HERTS & ESSEX SITE INVESTIGATION

The Old Post Office, Well Pond Green, Standon,
Ware, Herts. SG11 1NJ

Telephone No.: Ware (01920) 822233
Fax No.: Ware (01920) 822200

25th May 2007

Our Ref : NJD/7857

Conisbee
1-5 Offord Street,
London.
N1 1DH

For the attention of Ms. H. Hawker.

Dear Madam,

Re : 62 Marchmount Road, London. WC1N 1AB : Geotechnical Site Assessment

1.0 Introduction

1.01 Authorisation

In accordance with the e-mail instructions received from Ms. H. Hawker on the 4th April 2007, a Geotechnical Site Assessment was carried out on the 19th and 20th April 2007.

1.02 Aims

The aims of this Geotechnical Site Assessment were as follows;

- Determine details of the existing and proposed site including any existing and proposed structures, services, cover and vegetation.
- Determine details of the existing foundations, ground and groundwater conditions and root activity and to perform in-situ bearing capacity tests across the site in the areas of the proposed structure.
- To collect a range of disturbed samples and to carry out geotechnical laboratory testing upon them based on the ground conditions encountered and the existing and proposed site.
- To provide parameters and recommendations on the ground and groundwater conditions, tree, shrub and seasonal influences and heave potential, bearing capacity, shallow foundation and floor design, concrete protection from sulphate attack and excavations and dewatering.
- To provide recommendations for any remedial measures required and the need for and design of any further investigative works and testing required.

1.02 Limitations

The comments and opinions expressed in this report are based on our experience and the relevant guidelines available, the characteristics of the site and surrounding area observed during our visit, the subsoil conditions encountered on site, in the locations of our works and at the time of our visit, the subsequent geotechnical laboratory testing and geological, hydrogeological and topographical records.
Therefore, it is possible that certain relevant conditions were not encountered and that those encountered may change over time, that further investigative and site works and laboratory testing would provide relevant information and that existing guidelines may change and further guidelines may become applicable in the future.

2.0 Environment Setting

2.01 Geological Records

The site is shown on BGS Sheet 256, 1:50000 to be present on a Post-diversionary Thames river terrace drift deposit of Lynch Hill Gravel (Gravel, sandy and clayey in part), overlying a solid Eocene deposit of London Clay (Undivided; Clay, silty in part 71-110m).

2.02 Hydrogeological Records

The site is shown on NRA Sheet 40, 1:100000 to be present on a Minor Aquifer comprising of the Lynch Hill Gravel deposit of intermediate permeability, overlying a non-Aquifer comprising of the London Clay deposit of negligible permeability.
Minor Aquifers can be fractured or potentially fractured rocks, which do not have high primary permeability, or other formations of variable permeability including unconsolidated deposits. Although they will seldom produce large quantities of water for abstraction, they are important both for local supplies and in supplying base flow to rivers.
In this urban area the Minor Aquifer comprising of the Lynch Hill Gravel deposit is given a worst-case vulnerability class HU until proved otherwise, because it is in an urban area where soil information is based on fewer observations and the soil is more likely to have been disturbed or removed than elsewhere. In non-urban areas Minor Aquifers comprising of the Lynch Hill Gravel deposit and overlying non-Aquifers are given the vulnerability class I1. I, indicating soils of intermediate leaching potential which have a moderate ability to attenuate diffuse source pollutants or in which it is possible that some non-absorbed diffuse source pollutants and liquid discharges could penetrate the soil layer. 1, indicating soils which can possibly transmit a wide range of pollutants.
Non-Aquifers are formations which are generally regarded as containing insignificant quantities of groundwater. However, groundwater flow through such soils/rocks, although imperceptible, does take place and needs to be considered in assessing the risk associated with persistent pollutants. Some Non-Aquifers can yield water in sufficient quantities for domestic use. Non-Aquifers are considered to be negligibly permeable and so are not given a vulnerability class.
This information was collated primarily to aid assessment of groundwater vulnerability to contamination, but additionally gives an indication of the permeability of geological deposits and potential groundwater conditions.

2.03 Topographical Records

The site is shown on OS Landranger Sheet No.176, 1:50000 to be present in the S of St. Pancras, London Borough of Islington, N London, TQ 301 823 GB Grid.
The site is present between the 20-55m amsl contours on a river terrace of the River Thames on land sloping very gently down from WSW-ENE towards the low valley formally occupied by the lost River Fleet.
There are no surface water features present in close proximity.

3.0 Site Reconnaissance

3.01 Location Description

The property is present on the ENE side of Marchmount Street, which extends NNW-SSE, approximately SSE 50m from the junction with Tavistock Place in a terrace of Victorian 5 storey buildings with basement and ground floor businesses and residential upper floors and basement level rear yards. Many of the original buildings have been renovated/extended. A modern block of flats with communal open spaces are present to the ENE and a similar terrace is present opposite to the WSW.
There is no direct vehicle access to the property but restricted parking is present opposite and in a car park present at the SSE end of Marchmount Street. There is gated pedestrian access to the rear of the property via Handel Street or the Community Centre.
The surrounding land is essentially level.

