# GROUND INVESTIGATION REPORT

Athlone House
Hampstead Lane
Highgate
London N6

Client: Virtus Real Estate

J16075

June 2017




Geotechnical & Environmental Associates

# Document Control

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| **Report prepared by** | Matthew Legg BEng FGS | | |
| **Report checked and approved for issue by** | Steve Branch BSc MSc CGeol FGS FRGS MIEnvSc | | |

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This report has been issued by the GEA office indicated below. Any enquiries regarding the report should be directed to the office indicated or to Steve Branch in our Herts office.

| | | | |
|---|---|---|---|
| ✓ | Hertfordshire | tel 01727 824666 | mail@gea-ltd.co.uk |
| | Nottinghamshire | tel 01509 674888 | midlands@gea-ltd.co.uk |

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This report is intended as a Ground Investigation Report (GIR) as defined in BS EN1997-2, unless specifically noted otherwise. The report is not a Geotechnical Design Report (GDR) as defined in EN1997-2 and recommendations made within this report are for guidance only.

# CONTENTS

# EXECUTIVE SUMMARY

*This executive summary contains an overview of the key findings and conclusions. No reliance should be placed on any part of the executive summary until the whole of the report has been read. Other sections of the report may contain information that puts into context the findings that are summarised in the executive summary.*

## BRIEF

This report describes the findings of a ground investigation carried out by Geotechnical and Environmental Associates Limited (GEA) on the instructions of Virtus Real Estate, with respect to the refurbishment of the existing property. The work is to include a single storey rear extension and single level basement for a swimming pool, in addition to the refurbishment of a number of outbuildings and the construction of a new outhouse. GEA has previously carried out a desk study and several phases of ground investigation at the site (ref: J12224, dated July 2014 and J16075, dated June 2016). Supplementary investigation has now been carried out to provide further information on the ground conditions, with the additional investigation data used to revise and update the previous report.

## SUMMARY OF PREVIOUS DESK STUDY FINDINGS

At the time of the earliest map studied, dated 1870, the site was occupied by Fitzroy House and its associated grounds. By 1896, Fitzroy House had been replaced with what appears to be the existing manor house, although at that time it was known as Caen Wood Towers. The map dated 1896 also shows a large pond in the northwestern corner of the site. By 1935, the house had been extended northwards and a rectangular feature constructed within the grounds to the west, which on later maps is annotated as a tennis court. The site remained unchanged until between 1953 and 1964, when a large rectangular building was constructed adjacent to the eastern elevation of the original manor house. It is on the map dated 1964 that the site is first referred to as Athlone House and is stated as forming part of the Middlesex Hospital. The site remained unchanged until between 2006 and 2012, when the large rectangular building to the east was demolished, along with the northern extension to the existing building.

## GROUND CONDITIONS.

The ground investigation encountered a generally moderate thickness of made ground over the Claygate Member of the London Clay Formation, which was proved to the maximum depth investigated. In areas close to the house, the made ground was found to extend to depths of between 0.80 m and 1.80 m, whilst in the lawn areas surrounding the house, it extended to a maximum depth of 0.40 m. It generally comprised brown clayey silt with rootlets, gravel, brick, concrete, coal and timber fragments. The Claygate Member generally comprised an initial horizon of firm becoming stiff medium to high strength brown and orange-brown mottled grey silty very sandy clay, with pockets of clayey fine sand and sandy silt, to depths of 6.00 m and 7.30 m. This initial layer was underlain by stiff brownish grey silty sandy clay with partings and pockets of pale grey silt to depths of 7.30 m and 9.00 m, over stiff high strength dark grey silty clay to clayey silt to depths of 12.00 m and 15.00 m. This was in turn underlain by very stiff high strength to very high strength dark grey silty, locally sandy, clay with traces of selenite, which was proved to the maximum depth investigated, of 20.00 m. During drilling of the boreholes, groundwater was encountered at depths of 12.10 m and 12.50 m, corresponding to levels of between 101.43 m OD and 99.75 m OD. Subsequent monitoring of the standpipes installed in boreholes during several phases over a number of years has measured groundwater at depths of between 9.02 m and 10.21 m (104.42 m OD to 101.99 m OD). The made ground has been found to be generally free from significant contamination. However, a sample recovered from the base of the pond in the northwestern corner of the site was found to contain elevated concentrations of lead, sulphide, benzo(a)pyrene and TPH.

## RECOMMENDATIONS

The proposed basement will be excavated to a depth of 3.20 m, corresponding to a level of 109.55 m OD, in order to match the level of the existing basement. It is also understood that it is proposed to utilise the available space on the site by excavating the basement in an open cut, which is considered to be a suitable solution. For the low-rise outbuildings, spread foundations should be a suitable foundation solution, although founding depths will need to be determined in accordance with NHBC guidelines and will need to extend beyond the depth of any desiccation. Foundations at a minimum depth of 0.90 m may be designed to apply a net allowable bearing pressure of 100 $kN/m^2$. Further testing and remediation is likely to be required around the existing pond in the northwest of the site.

# Part 1: INVESTIGATION REPORT

This section of the report details the objectives of the investigation, the work that has been carried out to meet these objectives and the results of the investigation. Interpretation of the findings is presented in Part 2.

## 1.0 INTRODUCTION

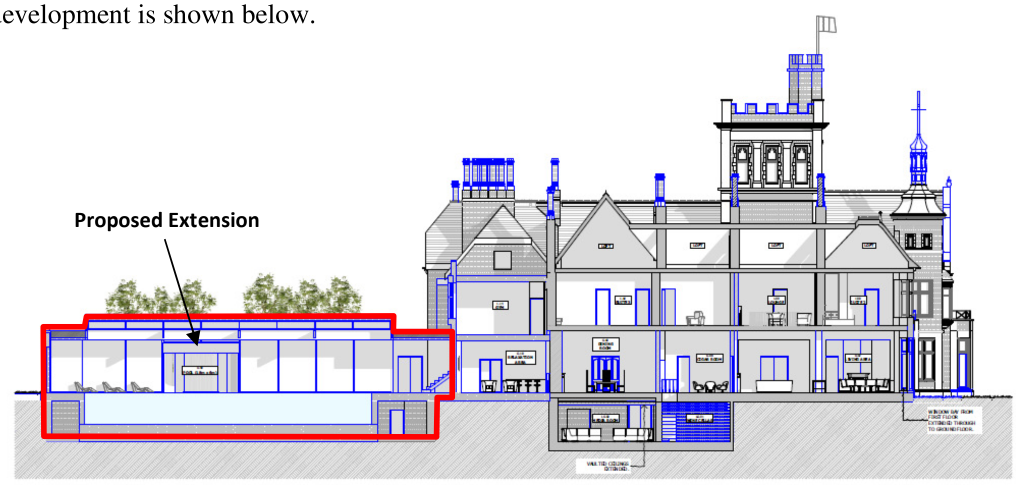
Geotechnical and Environmental Associates (GEA) has been commissioned by Virtus Real Estate, to carry out a ground investigation at the site of Athlone House, Hampstead Lane, N6 4RU. The site has been the subject of a desk study and a number of phases of investigation with the following reports being produced:

- *Report on Ground Water for Athlone House* - LBH Wembley (report ref LBH2921(a), dated September 2003);
- *A Groundwater Assessment Report* - RPS Health, Safety and Environment (report ref: FLC1578.002L, dated May 2004);
- *Desk Study and Basement Impact Assessment* – GEA (report ref J12224 report issue 4, dated July 2014); and
- *Ground Investigation and Basement Impact Assessment Report* – GEA (report ref J16075 report issue 2, dated June 2016). This second issue of the report reflected a number of changes to the proposed development.

Copies of the LBH Wembley and RPS reports were reviewed by GEA as part of the 2014 Desk Study and Basement Impact Assessment Report and the findings are discussed herein where appropriate. Supplementary investigation has now been carried out to provide further information on the ground conditions and existing foundations, with the additional investigation data used to revise and update the most recent report.

### 1.1 Proposed Development

It is understood that it is proposed to refurbish and extend the existing detached property, which will include the construction of a single storey rear extension that will house an indoor swimming pool within a single level basement. A cross-section through the proposed development is shown below.



In addition to the works to the main house, it is also proposed to refurbish and extend two buildings along the northern boundary, whilst also constructing a 2000 $m^2$ outhouse in the northwestern corner of the site.

This report is specific to the proposed development and the advice herein should be reviewed once the development proposals have been finalised.

## 1.2 Purpose of Work

The principal technical objectives of the work carried out were as follows:

- to review the previous desk study and ground investigation;
- to identify the configuration and bearing stratum of existing foundations;
- to provide advice for the design of soakaway drainage and new areas of hardstanding; and
- to provide additional advice with respect to the design of suitable foundations and retaining walls for the proposed development.

## 1.3 Scope of Work

In order to meet the above objectives, an intrusive ground investigation was carried out which comprised, in summary, the following activities:

- ten trial pits manually and mechanically excavated to depths of between 0.90 m and 2.30 m;
- groundwater monitoring of existing standpipes on three occasions over a one-month period;
- ground movement analysis of the proposed basement structure; and
- provision of a report presenting and interpreting the above data, together with our advice and recommendations with respect to the proposed development.

