E Report on Ground Investigation

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August 2013

50 AVENUE ROAD,

CAMDEN, LONDON NW8 6HS

REPORT ON A GROUND INVESTIGATION

Prepared for

Elliott Wood Partnership LLP

Acting on behalf of

The Shri Krishna Trust

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Ref: 13/20821
August 2013

Report on a Ground Investigation

At

50 Avenue Road, London, NW8 6HS

For

The Shri Krishna Trust C/O HSBC Trustee (C.I.) Limited

1.0 INTRODUCTION

At the request of the Elliott Wood Partnership, Consulting Engineers, acting on behalf of the The Shri Krishna Trust C/O HSBC Trustee (C.I.) Limited, a ground investigation was carried out in connection with a proposed residential development at the above site. A Phase 1 Preliminary Risk Assessment is presented under separate cover in a Site Analytical Services Limited report (Project No. 13/20821-1) dated August 2013 and a basement impact assessment is also presented in a Site Analytical Services Limited report (Project No. 13/20821-2) dated August 2013.

The information was required for the design and construction of foundations and infrastructure for the proposed development which includes the construction of a two storey basement beneath the footprint of the existing property and part of the garden. A study to assess whether any remediation was required for the protection of the end-user from the presence of potential contamination within the soils encountered was outside the scope of the present report.

Anticipated foundation loads for the proposed building are expected to be high and of the order of 200-300kN/m^2. Ground slab loadings are expected to be of the order of 10-15kN/m^2.

The recommendations and comments given in this report are based on the ground conditions encountered in the exploratory holes made during the investigation and the results of the tests made in the field and the laboratory. It must be noted that there may be special conditions prevailing at the site remote from the exploratory hole locations which have not been disclosed by the investigation and which have not been taken into account in the report. No liability can be accepted for any such conditions.

2.0 THE SITE AND LOCAL GEOLOGY

(National Grid Reference: TQ 270 837)

2.1 Site Description

The site is situated at an existing residential property at 50 Avenue Road located in the Swiss Cottage area of London, NW8 6HS.

The site comprises of an extensive three-storey detached house with a large rear garden and gated driveway from Avenue Road. The rear garden is mainly set to lawn space and flower beds and small shrubs. The garden is bordered by a low brick wall with some large trees present at the end of the garden. A small wooden summer house is present at the end of the garden. The main house has a large gated driveway at the front, including a small raised lawn and hedge.

The site is bordered by Avenue Road to the south-west, and further residential properties to the west, east and north. Primrose Hill is situated further to the east and the general area is essentially residential in use with some schools in the local area.

The site itself is essentially flat, although there is a general slight slope across the site from north-west to south-east away from Primrose Hill down towards the Thames Basin.

2.2 Geology

The 1:50000 Geological Survey of Great Britain (England and Wales) covering the area (Sheet 256, 'North London', Solid & Drift Edition) indicates the site to be underlain by the Eocene London Clay Formation. However, Superficial Head Deposits are located to the east and west of the site.

3.0 SCOPE OF WORK

3.1 General

The scope of the investigation was agreed by the Consulting Engineers and comprised:

- The drilling of one continuous flight auger borehole to a depth of 15m below ground level (Borehole 1).
- The drilling of one shell and auger (cable percussive) borehole to a depth of 15m below ground level (Borehole 2).
- The excavation of a single trial pit to 1.50m below ground level to expose the existing party wall foundations (Trial Pit 1).

- The placement of gas and groundwater monitoring standpipes to a depth of 10m below ground level in both of the boreholes.
- Sampling and in-situ testing as appropriate to the ground conditions encountered in the boreholes and trial pit.
- Interpretative reporting on foundation options for the proposed building works and infrastructure.
- A study into the possibility of the presence of toxic substances in the soil, together with comments on any remediation required was outside the scope of the present investigation.

3.2 Ground Conditions

The locations of the boreholes and trial pit are shown on the site sketch plan (Figure 1).

The exploratory holes revealed ground conditions that were generally consistent with the geological records and known history of the area and comprised up to 1.50m thickness of Made Ground overlying materials typical of the London Clay Formation.