3.02 Existing Site Description

The site comprises of No.62, which faces WSW and consists of a mid-terrace, 5 storey Victorian building of traditional brick and timber construction extending from the basement floor with a mansard floor and a later brick, concrete and timber 2 storey rear extension offset to the NNW and enclosing a basement level concrete paved yard accessed from the basement floor fire exit and steps present along the rear property boundary and leading to the bin stores and neighbouring open space.

Surface and foul water pipes and gullies are present along the rear wall of the main building flowing into 4" vitrified clay drains present in the manhole in the yard , the main combined run has an invert level of 0.45m and flows from ENE-WSW towards the front and beneath the main building. A rodding eye was present on the SSE flank wall of the rear extension and the associated 4" vitrified clay drain was encountered in TP3. No other services were apparent on site or were encountered in our excavation.
A mature, moderate water demand 12m Rowin tree is present in the neighbouring open space adjacent to the NE corner of the property.
There is a 1.7m step up in level from the rear yard to bin stores and neighbouring open space. The yard, bin stores and open space are essentially level

3.03 Proposed Site Description

We understand that the proposed works will include the construction of a full width, 2 storey extension occupying the majority of the existing rear yard.

4.0 Site Work

4.01 Site Plans

Details of the existing and proposed site, the trial pit and borehole locations and the predicted existing and mature tree influences are shown on site plans forming appendix one. The site was surveyed basically using tape measured offset and bisecting distances and levels and heights and should therefore be considered approximate. Therefore, it may be prudent to carry out further accurate surveying employing a TST and to employ an arboriculturalist to identify any trees and predict their influence. The predicted existing and mature tree influences were determined in accordance with NHBC 4.2.

4.02 Trial Pits and Boreholes

TP1, TP2 and TP3 were excavated externally by hand in the rear yard in order to expose the existing foundations of the various buildings within the area of the proposed extension and to describe the existing ground and groundwater conditions and root activity, to recover disturbed soil samples for subsequent geotechnical testing and to perform in-situ bearing capacity tests.
TP1 was excavated on the junction of the ENE rear wall of the main building and NNW flank wall of the chimney, TP2 on the junction of the NNW flank wall of a projecting building of the neighbouring property to the SSE and stepped sections of the SSE boundary wall and TP3 in the centre of the SSE flank wall of the rear extension.
Towards the NNW end of the line of the rear wall of the proposed extension a 50mm ø borehole was hand augered to a depth of 5.10m in order to describe the existing ground and groundwater conditions and root activity, to recover disturbed soil samples for subsequent geotechnical testing and to perform in-situ bearing capacity tests.
TP2 was excavated to reduce the required size of TP1, TP3 was relocated to avoid the rodding eye and BH1 was relocated to avoid the standing water present in BH3.
The details are recorded on the trial pit and borehole logs forming appendix two.

4.03 Bearing Capacity Tests

In-situ shear strength tests were carried out in the CLAY (1) using a hand held Shear Vane. The shear vane test is considered to be an approximate unconsolidated undrained test of shear strength.
The details are recorded on the trial pit and borehole logs forming appendix two.

5.0 Laboratory Testing

5.01 Geotechnical Laboratory Testing

All geotechnical testing undertaken was in accordance with BS:1377:1990 Method of Test for Soils for Civil Engineering purposes.
Geotechnical laboratory testing was carried out on all the disturbed samples of the CLAY (1) to determine their Moisture Contents and on selected samples to determine their Atterberg Limits (LL, PL, PI), soluble sulphate contents and pH values.
The results of the geotechnical laboratory testing form appendix three.

6.0 Geotechnical Analysis

6.01 Desiccation Profile

Driscoll's Method of Desiccation Analysis was applied to produce the desiccation profile indicating adverse tree influence. The Liquid Limit (LL) of the soil is multiplied by 0.4 and 0.5 and compared with its moisture content. A moisture content <0.4LL indicates significant desiccation, 0.4-0.5LL indicates slight desiccation and >0.5LL indicates no desiccation. The Liquid Limit is multiplied by one minus retention LL(1-r) to account for any granular content.
The method is applicable to natural soils to a depth of 3.00m. Beyond this depth overburden pressure is considered to exert increasing and unacceptable influence upon the results.
It should be noted that the Driscoll's Method of Desiccation Analysis applied is based on the London Clay deposit of which the CLAY (1) encountered form a part and the shear strengths and loading of the CLAY (1) present on site and experience of the London Clay deposit present are taken into account.

6.02 Predicted Tree and Seasonal Influences

Predicted Tree, Shrub and Seasonal Influences were determined in accordance with NHBC chapter 4.2 'Building near trees' April 2003 edition.
The CLAY (1) is a high volume change potential soil (I'p>40%).
The Rowin tree is of moderate water demand with a mature height of 12m.
A minimum foundation depth of 1.0m is recommended to avoid adverse seasonal influences.
It should be noted that the method of determining tree influence employed is empirical and non-site specific. Therefore any over sophistication of approach may give an impression of unwarranted accuracy.