The findings of the above investigation were presented in the most recent report. An supplementary phase of investigation has now been carried out in order to further investigate the shallow ground conditions to provide additional information for the design of spread foundations. The supplementary investigation comprised the following activities:

- a total of 19 manually excavated trial pits to expose additional existing foundations;
- a series of four opendrive percussive sampler boreholes advanced to a depth of 6.00 m;
- Standard Penetration Tests (SPTs) carried out at regular intervals within the boreholes to provide quantitative data on the strength of the shallow soils;
- laboratory testing of selected soil samples for geotechnical purposes and for the presence of contamination; and

- provision of a report presenting and interpreting the above data, together with our advice and recommendations with respect to the proposed development.

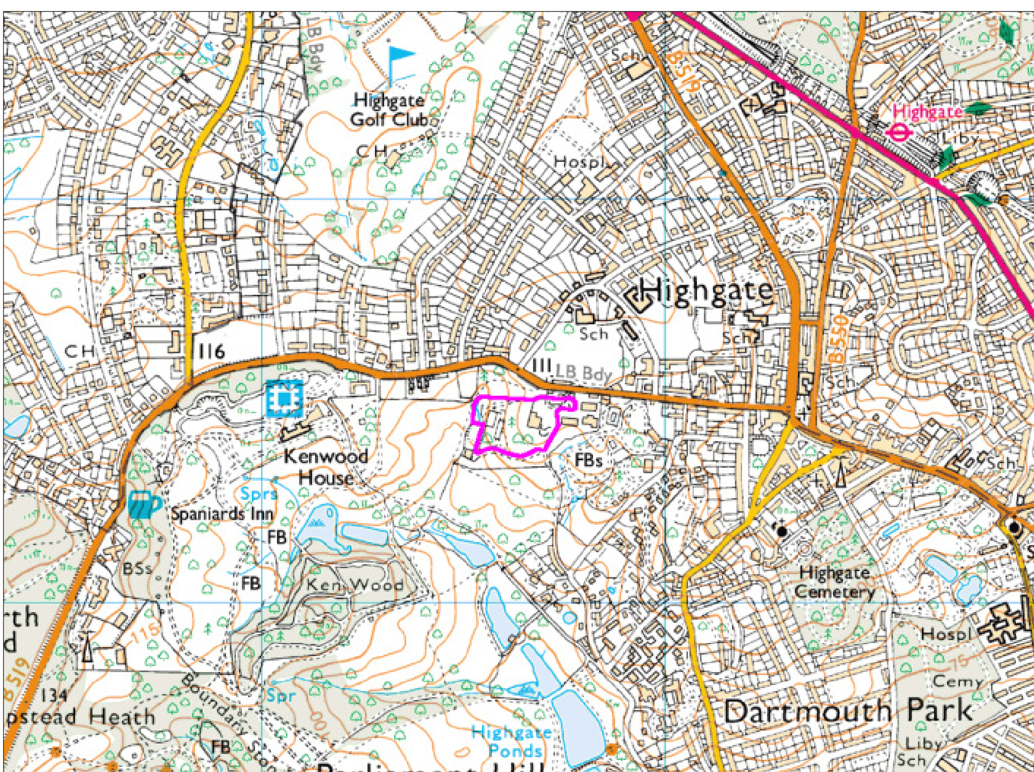
### 1.4 Limitations

The conclusions and recommendations made in this report are limited to those that can be made on the basis of the investigation. The results of the work should be viewed in the context of the range of data sources consulted, the number of locations where the ground was sampled and the number of soil, gas or groundwater samples tested; no liability can be accepted for information in other data sources or conditions not revealed by the sampling or testing. Any comments made on the basis of information obtained from the client or other third parties are given in good faith on the assumption that the information is accurate; no independent validation of such information has been made by GEA.

## 2.0 THE SITE

### 2.1 Site Description

The site is located in Highgate, north London, approximately 1 km to the southwest of Highgate London Underground station, along the northern boundary of Hampstead Heath. The site may be additionally located by National Grid Reference 527754,187097 and is shown on the map below.




The site covers a roughly rectangular area with maximum dimensions of 190 m north-south by 230 m east-west and occupies an area of approximately 2.9 hectares. It fronts onto Hampstead Lane to the north and is bordered to the east by a relatively recent residential development comprising three blocks of four-storey apartments, known as Caenwood Court, and to the south and west by Hampstead Heath, which to the west comprises the grounds of Kenwood House.

The site is currently occupied by Athlone House, a three-storey and four-storey former manor house, which is located in the eastern half of the site and includes a partial single level basement, which is below the majority of the western half of the building. The area to the northeast of the house is covered in hardstanding and was an area formerly occupied by another building that adjoined the house. With the exception of this area, and a paved terrace adjacent to the western elevation of the house, the remainder of the site is occupied by soft landscaped gardens. These comprise a large lawn around the western and southern elevations of the house and dense vegetation along the southern and western boundaries of the site.

A large number of species of deciduous and evergreen trees are present and stand at heights of up to 25 m. In the northwestern corner of the site, a small pond is present, which is surrounded by reeds. This is thought to represent a spring line and the remnants of a much larger pond shown on historical maps. The house and the area of hardstanding in the northeast are situated on relatively level plateaux, although the site level slopes relatively steeply down to the south and southwest beyond the house, in keeping with the topography of the surrounding area. The slopes are at an angle of between 4° and 6°.

## 2.2 Summary of Previous Desk Study Findings

### 2.2.1 Site History

At the time of the earliest map studied, dated 1870, the site was occupied by a Manor House known as Fitzroy House and its associated grounds; online information indicates that Fitzroy House was constructed between 1838 and 1839. By 1896, Fitzroy House had been replaced with what appears to be the existing house, although at this time it was known as Caen Wood Towers. Further online information indicates that this was constructed in 1872 but incorporated the original Fitzroy House. A portrait of Caen Wood Towers, dated 1880, can be seen below.

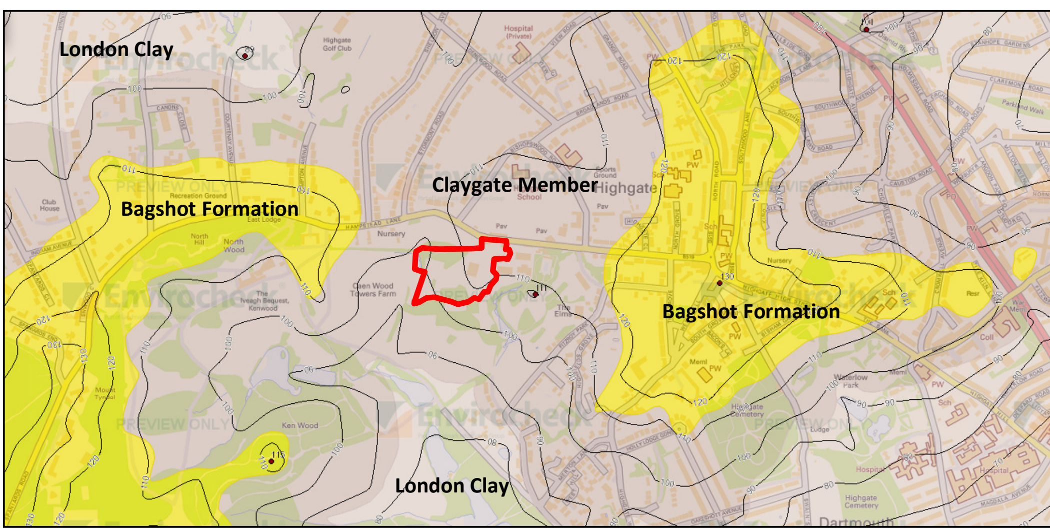
The map dated 1896 also shows a large pond in the northwestern corner of the site. The pond is still present, although is smaller than shown on the historical map, which also indicates that there was a boat house along the pond’s banks, which would give an indication of its size at this time.

By 1935, the house had been extended northwards and a rectangular feature constructed within the grounds to the west, which on later maps is annotated as a tennis court. It was also by this time that the pond in the northwestern corner of the site had been reduced to what appears to be the existing size. The site remained unchanged until between 1953 and 1964, when a large rectangular building was constructed adjacent to the eastern elevation of the original house. It is on the map dated 1964 that the site is first referred to as Athlone House, which was occupied by Middlesex Hospital. Online information[1] indicates that the Ministry of Health acquired the site in 1951 and the buildings were turned into a geriatric hospital. The rectangular building constructed to the east was used for nurses' accommodation. This information also indicates that Athlone House was used in World War I as a military convalescent hospital, known as the American Hospital for English soldiers.

Middlesex Hospital occupied the site until 2003 when it was sold to developers. The site configuration however remained unchanged until between 2006 and 2012, when a number of low-rise buildings to the north and east of the house, including the large rectangular building to the east, were demolished to make way for the construction of the existing apartment blocks to the east. The site has remained unaltered since that time until the present day.

### 2.2.2 Geology

The British Geological Survey (BGS) map of the area (sheet 256) indicates that the site is underlain by the Claygate Member of the London Clay Formation, as shown by the geological map extract below.




