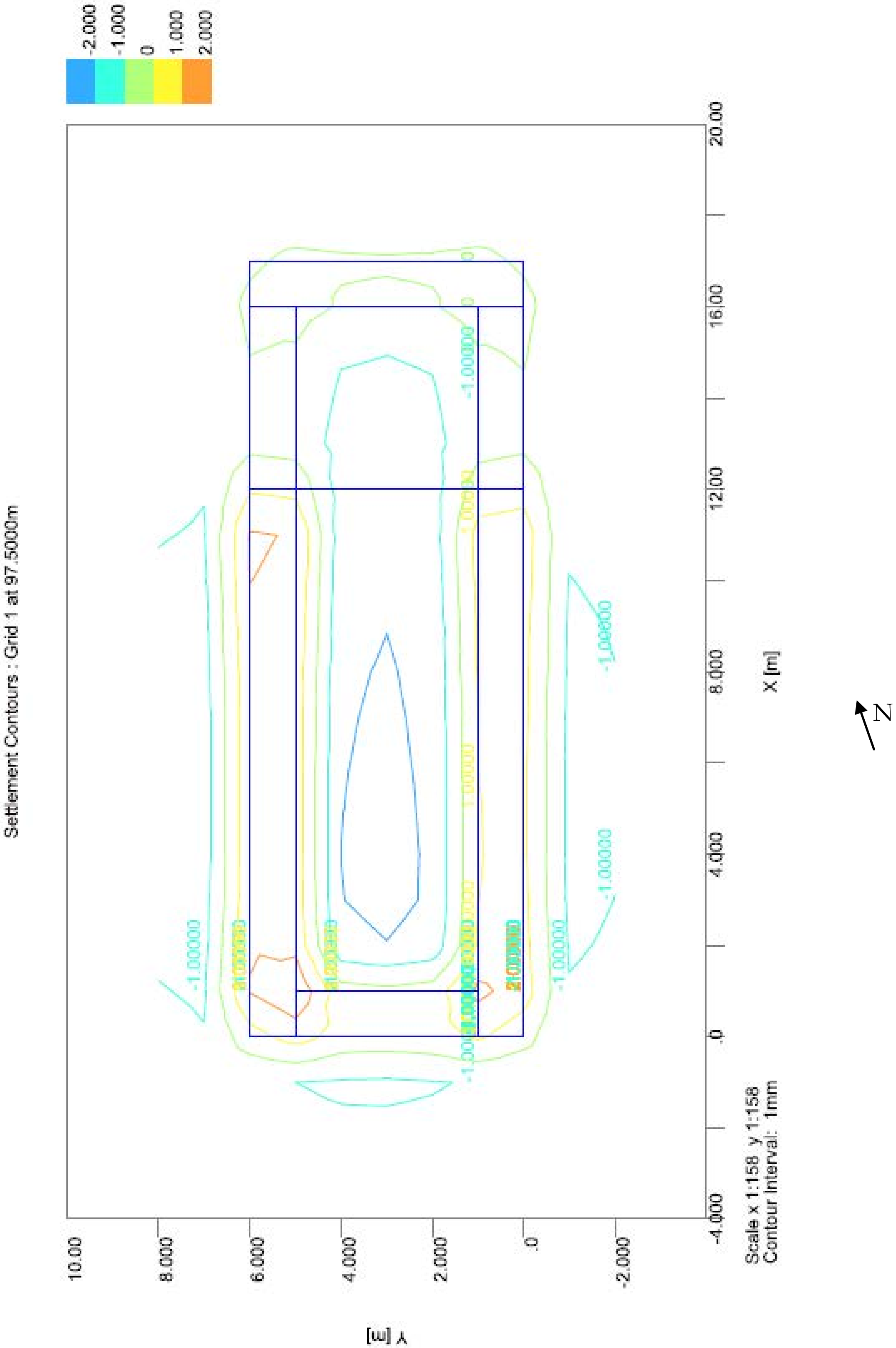
a) No.20: b) No.22 Ferncroft Avenue

Figure

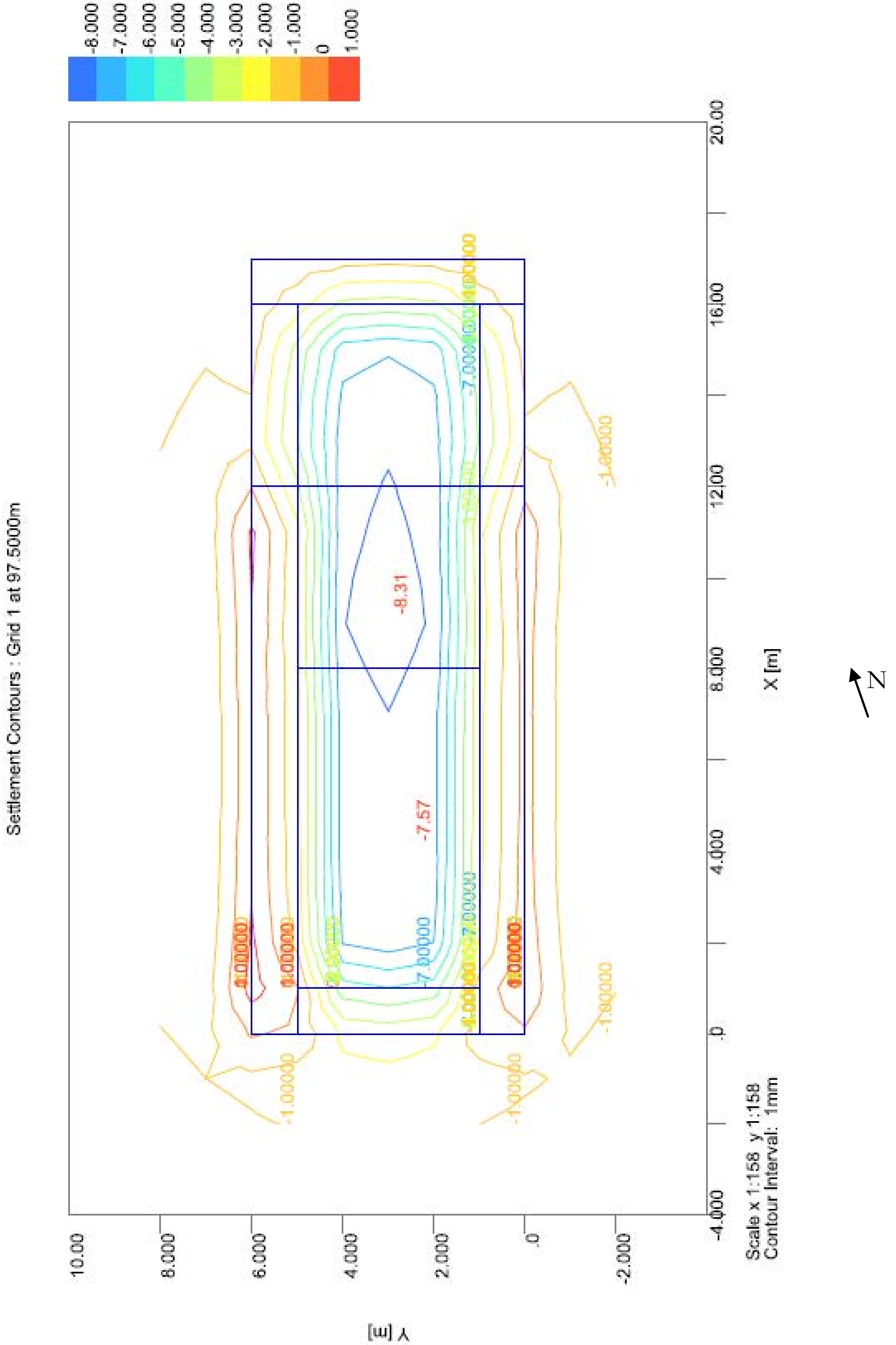
4



GCG	**Mr and Mrs Graff** 20a Ferncroft Avenue	**Figure** **5**
	Extract from The BGS Map North London, England and Wales Sheet 256. Solid and Drift Geology 1: 50,000.	



GCG	**Mr and Mrs Graff** 20a Ferncroft Avenue	**Figure** **6**
	Predicted ground movements due to transfer of loads during underpinning	



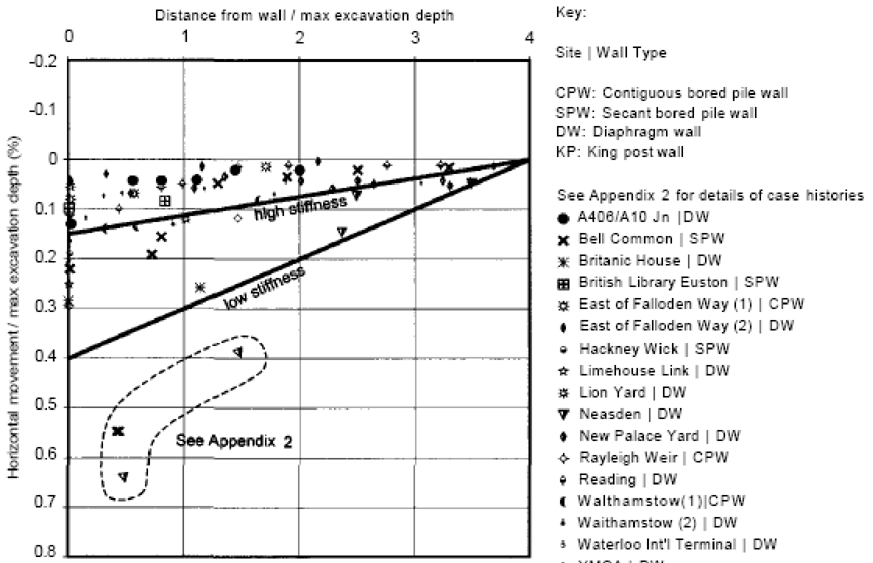
Mr and Mrs Graff

20a Ferncroft Avenue

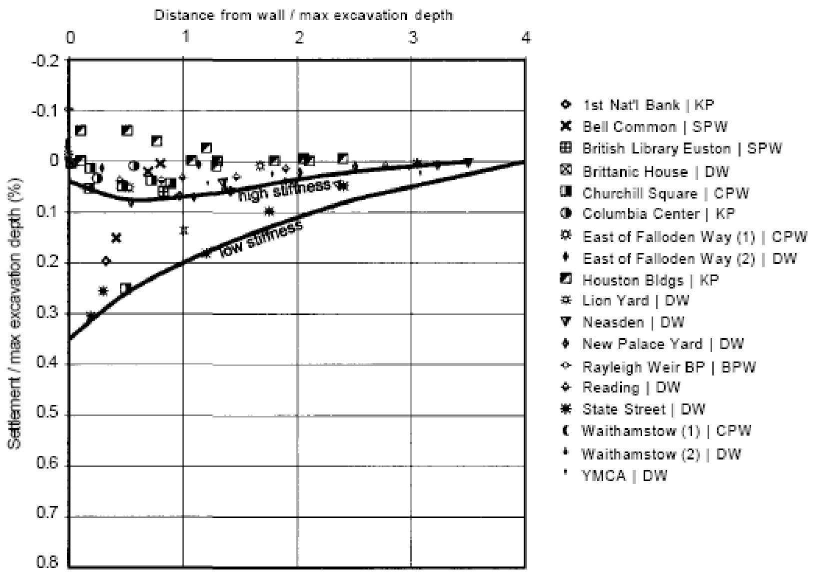
Estimated ground movements at completion of excavation for the new basement

