conisbee | Consulting Structural Engineers
Consulting Civil Engineers

Hillview, Vale of Health, Hampstead, NW3 1AN

Structural Report to Support Planning Application

London
1 – 5 Offord Street
London N1 1DH
Telephone 020 7700 6666

Norwich
6 Upper King Street
Norwich NR3 1HA
Telephone 01603 628 074

Cambridge
47 – 51 Norfolk Street
Cambridge CB1 2LD
Telephone 01223 656 058

design@conisbee.co.uk
www.conisbee.co.uk

Ref: 160372/A Dunsmore

Approved By: Terry Girdler (CARE)

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1.0 INTRODUCTION

1.1 Conisbee have been instructed by the owners of Hillview, Vale of Health to act as consulting structural engineers for the refurbishment of the property.

1.2 As part of our service a structural report has been requested to accompany the planning application being made by HEAT Architects.

1.3 This report addresses the current condition of the property and sets out the structural requirements to maintain overall structural integrity whilst ensuring that conservation principles are considered and implemented where possible.

1.4 The property is not listed but is in a conservation area.

1.5 This report has been written by Project Director Allan Dunsmore MICE MIStructE and approved by Head of Heritage, Terry Girdler, Conservation Accredited Engineer (CARE).

2.0 EXISTING PROPERTY

2.1 The existing house was constructed in 1879/80 and is of traditional Victorian construction. It currently has 4 storeys including a lower ground floor. The building is currently unoccupied and the fabric has been allowed to fall into a very poor state of repair over time and has also had some sub-standard repair work carried out to it. Over the years the property has suffered from gross instability.

2.2 The internal finishes have been removed and the walls, floors and roof structure are currently exposed.

2.3 The walls are typically 225mm solid masonry with timber floors and stud walls internally. The roof is of timber construction with a tiled finish.

2.4 The existing foundations are shallow corbelled masonry and are founded on a soft layer of silt which has been subject to settlement and movement over the years.

2.5 The foundations at the rear of the property have settled by 170mm in one corner causing severe deformation of the floors and of the rear brick facade. This can be demonstrated on the building survey drawing and the photographs appended to this report.

2.6 Throughout the property, the floor joists have been notched extensively for heating pipes and wiring and are not capable of safely carrying current domestic loadings. The joists are currently deflecting under their own self weight, are undersized for their spans and are also out of level due to the settlement of the foundation to the rear.

2.7 The roof timbers are in a poor state of repair and many of the key members exhibit severe wet rot where the roof has leaked. The roof timbers are currently supported from waling timbers which span between chimney breasts. Wet rot is prevalent in the supporting waling timbers which will need to be replaced. The primary timber roof structure has been altered for new rooflights at various points and is structurally deficient and a potentially dangerous structure. It is currently not fit for purpose.

The roof waling beams are also propped from floor joists which are not designed to transfer any load other than that of the floor.

Many of the roof timbers have been cut and removed creating weakness allowing for excessive deflection.

The roof has leaked in various places and the roof coverings are in a poor state of repair.

2.8 The rear elevation brickwork is in a poor state of repair and has been adversely affected by the 170mm settlement of the rear foundation on one side. This has caused deformation and cracking around the windows and the brick lintels have slipped and cracked. The gaps around the windows have made a direct route for water ingress into the rear of the property.

The brick bond to the rear elevation has had various repairs carried out over the years and there is no consistent bond due to the variable nature and quality of the repairs. These crude repairs also mask the defects to some extent.

Brickwork around the rear windows is loose and is unstable due to sub-standard repairs. Bricks here can easily be removed by hand.

Pointing both internally and externally is generally loose and friable making the wall very fragile. Internally, a combination of damp ingress and movement have made the internal plaster finish unstable and it has been largely removed.

The current openings in the rear elevation and in the central spine wall, combined with the poor state of the brickwork means that the lateral stability of the property is seriously compromised and the integrity of the building becomes reliant on the walls of adjacent properties in the terrace. This means that any sub-standard alterations to adjoining houses makes the whole terrace potentially subject to the book-end effect where lateral instability can be transferred through the houses to the end of terrace property.

Repairs to the rear elevation would only represent a short term solution to the problems which are now inherent. The appearance, durability and longevity of the structure would be enhanced greatly by re-construction to match original brick type and bond.

2.9 The front facade is in reasonable condition and will require minor repairs which will be part of the works. It will also need to be fully tied back to the new structure behind.

2.10 The existing rear extension is a poorly constructed off the shelf type conservatory structure with a pvc framed glazed roof.

3.0 PROPOSED REFURBISHMENT

3.1 The proposed refurbishment seeks to sensitively reinstate the structural integrity and independent stability of the property whilst being sympathetic to the aesthetics and character of the conservation area.