For detailed information on the ground conditions encountered in the boreholes and trial pit, reference should be made to the exploratory hole records presented in Appendix A.

The Made Ground extended down to depths of 1.10m below ground level in Boreholes 1 and 2 and to the full depth of investigation of 1.50m below ground level in the trial pit. The material generally consisted of a surface layer of topsoil underlain by a mixture of medium dense clayey silty sand and sandy silty clay with brick fragments and crushed concrete.

The underlying natural material initially consisted of firm and stiff mottled silty clay with some pockets and partings of silty fine sand that extended down to a depth of 2.00m below existing ground level in Boreholes 1 and 2. These materials were underlain by firm to stiff becoming stiff and then very stiff clay with occasional partings of silty fine sand and occasional small gypsum crystals. These deposits represent weathered London Clay and extended to respective depths of 12.00m and 10.70m below ground level in Boreholes 1 and 2.

The underlying material comprised of very stiff fissured silty clay with occasional partings of silty fine sand and scattered gypsum crystals. These materials are typical of the more competent unweathered London Clay Formation and extended down to the full depths of investigation of 15.00m below ground level in Boreholes 1 and 2.

3.3 Groundwater

Groundwater was not encountered during boring operations in Borehole 2 and Trial Pit 1 and the material remained essentially dry throughout. Groundwater was encountered as a seepage at a depth of 12.00m below ground level in Borehole 1.

It must be noted that the speed of boring is such that there may well be insufficient time for light seepages of groundwater to enter the boreholes and trial pit and hence be detected, particularly within more cohesive soils of low permeability.

Groundwater and gas monitoring standpipes were installed in Boreholes 1 and 2 to depths of approximately 10.0m below ground level.

Groundwater was subsequently found to have stabilised at a depth of 3.49m below ground level in the monitoring standpipe installed in Borehole 1 after a period of approximately seven to eight weeks. Groundwater was not encountered in the monitoring standpipe placed in Borehole 2 during the same period.

Isolated pockets of groundwater may be present perched within any less permeable material found at shallower depth on other parts of the site especially within any Made Ground.

It should be noted that the comments on groundwater conditions are based on observations made at the time of the investigation (June, July and August 2013) and that changes in the groundwater level could occur due to seasonal effects and also changes in drainage conditions.

4.0 IN-SITU AND LABORATORY TESTS

4.1 Standard Penetration Tests

The results of the Standard Penetration Tests carried out in the natural soils (London Clay) are shown on the exploratory hole records in Appendix A. SPT 'N' values range between 12 and 32 with a gradual increase in 'N' value with depth being apparent. Based on the methods outlined by Stroud and Butler (1975) the results indicate the London Clay to be of a firm becoming stiff consistency to depths.

The correlation between SPT 'N' value and measured cohesion is generally taken as a multiplier of between 4.5 and 5 on the SPT 'N' value according to Stroud and Butler (1975) and has generally proved to be reliable.

The results of the tests are shown on the appropriate borehole records and summary sheet presented in Appendix A.

4.2 Undrained Triaxial Test Results

Quick Undrained Triaxial Tests were made on five selected undisturbed 100mm diameter samples taken from Borehole 1.

The results of the tests are presented on Table 1, contained in Appendix B.

4.3 Mackintosh Probe and In-Situ Vane Tests

In essentially granular Made Ground, a single Mackintosh Probe test was made in Borehole 1 in order to assess the relative density of the materials. The result indicates that the granular Made Ground is in a generally loose state of compaction, the result being based on the generally accepted correlation as follows:

Mackintosh N75 X 0.38 = SPT 'N' Value

or

Mackintosh N300 X 0.1 = SPT 'N' Value

In the essentially cohesive natural soils encountered at the site, in-situ shear vane tests were made at regular depth increments in order to assess the undrained shear strength of the materials. The results indicate that the natural soils are of a generally firm becoming stiff and then very stiff consistency with increasing depth below ground level.

The results of the in-situ tests are shown on the appropriate exploratory hole records contained in Appendix A.