7.0 Geotechnical Results

7.01 Ground Soil Conditions

The ground encountered was uniform across the site and was consistent with that indicated BGS Sheet 256, consisting of the London Clay deposit underlying the pavement base.
London Clay Deposit- within TP1, TP2, TP3 a firm varying to firm to stiff light orange brown mottled grey slightly silty slightly sandy CLAY (1) was present from 0.40m to their close at 0.60-0.80m. Within BH1 the firm to stiff CLAY (1), which became stiff and dark grey brown with depth, with occasional fine orange silty Sand partings from 0.80m, was present from 0.40m to its close at 5.10m.

7.02 Existing Foundations and Pavements

The foundations exposed had slightly different detailing depending on loading but were all essentially shallow strip foundations seated at similar depths on the same soil with similar bearing capacities.
Within TP1A the foundation of the ENE rear wall of the main building consisted of 4 brick corbels over a 300mm Brick rubble CONCRETE strip, projecting 240mm and founded at 0.67m on the firm CLAY (1) with shear strengths of 66-68kN/m².
Within TP1B the foundation of the NNW flank wall of the chimney consisted of 2 brick corbels over a 300mm Brick rubble CONCRETE strip, projecting 320mm and founded at 0.67m on the firm CLAY (1) with shear strengths of 66-68kN/m².
Within TP2A,B&C the foundation of the NNW flank wall of the neighbouring building and stepped sections of the SSE boundary wall consisted of 2 brick corbels over a 200mm Brick rubble CONCRETE strip, projecting 240mm and founded at 0.44m on the firm to stiff CLAY (1) with shear strengths of 76-78kN/m².
Within TP3 the SSE flank wall of the rear extension consisted of 2 brick corbels over a 160mm Brick rubble CONCRETE strip, projecting 250mm and founded at 0.40m on the firm to stiff CLAY (1) with shear strengths of 78-80kN/m².
We probed 100mm back beneath the base of the foundations and the walls and foundations exposed appeared to be in reasonable condition.
Within TP1 and TP2 the pavement consisted of 30mm mass CONCRETE, over 130mm mass CONCRETE, over 240mm compact brown clayey silty sandy Brick rubble FILL base seated on the firm varying to firm to stiff CLAY (1).

Within TP3 and BH1 the pavement consisted of 50mm mass CONCRETE, over 350mm of the compact Brick rubble FILL base seated on the firm to stiff CLAY (1).
The pavements exposed and generally appeared to be in reasonable condition.

7.03 Root Activity

No live or decayed roots were encountered within our excavations.

7.04 Groundwater Conditions

On excavation standing water was present in TP1, TP2 and TP3 at 0.30m, 0.30m and 0.26m. After dewatering the water returned as moderate inflows in TP1 and TP2 and a slight inflow in TP3 from the base of the foundations and then stood at the same levels.

7.05 Bearing Capacity Tests

In-situ shear strengths of 66-138kN/m² were achieved in the CLAY (1).

7.06 Geotechnical Laboratory Testing

Geotechnical testing proved the CLAY (1) to be of high to very high plasticity, Plasticity Index (Ip)=48-52%, with 425μm sieve retention of 0%, giving Modified Plasticity Index (I'p)=48.0-52.0%, indicating it to be a high volume change potential soil (I'p)>40%.
A soluble sulphate in 2:1 water/soil extract concentration of 190mg/l and a pH value of 8.0 were achieved in the CLAY (1).

7.07 Desiccation Profile

The CLAY (1) encountered proved to be only slightly desiccated.

7.08 Predicted Tree Influence

Predicted Existing Tree Influence- extended to affect the rear ½ of the existing rear extension to depths of 0.0-0.5m and the rear NE corner of the proposed extension to depths of 0.0-0.2m.
Predicted Mature Tree influence- as above.

8.0 Conclusions and Recommendations

8.01 Ground Soil Conditions

Based on the geological records and our experience we would suggest that the Brick rubble FILL was placed as a base for the pavement and the CLAY (1) form part of the London Clay deposit.
Having been present for an extended period of time the Fill has minimal settlement potential and being shallow and non-cohesive it will not affect the design and construction of any proposed foundations/underpinning and floors.
The London Clay deposit is over-consolidated and typically extends to depth. It is uniform, of high volume change potential, very low permeability, relatively low compressibility and moderate to good bearing capacity in natural condition.
Basically, the CLAY (1) forms a reasonable bearing stratum for the simple and relatively lightweight existing structures in terms of bearing capacity and settlement potential, but it is vulnerable to adverse tree, shrub and seasonal influences causing soil volume change and to instability/bearing capacity reduction caused by pore water pressure changes.