3.2 The proposals do not affect the front elevation or the existing roof profile of the property. It is proposed to demolish the existing and construct a new rear extension which will enhance the space and appearance of the rear elevation.

3.3 In order to stabilise the foundations in the long term it is essential to underpin the entire property down to suitable natural ground. This will also incorporate a new lower ground floor slab for the main house footprint and for the new extension.

3.4 New concrete strip foundations will be constructed for the rear extension.

3.5 Other foundation options were considered including mini piling with a new suspended GF slab but this option would create more disruption for the neighbours and would also potentially create future issues with differential settlement of the adjacent terraces.

3.6 In order to form new floors that comply with current domestic loading and will help to stabilise the building, a new floor structure will be provided at GF, FF and 2F levels. This will consist of steel beams spanning between the party walls to trim around the stairs. New floor joists will be provided which span between the steel beams and the party walls. New floors will be tied into the walls using padstones with horizontal wall plates resin anchored to them.

Due to the existing joists being heavily notched and undersized it has not been possible to justify re-using these for the new floor construction.

The new floors will have plywood deck to provide a floor diaphragm, essential for long term overall stability and robustness of the refurbished building.

3.7 In order to establish integrity and provide independent stability, a steel frame is being constructed to the rear elevation of the building. This will provide lateral stability whilst allowing open access to the rear extension at lower ground floor level.

The steel frame will be supported from the new slab and foundations at lower ground floor.

Due to the poor construction and state of disrepair of the rear elevation we have advised that the safest way to construct the new foundations and steel frame in this area will be to demolish the existing brickwork and re-construct it.

It is proposed that this will be re-built in masonry with external leaf bond and details to match existing. This will bring the rear of the property in line with the requirements of the conservation area as well as providing long term stability and integrity to the rear of the house.

With the amount of structural work required to the floors, foundations and rear of the house, as well as the multitude of defects and poor construction, we believe that re-building the rear facade is the safest and most pragmatic way to progress the scheme whilst enhancing the appearance of this elevation.

There will be no detriment to adjacent properties during the works and appropriate sequencing and temporary works will be fully considered prior to works commencing.

Any alternative scheme of retention would result in a distressed rear facade of patched elements with loss of much of the original work. This would be visually unattractive and would not meet the structural requirements. By re-building the rear wall heat loss in the building can be improved which would not be possible with a patched existing wall.

3.8 It is proposed to keep the roof profile and finishes the same as existing but in order to bring the overall building fabric up to current standards and to provide a robust independent structure it is proposed to re-construct the roof using steel frame and timber infill.

The steel frame will be supported from the new 2nd floor roof structure and will provide a coherent load path for the structure rather than relying upon existing chimney breasts and undersized floor joists for support. The loads will be taken back into the party walls via the 2nd floor beams.

We have considered re-use of the existing roof timbers but due to their size and deterioration over the years they will only be suitable for some secondary timber box outs rather than primary structure.

The roof will also have a ply sheathing to provide a diaphragm and will contribute to the long term overall stability and integrity of the house. Traditional finishes will be used.

3.9 Existing chimneys and chimney breasts will be removed. Where removed the walls will be made good with brickwork and lime mortar to tie in with existing. There is no contribution from the existing chimney breasts to the overall stability of the property as they are debonded from the party walls.

4.0 CONCLUSIONS

THE VALE OF HEALTH CONSERVATION AREA

4.1 Hill View is not listed but is part of the conservation area and because it is considered to be a building which makes a positive contribution.

4.2 The reasons for a building having this designation can vary. They can be notable because of their value as local landmarks, or as particularly good examples of the local building tradition. Such buildings, whilst not statutorily listed are nevertheless important local buildings in their own right and make a positive contribution to the character and appearance of the Conservation Area. The general presumption should therefore be in favour of retaining such buildings.

4.3 Although not listed, the Government requires that proposals to demolish these buildings should be assessed against the same broad criteria as proposals to demolish listed buildings.

4.4 The structural defects identified in this report are all common faults that can be found in many traditional Victorian terraced houses. The scale of some of the defects is unusually severe, particularly the 170mm settlement of the foundations.

4.5 Previous very poor quality workmanship and loss of original fabric make this a very difficult building to repair in a manner that would immediately be described as good conservation. The building was of a speculative type and built originally to a budget. Buildings of this kind require continual maintenance and Hillview has been neglected over many years. Even if repairs were made, it will not be possible to bring Hillview to the standard it would be at, had the necessary maintenance been carried out over the years. Repairs to the existing fabric would have a very short term life.

4.6 We have carefully considered the heritage assets that are present in this building and feel that the most important elements are:

1. Its contribution to the “Vale of Health” street scene.
2. Its value as a building that will provide efficient and beneficial use.
3. A building that will be well maintained and ensure the adjacent buildings have suitable structural partners.