The geology in this area is generally horizontally bedded such that the boundary between the geological formations roughly follows the ground surface contour lines. The boundary between the Claygate Member and overlying Bagshot Beds is present approximately 350 m to the west and east of the site, at a level of between approximately 110 m OD and 115 m OD. The boundary between the Claygate Member and the upper unit of the London Clay is located approximately 300 m south of the site, at a level of 85 m OD, approximately 25 m below the site. The Claygate Member is described as typically comprising interbedded fine-grained sand, silt and clay.

A borehole was drilled by the BGS to a depth of 66.74 m (61.97 m OD) approximately 1 km to the southwest of the site. This borehole, generally referred to as 'the Hampstead Heath Borehole', placed the boundary between the Bagshot Formation and Claygate Member at a

1 Lost London Hospitals Website, *http://ezitis.myzen.co.uk/athlone.html*

level of 109 m OD, whilst the boundary between the Claygate Member and the underlying London Clay is indicated to be at a level of approximately 93.71 m OD. The boundary between these two strata can be very difficult to define, as their composition and description can be very similar.

The previous LBH Wembley Investigation comprised three boreholes, advanced to a maximum depth of 30.00 m (approximately 82.00 m OD) by means of cable percussion methods and a series of five mechanically excavated trial pits to depths of 3.50 m and 4.00 m. The investigation however included what is now the Caenwood Court site, so covered a wider area, with only a single cable percussion borehole drilled on the subject site in close proximity to the proposed basement. The borehole encountered a moderate thickness of made ground overlying the Claygate Member, which was reported to be proved to the maximum depth investigated. The Claygate Member was described as comprising an upper horizon of firm yellowish brown mottled grey silty sandy clay to a depth of 5.80 m (107.10 m OD), whereupon stiff grey fissured silty, locally sandy, clay with pockets and partings of silt and fine sand was encountered to a depth of between 18.00 m (94.90 m OD). Below that depth, very stiff grey fissured silty clay with occasional pockets and partings of silt and fine sand and was proved to the maximum depth investigated, of 21.50 m (91.40 m OD).

According to the BGS memoir, the Claygate Member comprises alternating beds of clayey silt, very silty clay, sandy silt and glauconitic silty fine sand. The lower part of the Claygate Member is generally more bioturbated. A bed of calcareous concretions is present near the base in many places. The London Clay Formation is homogenous, slightly calcareous silty clay to very silty clay, with some beds of clayey silt grading to silty fine grained sand.

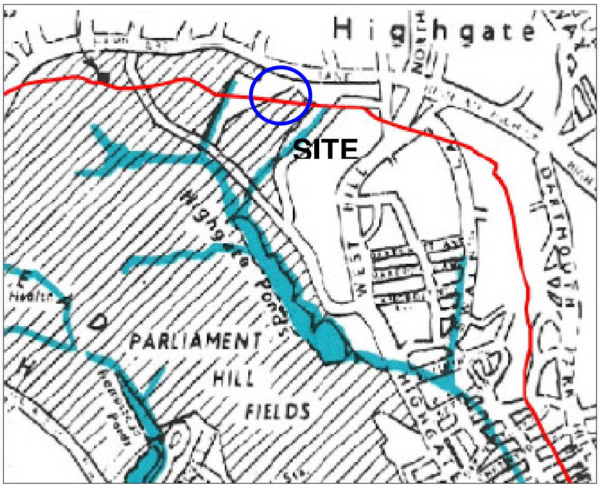
### 2.2.3 Hydrology and Hydrogeology

The Claygate Member is classified as a Secondary 'A' Aquifer, which refers to strata that contain permeable layers capable of supporting water supply at a local level and in some cases may form an important source of base flow for local rivers, as defined by the Environment Agency (EA).

Topographical maps show that the nearest surface water features are the small pond in the northwestern corner of the site, suspected of being associated with a former spring line, and a spring line that forms the source of a small stream that flows in a southerly direction towards a series of small ponds, which is situated approximately 150 m to the southeast of the site. Both of these features are at a level of between 100 m OD and 105 m OD and provide a good indication of the depth to the groundwater table below the site. The LBH Wembley investigation reportedly encountered groundwater at depths of 5.00 m (106.3 m OD) and 7.60 m (104.60 m OD). However, various reports, including the RPS report, indicate that the standpipes were found to be 'silted up' and that therefore the measured water levels were unlikely to be representative of true groundwater level.

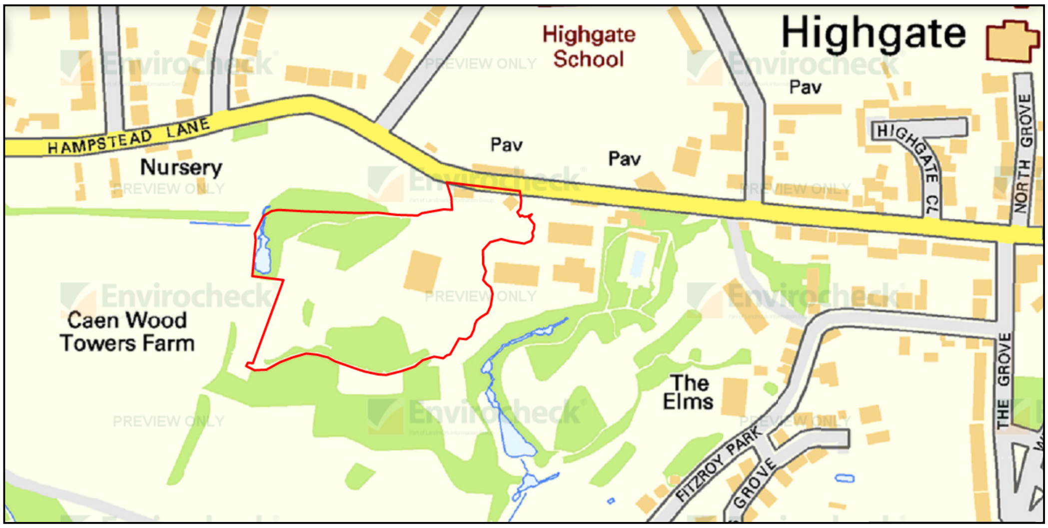
Approximately 300 m to the south of the site, in Hampstead Heath and Parliament Hill, is a further series of spring lines and ponds, which drain in a southerly direction, down the valley, towards both the Highgate and Hampstead Ponds, located approximately 400 m and 1.2 km south of the site respectively. The positions of these springs are likely to mark the boundary between the Claygate Member and underlying essentially impermeable London Clay. Within the area of Hampstead and Highgate, existing and historical springs are also present at the interface between the Claygate Member and the overlying more sandy Bagshot Beds. The springs at both geological boundaries have been the source of a number of London's "lost" rivers, notably the Fleet, Westbourne and Tyburn.

The extract of the Lost Rivers of London Map [2] included below indicates that the pond in the northwestern corner of the site and the spring to the southeast of the site, both historically formed sources of the River Fleet. This river flowed southwards from the springs through the Highgate Ponds and on through Kentish Town and Camden Town before flowing through Clerkenwell and issuing into the Thames below Blackfriars Bridge. Although the river no longer comprises an open watercourse, surface and near surface waters will still flow towards the former river course and in particular the former spring lines.




Left: London's Lost Rivers extract map

Below: Present day locations of spring lines




On the basis of all of the above, groundwater below the site is expected to be flowing in a generally southerly direction.

The Claygate Member is predominantly cohesive in nature and therefore groundwater flow is likely to be relatively slow, although horizons of sandier soils do occur in this stratum, resulting in the permeability ranging from "very low" to "high". The Claygate Member is only designated as a Secondary Aquifer because it contains such sand horizons, which provide more permeable layers for the storage of groundwater. Where such sand beds are not present, the Claygate Member behaves hydraulically more like the underlying London Clay, which accounts for the variable permeability described above. Published data for the

2 Nicholas Barton (2000) *London's Lost Rivers.* Historical Publications Ltd

permeability of the London Clay indicates the horizontal permeability to generally range between 1 x $10^{-10}$ m/s and 1 x $10^{-8}$ m/s, with an even lower vertical permeability.
The site is not within an area at risk from flooding, as defined by the EA, and although Hampstead Lane is listed as having suffered from surface water flooding during a 1975 storm event, the site itself is not shown to be in an area known to be at risk from surface water flooding, as shown by Figure 15 of the Arup guidance report and EA surface water flood maps.

## 3.0 SUMMARY OF PREVIOUS GEA INVESTIGATION FINDINGS

The previous GEA investigation comprised three boreholes advanced to depths of 15.00 m and 20.00 m using cable percussion drilling methods. Standard penetration tests (SPTs) were carried out at regular intervals in the boreholes and disturbed and undisturbed samples were recovered for subsequent laboratory examination, geotechnical testing and contamination analysis. The boreholes were supplemented by a series of five trial pits, mechanically excavated using a JCB 3CX excavator to depths of 2.80 m and 3.10 m in order to provide additional coverage of the site and to assess the stability of the shallow soils.

The investigation encountered a generally moderate thickness of made ground over the Claygate Member of the London Clay Formation, which was proved to the maximum depth investigated. In the areas close to the house, the made ground was found to extend to depths of between 0.80 m (111.65 m OD) and 1.80 m (110.45 m OD), whilst in the lawn areas surrounding the house, made ground extended to a maximum depth of 0.40 m (109.01 m OD).