8.02 Groundwater

Water was encountered TP1, TP2 and TP3 standing at 0.27-0.30m and as slight to moderate inflows from the base of the foundations seated in the very low permeability CLAY (1) which is considered to form part of a non-Aquifer of negligible permeability. No water was encountered within BH3, removed from the foundations to a depth of 5.10m. The site is present on essentially land, no surface water features are present in close proximity and our works were carried out during a wet seasonal period.

Considering the above we would suggest that the water inflows are derived from relatively limited volumes of infiltrating surface water and/or leaking drains water which has 'pooled' within the basement floor of he property and possibly terrace. Therefore, we would suggest that the a given depth of the CLAY (1) below and adjacent to the basement level foundations should be considered saturated affecting its stability, bearing capacity, bearing capacity determination and the design and construction of any foundations and excavations within it.
It should be noted that groundwater conditions were observed for insufficient periods of time relative to the permeability of the CLAY (1) to determine them conclusively during our works, groundwater conditions may fluctuate seasonally and the London Clay deposit is considered to form a non-Aquifer in terms of groundwater flow for abstraction rather than groundwater presence.

8.03 Tree and Seasonal Influences

None of the moisture contents of the CLAY (1) have been reduced to a degree great enough to be classified as significant desiccation. Significant desiccation is commonly considered to be a qualification of movement related damage to structures caused by soil volume decrease (shrinkage) in subsidence claims and to indicate the potential for movement related damage caused by soil volume increase (heave) as it dissipates.
In addition no live or decayed roots were encountered in our excavations, predicted existing tree influence did not extend to affect the areas of our excavations and did not extend below the foundations of the existing buildings and as the tree is mature its influence will not increase.
Therefore, we would suggest that the existing and mature tree influences are not currently adversely affecting ground conditions and are unlikely to during dry seasonal periods and hence will not affect foundation/underpinning, floor or pavement design.
NHBC 4.2 recommends a minimum foundation depth of 1.0m to avoid adverse seasonal influences. Given the conclusions of 8.02 we would suggest that this is not relevant on site as any seasonal effects will be overwhelmed by the standing water, the proposed extension will occupy the majority of the existing yard and the adjacent ground level is 1.7m above.
Should any planting be proposed we would recommend that any proposed underpinning/foundations and floor design should be modified be designed in accordance with NHBC 4.2.

8.04 Existing Foundations

The foundations exposed are typical of the type and age of the building present and we would suggest that they are original and have not been underpinned.
Therefore, having been present for an extended period of time any post construction settlements of the foundations will have occurred and considering their dimensions and the bearing capacities of the CLAY (1) relative to the loads imposed by the existing structures, bearing capacity or settlement failure is unlikely to have or to occur as a result of direct loading and as concluded in 8.03 they are not vulnerable to existing and mature tree influence or seasonal influences.
However, the foundations are likely to be at or approaching their limit in terms of bearing capacity and as such unable to support any additional proposed or redistributed loading and they may be vulnerable to disturbance during construction.

8.05 Proposed Foundation and Floor Design

Considering the information available we would suggest that shallow strip foundations/underpinning are employed on site and the CLAY (1) is employed as the bearing stratum. Ideally, these foundations would be designed to match the existing to minimise differential movements and settlements, avoid disturbing and loading the existing foundations and there bearing stratum and reduce the need to underpin the existing foundations.
We would recommend that any strip foundations/underpinning are taken a minimum of 300mm into the CLAY (1) to depths in accordance with NHBC 4.2, with the design modified where necessary to found below any Fill, existing services, significant desiccation, live root activity and 'softening' encountered during excavation. The foundations should be founded at the same level or below those of the adjoining foundations and any services to be retained or suitable service ducts be installed and the foundations should be designed to minimally or not to undermine or surcharge the adjoining foundations or there bearing stratum. Should it not be possible to match the foundations it may be prudent to allow for some differential movements/settlements between the existing and proposed foundations. Heave precautions are not required and ground bearing floor slabs can be installed.

8.06 Bearing Capacity

For shallow strip foundations determination of bearing capacity in a cohesive soil can be based on the lowest unconsolidated undrained shear strength achieved in the bearing stratum from foundation depth to foundation depth plus the width of the foundation (B).
The shear strengths noted are approximate undrained unconsolidated tests of shear strength, relating to the short-term condition of clays under load i.e. initial stability of the foundation. Due to their inherent inaccuracy it would be prudent to employ a higher factor of safety of 3.0, which is conservative for a residential building with shallow strip foundations.
The net safe bearing capacities (net q_s) of the CLAY (1) in undrained condition and unaffected by groundwater for a strip foundation can be calculated from Net $q_s = c_uN_c/3$ where c_u= shear strength or apparent cohesion and N_c= 5.14, a bearing capacity factor attributed to Prandtl and Reissner.

8.07 Concrete Sulphate Attack

We would recommend that the site is considered to be in a natural location and that the groundwater is considered to be mobile.
Within the CLAY (1), pH>5.5[d] and DS-1 for soluble sulphate in 2:1 water/soil extract concentrations of <500mg/l are applicable and therefore, AC-1[d] can be employed on site in accordance with BRE Special Digest 1.