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4.7 **The structural issues that have been identified and need to be addressed are**:

Foundations

The poor load carrying characteristics of the underlying silt on which the building sits leads us to the conclusion that a redesign of the foundation is necessary. From the completion of exploratory geotechnical boreholes and the subsequent Site Investigation Report which was carried out by Connaughts Site Investigations Ltd, it has been decided to adopt traditional mass concrete underpinning to strengthen the existing foundations. From information provided in the site investigation report sections of underpinning sections will be in the order of 1200mm to 1600mm deep in order that the structure is founded on a soil strata which is capable of supporting the loads applied.

Timber Floors and Roof

The overall condition of the timber floor has been compromised by previous repairs, interventions and deterioration. It can be seen that timbers have been placed alongside others in an attempt to strengthen structurally poor timbers. The installation of services (particularly water pipes) has resulted in the timbers being notched is a manner that severely reduces the strength of the floor. The roof has suffered from water ingress which has rotted the timbers. As the performance of a roof is fundamental to the future life of a building, it has been concluded that replacement of the roof in modern materials, to a structurally safe design is required. The existing external profile of the roof will be maintained in order to respect the street scene.

Rear Elevation

The condition of the brickwork to the rear elevation is poor. Previous repairs using cement mortar and replacement bricks can be seen. It is apparent that localised exfoliation of the bricks has taken place and this is a concern for future durability.

4.8 As the heritage asset provided by the rear elevation is considered nominal, it is proposed to rebuild the rear elevation. The replacement wall can then incorporate modern standards of insulation which will have a long term benefit to the thermal performance of the building. The visual appearance of the rear elevation can be carefully detailed and specified to satisfy conservation principles. For example, the use of replacement red brick segmental arches, sympathetic brick and mortar type, together with fenestration details can all be designed and agreed with the planning department to ensure like for like replacement and make a positive contribution to the conservation area that is currently not provided by the structurally defective elevation.

4.9 In conclusion, it is considered that as the structure of this building is not a good example of Victorian building, therefore its general (and often defective) fabric should not be considered to be a heritage asset. The most important contribution made by the building is its street elevation and this will be enhanced and maintained by the current proposal.

4.10 All of the proposed work will be carried out under the Party Wall etc Act 1996 thus ensuring proper safeguards for neighbouring properties.

4.11 A Construction Management Plan will also be put in place by a contractor prior to construction work commencing.

4.12 In its present state the building is not safe to inhabit.

4.13 The following section contains photographs of the property and which demonstrate the defects discussed in the body of this report.

5.0 RELEVANT PHOTOGRAPHS

Image 1: Image of cracks in lintel to rear wall due to deformation of the wall/foundation settlement

Image 2: Cracked lintel and exfoliation of brickwork to rear facade

Image 3: General view of rear elevation

Image 4: View of rear elevation and rear conservatory

Image 5: Rear elevation showing poor condition of brickwork, sub-standard repairs and different brick bond

Image 6: Internal view of rear elevation showing excessive deflection of floor adjacent to rear door

Image 7: Internal view of rear facade inadequately toothed in/tied to party wall – Image 1

Image 8: Internal view of rear facade inadequately toothed in/tied to party wall – Image 2

Image 9: Loose brickwork to window reveal in rear facade

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Image 10: Evidence of rot in floor joist ends and timber wall plate

Image 11: Image of rotten floor joist

Image 12: Image of rotten floor joist

Image 13: Undersized joists and evidence of rot where joists bear on external wall

Image 14: Internal brick wall at lower ground floor, brick wall and pier in poor condition

Image 15: Timber blocking embedded in brick pier approximately at mid-height

Image 16: Notching of joists and rot evident where joists bear on internal spine wall, general poor condition

Image 17: Opening bricked up and brickwork not toothed in properly

Image 18: Excessive notching of trimmer joist to stair void – Image 1

Image 19: Excessive notching of trimmer joist to stair void – Image 2

Image 20: Load-bearing stud (spine wall on upper floors) in poor condition and is structurally inadequate

Image 21: Image showing excessive deflection of floor adjacent to stair void

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Image 22: Rotten floor joists inadequately spliced – Image 1

Image 23: Rotten floor joists inadequately spliced – Image 2

Image 24: Image of rotten joist

Image 25: Image of rotten joist

Image 26: Joists inadequately sized

Image 27: Roof waling beam and rafter ends showing signs of rot – Image 1

Image 28: Roof waling beam and rafter ends showing signs of rot – Image 2

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Image 29: Excessive notching to timber prop supporting roof waling beam

Image 30: Inadequate halving joint configuration and evidence of rot in roof waling beam