Figure

7



(a) Horizontal movements



(b) Vertical movements

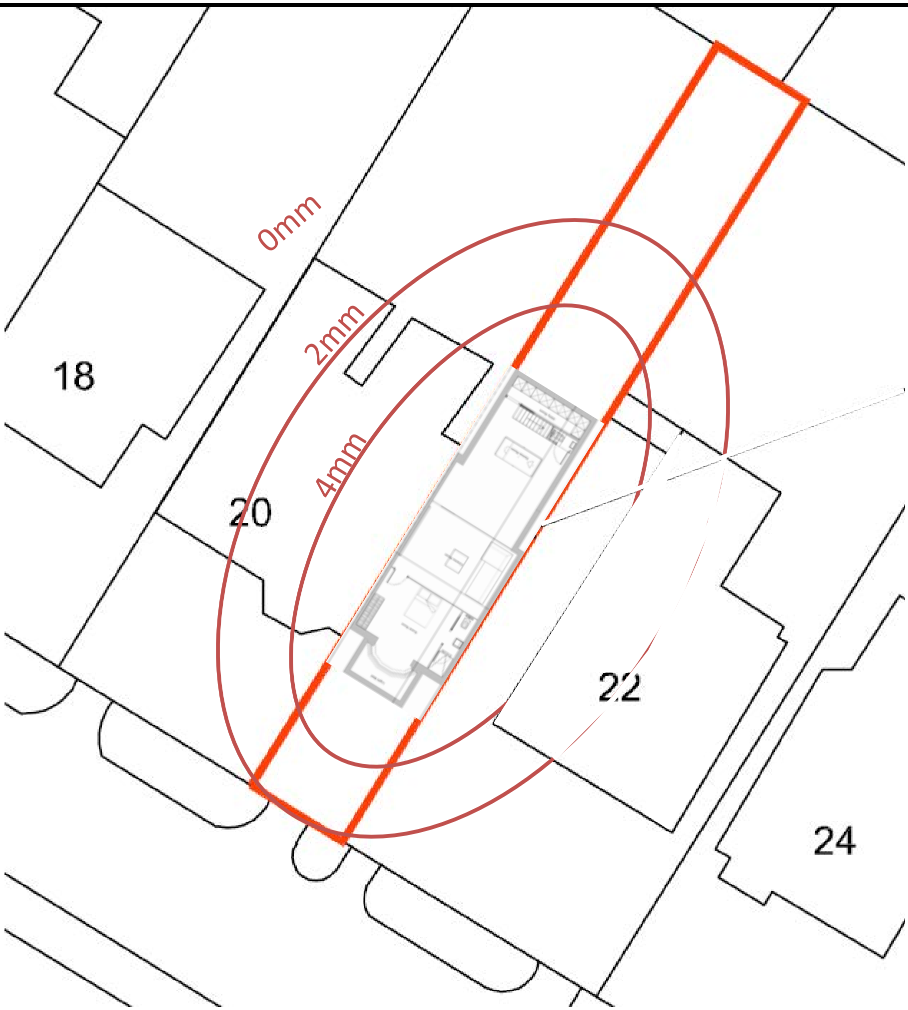
Mr and Mrs Graff

20a Ferncroft Avenue

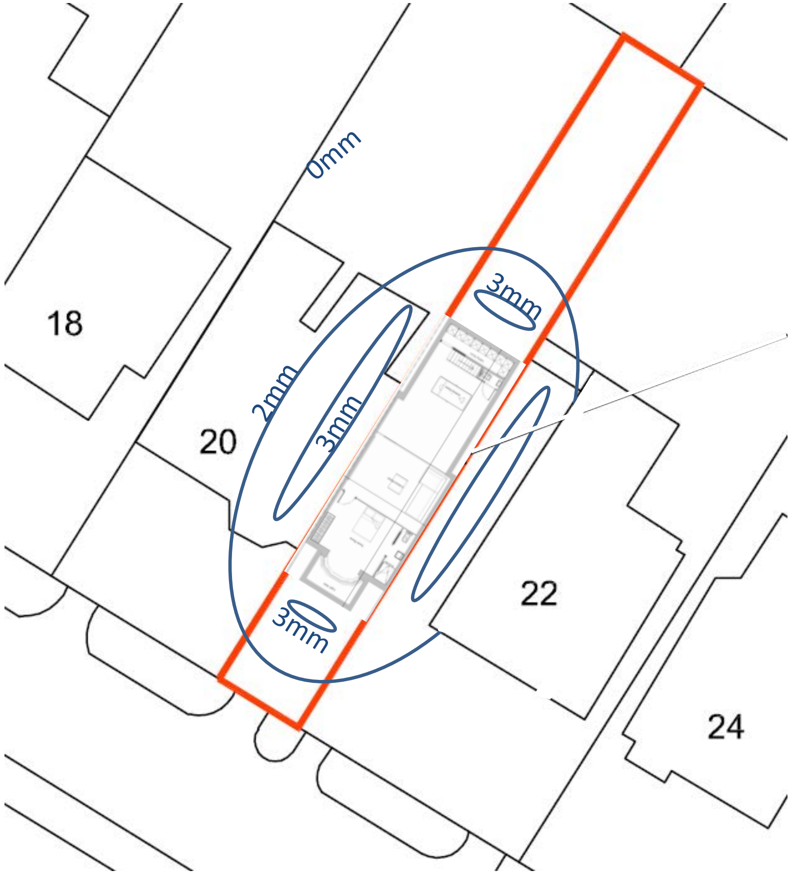
Field measurements of ground movements due to excavation in front of wall in stiff clay (CIRIA C760)

Figure

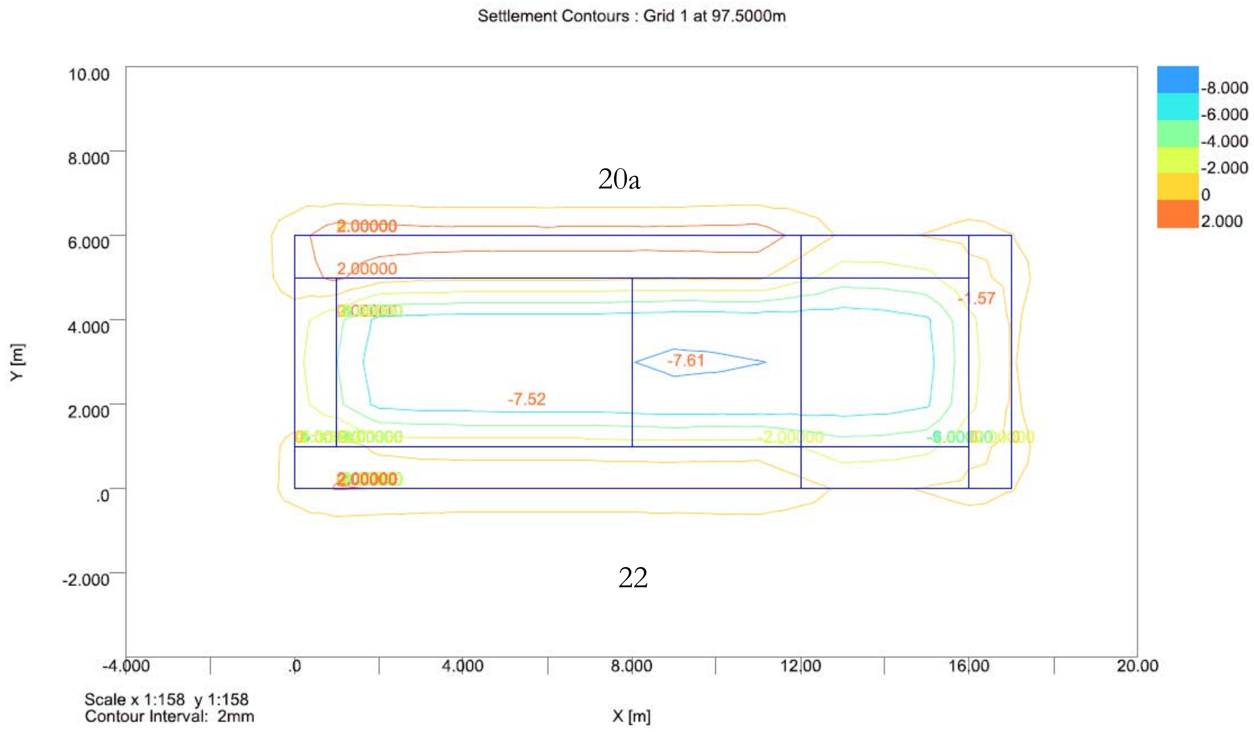
8



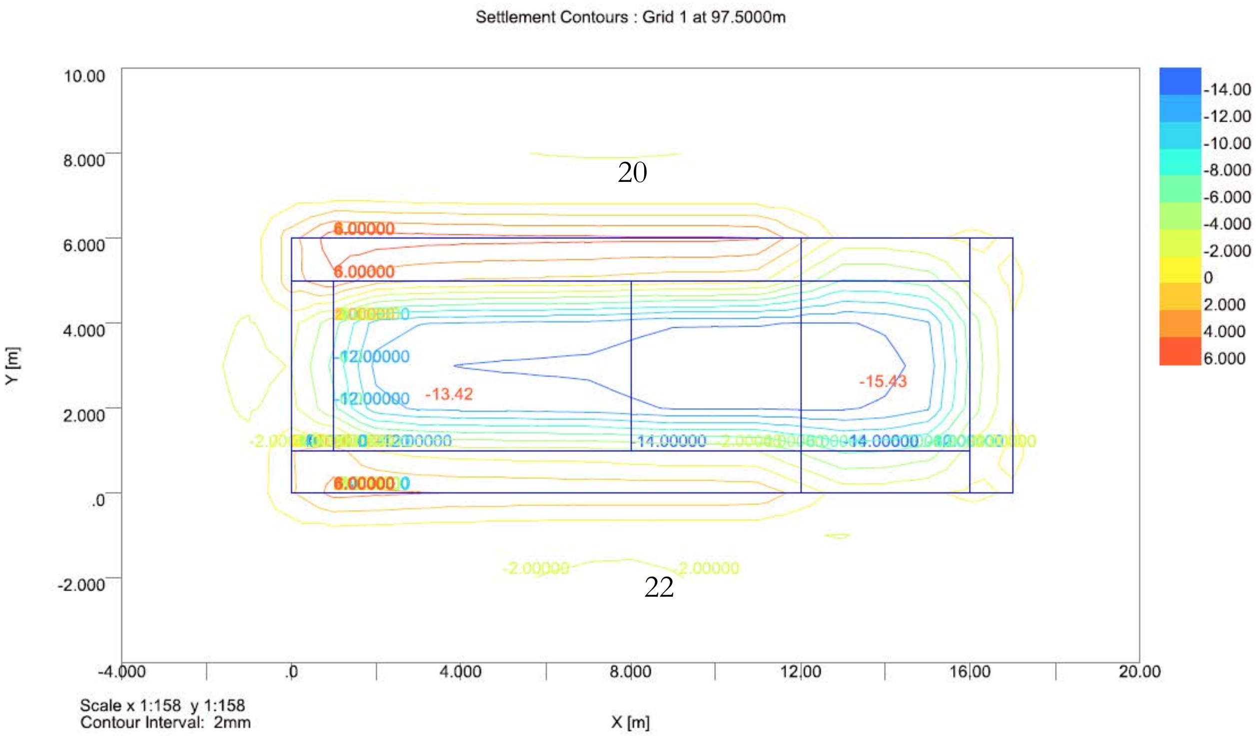
GCG	**Mr and Mrs Graff** 20a Ferncroft Avenue	**Figure**
	Ground movements induced around the site Total horizontal movements behind retaining walls	**9**



GCG	**Mr and Mrs Graff** 20a Ferncroft Avenue	**Figure** **10**
	Ground movements induced around the site Total vertical movements	



(a)



(b)

	Mr and Mrs Graff 20a Ferncroft Avenue	**Figure**
	Estimated loads at completion of construction. a) End of construction b) long term after completion	**11**

Category of damage	Description of typical damage (ease of repair is underlined)	Approximate crack width (mm)	Limiting tensile strain ε_{lim} (per cent)
0 Negligible	Hairline cracks of less than about 0.1 mm are classed as negligible.	< 0.1	0.0–0.05
1 Very slight	Fine cracks that can easily be treated during normal decoration. Perhaps isolated slight fracture in building. Cracks in external brickwork visible on inspection.	< 1	0.05–0.075
2 Slight	Cracks easily filled. Redecoration probably required. Several slight fractures showing inside of building. Cracks are visible externally and some repointing may be required externally to ensure weathertightness. Doors and windows may stick slightly.	< 5	0.075–0.15
3 Moderate	The cracks require some opening up and can be patched by a mason. Recurrent cracks can be masked by suitable linings. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired.	5–15 or a number of cracks > 3	0.15–0.3
4 Severe	Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Windows and frames distorted, floor sloping noticeably. Walls leaning or bulging noticeably, some loss of bearing in beams. Service pipes disrupted.	15–25 but also depends on number of cracks	> 0.3
5 Very severe	This requires a major repair involving partial or complete rebuilding. Beams lose bearings, walls lean badly and require shoring. Windows broken with distortion. Danger of instability.	usually > 25 but depends on number of cracks.	

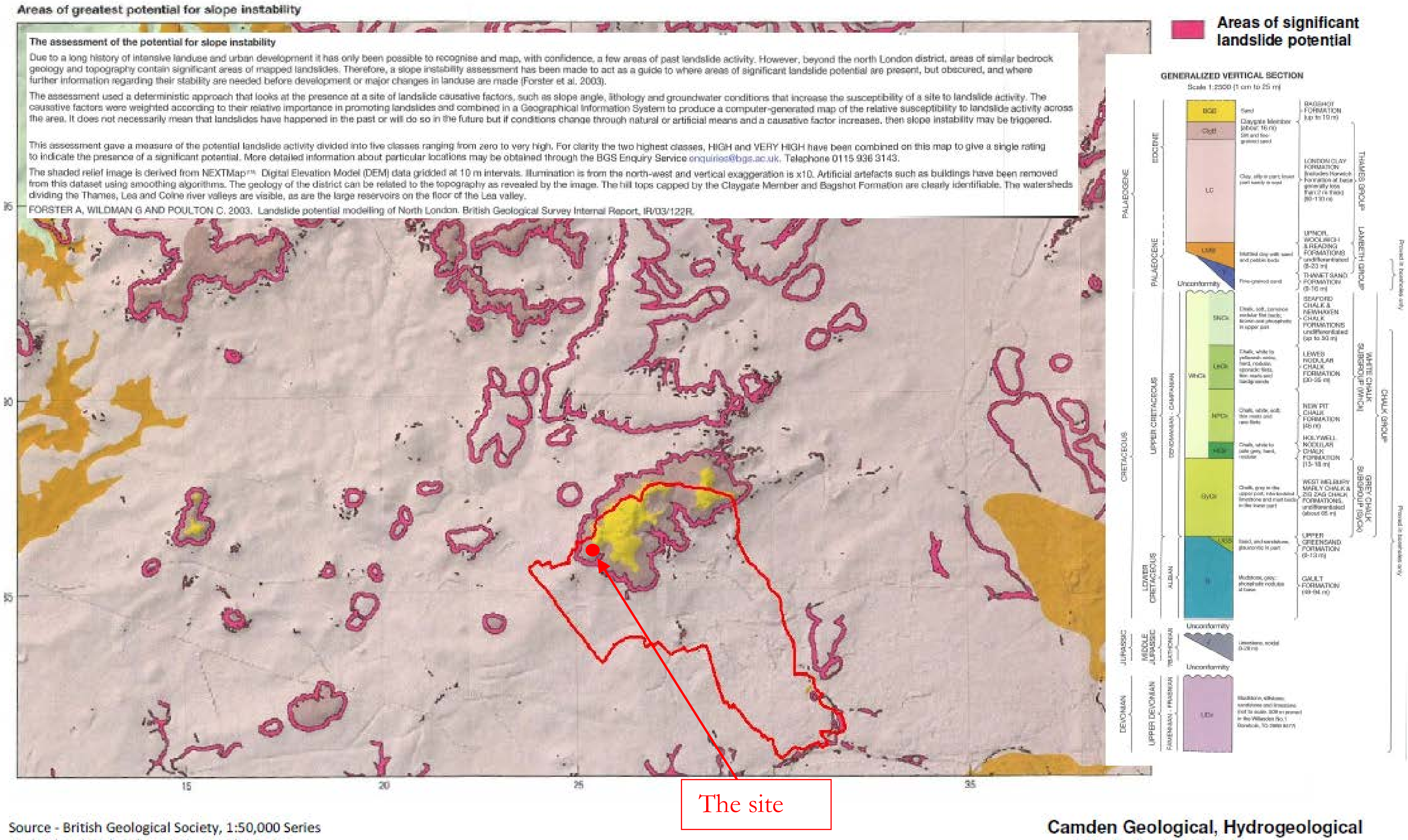
Mr and Mrs Graff

20a Ferncroft Avenue

Damage Category Table, Ciria C760

Figure

12



Source - British Geological Society, 1:50,000 Series
England and Wales Sheet 256 – North London

Camden Geological, Hydrogeological and Hydrological Study

GCG	**Mr and Mrs Graff** 20a Ferncroft Avenue	**Figure**
	Areas of significant landslide potential (Arup 2010)	**13**

A.1 Appendix A- Soil parameters used for PDisp calculations

The soil parameters for the ground movements analyses have been selected based on experience and on published information on the mechanical behaviour of the soil at the site (Hight et al. 2007, Gasparre et al. 2014).