4.4 One Dimensional Consolidation Tests

One dimensional consolidation tests were carried out on samples of cohesive soil recovered from 6.50m depth and 9.50m depth in Borehole 2, using the methods detailed in BS 1377-5: 1990 Clauses 3 and 4.

The results of the tests, including coefficients of consolidation and compressibility, are presented in both graphical and tabular formats in Appendix B.

4.5 Classification Tests

A single Atterberg Limit test was conducted on one sample taken at 6.00m depth from the near surface cohesive soils encountered in Borehole 1. The sample falls into Class CV according to the British Soil Classification System.

These are fine grained sandy and silty clay soils of very high plasticity and as such generally have a low permeability and a high susceptibility to shrinkage and swelling movements with changes in moisture content, as defined by the NHBC Standards, Chapter 4.2. The result indicated a Plasticity Index value of 51%, being well above the 40% boundary between soils assessed as being of medium swelling and shrinkage potential and those assessed as being of high swelling and shrinkage potential.

The test results are given in Table 2, contained in Appendix B.

4.6 Sulphate and pH Analyses

The results of the sulphate and pH analyses made on a single natural soil sample are presented on Table 3, contained in Appendix B. The results show the natural soil sample to have a water soluble sulphate content of 2.10g/litre associated with a slightly acidic pH value.

4.7 In-situ Falling Head Permeability or Soakage Tests

In order to assess the soil infiltration characteristics of the natural superficial soils at the site, an in-situ falling head permeability test was carried out in Borehole 2 using a combination of the methods detailed in Building Research Establishment Digest 365:1991 and British Standard 5930:1981.

4.8 Gas and Groundwater Monitoring Results

The standpipes installed in Boreholes 1 and 2 were monitored for gas and groundwater levels over a period of approximately seven to eight weeks and the results are presented on Tables 4, 4a and 4b, contained in Appendix B.

Groundwater was found to have stabilised at a depth of 3.49m below ground level in the monitoring standpipe installed in Borehole 1 over the monitoring period, although groundwater was not encountered in the monitoring standpipe placed in Borehole 2 during the same period.

During the first groundwater monitoring period groundwater was not recorded in Borehole 1. This may be due to slow recharge of the local groundwater following installation of the monitoring standpipe.

No significant putrifiable material that could give rise to on-site generation of methane gas was encountered in the boreholes.

4.8.1 Methane

Methane is a flammable asphyxiating gas, the flammable range being 5 to 15% by volume in air. If such a methane-air mixture is confined in some way and ignited it will explode. The 5% by volume concentration is termed the lower explosive limit (LEL). Methane is a buoyant gas having a density about two-thirds that of air.

Various guidelines have been published to help determine mitigation measures for landfill gas. `Landfill Gas' includes gas which may be generated in natural soils such as organic alluvium peat. Methane presents an explosion and asphyxiant hazard.

Building Research Establishment Report BR212 `Construction of New Buildings on Gas-Contaminated Land', states that if Methane concentrations in the ground are unlikely to exceed 1% by volume and a house or small building is constructed in accordance with its recommendations, then no further protection is required. The recommendations include installing granular under slab venting and sealing floor slabs.

CIRIA Report C665 (2007) "Assessing risks posed by hazardous ground gases to buildings" provides guidance on the monitoring and control of landfill gas. The report suggests a classification system which is summarised in Table 8.5 in the document and employs a method which uses both gas concentrations and borehole flow rates to define a characteristic situation for a site based on the Gas Screening Value (also named the limiting borehole gas volume flow) for methane and carbon dioxide.

4.8.2 Carbon Dioxide

Building Research Establishment Report BR212 `Construction of New Buildings on Gas-Contaminated Land', 1991 states that if carbon dioxide concentrations are above 1.5% by volume then protection should be considered to prevent gas ingress. If concentrations exceed 5% by volume, such protective measures are required. This has been superseded by CIRIA Report C665 (2007), states that if carbon dioxide concentrations are above 5% by volume then protection should be considered to prevent gas ingress.

Carbon Dioxide is a non-flammable toxic gas, which is about 1.5 times as heavy as air and is an asphyxiant hazard.