Image 31: Poor condition of brickwork and rot evident in roof waling beam and rafter ends

Image 32: Every second roof rafter has been cut at collar tie postion over full length of roof rendering the roof structcure inadequate and potentially dangerous

Image 33: Structural integrity of rafter compromised by excessive notching to roof light

Image 34: Poor condition of roof structure and inadequate trimming around roof lights

Image 35: Inadequate trimming to rafters around roof lights

Image 36: Splice in vertical timber prop to roof structure is inadequate and potentially dangerous

Image 37: Poor condition of waling beam and inadequate trimming to roof lights

Image 38: General image of poor condition of second floor joists, brickwork, roof waling beam, roof rafters and timber boarding to roof

Image 39: Image of poor condition of brickwork and roof structure to rear facade

Image 40: General view of poor state of roof construction and sub standard interventions for roof lights

Image 41: Rotten waling timber supporting roof joists. Also timber studs supported from undersized floor joists. Generally substandard roof construction is evident throughout

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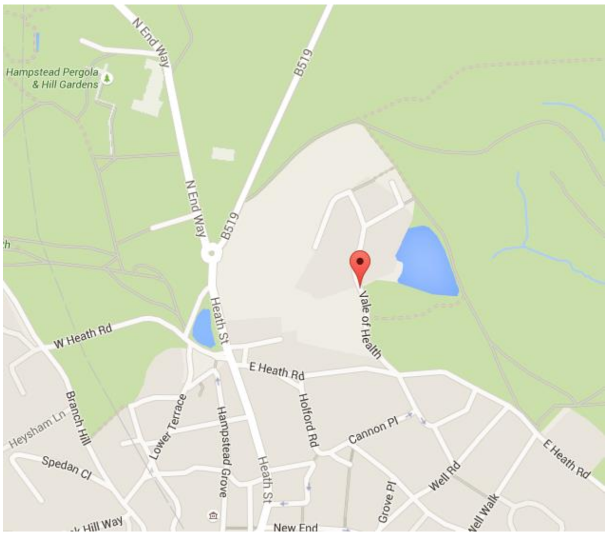
APPENDIX 1 CONNAUGHT SI REPORT

CONNAUGHTS
SITE INVESTIGATION LTD



Site Investigation Report
Hillview
Vale of Heath
Hampstead
London
NW3 1AN

Report No.: 0567i
Date: 27th July 2016

Client: Conisbee Consulting Engineers
(Denis Kealy)

Registered Office: 35 Green Lane, Leigh on Sea, Essex, SS9 5AP. Tel: 01702 528098
Registered in Cardiff. No. 6821685

Connaughts
Site Investigation Ltd

35 Green Lane
Leigh on Sea
Essex SS9 5AP

Phone: 01702 528098
Email: www.connaughts.aol.com

Conisbee Consulting Engineers Ltd
1-5 Offord Street
London
N1 1DH

Our Ref: SW/JW/0567i
Date: 26th July 2016

F.A.O: Denis Kealy

Dear Sir

Re: Hillview, Vale of Heath, Hampstead, London, NW3 1AN: Site Investigation Report

1.0 INTRODUCTION

In accordance with your instructions, we revisited the above site on 28th June 2016 following our initial site investigation conducted on the 17th and 18th May 2016. The purpose of our return visit was to provide additional information on the foundations to the existing property and structures along with the drilling of two boreholes to provide information on subsoil conditions at depth beneath the site.

The property Hillview, Vale of Heath, comprises four storey, mid terrace property of estimated Victorian / Edwardian age. The property contained a basement along with a number of sections which appeared to have been infilled at a later date than the original sections of the property. The property has a rear conservatory and a rear garden which leads to a large lake which is present approximately 20m from the property.

At the time of our site visit, the internal wall coverings and floor coverings had been removed in preparation for the proposed improvement works. These exposed potential indicators of structural movement to the property both in terms of walls and floors.

It is understood that the proposed redevelopment of the property is to comprise the lowering of internal floor levels along with the possible construction of a basement to the property. Presumably modernisation and structural alteration works will also be part of the proposals.

2.0 GEOLOGICAL INFORMATION

The geological survey map of the region shows the site to be situated in an area underlain by the Claygate Member, with the Bagshot Sands outcropping very close to the north, west and south of the site.

The Claygate Formation comprises a dark grey series of clays and sands which pass into layers of clays, silts and fine sands. In general this stratum represents a coarse particle size to the underlying London Clay Formation and with this, a typical lowering of plasticity (normally intermediate to high). In London, the thickness of the Claygate Formation is on average 16m. The Bagshot Formation which overlies the Claygate Formation typically comprises a fine to coarse SAND which can be clayey in part and attains a maximum thickness of 45m in the south west of London.