8.08 Excavations and Dewatering

The continuous strip underpinning suggested and excavations adjacent to the existing foundations will require some form of shoring as a safety measure.
Dewatering will be required for excavations adjacent and extending down to the existing foundations. Given the conclusions of 8.02 a simple sump and pump method may be adequate, but may take some time with the potential to destabilise adjacent basement ground bearing floor slabs seated on the Fill.

8.09 Further Works

Should you wish to confirm groundwater conditions, peizometers could be installed and monitored over an extended period of time. Given our conclusions we would suggest that this is unnecessary.
Should you wish to obtain more accurate measures of bearing capacity, we would suggest that a borehole is driven by cut down light percussion rig recovering undisturbed samples for triaxial testing. Given our conclusions we would suggest that this is unnecessary.
Should you wish to determine settlement potential, we would suggest that a borehole is driven by cut down light percussion rig recovering undisturbed samples for oedometer testing. Given our conclusions we would suggest that this is unnecessary.
Should you wish to determine whether dewatering is required and the most suitable form, the results of any peizometer and permeability testing could be utilised. Given our conclusions we would suggest that this is unnecessary.
Should you wish to determine whether shoring is required and the most suitable form, the results of any triaxial, bulk density, peizometer and permeability testing could be utilised. We would suggest that the underpinning contractor is consulted to determine whether this is necessary.
Should you consider that deep piled underpinning would be preferable to the shallow strip foundations/underpinning proposed, we would suggest that a 15m borehole is driven by cut down light percussion rig and the appropriate geotechnical testing carried out to allow pile design.

We hope that this report is sufficient for your requirements. Should you require any further information or works please do not hesitate to contact us.

Yours faithfully

N. J. Dunn
Contract Engineer

M. R. Smith M.Sc
Principal Engineer

HERTS & ESSEX SITE INVESTIGATIONS
The Old Post Office, Wellpond Green, Standon, Ware, Herts. SG11 2NJ
Telephone : Ware (01920) 822233
Fax : Ware (01920) 822200

Appendix No.	1
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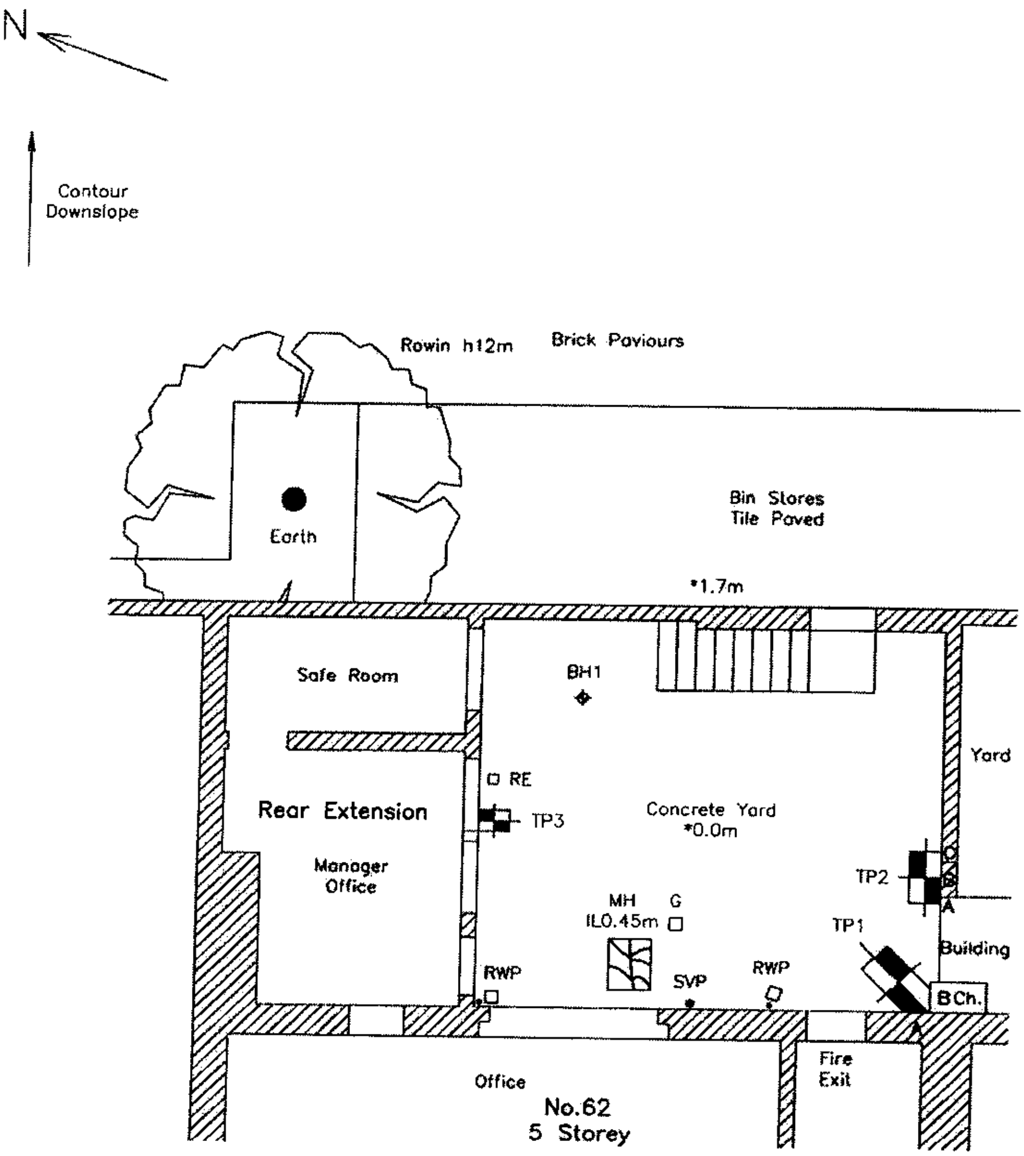
62 Marchmount Road, London. WC1N 1AB