It generally comprised brown clayey silt with rootlets, gravel, brick, concrete, coal and timber fragments.

The underlying Claygate Member generally comprised an initial horizon of firm becoming stiff medium to high strength brown and orange-brown mottled grey silty very sandy clay with pockets of clayey fine sand and sandy silt. This initial horizon extended to the maximum depth investigated in the trial pits, of 3.10 m (108.94 m OD), and to depths of 6.00 m (106.25 m OD) and 7.30 m (105.34 m OD) in the cable percussion boreholes. Stiff brownish grey silty sandy clay with partings and pockets of pale grey silt was then encountered to depths of 7.30 m (104.95 m OD) and 9.00 m (103.64 m OD) and was underlain by stiff high strength dark grey silty clay to clayey silt to depths of 12.00 m (101.44 m OD) and 15.00 m (97.25 m OD). This was underlain by very stiff high strength to very high strength dark grey silty, locally sandy, clay with traces of selenite, which was proved to the maximum depth investigated, of 20.00 m (92.64 m OD).

Desiccation of the clay soils was not encountered during the investigation and laboratory testing indicated the clay to be of moderate shrinkability.

During the drilling of the boreholes, groundwater was encountered at depths of 12.10 m and 12.50 m, corresponding to levels of between 101.43 m OD and 99.75 m OD. Subsequent monitoring of the standpipes installed in the boreholes was initially carried out over a period of three weeks after the intrusive work, with a second phase of monitoring carried out from July 2014, which included periodic monitoring and continuous groundwater monitoring using level loggers over a period of seven months. The results of the monitoring indicated groundwater at depths of between 9.62 m and 10.21 m, levels of between 101.99 m OD and 103.23 m OD and the full tabulated results and corresponding graphs are included in the appendix

# 4.0 EXPLORATORY WORK

In order to determine the configuration of existing foundations, the previous investigation comprised a series of nine manually and mechanically excavated trial pits excavated adjacent to various external elevations and within the existing basement. In addition to the trial pits, the standpipes installed as part of the previous investigation were monitored on three occasions, over a one-month period.

Following the above investigation, the second phase of investigation carried out in April and May 2017 comprised an additional four boreholes drilled to a depth of 6.45 m using an opendrive sampling rig. Standard penetration tests (SPTs) were carried out at regular intervals in the boreholes to provide quantitative data on the strength of the shallow soils.

An additional 19 trial pits was manually excavated adjacent to various existing structures in order to determine the configuration of the foundations, whilst two further trial pits were mechanically excavated using a JCB 3CX excavator to facilitate soakage testing. A series of five insitu CBR tests was carried out to provide design parameters for the design of new roads and hardstanding.

The Ordnance Datum (OD) levels shown on the exploratory records have been interpolated from spot heights shown on a site survey drawing (ref: 22929_01_P, dated January 2016), which has been provided by Engineers HRW.

## 4.1 Sampling Strategy

The scope of work was specified by Engineers HRW with input from GEA. The exploratory locations were agreed on site between GEA and Engineers HRW and were positioned by an engineer from GEA to avoid known buried services.

Four samples of made ground were subject to analysis for a range of common industrial contaminants and contamination indicative parameters. For this investigation the analytical suite for the soil included a range of metals, speciation of total petroleum hydrocarbons (TPH), polycyclic aromatic hydrocarbons (PAH), total cyanide and monohydric phenols. The soil samples were selected on the basis of observations made on site, to provide a general view of the chemical conditions of the soils that are likely to be involved in a human exposure and to provide advice in respect of re-use or for waste disposal classification. The samples were also screened for the presence of asbestos.

Samples of natural soil were submitted to a geotechnical testing laboratory and a number of classification and strength test undertaken, including Atterberg limit tests.

The laboratory testing was carried out at an MCERTs and UKAS accredited laboratories with the majority of the testing suite accredited to MCERTS standards. Details of the MCERTs accreditation and test methods are included in the Appendix together with the analytical results.

# 5.0 GROUND CONDITIONS

The additional boreholes and trial pits have confirmed the findings of the previous investigation in that, below a generally moderate thickness of made ground, the Claygate Member was encountered and proved to the maximum depth investigated.

## 5.1 Made Ground

Made ground extended to depths of between 0.25 m and 1.20 m and generally comprised dark brown and brown mottled orange-brown and grey silty clay with roots and rootlets, gravel, clinker, brick, coal, charcoal, metal, plastic, slate and timber fragments.

In Trial Pit IT1, which was excavated through the existing tarmac driveway in the northeast of the site, the tarmac was found to be 50 mm in thickness and underlain by brownish grey and dark grey sand with ash, clinker, gravel, brick and concrete fragments.

No visual or olfactory evidence of contamination was noted in the made ground. However, a single hand auger was advanced through the existing natural pond in the northwestern corner of the site, which found the base of the pond to comprise a very soft dark grey silty organic clay, which extended to a depth of 0.70 m. This material was noted to contain a strong hydrocarbon odour with visible staining. A sample of this soil, along with three representative samples of the made ground, were sent to an analytical laboratory for a suite of contamination testing and the results are discussed in Section 5.4.

## 5.2 Claygate Member

The Claygate Member generally comprised an initial horizon of firm orange-brown and brown mottled orange-brown and pale grey silty sandy clay with pockets and partings of sandy silt and silty fine sand, abundant mica and occasional pockets of pyrite. The initial horizon extended to depths of between 3.30 m and 5.00 m, whereupon stiff brownish grey and dark grey silty sandy clay with partings and pockets of pale grey and yellowish brown silty sand and sandy silt was encountered to the maximum depth investigated, of 6.45 m.

In Borehole No 104, which was advanced in close proximity of a number of mature trees in the northwestern corner of the site, the clay was noted to contain roots and rootlets to a depth of 1.00 m and was assessed as being potentially desiccated to a depth of around 2.00 m. Laboratory moisture content and plasticity index tests have generally confirmed this and have also indicated the clay to be of medium volume change potential and between intermediate and high plasticity. These soils were observed to be free of any evidence of contamination.

## 5.3 Groundwater

During the drilling of the additional boreholes, groundwater was encountered in Borehole Nos 104 and 105 only, which were advanced in the northwestern corner of the site from a much lower topographic level. Groundwater was encountered at a depth of 5.00 m (94.61 m OD), with standing levels of 2.50 m (97.11 m OD) and 3.57 m (96.04 m OD) recorded respectively.

The table below indicates the results of the groundwater monitoring carried out on the standpipes installed in Borehole Nos 101 and 103, which were monitored on three occasions during the previous investigation and on a single occasion during this additional investigation.

| Date | Borehole No | Depth to water (m) [Level (m OD)] |
|---|---|---|
| 14/04/2016 | 101 | 9.14 [104.30] |
| | 103 | 9.25 [103.00] |
| 28/04/2016 | 101 | 9.16 [104.28] |
| | 103 | 9.28 [102.97] |
| 12/05/2016 | 101 | 9.02 [104.42] |
| | 103 | 9.23 [103.02] |
| 07/04/2017 | 101 | 9.18 [104.26] |
| | 103 | 9.30 [102.95] |

## 5.4 Soil Contamination

The table below sets out the results of the contamination across each phase of investigation, including the four samples recovered as part of the most recent phase of investigation; all concentrations are in mg/kg unless otherwise stated.

| Determinant | Maximum concentration recorded (mg/kg) | Minimum concentration recorded (mg/kg) | Number of samples below detection limit | Normalised upper bound US95 |
|---|---|---|---|---|
| pH | 7.8 | 4.6 | - | - |
| Arsenic | 23 | 3.6 | None | 16.4 |
| Cadmium | 1.4 | <0.1 | 8 | 0.6 |
| Chromium | 42 | 12 | None | 33.5 |
| Copper | 14 | 7.7 | None | 137.7 |
| Mercury | 0.18 | <0.10 | 8 | 0.3 |
| Nickel | 34 | 5.5 | None | 20.1 |
| Lead | **910** | 17 | None | 141 |
| Selenium | 0.43 | <0.20 | 5 | 0.8 |
| Zinc | 690 | 33 | None | 289.6 |
| Total Cyanide | <0.5 | <0.5 | All | < 0.5 |
| Total Phenols | <0.3 | <0.3 | All | < 0.3 |
| Sulphide | **360** | 1.8 | None | 3.9 |
| Total TPH | **2100** | <10 | 7 | 64.7 |
| Naphthalene | 1.2 | <0.1 | 2 | 0.8 |
| Benzo(a)pyrene | 6.1 | <0.1 | 1 | 3.1 |
| Total PAH | 62.3 | <2 | 1 | 33 |
| Total organic carbon % | 3.7 | 0.3 | None | 2.4 |
| *Note:* Figure in **bold** indicates concentration in excess of risk-based soil guideline values, as discussed below | | | | |

The contamination testing has generally revealed low concentrations of the contaminants tested within the samples of made ground. However, the sample of soil recovered from the base of the pond in the northwestern corner of the site was found to contain elevated concentrations of lead, sulphide, benzo(a)pyrene and TPH. Loose chrysotile asbestos fibres were also encountered in the sample. Given that this was a different material to the made ground encountered across the site, these elevated concentrations were omitted from the US95 calculation, with all of the US95 values falling below the respective generic guideline values.