3.0 FIELDWORK

The site investigation works comprised the excavation of two trial pits on the existing building and structures along with the drilling of two window samples boreholes as well as the performing of two dynamic probe tests.

The boreholes were drilled to a depth of a depth of 5.00m (WS1) and 6,00m (WS2). Adjacent to these boreholes, a dynamic probe test (DP1 & DP2) was performed and taken to a depth of 8.00m within each probe hole to provide information on soil strength at depth. The boreholes were drilled using a Competitor Window Sampling drilling rig which progresses a borehole by the hammer drilling of 1m long steels cutting tubes within which is held a 1m long clear plastic liner which collects undisturbed samples. The diameter of the cutting tube is reduced regularly to allow for drilling to depths.

Insitu strength testing was conducted at 1.00m intervals within the window sample boreholes by the performing of a Standard Penetration Testing (SPT). This test comprises the recording of the number of blows taken to drive a steel cone into the soil from the drop of a 63.5kg hammer of a distance of 760mm. This test then records the four increments of 75mm to provide an N value with the first 150mm considered the seating blow and not included as part of the N value. As such, the actual test depth is 150mm greater than stated (i.e. SPT test conducted at 4.00m actually refers to an N value at 4.15m).

The dynamic probe tests were performed using exactly the same drop weight and drop height as the SPT tests and as such provides a continuous SPT test throughout the probe hole. To calculate an N Value for the soil, then for this test, the blow counts for thee increments of 100mm are taken and added together to provide an N value over 300mm. Within a cohesive soil, a number of publications have provided different factors to multiply N values by to obtain undrained shear strengths. The average factor which is used is a factor of 5.5. For example an N Value within a clay soil of 10 blows will correlate to 10 x 5.5 = 55kPa. This has been used to help interpret the soils encountered. Details of all subsoils encountered within the boreholes, along with samples and insitu strength testing conducted is discussed within Chapter 5.0 and recorded as borehole and dynamic probe logs within Appendix 2. All depths stated within this report and on the borehole logs are depths below the ground level surrounding the borehole positions.

Within the trial pits, foundation profiles were measured and recorded along with the taking of soil samples and insitu strength testing at foundation level. Within the hand augered borehole, small disturbed samples and insitu strength testing was conducted at 0.50m intervals. Insitu strength testing was conducted using the shear vane test in cohesive soils and the Mackintosh probe test in the granular soils encountered.

The shear vane test records the torque needed to shear a four bladed vane inserted into the soil. This is then converted to an undrained shear strength (kPa). In practice the shear vane readings tend to be a little higher than the tests obtained from laboratory testing such as triaxial compression testing. This tends to be due to the inconsistencies (veins, fissures etc) which are not always picked up in the shear vane test.

Mackintosh probe tests, within the granular soils, records the number of blows taken from the drop of a 10kg hammer over 500mm to drive a steel cone 300mm into the ground. This provides a guidance to the density of the subsoil. No published literature exists to convert Mackintosh probe tests to the more advanced Standard Penetration Tests although some practitioners use a factor of 0.1 as a rough guideline conversion figure.

Details of the foundations logged and the findings of the boreholes drilled are discussed within Chapter 5.0 and held as scaled diagrams and borehole logs within Appendix 2. The location of the borehole is marked on the site plan within Appendix 1.

4.0 LABORATORY TESTING

Selected soil samples taken from the window sample borehole was sent to Soil Property Testing for UKAS accredited soils testing in accordance with British Standards 1377: Testing of soils for civil engineering purposes.

Two samples of the underlying cohesive clay soil were tested for their moisture content and their plasticity using the Atterberg limits tests. Two samples extracted from the sample tubes were tested for their undrained shear strength using the multistage triaxial compression test. Two samples were also tested for their soluble sulphate and pH value. The results of the soil laboratory testing is discussed within Chapter 7.0 and held as results summaries and test sheets within Appendix 3.

5.0 TRIAL PIT INFORMATION

Trial pit 7 was excavated within a vault room beneath the steps to the front door to the property at a similar level to the basement which was approximately 1.70m below the pavement level. The trial pit exposed the foundations to the font wall (A-A) and side wall (B-B) with both foundations seen to be of similar construction, comprising brickwork with two step outs onto a crushed flint and brick strip foundation. The total projection of the foundations were measured at 105mm (A-A) and 125mm (B-B) with both seated at a depth of 0.575m below surrounding ground level onto a very soft, very low strength (V= 14-18kPa), green / brown, very silty, organic CLAY (alluvium).

A water seepage was encountered at foundation level within this trial pit. No roots were encountered.

