Site Analytical Services Ltd.

Site Investigations, Analytical & Environmental Chemists, Laboratory Testing Services.

Units 14 + 15, River Road Business Park,
33 River Road, Barking, Essex IG11 0EA

Directors: J. S. Warren, M.R.S.C., P. C. Warren, J. I. Pattinson, BSc (Hons). MSc
Consultants: G. Evans, BSc., M.Sc., P.G. Dip., FGS., MIEnvSc. A. J. Kingston, BSc C.Eng. MIMM
F. J. Gibbs, F.I.B.M.S. F.I.F.S.T., F.R.S.H. K. J. Blanchette

Tel: 0208 594 8134
Fax: 0208 594 8072
E-Mail: services@siteanalytical.co.uk

Your Ref:

Our Ref: **Ref: 16/25552-2**
November 2016

81 AVENUE ROAD,

LONDON, NW8 6HR

BASEMENT IMPACT ASSESSMENT

Prepared for

Elliott Wood Partnership LLP

Acting on behalf of

Mr B K Mirchandani

CONTENTS

1.0 INTRODUCTION

1.1 Project Objectives

The purpose of this assessment is to consider the effects of a proposed basement construction on the local slope stability, surface water and groundwater regime at the residential property at 81 Avenue Road, London, NW8 6HR.

The recommendations and comments given in this report are based on the information contained from the sources cited and may include information provided by the Client and other parties, including anecdotal information. It must be noted that there may be special conditions prevailing at the site 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.

This report does not constitute a full environmental audit of either the site or its immediate environs.

1.2 Planning Policy Context

The information contained within this BIA has been produced to meet the requirements set out by Camden Planning Guidance – Basements and Lightwells (CPG4) including Camden Development Policies DP27 – Basements and Lightwells (Ref 1) in order to assist London Borough of Camden with their decision making process.

As recommended by the Guidance for Subterranean Development (Ref 1) the BIA comprises the following steps

1. **Initial screening** to identify where there are matters of concern

2. **Scoping** to further define the matters of concern

3. **Site Investigation and study** to establish baseline conditions

4. **Impact Assessment** to determine the impact of the basement on baseline conditions

5. **Review and Decision Making** (to be undertaken by LBC)

1.3 Qualifications

The report has been prepared by Mr Tom Murray, a Fellow of the Geological Society (FGS) and Mr Andrew Garnham, a Fellow of the Geological Society (FGS) in coordination with Mr Mike Brice of Applied Geotechnical Engineering, a Chartered Geologist (CGEOL), Neil Smith of Applied Geotechnical Engineering, a Chartered Civil Engineer (CEng), Mr Ed Davenport of Elliott Wood Partnership Ltd and Ms Sarah Wadley of Elliott Wood Partnership Ltd.

2.0 SITE DETAILS

(National Grid Reference: TQ 270 837)

2.1 Site Location

81 Avenue Road is a residential property, located on the western side of Avenue Road, Camden at approximate postcode NW8 6HR. The residential dwelling has two levels of accommodation; ground and first floor. The residential property comprises a front driveway, a single storey garage and a rear garden, which contains an outdoor swimming pool. The site covers an approximate area of 0.15 Hectares with the general area being under the authority of Camden Council.

The site is located on the western side of Avenue Road with residential properties to the north-west and south-east and roadways to the north-east, south-west.