4.8.3 Carbon Monoxide

The occupational exposure standards for carbon monoxide are 30 ppm for long term exposure (8 hours calculated from the HSE Guidance Note EH40, 1991) and 200 ppm for short term exposure (15 minutes calculated from the HSE Guidance Note EH40, 1991) (CIRIA Report C665).

4.8.4 Hydrogen Sulphide

Hydrogen sulphide is toxic at low concentrations. The occupational exposure standard for hydrogen sulphide is 10 ppm for 8-hour time weighted average reference period and 15 ppm for short-term exposure (10 minutes reference period) (HSE Guidance Note EH40, 1991).

4.8.5 Results

The Gas Screening Value is calculated as follows:

The Gas Screening Value (litres of gas per hour) = maximum borehole flow rate (l/h) x maximum gas concentration (%)

On-site monitoring has shown emissions of methane in air of 0.0% and carbon dioxide in air of up to 5.3% recorded during the monitoring visits. The maximum borehole flow rate was 0.0 l/h.

Carbon Monoxide and Hydrogen Sulphide were not detected above the detection limits of the gas monitoring instrument in the boreholes monitored during the monitoring programme.

As such the Gas Screening Value for methane at site is 0.0 l/h and the Gas Screening Value for carbon dioxide at site is also 0.0 l/h. As such the worst case value for the site would be less than 0.01 litres of gas per hour. This typically equates to a Characteristic Situation 1; however, due to the concentration of Carbon Dioxide being in excess of 5%, it is recommended that the site be classified as Characteristic Situation 2 which requires the following precautions:

a) Reinforced concrete cast in-situ floor slab (suspended, non-suspended or raft) with at least 1200g DPM and underfloor venting

b) Beam and block or pre-cast concrete and 2000 g DPM / reinforced gas membrane and underfloor venting

All joints and penetrations sealed.

Employing the NHBC 'traffic light' characterisation system, the site would be classified as Amber 1 in accordance with CIRIA Report C665. Table 8.7 using the maximum gas concentration for carbon dioxide and which require the following precautions:

Low to intermediate gas regime identified, which requires low-level gas protection measures, comprising a membrane and ventilated sub-floor void to create a permeability contrast to limit the ingress of gas into buildings. Gas protection measures should be as prescribed in BRE Report 414 (Johnson, 2001). Ventilation of the sub-floor void should facilitate a minimum of one complete volume change per 24 hours.

For further information on design and construction details, discussions should be sought with a specialist contractor. Guidance may also be obtained from the BRE Report BR212 'Construction of New Buildings on Gas-Contaminated Land' and CIRIA Report C665 (2007). It may also be prudent to contact the local Environmental Health Officer in order to comply with the Local Authority requirements.

5.0 FOUNDATION DESIGN

5.1 General

It is proposed to construct a two storey basement beneath the footprint of the existing property and part of the garden. The majority of the basement is founded at approximately 8m below ground level with a deeper section containing a swimming pool at approximately 10m below ground level. Exact details of the finalised structure, layout and loadings were not available at the time of preparation of this report although foundation loads are expected to high and of the order 200-300kN/m^2, whilst ground slab loadings are expected to be of the order of 10-15kN/m^2.

5.2 Site Preparation Works

The CDM Co-ordinator should be informed of the site conditions and risk assessment undertaken to comply with the Construction Design Management (CDM) regulations. Site personnel are to be made aware of the site conditions in particular the presence of man made services below the site.

5.3 Conventional Spread Foundations

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

Due to the maximum depth of the proposed basement being at least 8m, conventional spread foundations are unlikely to prove practical. However, based on the ground and groundwater conditions encountered in the boreholes, it should be possible to support any shallower parts of the proposed development on basement raft foundations taken down below the Made Ground and any weak superficial soils and placed in the firm to stiff weathered London Clay deposits encountered at a depth of about 2.00m below existing ground level.

Such foundations placed within natural soils may be designed to allowable net bearing pressures of the order of 180kN/m^2 at 2.25m depth increasing linearly to about 220kN/m^2 at 6.50m depth in order to allow for a factor of safety of about three against general shear failure.