Given the limited information on the stiffness response of Claygate Members, it has been assumed that it is similar to the stiffness response of the upper lithological units of the London Clay Formation.

For the purposes of the ground movement analysis based on an isotropic soil model, the elastic (small strain) undrained stiffness of the London Clay (E_{uo}) can be taken as:

$$E_{uo}=975p' \qquad (1)$$

where the mean effective stress p' has conservatively been calculated considering a coefficient of earth pressure at rest Ko equal to 1.

The elastic drained stiffness (E'_o) of the Claygate Member has been estimated from the relationship:

$$E'_o= 0.75Eu \qquad (2)$$

For the analysis it has been assumed that the proposed works will give rise to strains in the more superficial strata of the Claygate Members, which will reduce its elastic stiffness. The stiffness reduction has been calculated based on the magnitude of the applied loads.

In summary, the following soil conditions and soil parameters have been assumed in the analyses:

Stratum	Level at top [mOD]	Undrained Stiffness Eu [MN/m^2]	Drained Stiffness E' [MN/m^2]
Made Ground	+102	-	10
Claygate Members	+101	5.8 +8z	0.75 Eu
London Clay	+90	14+6z1	0.75 Eu
Lambeth Group	+30	-	400
Rigid boundary	+20		

Where z is the depth below the top level of the top of the Claygate Member and z_1 is the depth below the London Clay.

Appendix B

Geotechnical Consulting Group Hydrogeological Impact Assessment Revision 1 dated March 2020

MR AND MRS GRAFF

20A FERNCROFT AVENUE

HYDROGEOLOGICAL IMPACT ASSESSMENT

REVISION 1

March 2020

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MR AND MRS GRAFF

20A FERNCROFT AVENUE

HYDROGEOLOGICAL IMPACT ASSESSMENT

REVISION 1

March 2020

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REVISION HISTORY

Revision	Date	Prepared by	Reviewed by	Description
Revision 0	2/12/2019	A. Gasparre Dott,Ing, PhD, DIC, CEng, MICE	J. A. Davis EurGeol CGeol BSc MSc DIC FGS	Issued for comments
Revision 1	5/03/2020	A. Gasparre Dott,Ing, PhD, DIC, CEng, MICE	J. A. Davis EurGeol CGeol BSc MSc DIC FGS	Reference to RedFrog added

MR AND MRS GRAFF

20A FERNCROFT AVENUE

HYDROGEOLOGICAL IMPACT ASSESSMENT

REVISION 1

March 2020

EXECUTIVE SUMMARY

The proposed redevelopment of 20A Ferncroft Avenue comprises the extension of the ground floor of the existing house and the construction of a new basement.

A hydrogeological study has been undertaken to assess the impact of the proposal on the local hydrogeology and on the adjacent structures.

The site is on ground sloping southwards and eastwards with an approximate gradient of 1: 15. It is underlain by Claygate Member, proved to 10m below ground level.

Groundwater at the site has been measured from August 2019 to September 2019. Perched water has been found to be present from 2.3m below ground level, while the groundwater table has been measured at approximately 5m depth. This is expected to flow southwards and eastwards following the topography of the ground. There are lost rivers in the proximity of the site, which are likely to represent the preferential pathway for groundwater flow.

The new basement would be above the groundwater level but could intercept perched water present across the site. Although it is unlikely that this water flows with a significant gradient, the new basement box could create a local barrier to any underground water flow.

This is unlikely to have any significant impact on the local hydrogeology and on the surrounding properties.

Water ingress could occur during construction and provision should be made to excavate in the dry.

MR AND MRS GRAFF

20A FERNCROFT AVENUE

HYDROGEOLOGICAL IMPACT ASSESSMENT

REVISION 1

March 2020

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LIST OF FIGURES

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1 Introduction

The proposed redevelopment of 20a Ferncroft Avenue comprises the extension of the ground floor of the existing house and the construction of a basement below the footprint of the house and its rear patio.

The Geotechnical Consulting Group (GCG) have been commissioned to estimate the impact of the proposed basement construction on the local hydrogeology.

This report discusses the issues related to groundwater and considers the land drainage design measures (if required) to minimise the potential risks of adverse effects of the project on groundwater and neighbouring properties.

Information on the proposal has been provided by Simpson Associates, the structural consultants for the project.

This report has been prepared as part of the requirements set by the DP27/CPG4 and LB Camden's 'Guidance for Subterranean Development'. It addresses the issues of the subterranean (ground water) flow screening chart.

2 The site and the proposed redevelopment

The site lies within the Frognal & Fitzjohn Ward of Camden Administrative Boundary and is located on the north side of Ferncroft Avenue, approximately 250m to the south of West Heath (Figure 1a).

It has a rectangular shape, approximately 45m x 6m in plan, orientated along a north-east to south-west direction. The ground level across the property reduces from a relative datum (RD) of approximately +52.5mRD at the rear to +50mRD at the front. It should be noted that this datum, provided in the structural drawings, is approximately 50m below the Ordnance Datum (OD).

The site includes an 8m long driveway at the front, a semidetached mansion house with a paved patio and a garden at the rear. Figure 1b shows a layout of the site.

The house is approximately 15m long and 6m wide and shares a party wall with the adjacent house on 20 Ferncroft Avenue. The rear section of the house steps up the front section by approximately 0.5m.

In the rear section of the house there is also a 1.5m diameter cylindrical cellar that extends to approximately 3m depth.

Figure 2 shows a plan of the existing ground floor and a north-east to south-west section through the house.

It is proposed to extend the ground floor of the existing house to the rear and create a new basement underneath the original footprint of the house and its rear patio. A light well will also be formed at the front of the house. Figure 3 shows a plan of the proposed basement and a section through the site.

The finished floor level of the new basement will be at approximately +48.5mRD (i.e. +98.5mOD) and will require an excavation of approximately 3m under the front of the house and 3.5m under the rear.

It is understood that the basement will be formed by underpinning the party walls of the house.

3 Topography and geology

The site is on the edge of a ridge, on ground sloping to the south-west and south-east an approximate gradient of 1:15 (Figure 1a). Relative to ordnance level the ground level at the front of the site is approximately +100mOD and is about +102.5mOD at the rear.

The ground and groundwater conditions have been established on the basis of record information (British Geological Survey, BGS, maps and record boreholes) and a site-specific ground investigation carried out by Risk Management, (Risk Management, 2019).

The 1:50,000 scale geological map (BGS, 1994, Sheet 256 – North London, Figure 4 shows that the site is underlain by Claygate Member, the uppermost Member of the London Clay Formation. The Claygate Member comprises interbedded layers of fine-grained sands, silts and clays. At less than 100m to the north of the site Bagshot Sand Formation overlies the Claygate Member.

Below the Claygate Member the stratigraphy below the site includes London Clay, Lambeth Group, Thanet Sand and Chalk.

The London Clay outcrops about 200m to the south east of the site at +90mOD. The thickness of the London Clay in this area is expected to exceed 60m.

The site specific ground investigation included the sinking of a borehole (BH1) to 10m depth in the front driveway, a Drive-in Sampler borehole (DIS2) to 5m depth in the rear patio and two trial pits along the front and rear walls of the house. The location of the investigation holes is shown in Figure 5.

The borehole logs consistently identify the presence of a thin layer of Made Ground over the Claygate Member, which is described as brown-orange and grey silty clay with pockets of silt in both boreholes. The clay is weathered, as suggested by the presence of selenite and the orange colour. A layer of sand is also identified in BH1 between 4.6m and 6.2m depth (i.e. approximately +96.4mOD to +94.8mOD).

A similar stratigraphy was identified in the two boreholes sunk at 22 Ferncroft Avenue in 2013, although a layer of sandy deposits was identified from +97mOD and was found to be 3m thick at the front of 22 Ferncroft Avenue and at least 1.5m at the rear, where its thickness was not proved.

At 18 Ferncroft Avenue, 20m to the west of the site, two additional boreholes confirmed the stratigraphy identified at 20a Ferncroft Avenue, although the sand layer was found to be 0.4m thick from approximately +95mOD.

The combined information from the available boreholes indicates that in the vicinity of the site the sandy deposit dips and reduces in thickness towards north and west.

4 Hydrogeological conditions and hazards

The OS topographic maps show the house sits on top of a broad watershed ridge dropping to the south west from Childs Hill. A map in the Lost Rivers of London (Burton and Myers 2006) and the RedFrog report (Arup, 2020), (Figure 6) and the 1920 1:10,560 scale BGS map (Figure 4a) show that numerous streams exist in the area of the site. These typically originate at geological boundaries.

About 200m to the east of the site a former stream runs southwards along the southern section of Redington Road, to feed a tributary of the Westbourne River further to the south. Further to the north-west there are springs of the Brent River as the ground on the western side of the ridge drops to the west into the drainage of the River Brent.

The RedFrog report also indicates the presence of a spring, from local knowledge, in a property along Hollycroft Road, to the north of the site. It is unclear as to whether this is a natural or an artificial feature.

Groundwater in the area of the site would be expected to flow off the watershed eastwards into the Westbourne catchment and westwards into the Brent catchment. Groundwater flow along the watershed ridge is likely to be insignificant.

Other streams are further than 100m from the site and flow into various drainage channels to form tributaries of the four main rivers within the LB Camden. All these springs would be expected to be culverted or filled in.

The site is more than 100m away from the Hampstead Chain Catchment. The closest ponds appear to be at approximately 800m to the north of the site (Leg of Mutton), although smaller ponds are known to be present within West Heath.

4.1 Aquifers

Within the London area there are two recognised principle aquifers. The major aquifer is a deep aquifer below the London Clay, while the shallow aquifer lies predominantly within the deposits above the London Clay. The London Clay acts as a barrier between the two aquifers.

4.1.1 Deep Aquifer

The deep aquifer lies within the Chalk and Thanet Sand Formation that extends under the London Basin. Historically, extraction of water from this aquifer for drinking and industrial purposes has caused a significant drop in the aquifer level. Since the mid-1960s, extraction of water from the deep aquifer has declined greatly, and as a result the water level has been recovering. Due to the implications that this rising groundwater level has for the infrastructure of London, the aquifer level is now monitored and the rise in its level is controlled by pumping (as described by the Environment Agency, 2017).

Currently, the deep aquifer beneath the site lies at approximately -10mOD (Figure 7). The London Clay and clay sub-units of the Lambeth Group that overlie the Thanet

Sand and Chalk are of very low permeability and of sufficient thickness that the proposed development will have no impact on the deep aquifer.

4.1.2 Shallow aquifer

The shallow aquifer lies within the superficial deposits above the London Clay. It is variable in both level and thickness, and is discontinuous. It has also been heavily modified by human activity throughout the history of London.

Groundwater in the shallow aquifer tends to flow above the underlying impermeable layers of clays following the underground topography of the area.

The presence of lost rivers or streams generally indicate the preferential ways of groundwater flow within the shallow aquifer.

4.2 Site conditions

As mentioned above, the lost rivers in the vicinity of the site represent the preferential directions of groundwater flow in the area.

Due to the nature of the Claygate Member, discontinuous and localised groundwater might also be present within the sandy bands above the more clayey layers.

Groundwater has been recorded during the investigation works and standpipes were installed to monitor groundwater at completion of the works.

During the investigation works groundwater was encountered at 5.5m depth in BH1 and 4.5m in DIS2, which, considering the ground levels of these holes, correspond to levels of +44.5mRD (or +94.5mOD) and +47mRD (or +97mOD).

Standpipes extended to 6m below ground level (bgl) in BH1 and 5mbgl in DIS2, with a response zone to 1m bgl.

Groundwater level readings were taken on three return visits and are provided in the table below in depths bgl and with reference to mOD and mRD. They are also plotted in Figure 8.

Date [-]	BH1 [mbgl]	DIS2 [mbgl]	BH1 [mRD)	DIS2 [mRD)	BH1 [mOD)	DIS2 [mOD)
21/08/2019	5.34	2.57	45.37	48.96	95.37	98.96
05/09/2019	5.39	2.71	45.32	48.82	95.32	98.82
18/09/2019	5.43	3.67	45.28	47.86	95.28	97.86

The readings indicate that:

- There in an apparent difference in groundwater levels between the front and rear of the site of 3-4m
- The readings in DIS2 vary with time while the readings in BH1 are constant.

The above features can be explained considering the depths of the standpipes and their response zones and the rainfall across the site.

Figure 9a shows that the first two readings were taken immediately after days of intense rain, while the last reading was taken after a few days of dry weather. The readings in DIS2 are therefore affected by rainfall. This is likely to be due to the fact that the response zone of this standpipe is entirely in the shallower clay deposits. It is possible that rainfall infiltrated into the standpipe and could not dissipate rapidly through the low permeability clay. However, it is also possible that the rainfall was effectively retained within the less permeable layer of the Claygate Member and this standpipe recorded this perched water. The response zone of the BH1 is immediately below a high permeability sandy layer, through which groundwater can flow and therefore the records are not significantly affected by rainfall. This is a typical feature of the Claygate Member.

Given the discontinuities of the clay layers within the Claygate Members any perched groundwater within these layers is also likely to be localised and discontinuous.

It should also be noted that although the monitoring was carried out over a summer period the rainfall between August and September 2019 was around the annual average, as shown in Figure 9b. This indicates that the groundwater readings can be taken as representative of an annual average.

4.3 Surface Flooding

The Environment Agency data indicates that the area of the site is at very low risk of flooding from surface water, rivers or sea and reservoirs (Figure 10a).

However Ferncroft Avenue is in the list of roads of Camden affected by flooding in 1975 (Figure 10b). Flooding occurred after intense rainfall and most likely it was due to poor drainage of surface water and run off.

The proposal will not increase the current ratio of paved/green areas. A SUDS system (attenuation tank) for the storage and attenuation of water run-off in the rear garden is to be provided. This will mitigate the current discharge of surface water into the local sewer alleviating the contribution of the site to potential flooding.

The site is not included in a Source Protection Zone and is also not in a sensitive land use or in a potentially contaminative industrial land use.