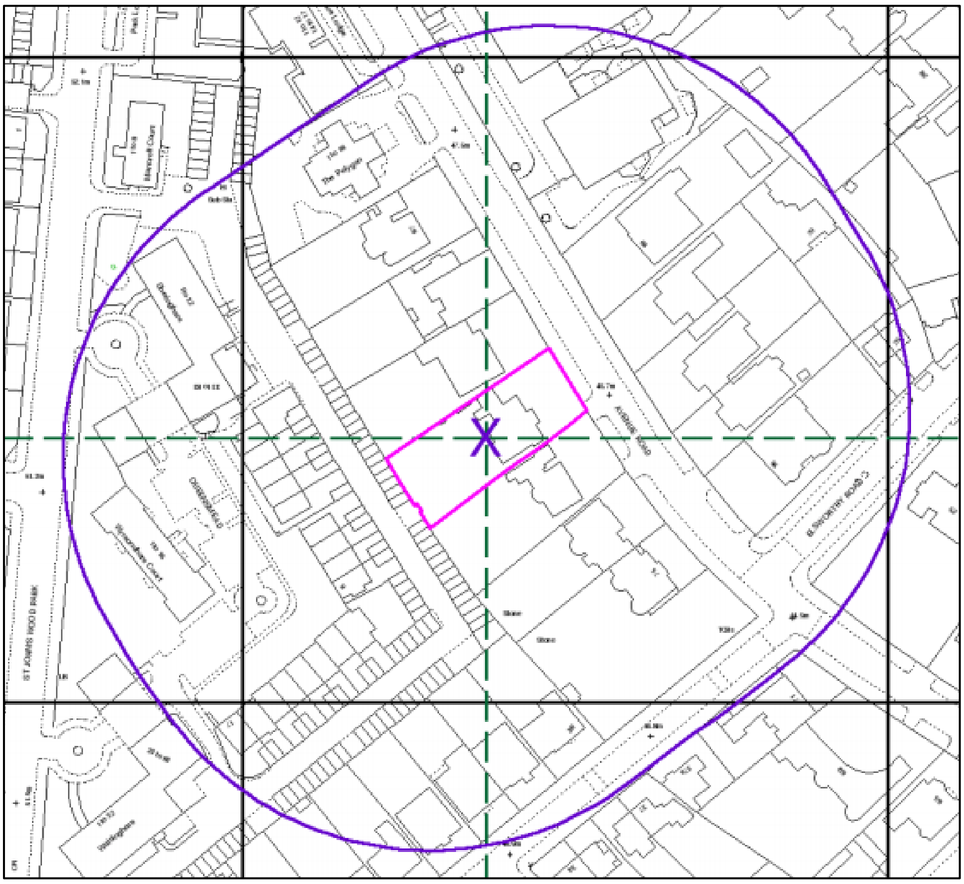

Figure 1. Site Location Plan

2.2 Site Layout and History

The site was attended on 16th August 2016 for the purposes of conducting the site walkover.

The site is accessed from Avenue Road and comprises a detached three storey residential property house with a driveway to the front and a large rear garden containing a swimming pool.

The front of the property is set mainly to paving slabs, with a small garden area between Avenue Road and the driveway.

The back garden consists of a large well-manicured lawn with flower beds and trees. There is also a large patio with a swimming pool to the rear of the house.

The existing ground level in the area of the site is believed to be broadly horizontal at an estimated level of approximately +46.4 mOD.

From the site walkover there were no obvious potentially contaminating activities on the site.

From historical map evidence it would appear that the site was first built on prior to 1871, with two periods of major changes taking place to the property between 1871-1896 and 1954-1960. The surrounding area has been residential throughout its history, although some industrial sites, including a garages and electrical sub-stations have been present within the area.

2.3 Geology

The 1:50000 Geological Survey of Great Britain (England and Wales) covering the area (Sheet 256, 'North London', Solid and Drift Edition) indicates the site to be underlain the London Clay Formation.

The British Geological Survey maintains an archive of historical exploratory borehole logs throughout the UK. SAS Limited has searched the database and have found that there are no boreholes located within 150m of the site.

- London Clay Formation: The London Clay Formation comprises clay, silt and sand and at this site location a thickness of between 70m and 100m is likely.

- Deeper strata is not of interest for this study.

Figure 2. Superficial and Bedrock geology of the Site (Ref. BGS Geoindex)

2.4 Hydrology and drainage

2.4.1 Rainfall and run-off

According to Mayes (1997) rainfall in the local area averages around 610mm and is significantly less than the national average of around 900mm.

Evapotranspiration is typically 450mm/year resulting in about 160mm/year as ‘hydrologically effective’ rainfall which is available to infiltrate into the ground or run-off as surface water flow.

According to publications regarding Lost Rivers of London (Barton, 1992) and (Talling, 2011), the site is within 100m of a former river or watercourse, which is a tributary to The Tyburn approximately 40m east of the site. The closest surface water feature is a drain located 395m north of the site.

Figure 3. Location of site relative to the 'Lost Rivers' of London (Source: Barton, 1992)

The area located immediately around the site is highly developed with more than 80% of the surface covered with hardstanding. Most of the rainfall in the area will run-off hard surface areas and be collected by the local sewer network.