Any soft or loose pockets encountered within otherwise competent formations should be removed and replaced with well compacted granular fill.

In addition, foundations may need to be taken deeper should they be within the zones of influence of either existing or recently felled trees and any proposed tree planting. The depth of foundation required to avoid the zone likely to be affected by the root systems of trees is shown in the recommendations given in NHBC Standards, Chapter 4.2, April 2010, "Building near Trees" and it is considered that this document is relevant in this situation.

5.4 Piled Foundations

In the event that the use of conventional spread foundations proves either impracticable or uneconomical due to the size and depth of foundation required, a piled foundation will be required. In these ground conditions, it is considered that some form of bored and in-situ cast concrete piled foundation with reinforced concrete ground beams should prove satisfactory.

The construction of a piled foundation is a specialist activity and the advice of a reputable contractor, familiar with the type of soil and groundwater conditions encountered at this site should be sought prior to finalising the foundation design. The actual pile working load will depend on the particular type of pile chosen and method of installation adopted.

To achieve the full bearing value a pile should penetrate the bearing stratum by at least five times the pile diameter.

Where piles are to be constructed in groups the bearing value of each individual pile should be reduced by a factor of about 0.8 and a calculation made to check the factor of safety against block failure.

Driven piles could also be used and would develop much higher working loads approximately 2.5 to 3 times higher than bored piles of a similar diameter at the same depth. However, the close proximity of adjacent buildings will in all probability preclude their use due to noise and vibration.

5.5 Retaining Walls

5.5.1 General

The results of the investigation indicated that Made Ground occurs to a depth of up to 1.50m below existing ground level. This is followed by firm, stiff and then very stiff clay deposits down to a depth of at least 15.0m below ground level. The general groundwater level is unknown, but based on water monitoring is believed to be at a maximum level of 3.49m below existing ground level.

Retaining walls should generally be designed as self-supporting cantilevered retaining walls. The excavations for a basement must not affect the integrity of adjacent structures and therefore will need to be supported. Two forms of support could be considered, these being temporary works i.e. sheet piling which could be removed after the earth retaining walls have been constructed or as permanent works incorporated into the final design.

Generally, cantilevered piled walls have an open face to embedded ratio of about one to two, i.e. a supported face three metres in height would require a penetration into the ground of about six metres below the base of the excavation. Should the piled retaining wall be purely an unsupported cantilever, then it is likely that quite deep section sheet piles or large diameter bored piles would be required.

The section of the sheet or the diameter of the piles could be reduced by installing a braced waling to the wall. Piles placed as part of the permanent works would be propped by the roof to the basement and would not be acting purely as a cantilevered support in the long term.

To reduce the likelihood of loss of ground if a sheet piled wall was adopted when removing the sheets, it is considered that the sheet piles should be incorporated into the final wall design. Assuming that the earth retaining wall will be propped, i.e. have its base slab and first floor slab cast in place soon after excavation, it is unlikely that full if any earth pressures will act on the wall while it is not propped. The greatest force acting on the wall, in the short term, is likely to be from the hydrostatic head should water percolate and be retained to the rear of the earth retaining structure.

Given the unknown depth of the proposed basement (and therefore unknown founding material), the design parameters for each element of soil recorded in the relevant exploratory holes are provided in Table A below. The depth of pile penetration can be calculated once structural details of the proposed basement are known.

Founding Material	Depth to top (m)	Description	Critical Angle of Shearing Resistance (°) (Φ'_{crit})[1]	Coefficient active pressure (Ka)	Coefficient passive resistance (Kp)
London Clay	1.10	Firm becoming stiff and then very stiff silty CLAY	20	0.49	2.04

Table A. Summary of design parameters for proposed basement foundation

Notes:

1. Calculated using guidance from BS8002
2. As the depth and structural details of the proposed basement are unknown these values should be used as guidance only.

5.5.2 Ground Movement

Excavation into the London Clay Formation will cause the soil to heave as the existing overburden pressure is removed. Short term heave may be reduced by proceeding with the excavation in stages and observing and recording any movement that occurs over a set period of time. It may therefore be advantageous to delay the construction until an adequate proportion of the uplift has occurred. Once this monitoring period has elapsed and a suitably qualified engineer is confident that the majority of uplift has occurred, basement construction can commence. These processes and other ways of dealing with ground movements are described at length in BS8004 (British Standard Code of Practice for Foundations).