5 Impact of the development and land drainage requirements

The excavation for the proposed basement will extend to an approximate level of +48mRD (i.e. +98mOD). It will remain approximately 2m above the groundwater level measured at the front part of the house, but 1.5m below the level measured at the rear. As mentioned above, the groundwater level measured at the rear of the site is likely to be perched water within the clay layers or rainwater retained in the standpipe. Given the nature of the Claygate Member, this is unlikely to be continuous across the site and to flow horizontally with a significant gradient. Any potential flow would be expected to follow the slope of the ground eastwards and westwards towards the lost rivers.

The new basement box could create a minor, local barrier to any potential flow of perched water within the most superficial layers across the site. However, its impact would be limited to a minor increase of water head on the uphill side (rear) of the new basement. Given the limited extent of the proposed basement, the depth of the measured perched water below ground and the stratigraphy, the construction of the new basement box is unlikely to have any significant impact on the local hydrogeology or on any existing neighbouring structures.

Nevertheless the new basement walls should be designed accounting for the presence of the perched water and considering potential increase of its level at the back of the retaining walls. For design, a potential burst of water pipes that might rapidly increase the water pressures on the retaining walls should also be considered.

The potential ingress of groundwater during construction should also be taken into account. Measures to deal with groundwater ingress will need to be adopted to form the underpinnings and the presence of perched water at levels higher than those measured should also be taken into consideration. As mentioned above any flow within these layers is likely to be localised and discontinuous and should be able to be controlled by simple pumping means.

In the permanent condition there will need to be a suitable internal construction to bring the structure to an acceptable standard with regard to moisture ingress.

6 Conclusions

The proposed redevelopment of 20A Ferncroft Avenue comprises the extension of the ground floor of the existing house and the construction of a new basement.

The results of the site investigations carried out in August 2019 indicate that the site is underlain by Claygate Member, proved to 10m below ground level.

Perched water has been found to be present from 2.3m below ground level, although discontinuous water ingress might be encountered also at shallower depths. The groundwater level has been measured at approximately 5m depth and is expected to flow southwards and eastwards following the topography of the ground.

The new basement would be above the groundwater level but could intercept perched water across the site. Although it is unlikely that this water flows across the site with a significant gradient, the new basement box could create a local barrier to this flow, which could cause a negligible increase of groundwater levels on the uphill side of the new basement. Any effect is likely to be small and localised within the boundaries of the site and is unlikely to have any significant impact on the local hydrogeology and on the surrounding properties.

The site is situated close to the watershed between the Westbourne an Brent drainage catchments. There are lost river in the vicinity of the site, but no other known ponds and wells. The site is outside the Hampstead pond chain catchment area.

The proposed construction will not increase the proportion of hard surfaced/paved areas, and a SUDS system is planned in the garden area.

Water ingress could occur during construction and provision should be made to excavate in the dry.

The proposal should have no impact on the deeper aquifer.

7 References

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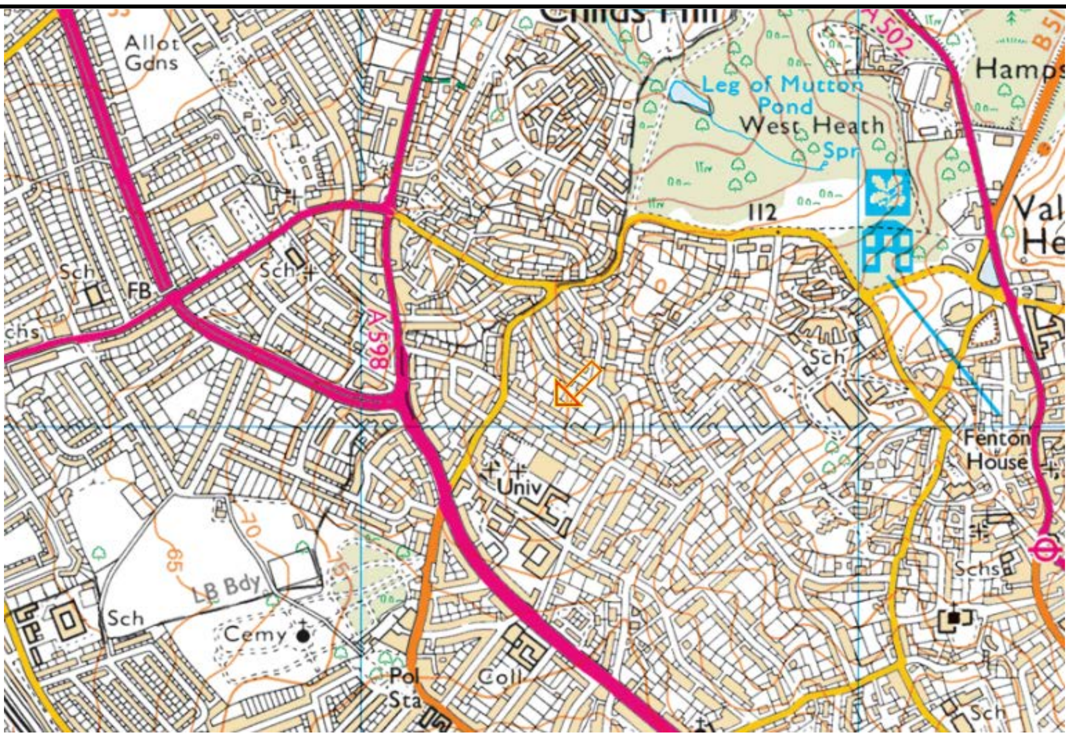
.

FIGURES



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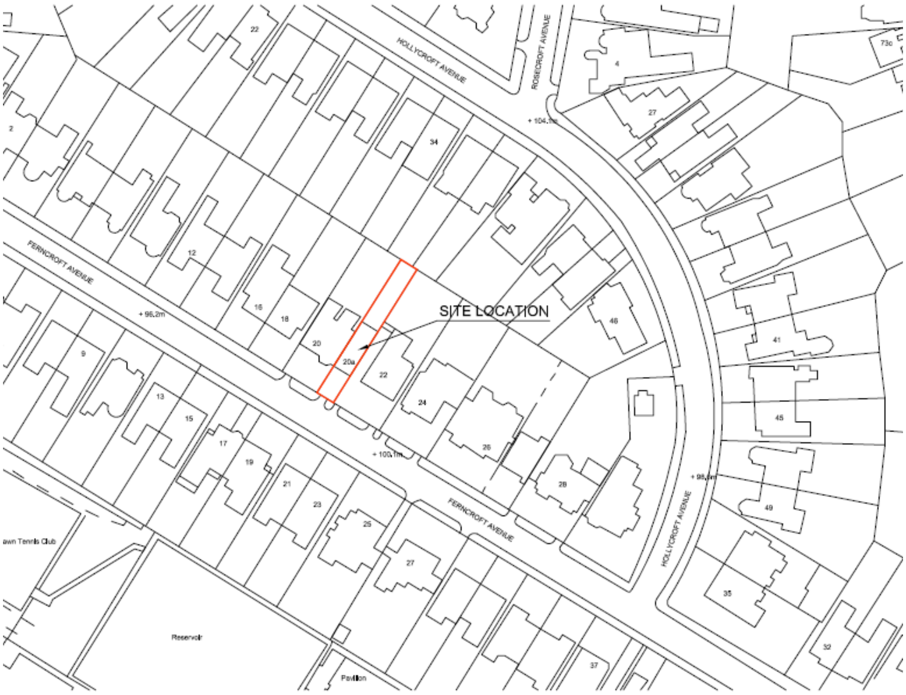
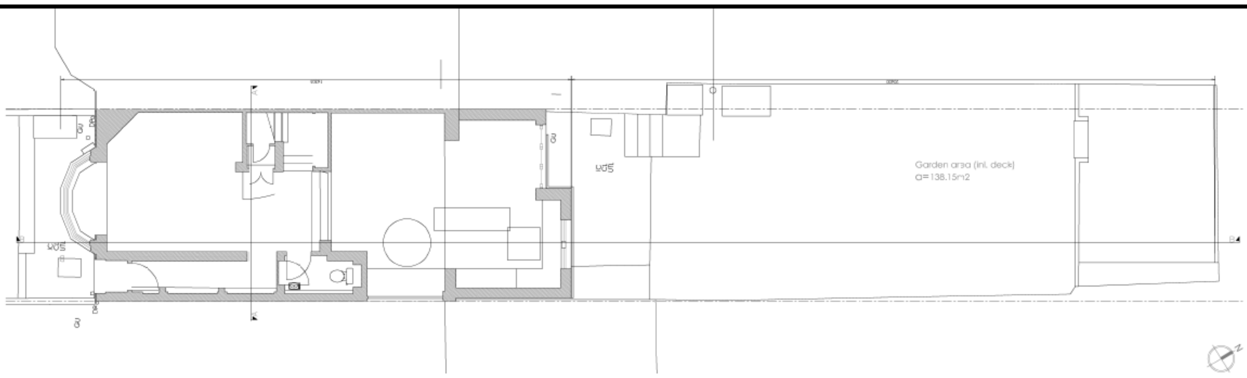


GCG	**Mr and Mrs Graff** 20a Ferncroft Avenue	**Figure** **1**
	The site a) Location b) site layout	



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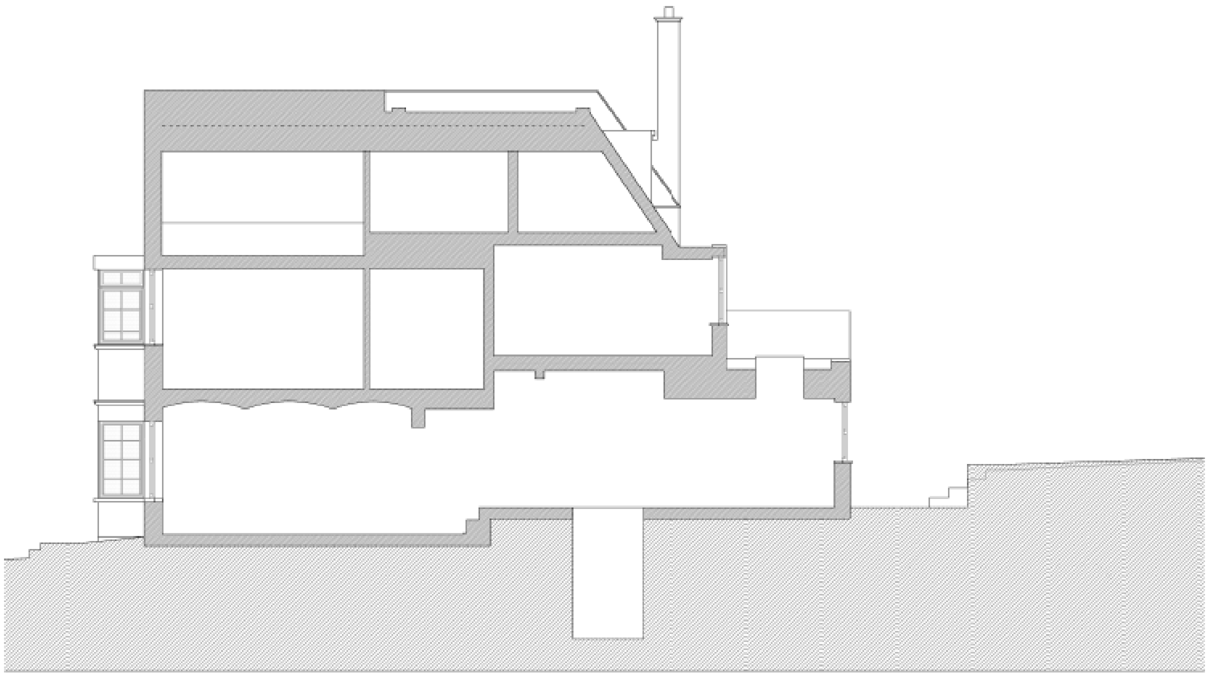
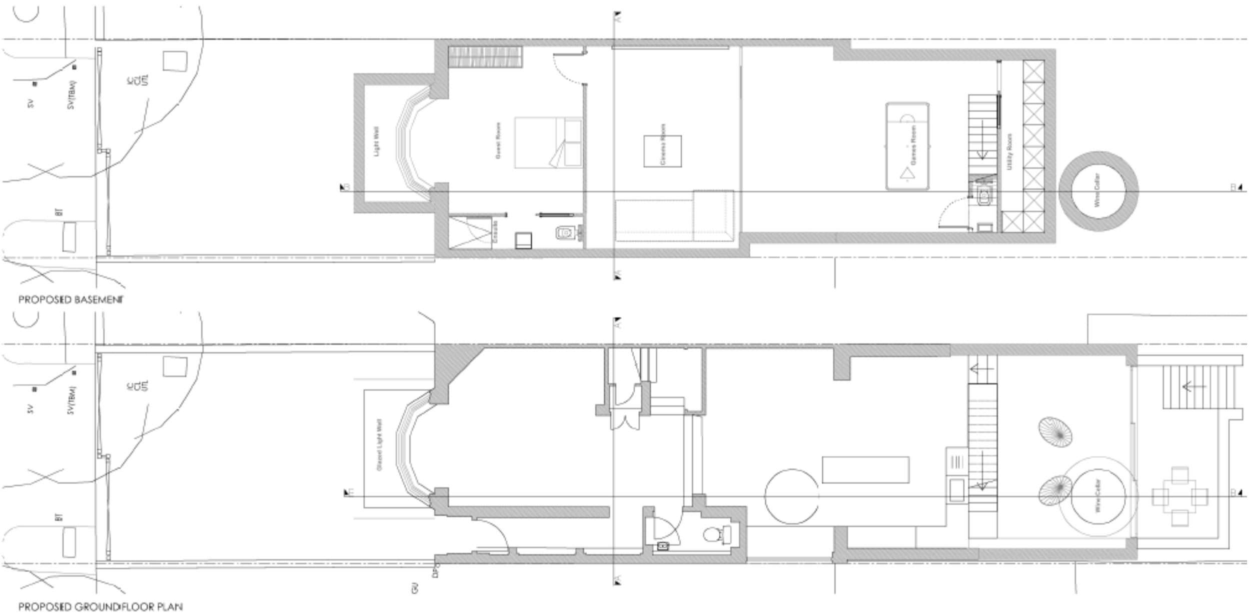


(b)

GCG	**Mr and Mrs Graff** 20a Ferncroft Avenue	**Figure**
	Existing lower ground floor (a) Plan and b) north-west to south-east section	**2**