Trial pit 8 was excavated internally within the property and exposed the construction of the sump beneath the wooden floor in this part of the basement and also the wall of the conservatory. This trial pit found the inner block wall of the conservatory to be built on top of the concrete slab which was present at 0.135m below the floor level. The rear wall of the conservatory could not be fully exposed and appeared to possibly extend below the slab level with a concrete base encountered at 0.82m. It is not clear whether this concrete is the strip footing to the wall or associated with the sump in this location.

The sump contain a side wall which was 200mm thick mass concrete with the base situated at 0.90m below the floor level. The surface of the sump has been covered with fibreglass presumably as a waterproof layer to hold any water which collects in this structure. This fibreglass layer prevented any further progress as this would have breached the sump.

No water inflows or root activity was encountered within this trial pit.

6.0 BOREHOLE FINDINGS

Borehole 1 (WS1) was drilled within the front room of the basement and encountered concrete (0.12m) over a brown / red, sandy gravel with much brick, concrete and flint pieces (MADE GROUND). Below this, at 0,50m, a very loose, brown, sandy, slightly gravelly clay with occasional brick and flint pieces (MADE GROUND) was encountered. This was underlain at 0.70m by a soft, very loose / very low strength, light grey, sandy, very silty CLAY / sandy, very clayey SILT with organic material and decayed roots. This soil was present to a depth of 3.00m where a firm becoming firm to stiff, medium strength, brown mottled grey, slightly sandy CLAY was encountered. This stratum was then encountered with minor lithological variations to the close of the borehole at a depth of 5.00m.

A steady inflow of water was encountered within this borehole at 4.00m, with the water level rising to 2.00m after 5 minutes and was found to stand at 1.00m on completion of all site works.

Borehole 2 (WS2) was drilled within one of the rooms towards the rear of the property at basement level. This borehole encountered a wooden floor and void to 0.30m over a brown / red, sandy gravel with much fine to coarse, brick, concrete and flint pieces (MADE GROUND). This was underlain at 1.20m by a soft, very loose / very low strength, brown / grey, slightly sandy, very silty CLAY / sandy, very clayey SILT with organic material. This soil was found to become slightly grey in colour with rare fine to medium black rounded flint gravels. This stratum as then present to a depth of 3.90m where a firm becoming firm to stiff, medium strength, brown mottled grey, slightly sandy CLAY was encountered. This stratum was then encountered with minor lithological variations to the close of the borehole at a depth of 6.00m.

No water inflows were encountered within this borehole which was cased to 3.00m. On removal of the casing, collapse of the borehole sides prevented any measurements of water levels within this borehole.

7.0 INSITU STRENGTH TESTING

Insitu strength testing within both boreholes comprised the performing of standard penetration testing at 1.00m intervals followed by the performing of continuing SPT testing within dynamic probe holes drilled to the side of the borehole locations.

Within **borehole 1**, the overlying alluvial clay soil was found to be very low strength (10-20kPa) becoming medium strength (40-75kPa) at a depth of 2.00m. The underlying very silty CLAY soil was of medium strength (40-75kPa) becoming high strength (75-150kPa) at 5.00m. The dynamic probing performed below this depth found the sandy CLAY to be medium strength at 5.50m and then gradually increased in strength becoming high strength by 6.10m and very high strength (150+ blows) below 7.10m and to the close of the probe hole at 8.00m.

Within **borehole 2**, the overlying alluvial clay soil was found to be very low strength (10-20kPa) becoming low strength (20-40kPa) at 2.00m and medium / high strength (75-150kPa) at 3.00m. The underlying very silty CLAY soil was of medium strength (40-75kPa) at 4.00m, becoming high strength (75-150kPa) at 5.00m.

The dynamic probing performed below this depth found the sandy CLAY to be low strength at 6.10m and then gradually increased in strength becoming medium strength by 6.30m and high strength (75-150kPa) by 6.70m and remained high strength to the close of the probe hole at 8.00m.

Borehole 1	SPT N Value	Soil Type	Conversion to shear strength (kPa)	Description
1.00m	3 blows	Alluvial CLAY	19kPa	Very low strength
2.00m	8 blows	Alluvial CLAY	52kPa	Medium strength
3.00m	7 blows	Alluvial CLAY	45kPa	Medium strength
4.00m	10 blows	CLAYGATE FM	65kPa	Medium strength
5.00m	17 blows	CLAYGATE FM	110kPa	High strength
6.00m	12 blows	CLAYGATE FM	78kPa	High strength
7.00m	26 blows	CLAYGATE FM	169kPa	Very high strength
7.70m	33 blow	CLAYGATE FM	214kPa	Very high strength
Borehole 2	**SPT N Value**	**Soil Type**	**Conversion to shear strength (kPa)**	**Description**
1.00m	3 blows	Alluvial CLAY	19kPa	Very low strength
2.00m	5 blows	Alluvial CLAY	32kPa	Low strength
3.00m	12 blows	Alluvial CLAY	78kPa	High strength
4.00m	9 blows	CLAYGATE FM	58kPa	Medium strength
5.00m	12 blows	CLAYGATE FM	78kPa	High strength
6.50m	9 blows	CLAYGATE FM	58kPa	Medium strength
7.00m	14 blows	CLAYGATE FM	91kPa	High strength
7.70m	22 blow	CLAYGATE FM	143kPa	High strength