In addition to the effects of heave, the deposits present below the projected formation levels have the potential to swell, which occurs when water penetrates into soils when it is exposed to the atmosphere. The laboratory tests carried out on samples of the London Clay Formation present below approximately 6.00m below ground level indicate that the soil has the potential to exert swelling pressures of the order of 125kPa to 300kPa. The contractor should take note of this condition and adopt appropriate measures to minimize these effects, such as covering the base of the excavation with a layer of concrete blinding as soon as possible after reaching formation level.

London clay is an overconsolidated deposit, having previously been subjected to an effective stress in its past history larger than the effective stress existing at the present time. This degree of overconsolidation is indicated by the overconsolidation ratio (OCR) and has been calculated from the one dimensional consolidation tests data shown in Appendix B using the methods outlined by Cassegrande (1936).

	Bulk Density (kN/m2)
Made Ground	15.0
London Clay	19.0

Depth (mbgl)	Effective Stress (kPa)	Past Maximum Stress (kPa)	OCR
6.5	120	400-500	3.3 to 4.2
9.5	180	450-550	2.5 to 3.0

Table B. Summary of input parameters and Overconsolidation Ratio

For the purposes of this calculation the water level has been assumed to be below the soils in question, so a pore pressure of zero has been assumed.

The results indicate the soil is lightly to heavily overconsolidated, where soils with an OCR greater than 4 are assessed as heavily overconsolidated.

It must be noted that the overconsolidation ratio does not have a constant value. Soils where overburden has been removed may be heavily overconsolidated in the upper regions and less so at depth. Additionally the overconsolidation ratio may vary with other post deposition processes such as fluctuations in the water table, desiccation, cementation and weathering, and secondary compression.

5.6 Basement Floor Slab

Due to the presence of cohesive soils assessed to be of high swelling and shrinkage potential and the proximity of trees close to the site, it is recommended that basement slabs should be designed as being fully suspended. It is recommended that NHBC Standards, Chapter 4.2, April 2010, "Building near Trees" be consulted during the design phase of the floor slab.

5.7 Excavations

Shallow excavations for foundations and services are likely to require nominal side support in the short term and groundwater is unlikely to be encountered in significant quantities once any accumulated surface water within the Made Ground has been removed. Deeper and longer excavations below approximately 1.0m below existing ground level will require close side support and some inflows of groundwater are likely to be encountered.

Although it will be necessary to control water during the construction period originating from the made ground, no particular difficulties are envisaged in removing such water by conventional internal pumping methods from open sumps.

The results of the in-situ permeability test indicated the apparent permeability of the materials at the site to be of the order of 8.4 x 10^{-6} m/sec. This value lies approximately midway in the range of published data for fissured and weathered clays and / or silty sands and is classed as low permeability material at the boundary between good and poor drainage characteristics.

Consequently, this value should be adopted for the design of any proposed dewatering system.

Normal safety precautions should be taken if excavations are to be entered.

5.8 Chemical Attack on Buried Concrete

The results show the natural soil sample to have a water soluble sulphate content of 2.10g/litre associated with a slightly acidic pH value.

In these conditions, it is considered that deterioration of buried concrete due to sulphate or acid attack is likely to occur unless precautions are taken. The final design of buried concrete according to Tables C1 and C2 of BRE Special Digest 1:2005 should be in accordance with Class DS-3 conditions.

In addition, segregations of gypsum were noted within the London Clay and scattered small gypsum crystals were also noted at depth. Consequently, it is considered that any buried concrete at depth may be attacked by such sulphates in solution and that it would be prudent to design any such deep buried concrete in accordance with full Class DS-3 conditions.

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Site Analytical Services Ltd.				REF: 13/20821
LOCATION:	50 Avenue Road, Camden, London, NW8 6HS			FIG: 1
TITLE:	Site Sketch Plan	DATE:	August 2013	SCALE: NTS