### 5.4.1 Generic Quantitative Risk Assessment

The use of a risk-based approach has been adopted to provide an initial screening of the test results to assess the need for subsequent site-specific risk assessments. To this end contaminants of concern are those that have values in excess of generic human health risk based guideline values, which are either the CLEA[3] Soil Guideline Value where available, or a Generic Screening Value calculated using the CLEA UK Version 1.06[4] software assuming a residential end use with plant uptake, or is based on the DEFRA Category 4 Screening values[5]. The key generic assumptions for this end use are as follows:

- that groundwater will not be a critical risk receptor;
- that the critical receptor for human health will be young female children aged zero to six years old;
- that the exposure duration will be six years;
- that the critical exposure pathways will be direct contact, indigestion of soil and dust, indigestion of soil adhering vegetables, indoor dust ingestion, skin contact with indoor dust, and inhalation of indoor and outdoor dust and vapours; and
- that the building type equates to a two-storey small terraced house.

It is considered that these assumptions are acceptable for this generic assessment of this site. The tables of generic screening values derived by GEA and an explanation of how each value has been derived are included in the Appendix.

Where contaminant concentrations are measured at concentrations below the generic screening value it is considered that they pose an acceptable level of risk and thus further consideration of these contaminant concentrations is not required. However, where concentrations are measured in excess of these generic screening values there is considered to be a potential that they could pose an unacceptable risk and thus further action will be required which could include;

- additional testing to zone the extent of the contaminated material and thus reduce the uncertainty with regard to its potential risk;
- site specific risk assessment to refine the assessment criteria and allow an assessment to be made as to whether the concentration present would pose an unacceptable risk at this site; or
- soil remediation or risk management to mitigate the risk posed by the contaminant to a degree that it poses an acceptable risk.

The significance of the contamination results is considered further in Part 2 of the report.

3 *Updated Technical Background to the CLEA Model (Science Report SC050021/SR3) Jan 2009* and Soil Guideline Value reports for specific contaminants; all DEFRA and Environment Agency.

4 Contaminated Land Exposure Assessment (CLEA) Software Version 1.06 Environment Agency 2009

5 CL:AIRE (2013) *Development of Category 4 Screening Levels for Assessment of Land Affected by Contamination* Final Project Report SP1010 and DEFRA (2014) *Development of Category 4 Screening Levels for Assessment of Land Affected by Contamination* Policy Companion Document SP1010

## 5.5 Existing Foundations

The findings of the trial pits are summarised in the table below and sketches and photographs of each pit are included in the Appendix.

| Trial Pit No | Structure | Foundation detail | Bearing Stratum |
|---|---|---|---|
| 1 | Southeastern vault to existing basement Eldon Grove | Brick footing<br>Top 300 mm below basement level (109.25 m OD)<br>Base 0.47 m (109.08 m OD)<br>Lateral projection two brick corbels - 90 mm | Brown mottled dark orange-brown clayey silty SAND |
| 2 | Internal basement wall | Concrete strip<br>Top 230 m below basement level (109.05 m OD)<br>Base 0.9 m (108.65 m OD)<br>Lateral projection 260 mm | Brown mottled dark orange-brown clayey silty SAND |
| 3 | Internal basement wall | Concrete strip<br>Top 440 mm below basement level (109.11 m OD)<br>Base 0.85 m (108.70 m OD)<br>Lateral projection 430 mm | Brown mottled dark orange-brown clayey silty SAND |
| 4 | Western elevation of existing house | Likely to be a concrete footing<br>Top Not Proved<br>Base Not Proved<br>No lateral projection | Trial Pit terminated in made ground at 2.3 m (110.47 m OD) and base of footing not proved |
| 5 | Western elevation of existing house | Trial pit terminated on suspected relic drain | Not Proved |
| 5A | Western elevation of existing house | Brick footing<br>Top GL<br>Base 1.20 m (111.45 m OD)<br>No lateral projection | Orange-brown very clayey silty fine SAND |
| 6 | Northern elevation of existing house | Basement void discovered 400 mm below ground level.<br>Void extended to 2.50 m (110.09 m OD) | Not Proved. |
| 7 | Southern elevation of existing house | Concrete footing<br>Top 1.10 m (111.48 m OD)<br>Base 1.60 m (110.98 m OD)<br>Lateral projection 150 mm | Orange-brown mottled pale grey very clayey silty fine SAND with bands of silty very sandy CLAY |
| 8 | Eastern elevation of existing house | Concrete strip<br>Top 660 mm (111.95 m OD)<br>Base 1.10 m (111.45 m OD<br>Lateral projection 300 mm | Firm orange-brown silty very sandy CLAY |
| 9 | Single storey sun room on southern elevation | Concrete strip<br>Top 370 mm (112.08 m OD)<br>Base 0.80 m (111.65 m OD)<br>Lateral projection 300 mm | Orange-brown mottled pale grey very clayey silty fine SAND with bands of very sandy silty CLAY |
| 10 | Outbuilding along the northeastern corner of the site | Loosely cemented brick rubble strip footing<br>Top 380 mm<br>Base 0.60 m (112.50 m OD)<br>Lateral projection 150 mm | Made Ground |
| 11 | Outbuilding along the northeastern corner of the site | Concrete strip footing<br>Top 80 mm<br>Base 0.30 m (112.80 m OD)<br>Lateral projection 140 mm | Firm orange-brown silty CLAY |
| 12 | Northeastern boundary wall | Concrete strip footing<br>Top 300 mm<br>Base 1.00 m (112.70 m OD)<br>Lateral projection 280 mm | Made Ground |

| Trial Pit No | Structure | Foundation detail | Bearing Stratum |
|---|---|---|---|
| **13** | Outbuilding along the northeastern corner of the site | Concrete strip footing<br>Top 660 mm<br>Base 1.06 m (112.22 m OD)<br>Lateral projection 170 mm | Firm orange-brown silty sandy CLAY |
| **14** | Outbuilding along the northeastern corner of the site | Concrete strip footing<br>Top 200 mm<br>Base 0.40 m (112.93 m OD)<br>Lateral projection 120 mm and 220 mm | Made Ground |
| **15** | Retaining wall along eastern elevation of tennis court | Concrete strip footing<br>Top 100 mm<br>Base 0.30 m (99.49 m OD)<br>Lateral Projection 50 mm | Firm orange-brown silty sandy CLAY |
| **16** | Retaining wall along eastern elevation of tennis court | Concrete strip footing<br>Top 100 mm<br>Base 0.40 m (99.21 m OD)<br>Lateral projection 170 mm | Firm orange-brown silty sandy CLAY |
| **17** | Retaining wall along eastern elevation of tennis court | Concrete strip footing<br>Top 100 mm<br>Base 0.50 m (99.11 m OD)<br>Lateral projection 200 mm | Firm orange-brown silty sandy CLAY |
| **18** | Retaining wall along southern elevation of tennis court | Concrete strip footing<br>Top 100 mm<br>Base 0.50 m (99.11 m OD)<br>Lateral projection 200 mm | Firm orange-brown silty sandy CLAY |
| **19 A-A** | Northeastern corner of isolated basement | Concrete strip footing<br>Top 600 mm<br>Base 0.80 m (109.43 m OD)<br>Lateral projection 270 mm | Firm orange-brown silty sandy CLAY |
| **19 B-B** | Northeastern corner of isolated basement | Concrete strip footing<br>Top 600 mm<br>Base 0.80 m (109.43 m OD)<br>Lateral projection 70 mm | Firm orange-brown silty sandy CLAY |
| **20** | Internal basement wall | Concrete strip footing<br>Top 500 mm<br>Base 0.80 m (108.73 m OD)<br>Lateral projection 550 mm | Firm orange-brown silty sandy CLAY |
| **21 A-A** | Southeastern corner of internal basement courtyard | Concrete strip footing<br>Top 260 mm<br>Base 0.45 m<br>Lateral projection 250 mm | Firm orange-brown silty sandy CLAY |
| **21 B-B** | Southeastern corner of internal basement courtyard | Concrete strip footing<br>Top 500 mm<br>Base 0.70 m<br>Lateral projection 350 mm | Firm orange-brown silty sandy CLAY |
| **22** | Internal basement wall | Concrete strip footing<br>Top 400 mm<br>Base 0.75 m (108.80 m OD)<br>Lateral projection 420 mm | Firm orange-brown silty sandy CLAY |
| **23** | Internal basement wall | Concrete strip footing<br>Top 400 mm<br>Base 0.80 m (108.75 m OD)<br>Lateral projection 170 mm | Firm orange-brown silty sandy CLAY |
| **24** | Internal basement wall | Concrete strip footing<br>Top 600 mm<br>Base 0.70 m (108.85 m OD)<br>Lateral projection 230 mm | Firm orange-brown silty sandy CLAY |
| **25** | Eastern elevation of main house | Mass concrete<br>Top 900 mm<br>Base not proved<br>Lateral projection not proved | Not proved |

| Trial Pit No | Structure | Foundation detail | Bearing Stratum |
|---|---|---|---|
| **26** | Internal ground floor elevation | Concrete strip footing<br>Top 860 mm<br>Base 0.95 m<br>Lateral projection 200 mm | Firm orange-brown silty sandy CLAY |
| **27** | Internal ground floor elevation | Concrete strip footing<br>Top 700 mm<br>Base 0.95 m<br>Lateral projection 250 mm | Brown clayey silty fine to medium SAND |
| **28** | Internal ground floor elevation | Concrete strip footing<br>Top 1020 mm<br>Base not proved – greater than 1.30 m<br>Lateral projection 500 mm | Firm orange-brown silty sandy CLAY |

# Part 2: DESIGN BASIS REPORT

This section of the report provides an interpretation of the findings detailed in Part 1, in the form of a ground model, and then provides advice and recommendations with respect to foundation options and contamination issues.