2.4.2 Drainage

Surface drainage from the site is assumed to be directed to drains flowing downhill north-west to south-east along Avenue Road.

2.4.3 Flood Risk

River or Tidal flooding

The site is currently not located within 1 kilometre of an area at risk from extreme flooding from rivers or sea without defences (Zone 2) or an area at risk from rivers or sea without defences (Zone 3). The EA's website also shows that this area does not fall within an area at risk of flooding from reservoirs.

Surface water flooding

According to Environment Agency Surface Water Flood maps of the area the site is at a low risk from surface water flooding, but there is an area of high risk approximate 50m north of the site.

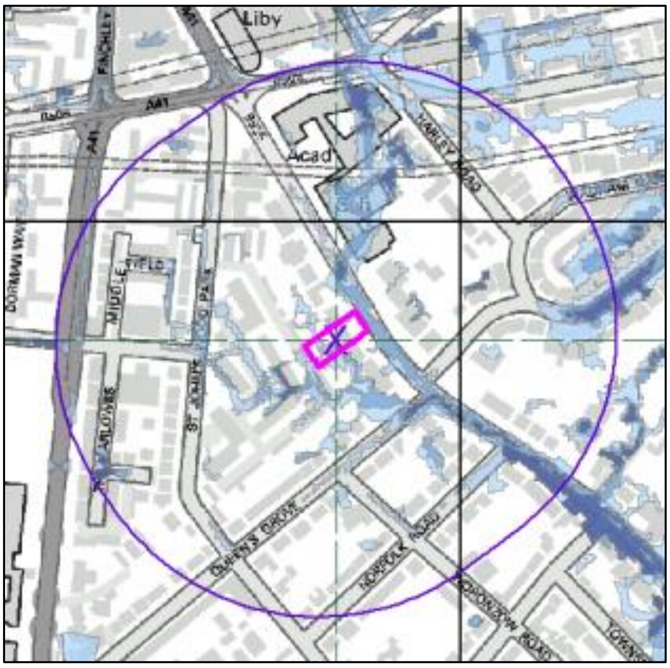

Figure 4. Extract from the Environment Agency's 'Risk of Flooding from Surface Water'. Ordnance Survey Crown copyright 2015. All rights reserved.

Sewer flooding

The London Regional Flood Risk Appraisal (2009) advises that foul sewer flooding is most likely to occur where properties are connected to the sewer system at a level below the hydraulic level of the sewage flow, which in general are often basement flats or premises in low lying areas. There is no record of sewer flooding having occurred at 81 Avenue Road and therefore the risk of sewer flooding is considered low.

2.5 Hydrogeology

The Environment Agency Groundwater Protection Policy uses aquifer designations that are consistent with the Water Framework Directive. These designations reflect the importance of aquifers in terms of groundwater as a resource (drinking water supply) and also their role in supporting surface water flows and wetland ecosystems.

The Bedrock geology underlying the site (London Clay) has been classified as Unproductive Strata; rock layers or drift deposits with low permeability that have negligible significance for water supply or river base flow.

Groundwater levels within London Clay and across the site have been monitored as part of this study and the results are described in Section 4.0 below.

Other hydrogeological data obtained from the Phase 1 Preliminary Risk Assessment (PRA) (SAS Report Ref: 16/25552) for the site include:

- The underlying soil classification of the site is of high leaching potential.
- A Zone II (Outer Protection Zone) is present on-site.
- There are 4 non potable water abstraction licences within 1 kilometre of the site.
- There are 3 potable water abstraction licences within 1 kilometre of the site.
- The closest is located 381m to the north of the site with the abstraction of water for Municipal Grounds: Spray Irrigation - Direct. The permitted start date for this licence is 5th December 2013.

2.6 Previous Reports

The results from a Phase 1 Preliminary Risk Assessment and Phase 2 Intrusive Investigation are presented under separate cover in Site Analytical Services Limited reports (Project No’s. 16/25552 & 16/25552-1) dated September 2016. The findings from these reports are described in this basement impact assessment.

2.7 Proposed Development

At the time of reporting of September 2016, it is proposed to demolish the existing property and construct a new three storey dwelling with a single storey basement beneath the footprint of the property extending into the front and rear garden.