8.0 LABORATORY TESTING RESULTS

Four samples of the underlying cohesive soils were tested for their **plasticity** using the Atterberg limits test from the window sample boreholes. This testing found the alluvial CLAY tested to be of high plasticity and of a high organic content, with one sample being classified as an organic clay. The underlying silty CLAY (Claygate Formation) was also tested, with the sample taken at 5.28m finding this clay to be of high plasticity.

BH	Depth	MC	LL	PL	PI	Class	Retention	Comments
WS1	1.00m	38	52	19	33	CHO	0%	1. No Des 2. No Des
	2.00m	39	54	21	33	CH	0%	1. No Des 2. No Des
WS2	2.00m	32	51	17	34	CH	0%	1. No Des 2. No Des
	5.28m	33	62	22	40	CH	0%	
MC: moisture content LL: Liquid Limit PL: Plastic limit PI: Plastic Index								

Desiccation analysis using Driscoll's relationships found no evidence for any significant levels of desiccation which is consistent with the field tests and field descriptions.

Triaxial compression testing of undisturbed samples extracted from the U100 sample at various depths was conducted to determine the undrained shear strength of the cohesive soil at this depth. This test was conducted at overburden pressures to replicate the pressure conditions the samples would have been in within the ground.

BH	Depth	MC	Bulk Density Mg/m3	Dry Density Mg/m3	Deviator Stress (kPa)	Shear Stress (cu)	Soil Strength Descriptions using BS5930 and (BS 14688)
BH1	3.80m	34	1.94	1.45	87 97 107	43kPa 48kPa 54kPa	Cu: 39.2 Ø: 2.4
BH2	5.28m	33	1.87	1.41	92 94 98	46kPa 47kPa 49kPa	Cu: 44.5 Ø: 0.6

This testing was consistent with the insitu strength testing (N values and converted shear strengths) with the testing indicating the overlying organic clay at 3.80m to be of medium strength and the underlying silty CLAYS also to be of medium strength.

Two samples of the underlying clay soil were tested for their **soluble sulphate** content and **pH value** at depths of 2.00m and 5.28m. The sample tested at 2.00m, was found to contain 0.03g/l soluble sulphate and had a pH value of 4.5 while the sample tested at 5.23m was found to contain 0.17g/l soluble sulphate and had a pH value of 6.1.

British Standards guidelines for assessing the aggressive chemical environment provide classification of sites based on SO_4 levels. To convert SO_3 to SO_4 levels a factor of 1.2 must be applied followed by multiplying by 1000 to convert from g/l to mg/kg. Applying these factors gives SO_4 results of 36mg/kg at 2.00m and 204mg/kg at 5.23m. Applying these results to the standards chart indicates that the clay soil on this site has a very low sulphate level with both samples falling into the DS1 concrete class (<500mg/kg).

9.0 COMMENTS

The geological survey map of the area suggested that the site was underlain by the Claygate Formation of Eocene age. This is broadly consistent with the soil profile encountered on site which comprised an overlying layer of variable MADE GROUND over an organic alluvial CLAY. The Claygate Formation (slightly sandy CLAY) was encountered below 3.00m (WS1) and 3.90m (WS2) and was then present to the close of the boreholes / probe holes at 8.00m. The presence of the alluvial clays to a depth of 3.00-3.90m is most likely associated with the lake present to the rear of the site.

Water levels were difficult to determine within the boreholes with collapse of borehole sides preventing readings from being taken on completion of drilling in borehole 2. Within borehole 1 a steady inflow was encountered at 4.00m with the water level found to rise to 2.00m after 5 minutes and stand at 1.00m on completion of all site works.

This would seem to indicate that groundwater is not present at shallow depths, although it should be noted that inflows were encountered within trial pit 7 and with a number of the previous trial pits drilled during the previous investigation works.

It is understood that the proposed development on this site is for the redevelopment of the property to comprise the lowering of internal floor levels along with the possible construction of a basement to the property. Presumably modernisation and structural alteration works will also be part of the proposals.