## 6.0 INTRODUCTION

It is understood that it is proposed to refurbish the existing detached property, which will include the construction of a single storey rear extension that will house an indoor swimming pool within a single level basement. New landscaping works also form part of the proposals, in addition to the refurbishment of the existing single storey out-buildings along the northern boundary of the site, whilst a 2000 $m^2$ outhouse will be constructed in the northwestern corner of the site.

## 7.0 GROUND MODEL

The previous desk study revealed that the site has not had a potentially contaminative history, having been occupied by two manor houses, the first of which was constructed in the 1830s. The use of the existing building for geriatric care between 1950 and 2003 is not considered to be a contaminative usage. On the basis of the intrusive investigations, the ground conditions at this site can be characterised as follows:

- below a generally moderate thickness of made ground, the Claygate Member of the London Clay Formation is present and was proved to the maximum depth investigated;
- made ground extends to depths of between 0.25 m and 1.80 m and generally comprises brown clayey silt with rootlets, gravel, brick, concrete, coal and timber fragments;
- the underlying Claygate Member initially comprises firm becoming stiff medium to high strength brown and orange-brown mottled grey silty very sandy clay with pockets of clayey fine sand and sandy silt, which extends to depths of between 3.30 m and 7.30 m;
- in northwest of the site, in close proximity of mature trees, desiccation of the initial clay soils has been encountered to a depth of 2.00 m;
- the initial horizon is underlain by stiff brownish grey silty sandy clay with partings and pockets of pale grey silt to depths of 7.30 m and 9.00 m, whereupon stiff high strength dark grey silty clay to clayey silt is present to depths of 12.00 m and 15.00 m;
- below these depths the Claygate Member very stiff high strength to very high strength dark grey silty, locally sandy, clay with traces of selenite, which was proved to the maximum depth investigated, of 20.00 m;
- groundwater monitoring visits have measured groundwater at depths of between 9.02 m and 10.21 m, levels of between 104.42 m OD and 101.99 m; and

- contamination testing has generally indicated the absence of contamination within the made ground; however elevated concentrations of lead, sulphide, benzo(a)pyrene and TPH have been measured within the soils at the base of the natural pond in the northwestern corner of the site.

# 8.0 ADVICE AND RECOMMENDATIONS

It is understood that the proposed basement will be excavated to a depth of 3.20 m, a level of 109.55 m OD, in order to match the level of the existing basement. It is also understood that is it is proposed to utilise the available space on the site by excavating the basement in an open cut.

For the low-rise developments, spread foundations should be a suitable foundation solution, although founding depths will need to be determined in accordance with NHBC guidelines and will need to extend beyond the depth of any desiccation.

## 8.1 Basement Excavation

The formation level for the proposed basement will be within the Claygate Member at a level of approximately 109.55 m OD. On the basis of the groundwater observations, which has indicated the groundwater table to be present and fluctuate between levels of 104.42 m OD and 101.99 m, groundwater will remain between approximately 5.00 m and 7.00 m below the basement excavation.

There are a number of methods by which the sides of the excavation could be supported in the temporary and permanent conditions. The choice of wall may be governed to a large extent by whether it is to be incorporated into the permanent works and have a load bearing function. The final choice will depend to a large extent on the need to protect nearby structures from movements, the required overall stiffness of the support system, and the need to control groundwater movement through the wall in the temporary condition.

Given the anticipated absence of significant groundwater inflows and the available open space, the construction of insitu retaining walls within an open cut excavation, with the sides battered to a safe angle, is considered to be suitable option. Slopes within the made ground and underlying Claygate Member should be excavated at 1 (vertical) to 2 (horizontal), although care should be taken to protect the sides of any unsupported cut slopes during periods of rainfall and any run-off from construction operations until the retaining walls have been installed. Movement of plant at the top of any open cut should be prevented and daily inspections of the cut faces should be carried out to check stability.

The ground movements associated with the excavation will depend on the method of excavation and support and the overall stiffness of the basement structure in the temporary condition. Thus, a suitable amount of propping will be required to provide the necessary rigidity and the timing of the provision of support to the wall will have an important effect on movements. The movements associated with the proposed basement is however considered further in Part 3 of this report.

### 8.1.1 Retaining Walls

The following parameters are suggested for the design of the permanent basement retaining walls.

| Stratum | Bulk Density (kg/m³) | Effective Cohesion (c' – kN/m²) | Effective Friction Angle (Φ' – degrees) |
|---|---|---|---|
| Made ground | 1800 | Zero | 27 |
| Claygate Member | 2000 | Zero | 25 |

Groundwater is unlikely to be encountered within the excavation and at this stage, it is thought that the retaining walls may be designed assuming the groundwater level to be below the excavation. Consideration will however need to be given to the risk of groundwater and

surface water collecting behind the retaining walls. The advice in BS8102:2009[6] should be followed in the design of the basement retaining walls and with regard to waterproofing requirements.

### 8.1.2 Heave

The proposed development will require a maximum excavation depth of approximately 3.20 m, which will result in a net unloading of around 60 kN/m². The unloading will result in heave of the underlying clay soils, although these movements will to certain extent be counteracted by the applied loads from the new extension and swimming pool structure. Further consideration is given to heave movements in Part 3 of this report.

## 8.2 Spread Foundations

The excavation of the basement extension will result in a formation level in the Claygate Member. It should be possible to adopt moderate width pad or strip foundations in the firm sandy clay and clayey sand at a depth below 3.20 m (109.55 m OD), which may be designed to apply a net allowable bearing pressure of 140 kN/m².

Some parts of the proposed extension do not include a basement and spread foundations excavated from existing ground level are likely to need to extend to approximately 1.80 m in order to bypass the made ground and bear within the natural clayey sandy soils of the Claygate Member. At this depth, foundations may be designed to apply a net allowable bearing pressure of 110 kN/m².

New foundations for the proposed refurbished outbuildings and the new outhouse will need to be designed to bear at a minimum depth of 0.90 m, assuming that restrictions are applied on the planting of shrubs in the vicinity of foundations, or at a depth of 1.25 m if there is unrestricted planting of shrubs in the new development, subject also to the further restrictions on new tree planting as detailed in the NHBC guidelines. Foundations at these depths, bearing on either the clay of the Claygate Member, may be designed to apply a net allowable bearing pressure of 100 kN/m².

Foundations will need to be deepened in the vicinity of existing and proposed trees and National House Building Council (NHBC) guidelines should be followed in this respect. Medium volume change potential clay should be assumed. Where trees are to be removed the required founding depth should be determined on the basis of the existing tree height if it is less than 50% of the mature height and on the basis of full mature height if the current height is more than 50% of the mature height. Where a tree is to be retained the final mature height should be adopted. Due allowance should be made for future growth of the trees. The requirement for compressible material alongside foundations should be determined by reference to the NHBC guidelines.

6 BS8102 (2009) *Code of practice for protection of below ground structures against water from the ground*

Notwithstanding NHBC guidelines, all foundations should extend beyond the zone of desiccation. In the area of the proposed new outhouse in the northwest of the site, foundations may need to extend to a depth of 2.00 m in order to extend beyond the observed desiccation. It would be prudent to have all foundation excavations inspected by a suitably experienced engineer to confirm that any desiccation and roots and rootlets have been suitably bypassed.

All of the above bearing pressures include an adequate factor of safety to keep settlements within tolerable limits, however foundations that span both sand and clay soils will need to be suitably reinforced in order to protect against differential settlements.

## 8.3 Piled Foundations

If piles were to be considered, for the ground conditions at this site, some form of bored pile is likely to be the most appropriate type. A conventional rotary augered pile may be appropriate, with temporary casing installed to maintain stability and prevent groundwater inflows, although it is likely that groundwater will be encountered at depth within the Claygate Member. Therefore the use of bored piles installed using continuous flight auger (cfa) techniques, which would not require the provision of casing, are likely to be the most appropriate choice of pile.