The proposed basement dig level is understood to be up to 6.95m below the garden level (39.45mOD).

2.8 Results of Basement Impact Assessment Screening

A screening process has been undertaken for the site and the results are summarised in Table 1 below:

Table 1: Summary of screening results

Item	Description	Response	Comment
Sub-terranean (Ground water Flow)	1a. Is the site located directly above an aquifer.	No	The site has been classified as being situated above an unproductive (negligibly permeable) formation (London Clay) that is generally regarded as containing insignificant quantities of groundwater.
	1b. Will the proposed basement extend beneath the water table surface.	Unknown – to be confirmed by Ground Investigation	Given the presence of a non-aquifer below the site it is unlikely that groundwater will be encountered during any excavations for the proposed basement, however this will be confirmed by the ground investigation.
	2. Is the site within 100m of a watercourse, well (used / disused) or potential spring line.	Yes	According to publications regarding Lost Rivers of London (Barton, 1992) and (Talling, 2011), the site is within 100m of a former river or watercourse, which is a tributary to The Tyburn approximately 40m east of the site. The closest surface water feature is a drain located 395m north of the site. From the British Geological Society 'Geoindex' the nearest water well is located approximately 455m east of the site.
	3. Is the site within the catchment of the ponds chains on Hampstead Heath	No	With reference to the Camden Geological, Hydrogeological and Hydrological Study, the site is not within the catchment of the pond chains on Hampstead, nor the Golder's Hill Chain.
	4. Will the proposed basement development result in a change in the proportion of hard surfaced / paved areas.	No	The amount of hardstanding on-site is not expected to change.
	5. As part of site drainage, will more surface water (e.g. rainfall and run-off) than at present be discharged to the ground (e.g. via soakaways and/or SUDS).	No	Existing drainage paths are to be utilised where possible. Whether soakaways/SUDS are used on the proposed development is to be confirmed (beyond the scope of this report). An appropriately qualified engineer should be engaged to ensure mandatory requirements are met.

Item	Description	Response	Comment
	6. Is the lowest point of the proposed excavation (allowing for any drainage and foundation space under the basement floor) close to, or lower than, the mean water level in any local pond or spring line.	Yes	According to publications regarding Lost Rivers of London (Barton, 1992) and (Talling, 2011), the site is within 100m of a former river or watercourse, which is a tributary to The Tyburn approximately 40m east of the site. The closest surface water feature is a drain located 395m north of the site. From the British Geological Society 'Geoindex' the nearest water well is located approximately 455m east of the site.
Slope Stability	1. Does the existing site include slopes, natural or man-made greater than 7 degrees (approximately 1 in 8).	No	The site is essentially flat.
	2. Will the proposed re-profiling of landscaping at the site change slopes at the property boundary to more than 7 degrees (approximately 1 in 8).	No	Re-profiling of landscaping at the site is not proposed.
	3. Does the development neighbour land, including railway cuttings and the like, with a slope greater than 7 degrees (approximately 1 in 8).	No	The surrounding area drops to the south-east, but from survey information and with reference to Figure 16 from Camden CPG 4, this is at angles of less than 7 degrees.
	4. Is the site within a wider hillside setting in which the general slope is greater than 7 degrees (approximately 1 in 8).	No	There is a general slope across the surrounding area from north-west to south-east towards the Thames Basin, but with reference to Figure 16 from Camden CPG 4, this is at angles of less than 7 degrees.
	5. Is the London Clay the shallowest strata at the site.	Yes	With reference to available BGS records, the London Clay Formation is expected to be encountered from ground level.
	6. Will any trees be felled as part of the development and/or are any works proposed within any tree protection zones where trees are to be retained.	No	It is understood that no trees are to be felled as part of the development.
	7. Is there a history of seasonal shrink-swell subsidence in the local area and/or evidence of such effects at the site.	Yes	The site lies above the London Clay Formation well known as having a high tendency to shrink and swell.
	8. Is the site within 100m of a watercourse or a potential spring line.	Yes	According to publications regarding Lost Rivers of London (Barton, 1992) and (Talling, 2011), the site is within 100m of a former river or watercourse, which is a tributary to The Tyburn approximately 40m east of the site. The closest surface water feature is a drain located 395m north of the site. From the British Geological Society 'Geoindex' the nearest water well is