Bearing capacity figures have been provided to give a guide to the anticipated bearing capacities of the soil based on the converted N Values and triaxial strength tests conducted. Where both tests are present, then the triaxial strength test will take priority due to the accuracy of this test over SPT conversion factors. All bearing capacity figures provided are based on an assumed 1.00m wide strip foundation unaffected by groundwater clearly greater bearing capacities will be achieved by wider / larger foundations. Soil bearing capacities have been provided but should be calculated by a qualified engineer with knowledge of the anticipated design loadings.

Depth	Soil Type	Test Result (N Value / Triaxial)	Undrained shear strength	Bearing Capacity
1.00-120m	Alluvial Clay	3 blows (N)	18kPa	36kN/m^2
2.00m	Alluvial Clay	5-8 blows (N)	32-52kPa	60-100kN/m^2
3.00m	Alluvial Clay	7-12 blows (N)	45-78kPa	90-150kN/m^2
4.00m	Claygate FM	9-10 blows (N)	58-65kPa	115-130kN/m^2
5.00m	Claygate Fm	12-17 blows (N)	78-110kPa	150-220kN/m^2
6.50m	Claygate FM	11-16 blows (N)	71-104kPa	140-210kN/m^2
7.50m	Claygate FM	18-30 blows (N)	117-195kPa	230-300+kN/m^2

The strength of the clay soil at shallow depths shows a low bearing capacity is achievable within the alluvial clay at shallow depth but increases with depth. Sufficient information is held within this report for the design of any new foundations and to enable a decision over the most suitable foundation solution.

10.0 CERTIFICATION

The conclusions and recommendations given within this report, are based upon the stated development plans for the site. If the site is to be developed for a more or less sensitive use then a different interpretation may be appropriate.

This report relies upon the co-operation of other organisations and the free availability of information and total access. Therefore, no responsibility can be accepted for conditions arising from information, which was not available to the investigation team as a result of information being withheld or access prevented.

The analyses and opinions expressed in the report are based upon data obtained from the site investigation. Responsibility cannot be accepted for variation in ground conditions between and around exploratory points not revealed by the data or at the time of the investigation.

The report may suggest an opinion on the nature of the strata or conditions between exploratory points and below the maximum depth of investigation. However, this is for guidance only and no liability can be accepted for its accuracy.

Signed

JWoodward

James Woodward BSc(Hons) DipHE
For and on behalf of
CONNAUGHTS SITE INVESTIGATION LTD

Signed

M Pickering

Mark Pickering FGS
For and on behalf of
CONNAUGHTS SITE INVESTIGATION LTD

APPENDIX 2 CVS

| Team & Experience |

Allan Dunsmore - Director
BEng (Hons) CEng MICE MIStructE

Allan's experience spans many key sectors including residential, commercial, arts and culture and education.

Current projects in his team include a major refurbishment at Queen Mary University, mixed-tenure residential schemes for Peabody and One Housing Group, both in central London, a complex mixed-use scheme for CIWEM which involves a facade retention and new build over old mail tunnels along with a new-build studio theatre for the School of Drama at Royal Holloway University of London. Allan also heads the team for our recent appointment to City University Framework.

Allan has longstanding working relationships with a number of high-profile / design-led architectural practices. He collaborates closely with them to fully realise the architectural intent of their projects.

His prizewinning projects include the Laban Dance Centre with architect Herzog and De Meuron which won the Stirling Prize in 2003 and St Marylebone School Performing Arts Faculty which won a RIBA National award 2008, and Concrete Society best building award 2008. Lowther Children's Centre also won a RIBA award in 2010 and the Artist's House; a private residential scheme, won the Stephen Lawrence prize in 2010. Allan is currently Project Director, working with Igloo and Lambeth on the Somerleyton Road scheme in the heart of Brixton.

Allan is the Conisbee Supervising Civil Engineer and is a regular reviewer for candidates wishing to sit the IStructE chartered member exam.

Terry Girdler – Associate, Head of Heritage Engineering
BSc (Hons) Eng, MSc, CEng, FICE, MIStructE
Conservation accredited engineer (CARE)

Renowned in the UK for his conservation engineering work on historic buildings and structures, Terry is one of a very small number of engineers nationally on the Conservation Accreditation Register for Engineers. He has contributed to the repair and preservation of many of the nation's most prestigious buildings and structures – from Hampton Court Palace, Stonehenge, and The Iron Bridge at Colebrook Dale, to Hadrian's Wall, and the Monument in London.

Terry joined Conisbee in 2009 as head of heritage engineering providing appropriate and innovative solutions to a range of conservation and refurbishment projects.

His career has included working for the Department of the Environment's Directorate of Ancient Monuments and as chief structural and civil engineer at English Heritage with responsibility for the engineering work carried out on all of its 409 sites.

In addition to giving regular lectures, he is also on steering committees for research and advice with organisations such as CIRIA, ICOMOS and the Historic Bridges and Infrastructure Awards.