Existing Site Plan



SCALE 1:100

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Appendix No.	1
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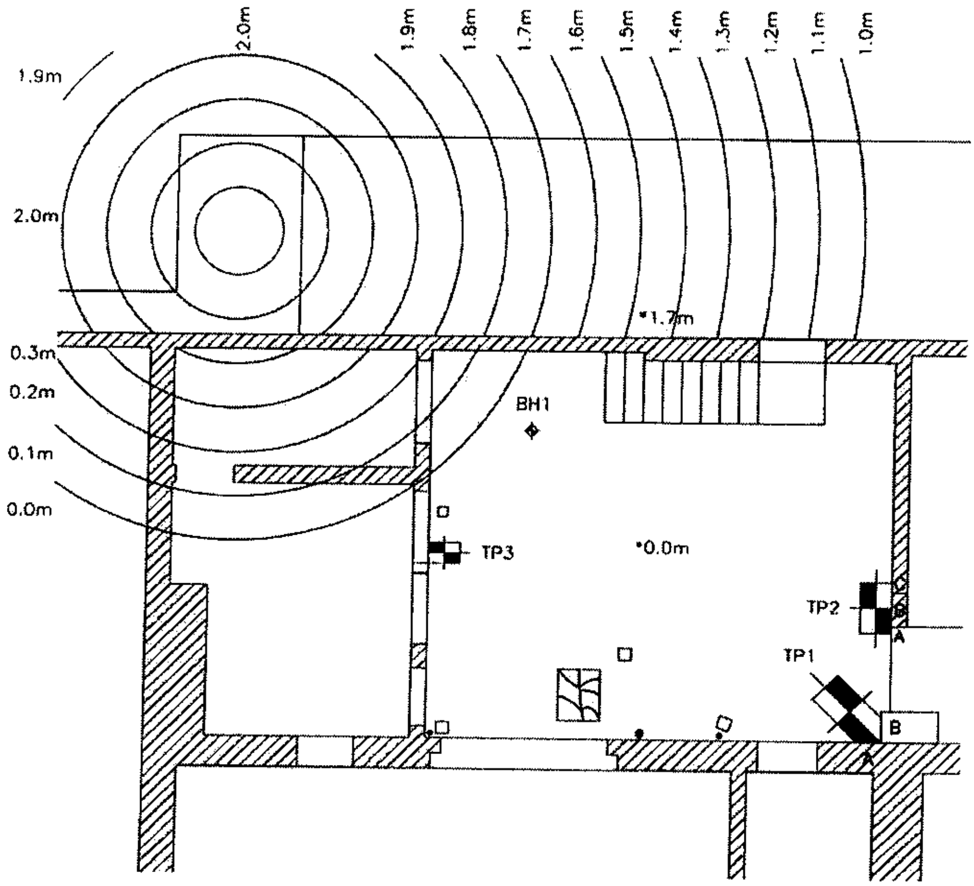
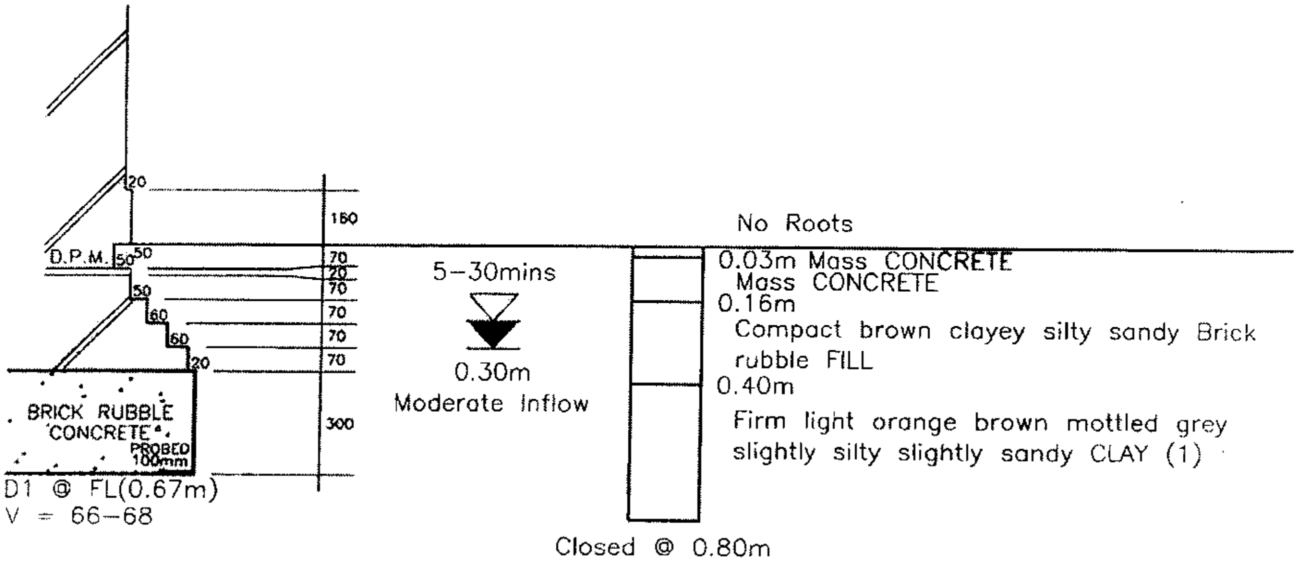
62 Marchmount Road, London. WC1N 1AB