Item	Description	Response	Comment
			located approximately 455m east of the site.
	9. Is the site within an area of previously worked ground.	No	According to the records held by the BGS the site is not underlain by any worked ground, made ground, infilled ground or landscaped ground
	10. Is the site within an aquifer. If so, will the proposed basement extend beneath the water table such that dewatering may be required during construction.	No	The site has been classified as being situated above an unproductive (negligibly permeable) formation (London Clay) that is generally regarded as containing insignificant quantities of groundwater.
	11. Is the site within 50m of the Hampstead Heath Ponds	No	With reference to the Camden Geological, Hydrogeological and Hydrological Study, the site is not within the catchment of the pond chains on Hampstead, nor the Golder's Hill Chain.
	12. Is the site within 5m of a highway or pedestrian right of way.	Yes	The site lies within 5m of Avenue Road.
	13. Will the proposed basement significantly increase the differential depth of foundations relative to neighbouring properties.	Unknown	It is not known whether the neighbouring buildings have basements, though it is noted there is a passageway to the left of No 83 at a level of 44.3mOD, approximately 2.1m below general ground level suggesting a sub-structure of some form is present beneath this house. It will be assumed that a basement is not present beneath No 79, this is potentially conservative.
	14. Is the site over (or within the exclusion zone of) any tunnels, e.g. railway lines.	Unknown / outside scope of report	A full statutory service search was outside the scope of this report and must be completed prior to any excavations.
Surface Water and Flooding	1. Is the site within the catchment of the ponds chains on Hampstead Heath	No	With reference to the Camden Geological, Hydrogeological and Hydrological Study, the site is not within the catchment of the pond chains on Hampstead, nor the Golder's Hill Chain.
	2. As part of the proposed site drainage, will surface water flows (e.g. volume of rainfall and peak run-off) be materially changed from the existing route.	No	The amount of hardstanding on-site is not changing therefore surface water will not be impacted by the development.
	3. Will the proposed basement development result in a change in the proportion of hard surfaced / paved external areas.	No	The amount of hardstanding on-site is not expected to increase.

Item	Description	Response	Comment
	4. Will the proposed basement result in changes to the profile of the inflows (instantaneous and long-term) of surface water being received by adjacent properties or downstream watercourses.	No	As no changes are occurring above the ground, surface water will not be impacted by the development.
	5. Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream watercourses.	No	As no changes are occurring above the ground at the location of the basement, surface water will not be impacted by the development.
	6. Is the site in an area known to be at risk from surface water flooding, such as South Hampstead, West Hampstead, Gospel Oak and King's Cross, or is it at risk from flooding, for example because the proposed basement is below the static water level of a nearby surface water feature	Yes	Avenue Road flooded during the 2002 flood event. According to modelling by the Environment Agency, there is a 'Low' risk of surface water flooding for No.81 and the surrounding area.

The Screening Exercise has identified the following potential issues which will be carried forward to the Scoping Phase

Subterranean Groundwater Flow

- Will the proposed basement extend beneath the water table surface?
- Is the site within 100m of a watercourse, well (used / disused) or potential spring line?
- Is the lowest point of the proposed excavation (allowing for any drainage and foundation space under the basement floor) close to, or lower than, the mean water level in any local pond or spring line?

Slope Stability

- Is the London Clay the shallowest strata at the site?
- Is there a history of seasonal shrink-swell subsidence in the local area and/or evidence of such effects at the site?
- Is the site within 100m of a watercourse or a potential spring line?
- Is the site within 5m of a highway or pedestrian right of way?
- Will the proposed basement significantly increase the differential depth of foundations relative to neighbouring properties?

Surface water and flooding

- Is the site in an area known to be at risk from surface water flooding?

3.0 SCOPING PHASE

This purpose of the scoping phase is to assess in more detail the factors to be investigated in the impact assessment. Potential impacts are assessed for each of the identified impact factors and recommendations are stated.

A conceptual ground model is usually complied at the scoping stage however, because the ground investigation has already been undertaken for this project, the conceptual ground model including the findings of the ground investigation is described under Chapter 4.