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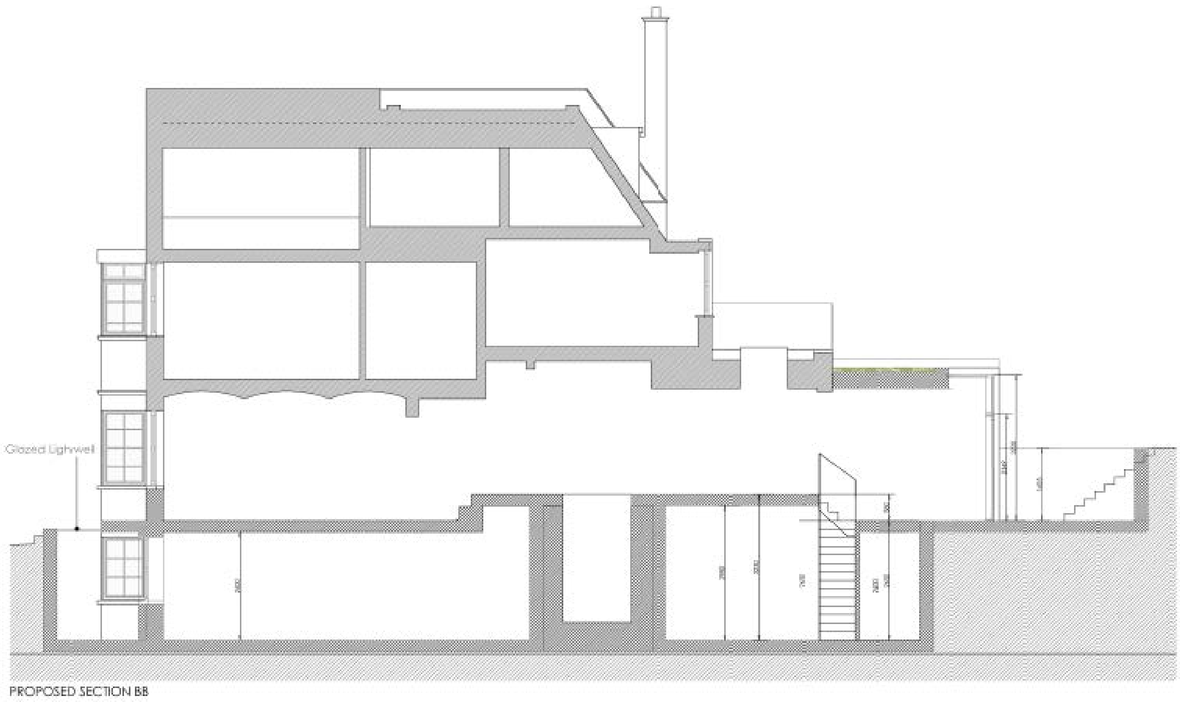
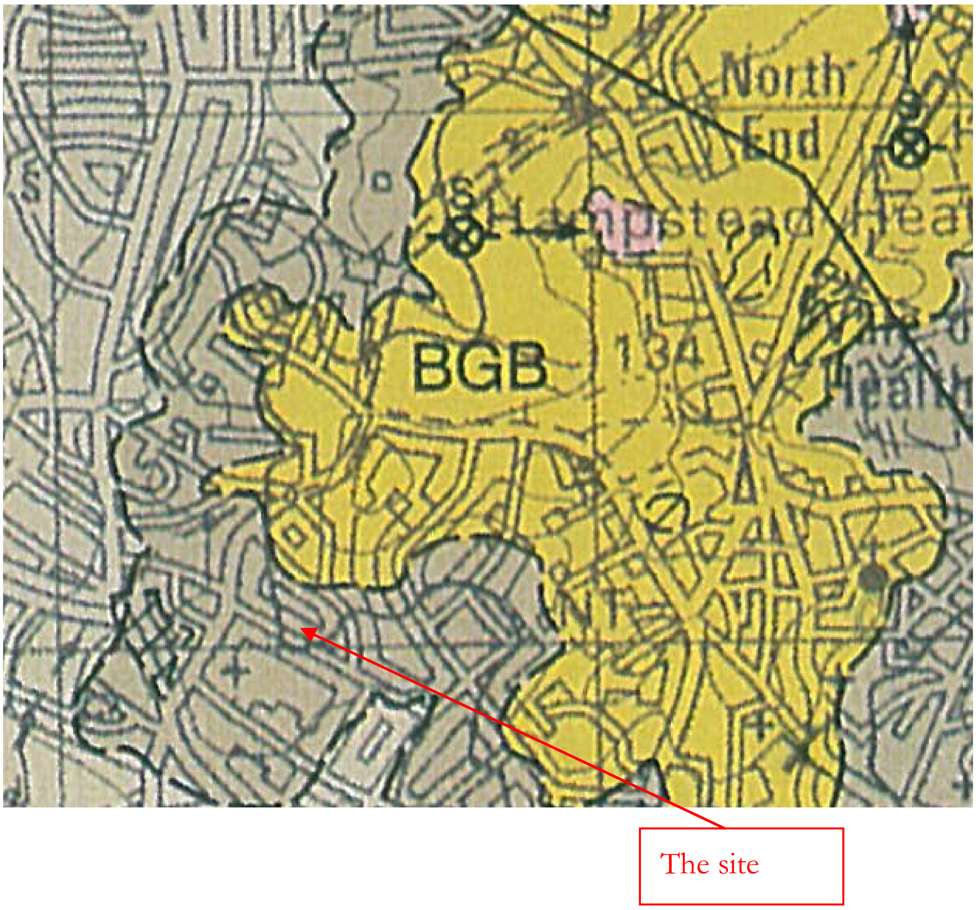


(b)

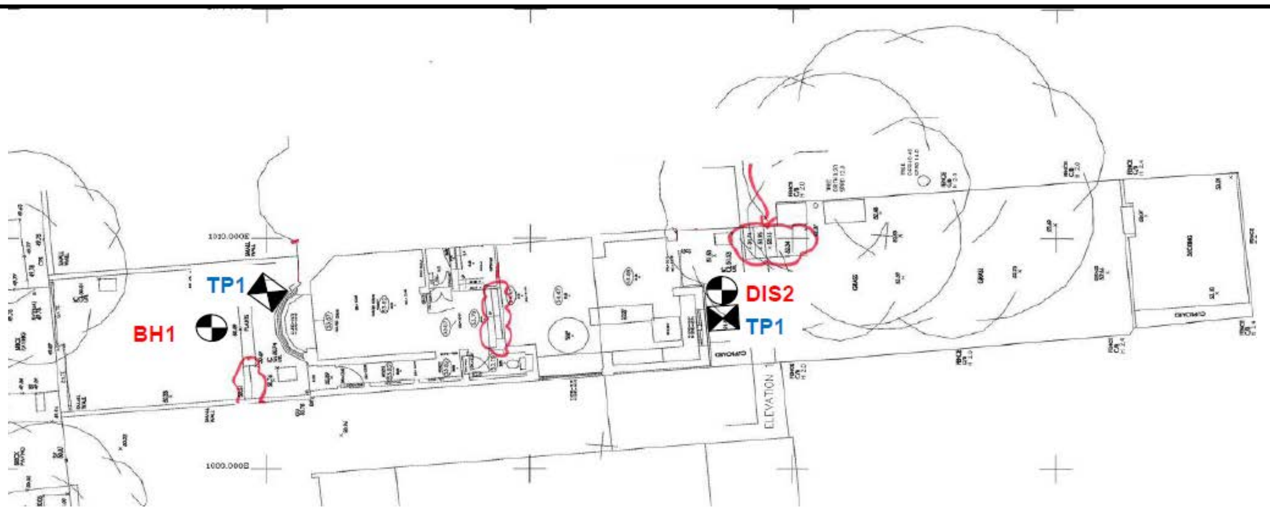
	Mr and Mrs Graff 20a Ferncroft Avenue	**Figure** **3**
	Proposed basement (a) Plans of the basement and ground floor and (b) north-south section	