The following table of ultimate coefficients may be used for the preliminary design of bored piles, based on the SPT & Cohesion / level graph in the appendix.

| Stratum | Level m OD | kN / m² |
|---|---|---|
| Ultimate Skin Friction | | |
| Made Ground and Claygate Member | All soil above 109.55 | Ignore (Basement excavation) |
| Claygate Member (clay - $\alpha$ = 0.6) | 109.55 to 92.75 | Increasing linearly from 35 to 115 |
| Ultimate End Bearing | | |
| Claygate Member | 97.75 to 92.75 | Increasing linearly from 1350 to 1710 |

On the basis of the above coefficients and a factor of safety of 3, it has been estimated that a 450 mm diameter pile founding at a depth of 15 m below ground level, with a toe level of approximately 97.75 m OD, should provide a safe working load of about 400 kN. Alternatively, a 450 mm diameter pile founding at a depth of 20 m below ground level (92.0 m OD) should provide an increased safe working load of 665 kN.

In the design of piled foundations, the effect of potential future shrinkage and swelling of the clay should be taken into account. In designing for compressive loads it should be assumed that further desiccation, and hence shrinkage of the clay, could continue where trees are to remain. Pile shaft adhesion within the theoretical maximum future desiccated thickness should therefore be ignored, and this thickness should be determined by reference to the NHBC guidelines in line with the advice given above for spread foundations. Heave of the clay soils could also occur due to future swelling as a result of trees being removed. This would exert a tensile uplift force on the piles, unless piles are effectively isolated from the surrounding soil by means of a slip layer or sleeve around the pile shaft.

On completion of construction the uplift forces would, to some extent, be counteracted by the applied loads. However, since the full structural loads may well be less than the potential uplift forces the piles would, in the absence of sleeving, need to be sufficiently "anchored" below the desiccated zone to withstand the uplift forces. Adequate reinforcement would need to be provided to accommodate the resulting tension.

The above examples are not intended to constitute any form of recommendation with regard to pile size or type, but merely serve to illustrate the use of the above coefficients. Specialist piling contractors should be consulted with regard to the design of a suitable piling scheme for this site.

## 8.4 Shallow Excavations

On the basis of the trial pit findings, it is considered likely that it will be feasible to form relatively shallow excavations that extend through the made ground and terminate within the underlying sand without the requirement for lateral support, although localised instabilities may occur from within the made ground. Where personnel are required to enter excavations, a risk assessment should be carried out and temporary lateral support or battering of the excavation sides will be required in order to comply with normal safety requirements.

Although no shallow groundwater inflows have been encountered during the investigation, it is conceivable that seepages may be encountered from perched water tables within the made ground, particularly within the vicinity of existing foundations. Such inflows should be suitably controlled by sump pumping.

## 8.5 Basement and Ground Floor Slabs

Following the excavation of the proposed basement, it is likely that the basement floor slab will need to be suspended over a void in order to accommodate the likely heave movements, unless the slab can be designed to accommodate these pressures. An analysis of potential heave movements has been carried out and is discussed in Part 3 of this report.

For new ground floor slabs, it should be possible to adopt ground bearing slabs directly on the firm silty sandy clay of the Claygate Member, subject to a proof rolling exercise and the infilling of any soft spots with suitably compacted granular fill. If, however, new slabs are within the zone of influence of existing and / or proposed trees, they will need to be suspended over a void in accordance with NHBC guidelines.

## 8.6 Effect of Sulphates

Generally low concentrations of total sulphate were measured in selected samples of made ground and natural soil recovered as part of the previous investigation and therefore indicate buried concrete can be designed in accordance with Class DS-1 conditions of Table C2 of BRE Special Digest 1: SD1 Third Edition (2005). The measured pH conditions are near neutral and therefore on the basis of static groundwater conditions being assumed for buried concrete an ACEC classification of AC-1s may be adopted.

The guidelines contained in the above digest should be followed in the design of foundation concrete.

## 8.7 Pavement Design

The results of insitu CBR testing indicate that a CBR value of 3% may be adopted for the design of new roads and pavements. Laboratory testing has indicated that the shallow soils are not frost susceptible.

## 8.8 Soakaway Design

The soakage testing undertaken in Trial Pit Nos IT1 and IT2 has indicated design soakage rates of $2.6 \times 10^{-6}$ m/s and $5.2 \times 10^{-6}$ m/s respectively, which although not particularly high rates of soakage, should be adequate for the use of shallow soakaways. Sufficient storm capacity will need to be ensured in the design of any soakaways, following the guidance in BRE Digest 365.

## 8.9 Site Specific Contamination Risk Assessment

The desk study has indicated that the site has not had a contaminative history and no sources of contamination were identified on site or in the immediate surrounding area. The contamination testing has also generally identified the made ground across the site to not contain elevated levels of the contaminants tested and as such site wide remediation is not considered to be required. The made ground is unlikely to be a suitable material to be classed as a topsoil and so if new areas of planting form part of the proposals, it is likely to be required that certified clean topsoil will need to be imported.

Beyond the general made ground, the material at the base of the natural pond in the northwest corner of the site was noted as being discoloured and to have a hydrocarbon odour during sampling. Contamination testing has indicated this material to contain elevated concentrations of lead, sulphide, benzo(a)pyrene and TPH. The proportions of the various carbon chain lengths suggest that the hydrocarbon contamination represents a heavy heating oil and it is possible that the other elevated contaminants originate from the same source, through the combustion or partial combustion of some form of hydrocarbon fuel. The pond is a natural pond originating from a spring that forms the source of one of the tributaries to the River Fleet, one of London's "lost rivers" and is therefore a sensitive receptor. Whilst the exact source of the contamination is unknown, it is thought that various surface water drains feed into the pond, so it is possible that the contamination is from some form of relic spillage that has washed down the existing drainage routes and into the pond.

It is understood that it may potentially be proposed to create a freshwater swimming pond at the existing location of the pond. This will probably require some form of lining material to be installed. Should this be the case, then it may be that existing material at the base of the pond could be excavated and disposed off-site and the new lining material installed. However, regardless of the proposals, this affected material will need to be excavated and disposed of in order to prevent ongoing contamination of the water. Should it not be proposed to transform the pond into a freshwater swimming pond, further testing will be required to determine the extent of the contamination. Further groundwater testing will be required in order to determine the required treatment of the water. It is possible that chemical treatment using Oxygen Reducing Compounds (ORC) will be required. It is recommended that once the exact proposals have been finalised, that the requirement for additional testing is reviewed and a remediation method statement compiled.

## 8.10 Waste Disposal

Under the European Waste Directive, waste is classified as being either Hazardous or Non-Hazardous and landfills receiving waste are classified as accepting hazardous or non-hazardous wastes or the non-hazardous sub-category of inert waste in accordance with the Waste Directive. Waste classification is a staged process and this investigation represents the preliminary sampling exercise of that process. Once the extent and location of the waste that is to be removed has been defined, further sampling and testing may be necessary. The results from this ground investigation should be used to help define the sampling plan for such further testing, which could include WAC leaching tests where the totals analysis indicates the soil to be a hazardous waste or inert waste from a contaminated site. It should however be noted that the Environment Agency (EA) guidance WM3[7] states that landfill WAC analysis, specifically leaching test results, must not be used for waste classification purposes.

Any spoil arising from excavations or landscaping works, which is not to be re-used in accordance with the CL:AIRE[8] guidance, will need to be disposed of to a licensed tip. Waste going to landfill is subject to landfill tax at either the standard rate of £ £86.10 per tonne (about £155 per $m^3$) or at the lower rate of £2.70 per tonne (roughly £5 per $m^3$). However, the classifications for tax purposes and disposal purposes differ and currently all made ground and topsoil is taxable at the 'standard' rate and only naturally occurring soil and stones, which are accurately described as such in terms of the 2011 Order, would qualify for the 'lower rate' of landfill tax.

Based upon on the technical guidance provided by the EA it is considered likely that the soils encountered during this ground investigation, as represented by the 9 chemical analyses carried out, would be generally classified as follows;

| Soil Type | Waste Classification (Waste Code) | WAC Testing Required Prior to Landfill Disposal? | Comments |
|---|---|---|---|
| Made ground | Non-hazardous (17 05 04) | No | |
| Langley Silt / Taplow Gravel | Inert (17 05 04) | Should not be required but confirm with receiving landfill | - |

Under the requirements of the European Waste Directive all waste needs to be pre-treated prior to disposal. The pre-treatment process must be physical, thermal, chemical or biological, including sorting. It must change the characteristics of the waste in order to reduce its volume, hazardous nature, facilitate handling or enhance recovery. The waste producer can carry out the treatment but they will need to provide documentation to prove that this has been carried out. Alternatively, the treatment can be carried out by an approved contractor. The Environment Agency has issued a position paper[9] which states that in certain circumstances, segregation at source may be considered as pre-treatment and thus excavated material may not have to be treated prior to landfilling if the soils can be segregated onsite prior to excavation by sufficiently characterising the soils insitu prior to excavation.

The above opinion with regard to the classification of the excavated soils is provided for guidance only and should be confirmed by the receiving landfill once the soils to be discarded have been identified.

The local waste regulation department of the EA should be contacted to obtain details of tips that are licensed to accept the soil represented by the test results. The tips will be able to provide costs for disposing of this material but may require further testing.