Subterranean (Groundwater Flow)

Potential Issue (Screening Question)		Potential impacts and actions
1b	Will the proposed basement extend beneath the water table surface?	**Potential impact:** Local restriction of groundwater flows (perched groundwater or below groundwater table). **Action:** Ground investigation required, the review.
2	Is the site within 100m of a watercourse, well (used / disused) or potential spring line?	**Potential impact:** The flow from a spring, well or watercourse may increase or decrease if the groundwater flow regime is affected by the proposed basement **Action:** Review hydrogeology of the site and undertake a ground investigation.

Slope Stability

Potential Issue (Screening Question)		Potential impacts and actions
5	Is the London Clay the shallowest strata at the site?	**Potential impact:** The London Clay is prone to seasonal shrink-swell (subsidence and heave). **Action:** Ground investigation required, the review.
7	Is there a history of seasonal shrink-swell subsidence in the local area and/or evidence of such effects at the site?	**Potential Impact:** Ground movements will occur during and after the basement construction. **Action:** Ground investigation required, then review.
8	Is the site within 100m of a watercourse or a potential spring line?	**Potential impact:** The flow from a spring, well or watercourse may increase or decrease if the groundwater flow regime is affected by the proposed basement **Action:** Review hydrogeology of the site and undertake a ground investigation.
11	Is the site within 5m of a highway or a pedestrian right of way?	**Potential impact:** Excavation of basement causes loss of support to footway/highway and damage to the services beneath them. **Action:** Ensure adequate temporary and permanent support by use of best practice working methods.

Potential Issue (Screening Question)		Potential impacts and actions
12	Will the proposed basement substantially increase the differential depth of foundations relative to neighbouring properties?	**Potential impact:** Loss of support to the ground beneath the new foundations to neighbouring properties if basement excavations are inadequately supported. **Action:** Ensure adequate temporary and permanent support by use of best practice methods.

Subterranean (Surface Water Flooding)

Potential Issue (Screening Question)		Potential impacts and actions
5	Is the site in an area known to be at risk from surface water flooding?	**Potential impact**: Flooding occurs during the excavation of the basement get flooded following construction **Action**: A groundwater exception test should be carried out prior to any construction works.

These potential impacts have been further assessed through the ground investigation, as detailed in Section 4 below.

4.0 EXISTING SITE INVESTIGATION DATA

4.1 Records of site investigations

Ground conditions at the site were investigated by Site Analytical Services Limited in August and September 2016 (Report Reference 16/25552-1). The ground conditions revealed by the investigation are summarised in the following table.

Strata	Depth to top of strata (mbgl)	Level to top of strata (mOD)	Depth to base of strata (mbgl)	Level to base of strata (mOD)	Description
Made Ground	0.00	46.43 to 46.35	0.60 to 1.60	45.83 to 44.80	Stone slabs and concrete over silty sandy clay containing brick and concrete fragments.
Superficial Head Deposits	0.60 to 1.60	45.83 to 44.80	3.20 to 3.50	43.20 to 42.93	Firm silty sandy gravelly clay.
London Clay Formation	3.20 to 3.50	43.20 to 42.93	15.00 (base of boreholes)	31.43 to 31.35	Stiff becoming very stiff silty sandy clay with gypsum crystals.

Table A: Summary of Ground Conditions in Exploratory Holes

Groundwater was not encountered in the boreholes and the soils remained essentially dry throughout.

It must be noted that the speed of excavation is such that there may well be insufficient time for further light seepages of groundwater to enter the borehole and hence be detected, particularly within more cohesive soils.

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

Groundwater was not subsequently encountered within the monitoring standpipes within Boreholes 1 and 2, but was encountered at a depth of 2.30mbgl (44.13mOD) within the standpipe in BH3 after a period of approximately three weeks.

It should be noted that the comments on groundwater conditions are based on observations made at the time of the investigation (July and August 2016) and that changes in the groundwater level could occur due to seasonal effects and also changes in drainage conditions.

5.0 FOUNDATION DESIGN

5.1 General

At the time of reporting, September 2016, it is proposed to demolish the existing property and construct a new three storey dwelling with a single storey basement beneath the footprint of the property extending into the front and rear garden.

The proposed basement dig level is understood to be up to 6.95m below the garden level (39.45mOD).