Mr and Mrs Graff
20a Ferncroft Avenue

BGS Geological map, 1992

Figure
4



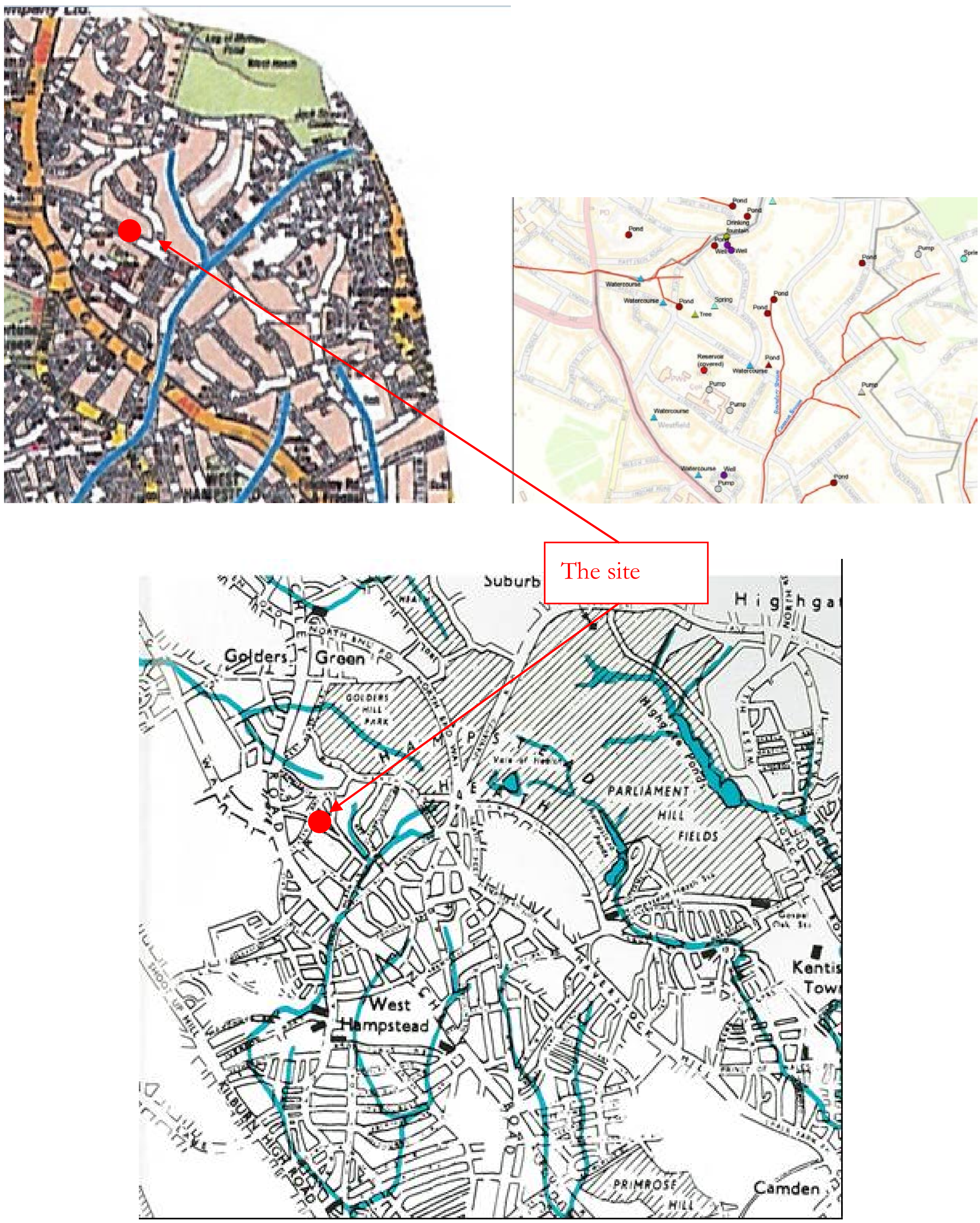
Mr and Mrs Graff

20a Ferncroft Avenue

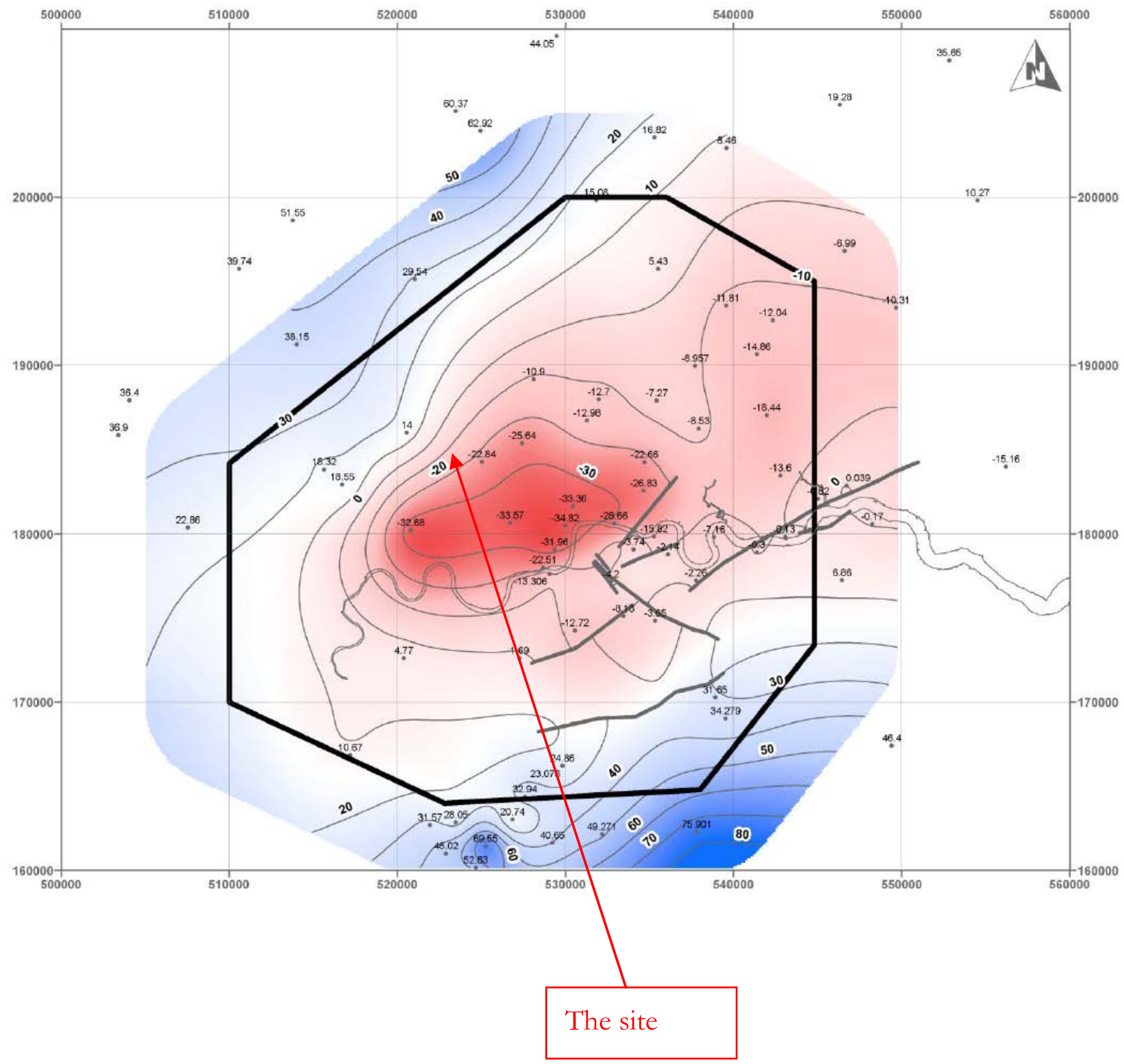
Location of investigation points

Figure

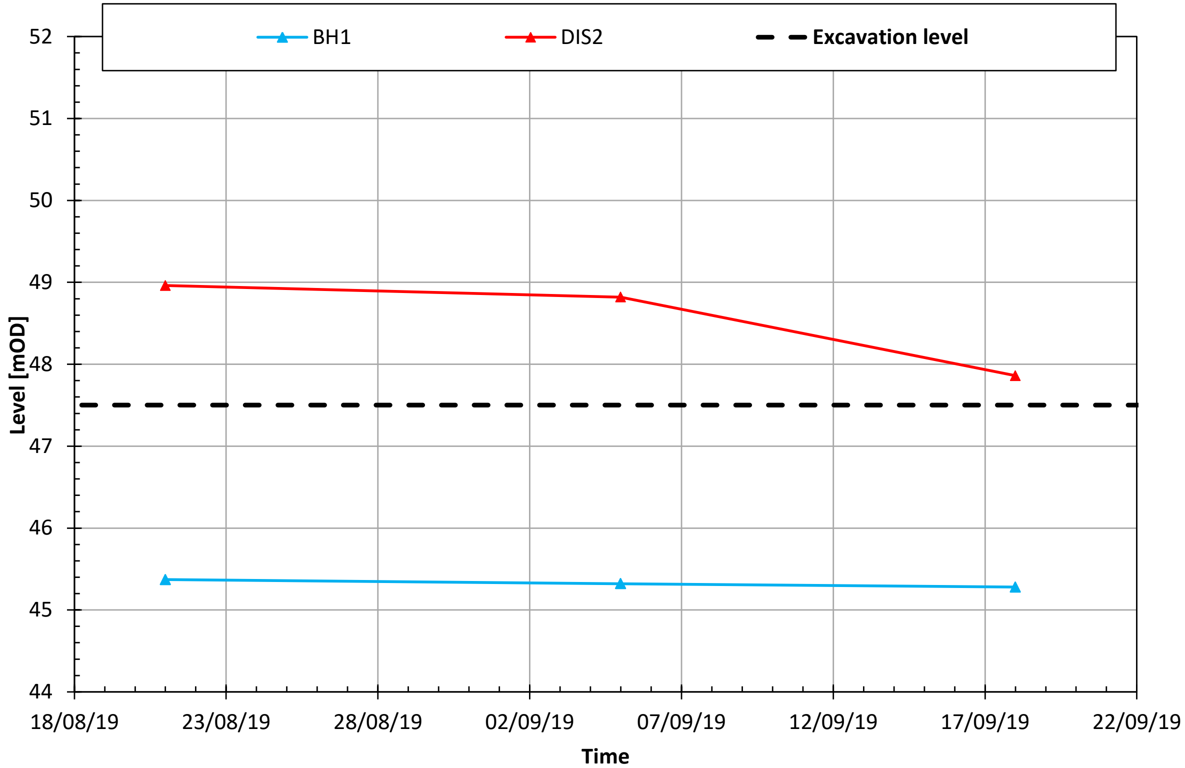
5



	Mr and Mrs Graff 20a Ferncroft Avenue	**Figure**
	Extract from the Lost Rivers of London (Barton 1992)	**6**



GCG	**Mr and Mrs Graff** 20a Ferncroft Avenue	**Figure**
	Level of deep aquifer Environment Agency 2016	**7**

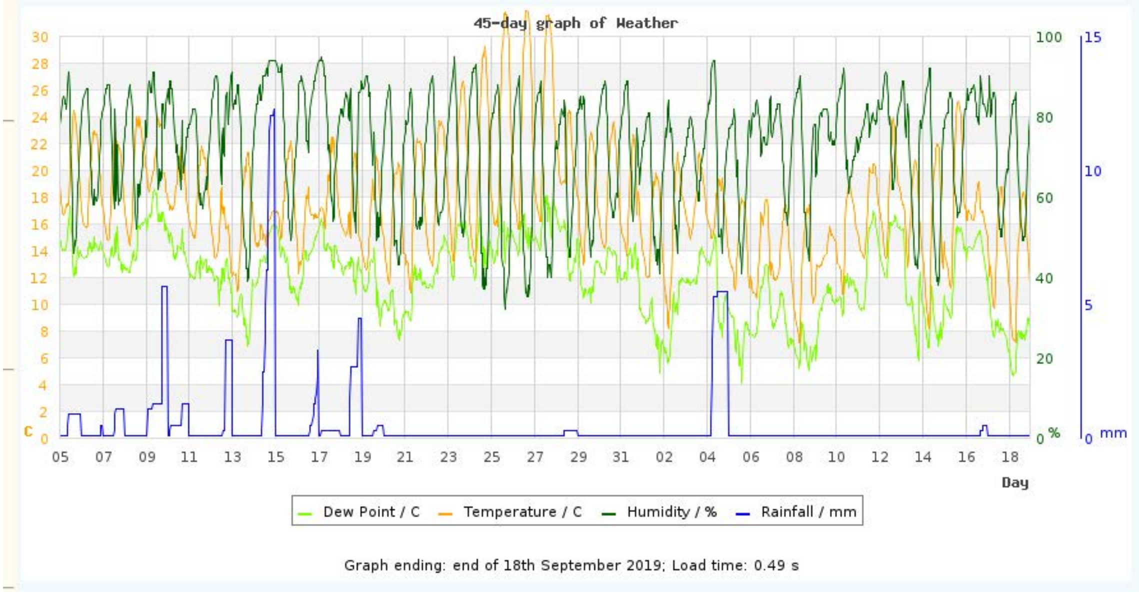


Mr and Mrs Graff

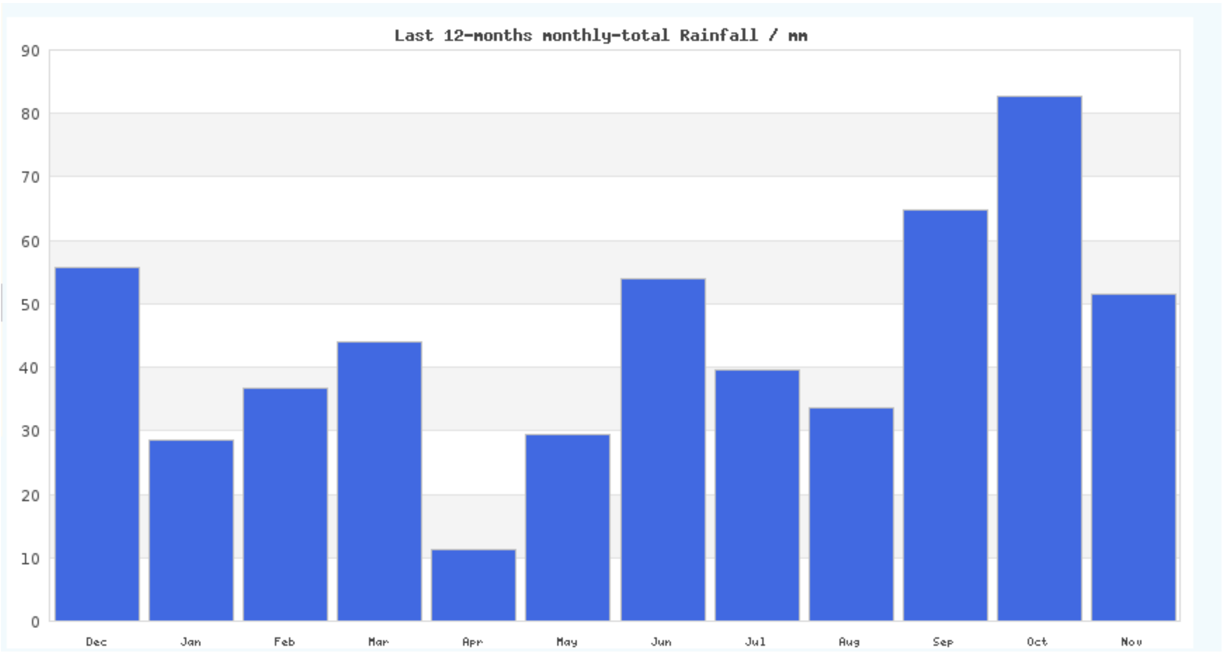
20a Ferncroft Avenue

Groundwater levels across the site

Figure 8



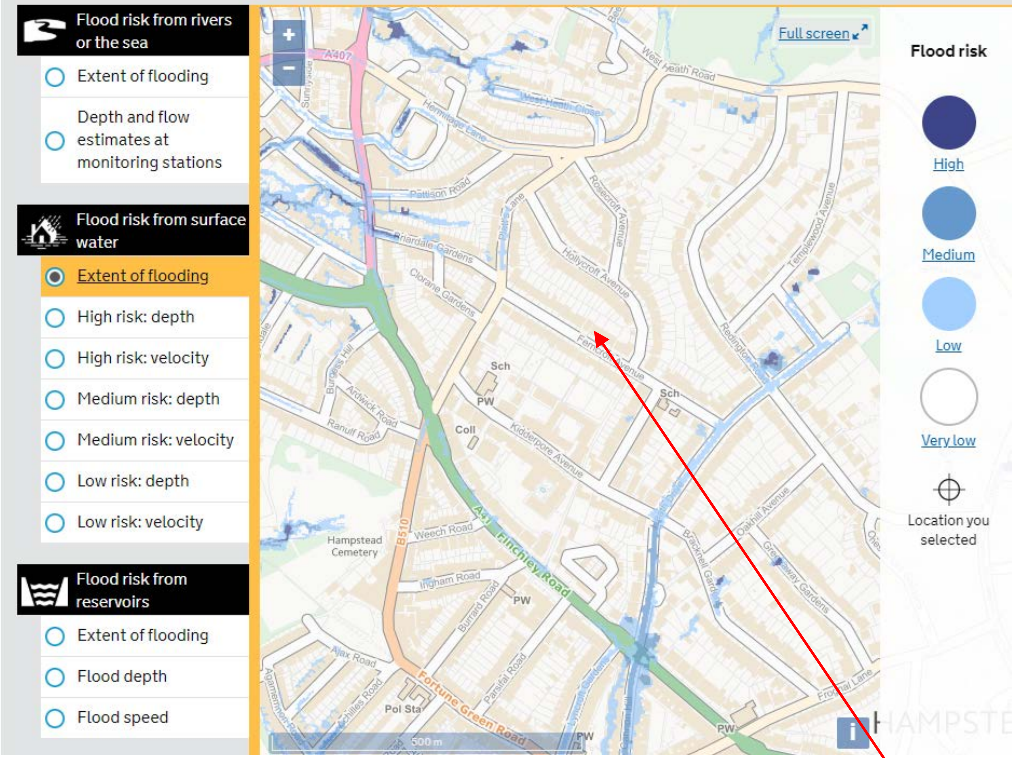
(a)



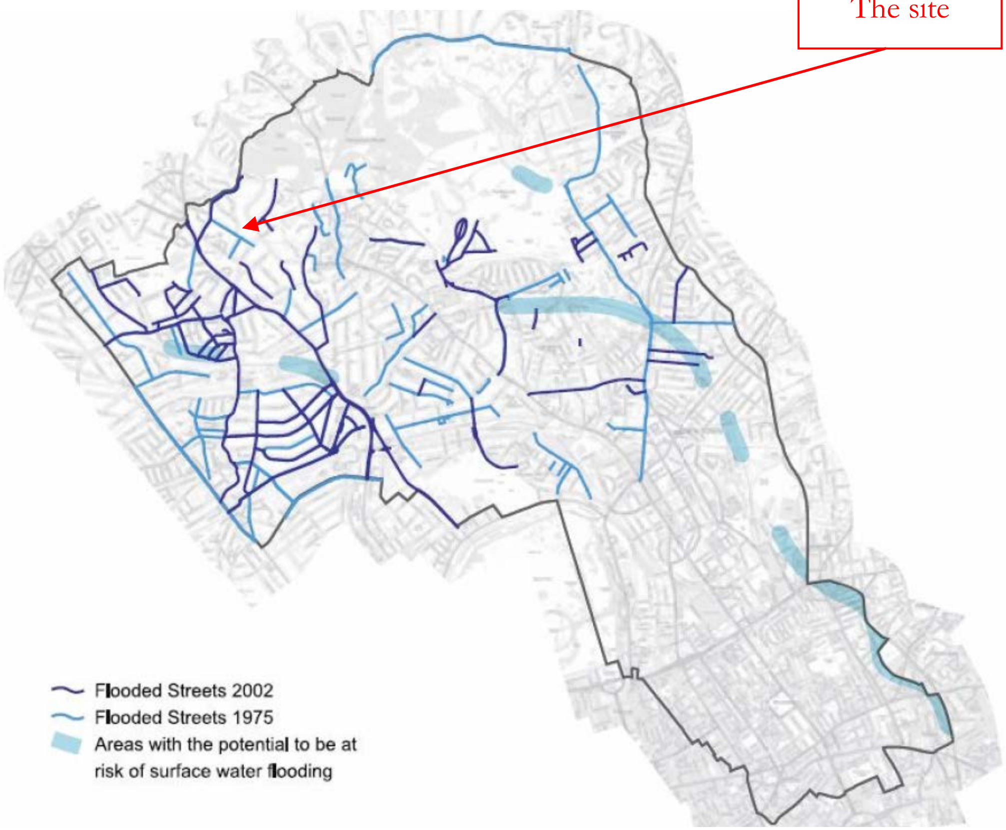
(b)

	Mr and Mrs Graff 20a Ferncroft Avenue	**Figure**
	Rainfall levels a) March to April 2019 b) April 2018 to April 2019	**9**

(a)



(b)



GCG	**Mr and Mrs Graff** 20a Ferncroft Avenue	**Figure**
	Surface flooding risk	**10**

Appendix C

SIMPSON Retaining Wall Calculations Summary dated March 2020

BRIEF

Simpson has been instructed to prepare the structural engineering design for the arrangements for the basement extension of the 20a Ferncroft Avenue property in London. The planned extension will require the design of RC walls capable of withstanding the applied earth and surcharge pressures during both the construction phase and the final permanent arrangement.