Predicted Eisting and Mature Tree Influence



SCALE 1:100

Trial Pit OneA



NOTES

D = Disturbed Sample
V = Shear Strength kN/m²
▼ = Water Struck
▽ = Water Standing

Scale 1 : 20

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62 Marchmount Street, London. WC1N 1AB
Existing Foundation Details

Appendix No. 2
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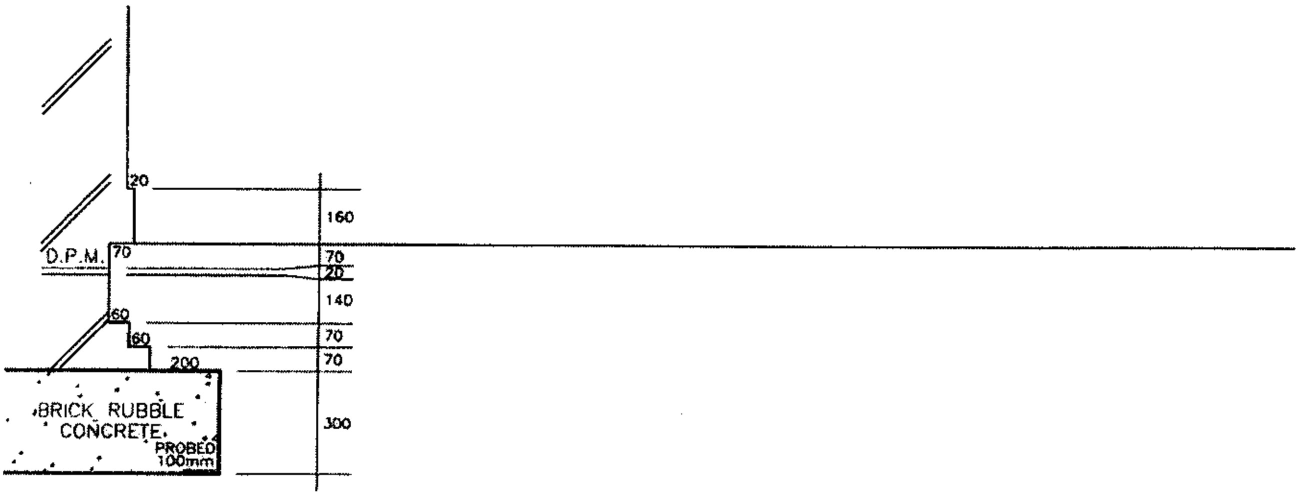
Trial Pit OneB



Scale 1 : 20

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62 Marchmount Street, London. WC1N 1AB
Existing Foundation Details