7 Environment Agency 2015. *Guidance on the classification and assessment of waste.* Technical Guidance WM3 First Edition
8 CL:AIRE March 2011. *The Definition of Waste: Development Industry Code of Practice* Version 2
9 Environment Agency 23 Oct 2007 *Regulatory Position Statement Treating non-hazardous waste for landfill - Enforcing the new requirement*

# Part 3: GROUND MOVEMENT ANALYSIS

This section of the report comprises an analysis of the ground movements arising from the proposed basement and foundation scheme discussed in Part 2 and the information obtained from the investigation, presented in Part 1 of the report.

## 9.0 INTRODUCTION

The sides of an excavation will move to some extent regardless of how they are supported. The movement will typically be both horizontal and vertical and will be influenced by the engineering properties of the ground, groundwater level and flow, the efficiency of the various support systems employed during underpinning and the efficiency or stiffness of any support structures used.

An analysis has been carried out of the likely movements arising from the proposed excavation and the results of this analysis have been used to predict the effect of these movements on surrounding structures.

### 9.1 Construction Methodology

It is proposed to excavate the basement in an open cut excavation. Once the excavation has been completed and the sides battered to a safe angle, the basement slab will be cast. The ground behind the walls will then be backfilled and compacted in layers. On the southern extent of the excavation, at the interface with the existing basement, sufficient propping will be installed to maintain stability of the existing house.

## 10.0 GROUND MOVEMENTS

An assessment of ground movements within and surrounding the excavation has been undertaken using the P-Disp computer programs licensed from the OASYS suite of geotechnical modelling software from Arup. This program is commonly used within the ground engineering industry and is considered to be appropriate tools for this analysis.

The analysis of potential ground movements within the excavation, as a result of unloading of the underlying soils, has been carried out using the Oasys P-Disp Version 19.3 – Build 12 software package and is based on the assumption that the soils behave elastically, which provides a reasonable approximation to soil behaviour at small strains.

For the purpose of these analyses, the corners have been defined by x and y coordinates, with the x-direction parallel with the orientation east-west, whilst the y-direction is parallel with the orientation of north-south. Vertical movement is in the z-direction.

The full outputs of all the analyses can be provided on request but samples of the output movement contour plots are included within the appendix.

### 10.1 Ground Movements Surrounding the Basement

As it is proposed to construct the basement in an open cut, there will be no vertical movement associated with the installation of a retaining wall, while horizontal movements, arising from the relief of the horizontal stresses are likely to be negligible. In any case, for a 3.0 m deep excavation in sandy clay, no movement would be expected to occur beyond 6.0 m away from the basement retaining walls, as described in various published literature, including Peck (1969)[10]. As no structures are present within 30 m of the excavation, that no vertical and horizontal movements will occur below the existing house as the existing basement retaining walls are being retained and re-used and due to the open cut nature of the excavation, there is not considered to be any likely impact from horizontal and vertical movements outside of the excavation. Heave movements from the unloading of the clay soils will take place outside of the excavation, as discussed below.

## 10.2 Ground Movements within the Excavation (Heave)

Unloading of the Claygate Member will take place as a result of the excavation of the proposed lower ground floor extension. The reduction in vertical stress will cause heave to take place. Undrained soil parameters have been used to estimate the potential short term movements, which include the "immediate" or elastic movements as a result of the existing building and basement excavation. Drained parameters have been used to provide an estimate of the total long-term movement.

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 $E_u$ and E', the undrained and drained stiffness respectively, to values of undrained cohesion, as described by Padfield and Sharrock[11] and Butler[12] and more recently by O'Brien and Sharp[13]. Relationships of $E_u = 500\ C_u$ and $E' = 300\ C_u$ for the cohesive soils and 2000 x SPT 'N' for granular soils have been used to obtain values of Young's modulus. More recent published data[14] indicates stiffness values of 750 x Cu for the London Clay and a ratio of E' to Cu of 0.75, but it is considered that the use of the more conservative values provides a sensible approach for this stage in the design.

The excavation of a 3.20 m thickness of soil across the majority of the proposed extension will result in a net unloading of roughly 60 kN/m, assuming a unit weight of 18 $kN/m^3$ for the made ground and 20 $kN/m^2$ for the Claygate Member. The existing basement, which is being retained and lowered by approximately 600 mm, forms the southeastern corner of the proposed basement and therefore the unloading in this corner is in the order of 14 $kN/m^2$. The basement will be founded on a basement raft foundation, which will apply a pressure of 48 $kN/m^2$, as indicated by the structural engineers.

A rigid boundary for the analysis has been set at a depth of 60.0 m below ground level (52.75 m OD), which will be within the London Clay. Below this depth the clay is considered to be essentially incompressible.

The P-Disp analysis indicates that the heave resulting from the excavation of the proposed basement will be up to 10 mm within the centre of the excavation and reducing to approximate 5 mm toward the edges. These movements would be expected to be complete by the end of the excavation and construction period, although depending on the time-scales of the excavation and subsequent construction, these movements may not be fully realised. Taking into account the loads of the proposed extension, the analysis has shown that in the

10 Peck R B (1969) *Deep excavations and tunnel-lining in soft ground*, State-of-the-Art Report, Proc 7th Int. Conf. SMFE, Mexico
11 Padfield CJ and Sharrock MJ (1983) *Settlement of structures on clay soils.* CIRIA Special Publication 27
12 Butler FG (1974) *Heavily overconsolidated clays: a state of the art review.* Proc Conf Settlement of Structures, Cambridge, 531-578, Pentech Press, Lond
13 O'Brien AS and Sharp P (2001) *Settlement and heave of overconsolidated clays - a simplified non-linear method.* Part Two, Ground Engineering, Nov 2001, 48-53
14 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

long term, the majority of the short-term movement will be recovered, with total heave movements at the centre of the excavation likely to actually be approximately 3 mm reducing to 1 mm at the edges. In the southeastern corner however, where the existing basement is present, up to 5 mm of settlement can be expected due to the limited unloading of this area coupled with the proposed loads.

The results of the P-Disp analysis also indicate the likely impact of the proposed basement construction beyond the site boundaries. On the basis of the analysis, total vertical movements outside the proposed extension to the east are unlikely to exceed 1 mm of heave, whilst beyond the southeastern corner between 1 mm and 2 mm of settlement maybe expected to occur. These movements however occur within 2 m of the edge of the excavation and are therefore considered to be very small and will not have detrimental impact on any surrounding structures.

The potential movements are summarised in the table below.

| Location | Movement (mm) | |
|---|---|---|
| | Short-term Movement (Excavation Phase) | Total Movement |
| Centre of excavation | 10 (heave) | 3 (heave) |
| Edge of excavations | 5 to 6 (heave) | 1 to 2 (heave) |
| Southeastern corner | 1 to 3 (heave) | 5 to 6 |

If a compressible material is used beneath the slab, it will need to be designed to be able to resist the potential uplift forces generated by the ground movements. In this respect potential heave pressures are typically taken to equate to around 40% of the total unloading pressure.

## 11.0 DAMAGE ASSESSMENT

The potential heave movements predicted by P-Disp have been used to carry out an assessment of the likely longer term damage to closest elevations of the existing house. The methodology contained within CIRIA 580 has been used with deflection ratios calculated from the line plots appended over their respective wall lengths and for the roughly 12 m high neighbouring buildings. The calculated deflection ratios fall well below 1 in 400, which would be well within the 0.05 % strain that is within Burland Category 0 – negligible.

## 12.0 OUTSTANDING RISKS AND ISSUES

This section of the report aims to highlight areas where further work is required as a result of limitations on the scope of this investigation, or where issues have been identified by this investigation that warrant further consideration. The scope of risks and issues discussed in this section is by no means exhaustive, but covers the main areas where additional work may be required.

The ground is a heterogeneous natural material and variations will inevitably arise between the locations at which it is investigated. This report provides an assessment of the ground conditions based on the discrete points at which the ground was sampled, but the ground conditions should be subject to review as the work proceeds to ensure that any variations from the Ground Model are properly assessed by a suitably qualified person.

Additional contamination testing is likely to be required in and around the existing pond in the northwestern corner of the site, although this will need to be reviewed once the exact proposals have been finalised.

Should spread foundations be proposed in the zone of influence of trees, that beyond determining the required founding depth in accordance with NHBC requirements, that foundation excavations are inspected by a suitably qualified engineer.

# APPENDIX

Borehole Records

Trial Pit Records

Soakage Test Results

Geotechnical Test Results

Contamination Test Results

Generic Risk-Based Guideline Values

Site Plan

**P-DISP ANALYSIS**
Short Term Movement
Total Movement

P-Disp Displacement Graphs

Proposed Development Drawings

Topographical Site Survey

Existing and Proposed Drainage Plan

Proposed Landscaping Plan

Geotechnical & Environmental Associates (GEA) is an engineer-led and client-focused independent specialist providing a complete range of geotechnical and contaminated land investigation, analytical and consultancy services to the property and construction industries.

We have offices at

Widbury Barn
Widbury Hill
Ware
Hertfordshire
SG12 7QE
tel 01727 824666
mail@gea-ltd.co.uk

Church Farm
Gotham Road
Kingston on Soar
Notts
NG11 0DE
tel 01509 674888
midlands@gea-ltd.co.uk

Enquiries can also be made on-line at

www.gea-ltd.co.uk

where information can be found on all of the services that we offer.

Geotechnical & Environmental Associates