The purpose of this calculation report is to assess the forces that will be acting on the earth retaining structures during the construction and the final permanent arrangement. Determine the wall sizes and rebar required for the structure to pass both equilibrium and strength checks in accordance to the relevant design standards.

DESIGN PHILOSOPHY

The retaining wall design will be in accordance to the relevant British standards and Eurocodes.

The minimum concrete grade used in the design of the RC walls is C32/40 and the minimum steel grade for the reinforcement is 500N/mm^2.

The soil information used for the structural checks was obtained from the SI report prepared by Risk Management Ltd – Project number RML 7096 “Site Investigation at 20Fercroft Avenue, Hampstead” dated September 2019.

The density of masonry is taken as 21kN/m^3.

There are 5No locations around the property that will be looked at to consider all possible retaining wall scenarios. For each location both the construction and permanent phases will be considered. Refer to the “Retaining Wall Design Outline Mark-up” within this calculation report for information on the retaining wall locations within the site.

1. **Retaining Wall No 1**: Located at the front of the building, the retaining wall is located outside of the existing building footprint.

    a. Temporary – There are no loads from the existing structure which are acting directly on the retaining wall. During the temporary phase the ground water will be drained, therefore the water table is not considered for the design. The retained clay soil is taken as consolidated as the SI report describes it as FIRM, therefore the consolidated depth z_c of the clay will not be acting directly on the wall. The forces acting on the wall considered for this phase are as follows.
        i. Horizontal pressure from the retained soil (reduced because of clay consolidation).
        ii. Horizontal pressure from the surcharge of the construction equipment and vehicles nearby.
        iii. Horizontal pressure from water in fissures (occurs in the event of precipitation).
    The wall will be built in sections and with the clay dug out locally where work is being executed. Construction of the wall will begin next to the existing structure to which it will be tied in. Therefore checking for sliding is unnecessary.

    b. Permanent – The water table is considered to be at 1m below ground level throughout the site, 1375mm of retained water. In its permanent state the wall will be pinned top and bottom. The top soil consolidation is not considered in the permanent state. The forces acting on the wall considered for this stage are as follows:
        i. Horizontal pressure from the retained soil.
        ii. Horizontal pressure from the surcharge of vehicles nearby.
        iii. Horizontal pressure from retained water in soil.
        iv. Vertical reaction from the internal slab acting on the toe of the wall.
    The wall will be tied into the main building structure. Sliding and Overturning will not be considered.

2. **Retaining Wall No 2**: Located at the rear of the building, the retaining wall is part of the new garden extension and external to the footprint of the existing and proposed building.

    a. Temporary – There are no loads from the existing structure which are acting directly on the retaining wall. During the temporary phase the ground water will be drained, therefore the water table is not considered for the design. The retained soil is made ground as shown in the SI report, while the base soil is clay. No vehicle surcharge will be considered as the rear of the building is not accessible to vehicles. The forces acting on the wall considered for this phase are as follows.

i. Horizontal pressure from the retained soil.

b. Permanent – The water table is considered to be at 1m below ground level throughout the site, 885mm of retained water. In its permanent state the wall will be pinned at the bottom. The forces acting on the wall considered for this stage are as follows:

i. Horizontal pressure from the retained soil.
ii. Horizontal pressure from the surcharge garden decorations/furniture.
iii. Vertical reaction on toe from domestic live loads and finishes.
iv. Horizontal pressure from retained water in soil.

The wall will be tied into the main building structure. Sliding and Overturning will not be considered.

3. **Retaining Wall No.3**: Located on the N-W side of the building bordering the neighboring property, will be supporting the party wall as well as preventing soil from entering the basement.

a. Temporary – The wall will be supporting the foundation loads of the existing structures, only the permanent actions will be considered during this stage. During the temporary phase the ground water will be drained, therefore the water table is not considered for the design. The consolidated clay is conservatively ignored for the design of the retaining walls which are underpinning the existing foundations. The soil above the top level of the RC wall is taken as a dead load surcharge (see engineering sketches in report for more information). There is no live load surcharge acting on the wall. The forces acting on the wall considered for this phase are as follows.

i. Horizontal pressure from the retained soil (consolidation ignored).
ii. Horizontal pressure from the surcharge of ground beyond the top level of the RC wall.
iii. The vertical reaction from the existing foundations (Dead Load only), from walls, floors and roof.

b. Permanent – The water table is considered to be at 1m below ground level throughout the site, 1250mm of retained water. In its permanent state the wall will be pinned top and bottom. The top soil consolidation is not considered in the permanent state. The forces acting on the wall considered for this stage are as follows:

i. Horizontal pressure from the retained soil.
ii. Horizontal pressure from the surcharge of ground beyond the top level of the RC wall..
iii. Horizontal pressure from retained water in soil.
iv. The vertical reaction from the existing foundations (Dead and Live Load), from walls, floors and roof.
v. Vertical reaction from the internal slab acting on the toe of the wall.

The wall will be tied into the main building structure. Sliding and Overturning will not be considered.

4. **Retaining Wall No 4**: Located on the S-E side of the building bordering the garage and the driveway, will be supporting the external cavity wall as well as preventing soil from entering the basement.

a. Temporary – The wall will be supporting the foundation loads of the existing structures, only the permanent actions will be considered during this stage. During the temporary phase the ground water will be drained, therefore the water table is not considered for the design. The consolidated clay is conservatively ignored for the design of the retaining walls which are underpinning the existing foundations. The soil above the top level of the RC wall is taken as a dead load surcharge (see engineering sketches in report for more information). The forces acting on the wall considered for this phase are as follows.

i. Horizontal pressure from the retained soil (consolidation ignored).
ii. Horizontal pressure from the surcharge of ground beyond the top level of the RC wall.
iii. Horizontal pressure from the surcharge of vehicles nearby.
iv. The vertical reaction from the existing foundations (Dead Load only), from walls, floors and roof.

b. Permanent – The water table is considered to be at 1m below ground level throughout the site, 1250mm of retained water. In its permanent state the wall will be pinned top and bottom. The top soil consolidation is not considered in the permanent state. The forces acting on the wall considered for this stage are as follows:
   i. Horizontal pressure from the retained soil.
   ii. Horizontal pressure from the surcharge of ground beyond the top level of the RC wall.
   iii. Horizontal pressure from retained water in soil.
   iv. Horizontal pressure from the surcharge of vehicles nearby.
   v. The vertical reaction from the existing foundations (Dead and Live Load), from walls, floors and roof.
   vi. Vertical reaction from the internal slab acting on the toe of the wall.

   The wall will be tied into the main building structure. Sliding and Overturning will not be considered.

5. **Retaining Wall No 5**: Located to the rear of the building it will become the permanent rear wall of the new basement. The design conditions/parameters are similar to those of retaining wall No 1. The differences with retaining wall No 1 are described below.

   a. Temporary – Same design parameters as R.W. No 1, with the exception of the absence of vehicle access to the rear of the building.
      i. Horizontal pressure from the retained soil (reduced because of clay consolidation).
      ii. Horizontal pressure from water in fissures (occurs in the event of precipitation).

   b. Permanent – Same design parameters as R.W. No 1, with the exception of a single story of the new building extension above causing additional surcharge load. The surcharge from the house is comparable to that used for the vehicles in the design of R.W. No 1.
      i. Horizontal pressure from the retained soil.
      ii. Horizontal pressure from house extension above.
      iii. Horizontal pressure from retained water in soil.
      iv. Vertical reaction from the internal slab acting on the toe of the wall.

      The wall will be tied into the main building structure. Sliding and Overturning will not be considered.

DESIGN INFORMATION

Soil information:

- FIRM/STIFF Silty-CLAY (Most of the site down to 4.0m from the ground level. Refer to the SI report and attached borehole reference drawing).

  Allowable Bearing Pressure = 150 kN/m^2

  Density of saturated soil = 19 kN/m^2

  Wall adhesion = 25 kN/m^2 (conservatively taken the passive wall adhesion – See SI report)

  Internal Angle of Friction = 20^o

- Top Layer MADE GROUND (Crushed road stone or Clinker and Brick Fill first 700mm. Refer to the SI report and attached borehole reference drawing).

  Allowable Bearing Pressure = N/A (new foundations to be taken down to natural soil)

  Density of saturated soil = 19 kN/m^2

  Internal Angle of Friction = 30^o

Loading:

The density of masonry is taken as $21kN/m^3$.

Wind loading is not relevant to this design and has been ignored.

Vehicle and/or equipment surcharge is taken as $10kN/m^2$.

Retaining Wall No 1: (These loading are also valid for R.W.5 – The R.W.1 design will also be valid for R.W.5)

Temporary Loading

- Horizontal pressure from the soil is calculated using Tekla TEDDS software. Refer to the calculations in the following section.
- The variable load surcharge (Non heavy-duty vehicle loading) is 5.0 kN/m^2.
- Horizontal pressure caused by water captured in the clay fissures = $0.5 \times 1.15^2 m \times 10kN/m^3 = 6.0kN/m$ (approximately)

Permanent Loading

- Horizontal pressure from the soil is calculated using Tekla TEDDS software. Refer to the calculations in the following section.
- The variable load surcharge (Non heavy-duty vehicle loading) is 5.0 kN/m^2.
- A permanent surcharge of 15 kN/m^2 (R.W. 5 load case)
- Horizontal pressure from the retained water is calculated using Tekla TEDDS software. Refer to the calculations in the following section.
- Loading on toe caused by the new slab and wall. Wall = 24 x 0.2m x 2.3m = 11.04 kN/m and the Slab = 24 x 0.25m x 1.2m = 7.2 kN/m and LL = 1.5 kN/m^2 x 1.2m = 1.8 kN/m

Retaining Wall No 2:

Temporary Loading

- Horizontal pressure from the soil is calculated using Tekla TEDDS software. Refer to the calculations in the following section.

Permanent Loading

- Horizontal pressure from the soil is calculated using Tekla TEDDS software. Refer to the calculations in the following section.
- The variable load surcharge (conservative loading for garden furniture) is 10 kN/m^2.
- Horizontal pressure from the retained water is calculated using Tekla TEDDS software. Refer to the calculations in the following section.
- Half of the extent of the proposed slab was modeled into the permanent stage TEDDS model to mimic the behavior of the completed structure without the need to add point load reaction on the toe.

Retaining Wall No 3:

Temporary Loading

- Horizontal pressure from the soil is calculated using Tekla TEDDS software. Refer to the calculations in the following section.
- The permanent load surcharge = 0.6m x $19kN/m^3$ x Ka= 5.7 kN/m^2 (approximated)
- The vertical load from the existing structure – Wall = $21kN/m^3$ x 0.2m x 9.0m = 37.8kN/m – Floors = $0.75kN/m^2$ x (6.0 / 2)m x 5 = 11.25kN/m – Roof = $0.8kN/m^2$ x 1.41 x (6.0 / 2)m = 3.4kN/m so a total of 52.5kN/m.

Permanent Loading

- Horizontal pressure from the soil is calculated using Tekla TEDDS software. Refer to the calculations in the following section.
- The permanent load surcharge = 0.6m x $19kN/m^3$ x Kp= 5.7 kN/m^2 (approximated)
- Horizontal pressure from the retained water is calculated using Tekla TEDDS software. Refer to the calculations in the following section.

- The vertical load from the existing structure – DL of 52.5kN/m and LL = 1.5kN/m^2 x (6.0 / 2)m x 5 = 22.5kN/m.
- Loading on toe caused by the new slabs and wall. Wall = 24 x 0.2 x 2.3 = 11.04 kN/m and the Slabs = (24kN/m^3 x 0.25m x 3m) + (24kN/m^3 x 0.3m x3m) = 39.6 kN/m and LL = 1.5 kN/m^2 x 3.0m x 2 = 9.0kN/m

<u>Retaining Wall No 4:</u>

Temporary Loading

- Horizontal pressure from the soil is calculated using Tekla TEDDS software. Refer to the calculations in the following section.
- The permanent load surcharge = 0.6m x 19kN/m^3 x Ka = 4.75 kN/m^2 (approximated)
- The variable load surcharge (Non heavy-duty vehicle loading) is 5.0 kN/m^2.
- The vertical load from the existing structure – Wall = 21kN/m^3 x 0.2m x 9.0m = 37.8kN/m – Floors = 0.75kN/m^2 x (6.0 / 2)m x 2 = 4.5kN/m – Roof = 0.8kN/m^2 x 1.41 x (6.0 / 2)m = 3.4kN/m so a total of 45.7kN/m.

Permanent Loading

- Horizontal pressure from the soil is calculated using Tekla TEDDS software. Refer to the calculations in the following section.
- The permanent load surcharge = 0.6m x 19kN/m^3 x Kp= 4.75 kN/m^2 (approximated)
- Horizontal pressure from the retained water is calculated using Tekla TEDDS software. Refer to the calculations in the following section.
- The variable load surcharge (Non heavy-duty vehicle loading) is 5.0 kN/m^2.
- The vertical load from the existing structure – DL of 45.7kN/m and LL = 1.5kN/m^2 x (6.0 / 2)m x 2 = 9.0kN/m.
- Loading on toe caused by the new slabs and wall. Wall = 24 x 0.2 x 2.3 = 11.04 kN/m and the Slabs = (24kN/m^3 x 0.25m x 3m) + (24kN/m^3 x 0.3m x3m) = 39.6 kN/m and LL = 1.5 kN/m^2 x 3.0m x 2 = 9.0kN/m

<u>Retaining Wall No 5:</u> Refer to Retaining Wall No 1.

CALCULATIONS & WALL DIMENSIONS

The calculations and structural/stability cheeks were performed with the aid of the engineering design software "Tekla TEDDS". The results obtained from the analysis and designs are tabulated in the summaries below. All of the design information laid out in the previous chapters was used as inputs for the TEDDS analysis.