Appendix No. 2
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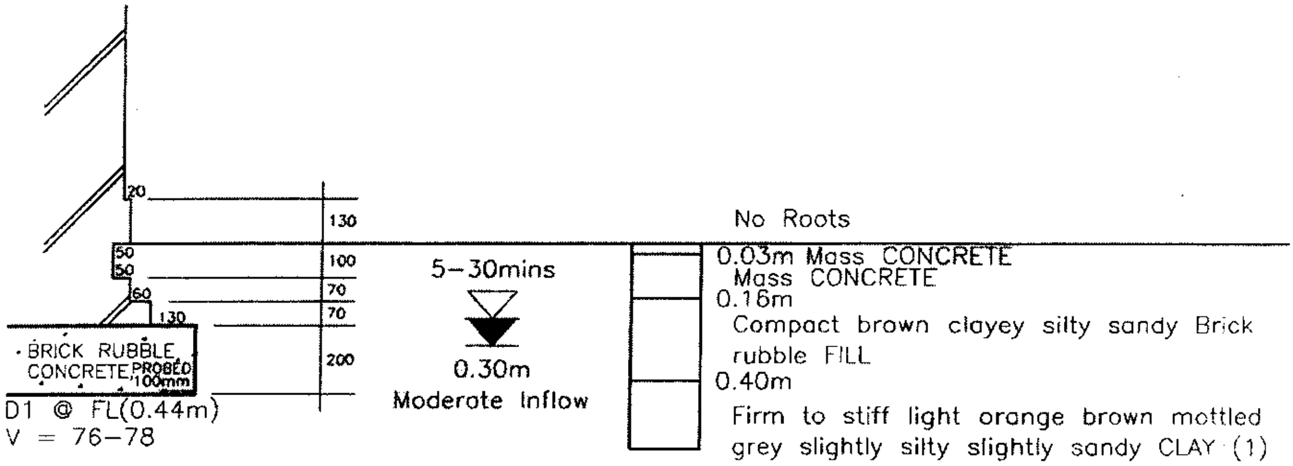
Trial Pit TwoA,B&C



Closed @ 0.60m

NOTES
D = Disturbed Sample
V = Shear Strength kN/m^2
▼ = Water Struck
▽ = Water Standing

Scale 1 : 20

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62 Marchmount Street, London. WC1N 1AB
Existing Foundation Details

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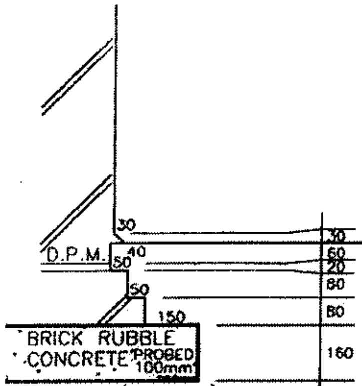
Trial Pit Three



D1 @ FL(0.40m)
V = 76–80

5–30mins
0.26m
Slight Inflow

No Roots

0.05m Mass CONCRETE
Compact brown clayey silty sandy Brick rubble FILL

0.40m
Firm to stiff light orange brown mottled grey slightly silty slightly sandy CLAY (1)

Closed @ 0.60m

Scale 1 : 20

NOTES
D = Disturbed Sample
V = Shear Strength kN/m^2
▼ = Water Struck
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Appendix No. 3
Sheet No. 1
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Date Apr 2007

Location : 62 Marchmount Road, London. WC1N 1AB

Moisture Content, Shear Strength, 'N'-value and Atterberg Limits test results with Ground, Groundwater, Root Activity and Driscoll's Method of Desiccation analysis profiles.

Borehole No.	Sample Depth (m)	Corrected Depth (m)	Sample	Natural Moisture Content (%)	Shear Strength (kN/m²)	N-Value	Ground Water (m)	Root Activity (m)	Stratum	Liquid Limit 0.4,0.5LL (%)	Plastic Limit PL+2% (%)	Plasticity Index (%)	Modified Plasticity Index (%)	Group Symbol	Desiccation Profile	Retention (%)
									0.40							
1	0.50		D1	28	78		No	No	1						Slight	
1	1.00		D2	29	80		No	No	1	27.6 69 34.5	23.0 21	48	48.0	CH	Slight	0
1	1.50		D3	30	84		No	No	1						Slight	
1	2.00		D4	31	102		No	No	1						Slight	
1	2.50		D5	31	124		No	No	1	30.8 77 38.5	27.0 25	52	52.0	CV	Slight	0
1	3.00		D6	30	128		No	No	1						Slight	
1	3.50		D7	29	126		No	No	1						Slight*	
1	4.00		D8	31	120		No	No	1						Slight*	
1	4.50		D9	30	116		No	No	1	30.8 77 34.5	27.0 25	52	52.0	CV	Slight*	0
1	5.00		D10	30	130		No	No	1 5.10						Slight*	
Trial Pit No.																
1	0.67		D1	31	66		0.30 No*	No	0.40 1 0.80						Slight*	
2	0.44		D1	30	76		0.30 No*	No	0.40 1 0.60						Slight*	
2	0.40		D1	29	78		0.26 No*	No	0.40 1 0.60						Slight*	

Remarks : No*-water probably not derived from groundwater.
Slight*-method of desiccation analysis not applicable below 3.00m. Guidance only.

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Appendix No. 3

Sheet No. 2

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Date Apr 2007

LOCATION 62 Marchmount Street, London. WC1N 1AB

SULPHATE ANALYSIS TEST RESULTS

Borehole No.	Depth (m)	Sample	Stratum	Concentrations of Soluble Sulphate			Design Sulphate Class	pH
				Soil		Groundwater		
				Total Potential SO_4 (%)	SO_4 in 2:1 water:soil (mg/l)			
1	1.00	D2	1		190		DS-1	8.0

Remarks :