Retaining Wall No.1 & No.5
Temporary Phase

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.10

Analysis summary

Description	Unit	Capacity	Applied	F o S	Result
Sliding stability	kN/m	37.55	23.6	1.591	PASS
Overturning stability	kNm/m	22	21.6	1.019	PASS
Bearing pressure	kN/m^2	150	84.2	1.781	PASS

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Stem p0 rear face - Flexural reinforcement	mm^2/m	565.5	367.5	0.65	PASS
Stem p0 - Shear resistance	kN/m	123.0	20.1	0.16	PASS
Base bottom face - Flexural reinforcement	mm^2/m	565.5	480.5	0.85	PASS
Base - Shear resistance	kN/m	146.7	30.2	0.21	PASS
Transverse stem reinforcement	mm^2/m	392.7	300.0	0.76	PASS

Retaining Wall No.1 & No.5
Permanent Phase

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.10

Analysis summary

Description	Unit	Capacity	Applied	F o S	Result
Bearing pressure	kN/m^2	150	48	3.124	PASS

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Stem max front face - Flexural reinforcement	mm^2/m	565.5	367.5	0.65	PASS
Stem p0 rear face - Flexural reinforcement	mm^2/m	565.5	367.5	0.65	PASS
Stem p0 - Shear resistance	kN/m	123.0	74.3	0.60	PASS
Stem p1 front face - Flexural reinforcement	mm^2/m	565.5	367.5	0.65	PASS
Stem p1 - Shear resistance	kN/m	123.0	29.6	0.25	PASS
Base top face - Flexural reinforcement	mm^2/m	565.5	518.1	0.92	PASS
Base bottom face - Flexural reinforcement	mm^2/m	565.5	480.5	0.85	PASS
Base - Shear resistance	kN/m	146.7	18.1	0.12	PASS
Transverse stem reinforcement	mm^2/m	392.7	300.0	0.76	PASS
Transverse base reinforcement	mm^2/m	392.7	113.1	0.29	PASS

Retaining Wall No. 2
Temporary Phase

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.10

Analysis summary

Description	Unit	Capacity	Applied	F o S	Result
Sliding stability	kN/m	32.2	14.1	2.283	PASS
Overturning stability	kNm/m	15	11.5	1.308	PASS
Bearing pressure	kN/m^2	150	53.9	2.785	PASS

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Stem p0 rear face - Flexural reinforcement	mm^2/m	565.5	367.5	0.65	PASS
Stem p0 - Shear resistance	kN/m	123.0	10.3	0.08	PASS
Base top face - Flexural reinforcement	mm^2/m	565.5	518.1	0.92	PASS
Base bottom face - Flexural reinforcement	mm^2/m	565.5	480.5	0.85	PASS
Base - Shear resistance	kN/m	146.7	20.4	0.14	PASS
Transverse stem reinforcement	mm^2/m	392.7	300.0	0.76	PASS
Transverse base reinforcement	mm^2/m	392.7	113.1	0.29	PASS

Retaining Wall No.2
Permanent Phase

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.10

Analysis summary

Description	Unit	Capacity	Applied	F o S	Result
Bearing pressure	kN/m^2	150	29.5	5.085	PASS

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Stem p0 rear face - Flexural reinforcement	mm^2/m	565.5	367.5	0.65	PASS
Stem p0 - Shear resistance	kN/m	123.0	33.6	0.27	PASS
Base top face - Flexural reinforcement	mm^2/m	565.5	367.5	0.65	PASS
Base bottom face - Flexural reinforcement	mm^2/m	754.0	357.9	0.47	PASS
Base - Shear resistance	kN/m	114.8	22.2	0.19	PASS
Transverse stem reinforcement	mm^2/m	392.7	300.0	0.76	PASS
Transverse base reinforcement	mm^2/m	392.7	150.8	0.38	PASS

Retaining Wall No. 3
Temporary Phase

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.10

Analysis summary

Description	Unit	Capacity	Applied	F o S	Result
Sliding stability	kN/m	48.7	40.2	1.211	PASS
Overturning stability	kNm/m	63.2	45	1.402	PASS
Bearing pressure	kN/m^2	150	132.8	1.129	PASS

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Stem p0 rear face - Flexural reinforcement	mm^2/m	565.5	412.7	0.73	PASS
Stem p0 - Shear resistance	kN/m	132.6	34.7	0.26	PASS
Base bottom face - Flexural reinforcement	mm^2/m	754.0	480.5	0.64	PASS
Base - Shear resistance	kN/m	146.7	73.1	0.50	PASS
Transverse stem reinforcement	mm^2/m	392.7	330.0	0.84	PASS
Transverse base reinforcement	mm^2/m	392.7	150.8	0.38	PASS

Retaining Wall No.3
Permanent Phase

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.10

Analysis summary

Note: The retaining wall is pinned top and bottom. The base has been expanded with the addition of the ground bearing slab therefore no equilibrium checks are required.

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Stem max front face - Flexural reinforcement	mm^2/m	565.5	412.7	0.73	PASS
Stem p0 rear face - Flexural reinforcement	mm^2/m	565.5	412.7	0.73	PASS
Stem p0 - Shear resistance	kN/m	132.6	49.9	0.38	PASS
Stem p1 front face - Flexural reinforcement	mm^2/m	565.5	412.7	0.73	PASS
Stem p1 - Shear resistance	kN/m	132.6	16.7	0.13	PASS
Base bottom face - Flexural reinforcement	mm^2/m	754.0	480.5	0.64	PASS
Base - Shear resistance	kN/m	146.7	57.2	0.39	PASS
Transverse stem reinforcement	mm^2/m	392.7	330.0	0.84	PASS
Transverse base reinforcement	mm^2/m	392.7	150.8	0.38	PASS

Job No	Description	Calc No:	9
P18-461	20a FERNCROFT AVENUE	Date:	12.03.2020
		By:	AT
		Checked:	-

Retaining Wall No. 4
Temporary Phase

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.10

Analysis summary

Description	Unit	Capacity	Applied	F o S	Result
Sliding stability	kN/m	52.5	47.7	1.101	PASS
Overturning stability	kNm/m	67.5	54.7	1.236	PASS
Bearing pressure	kN/m^2	150	135.2	1.109	PASS

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Stem p0 rear face - Flexural reinforcement	mm^2/m	565.5	412.7	0.73	PASS
Stem p0 - Shear resistance	kN/m	132.6	40.8	0.31	PASS
Base bottom face - Flexural reinforcement	mm^2/m	754.0	480.5	0.64	PASS
Base - Shear resistance	kN/m	146.7	83.1	0.57	PASS
Transverse stem reinforcement	mm^2/m	392.7	330.0	0.84	PASS
Transverse base reinforcement	mm^2/m	392.7	150.8	0.38	PASS

Retaining Wall No.4
Permanent Phase

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.10

Analysis summary

Note: The retaining wall is pinned top and bottom. The base has been expanded with the addition of the ground bearing slab thereofre no equilibrium checks are required.

Design summary

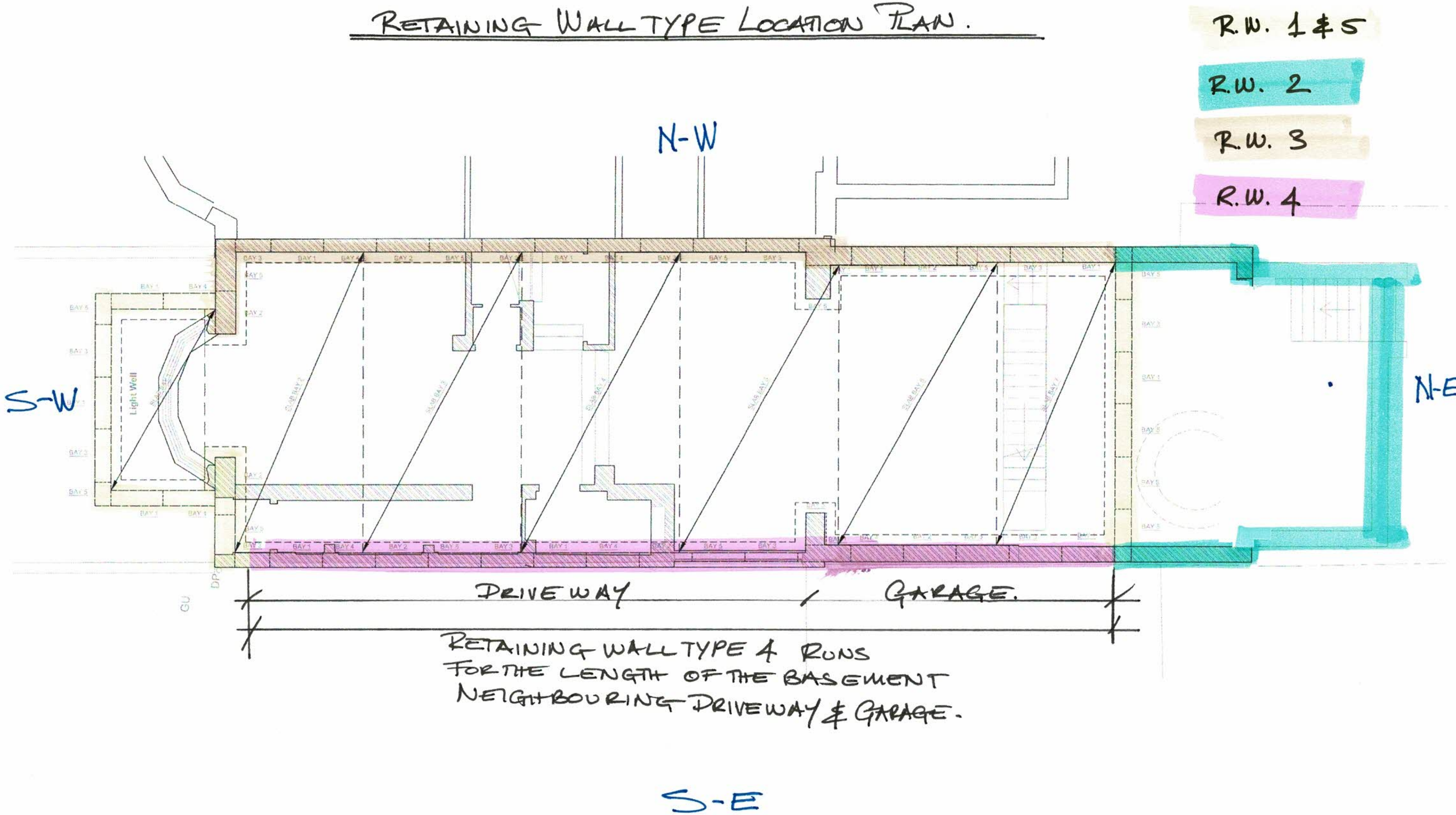
Description	Unit	Provided	Required	Utilisation	Result
Stem max front face - Flexural reinforcement	mm^2/m	565.5	412.7	0.73	PASS
Stem p0 rear face - Flexural reinforcement	mm^2/m	565.5	412.7	0.73	PASS
Stem p0 - Shear resistance	kN/m	132.6	55.4	0.42	PASS
Stem p1 front face - Flexural reinforcement	mm^2/m	565.5	412.7	0.73	PASS
Stem p1 - Shear resistance	kN/m	132.6	20.0	0.15	PASS
Base top face - Flexural reinforcement	mm^2/m	565.5	518.1	0.92	PASS
Base bottom face - Flexural reinforcement	mm^2/m	565.5	480.5	0.85	PASS
Base - Shear resistance	kN/m	146.7	50.9	0.35	PASS
Transverse stem reinforcement	mm^2/m	392.7	330.0	0.84	PASS
Transverse base reinforcement	mm^2/m	392.7	113.1	0.29	PASS

APPENDIX

- Retaining wall Designed Sections Locations on Plan
- Preliminary Design Information Work Sheets (RW2 omitted because of design simplicity)

ALL DIMS IN MM U.N.O.

RETAINING WALL TYPE LOCATION PLAN.



RETAINING WALL TYPE 4 RUNS FOR THE LENGTH OF THE BASEMENT NEIGHBOURING DRIVEWAY & GARAGE.

Appendix D

Campbell Reith's Basement Impact Assessment updated reply to Audit Query Tracker

20a Ferncroft Avenue, NW3 7PH
BIA – AUDIT
Reply to Audit Query Tracker

Query No.	Response	
Query No. 1 Stability Clarification required with respect to excavation depth and nature of basement retaining walls.	The proposed foundations at the new extension and rear garden terrace are set 1.85m below existing ground level, reference to NHBC standards chapter 4.2 building near trees and a review of proposed foundations depths adjacent to the Cherry Tree T1 to be removed require a minimum foundation depth of 1.85m. The proposed main basement retaining wall excavations are approximately 3m and 3.5m below the existing stepped ground floor levels.	
Query No. 2 Stability Retaining wall calculations to be revised to reflect recommendations in hydrogeological assessment.	The retaining wall calculations are based on existing ground water levels, but as recommended within the Hydrological assessment the retaining walls will be designed for a higher water level with the ground water level set 1m below top of retained ground level.	
Query No. 3 Stability Building damage assessment to be reviewed to ensure consistent with anticipated ground movements	GCG: the building damage assessment accounts for the predicted ground movements, which include settlements of the perimeter walls due to the underpinning process (i.e. 5-10mm localised under the walls) and settlements and horizontal movements due to excavation (i.e. up to 5mm and reducing with distance from the retaining walls – see figures 9 and 10). The trigger levels quoted in the report refer to measurements taken on the retaining structures and account for the movements described above. They are conservatively assessed in the interest of the neighbouring properties and assume that good workmanship is adopted throughout the construction process at 20a Ferncroft Avenue.	

Query No. 4 Stability Consideration to be given to impact of tree removal.	A paragraph has been added to the GMA. The new structures at 20a Ferncroft Avenue will be designed accounting for the removal of the Cherry tree and the presence of the other trees in the garden, in accordance with NHBC standards. The removal of the Cherry tree has the potential to cause ground heave that could affect 20 Ferncroft Avenue. However, the tree to be removed is currently in poor condition, with restrained root grow. It is therefore likely that its current water demand is very limited and its removal would have small effects on the ground and nearby structures.	
Query No. 5 Hydrogeology / Hydrology Impact of infiltration tank to be considered	The hydrogeological report refers to a SUDS system, which coincides with the attenuation tank mentioned in the main text of the BIA. This has been clarified in the main text of the BIA. The impact of the SUDS/attenuation tank is discussed in the hydrogeological report (sections 4.3 and 6).	