5.2 Site Preparation Works

The main contractor should be informed of the site conditions and risk assessments should be undertaken to comply with the Construction Design Management (CDM) regulations. Site personnel are to be made aware of the site conditions. It is recommended that extensive searches of existing man-made services are undertaken over the site prior to final design works.

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 stratum of adequate bearing characteristics.

Based on the ground and groundwater conditions encountered in the boreholes it should be possible to support the proposed new development on conventional strip or basement raft foundations taken down below the Made Ground and any weak superficial soils and placed in the natural firm sandy silty clay deposits which occur at depths of between approximately 3.20m (43.20mOD) and 3.50m (42.93mOD) below ground level over the site. Foundations should be placed in the natural deposits at a minimum depth of 1.00m below final ground level in order to avoid the zone affected by seasonal moisture content changes.

Using theory from Terzaghi (1943), for the proposed depth of the basement, then strip foundations placed within natural soils may be designed to allowable net bearing pressures of approximately 295kN/m^2 at 5.00m depth increasing to 320kN/m^2 at 7.00m depth in order to allow for a factor of safety of 2.5 against general shear failure. The actual allowable bearing pressure applicable will depend on the form of foundation, its geometry and depth in accordance with classical analytical methods, details of which can be obtained from "Foundation Design and Construction", Seventh Edition, 2001 by M J Tomlinson (see references) or similar texts.

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 both 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, then 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

Several methods of retaining wall construction could be considered. These may include retaining structures cast in an underpinning sequence, or the use of temporary or sacrificial works to facilitate the retaining structure's construction. The excavation of the basement must not compromise the integrity of adjacent structures.

The full design of temporary and permanent retaining structures is beyond the scope of this report. However, the following design parameters for each element of soil recorded in the relevant exploratory holes are provided in Table B below to assist the design of these structures.

Stratum	**Depth to top (m)**	**Bulk Density (Mg/m3) (ɣ)**	**Effective Angle of Internal Friction (Φ)**
Superficial Head Deposits	0.60 to 1.60	2.00	28
London Clay Formation	3.20 to 3.50	2.00	21

Table B. Retaining Wall Design Parameters

The designer should use these parameters to derive the active and passive earth pressure coefficients ka and kp. The determination of appropriate earth pressure coefficients, together with factors such as the pattern of the earth pressure distribution, will depend upon the type/geometry of the wall and overall design factors.

5.6 Chemical Attack on Buried Concrete

The results show the soil samples tested to have water soluble sulphate contents up to 2.39g/litre associated with near neutral pH values.

In these conditions, it is considered that deterioration of buried concrete due to sulphate or acid attack is likely to occur. 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 Formation. 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 concrete in accordance with full Class DS-3 conditions.

6.0 BASEMENT IMPACT ASSESSMENT

6.1 Summary

The screening identified a number of potential impacts. The table below summarises the previously identified potential impacts and the additional information that is now available from the site investigation in consideration of each impact.

Potential Impact	Site Investigation conclusions	Impact sufficiently addressed without further justification?
The proposed basement extends beneath the water table surface.	Groundwater was not subsequently encountered within the monitoring standpipes within Boreholes 1 and 2, but was encountered at a depth of 2.30mbgl (44.13mOD) within the standpipe in BH3 after a period of approximately three weeks. It is likely that the water encountered within the standpipe of BH3 is not representative of the true groundwater level and is likely caused by perched water from the Made Ground or surface water infiltration.	Yes
The site is within 100m of a watercourse, well (used / disused) or potential spring line	The site lies within 40m of the one of the tributaries of the former River Tyburn.	No – see below for further details.
The lowest point of the proposed excavation is close to, or lower than, the mean water level in any local pond or spring line		
There a history of seasonal shrink-swell subsidence in the local area and/or evidence of such effects at the site.	The London Clay was proven below the site and was recorded as having a high susceptibility to shrinkage and shrinkage. However, the base of proposed basement will extend well below the potential depth of root action.	Yes
The site is within 5m of a highway or pedestrian right of way.	The proposed basement is not to be extended below Avenue Road and therefore it is suggested that the impact on these access roads is likely to be minimal. There is nothing unusual in the proposed development that would give rise to any concerns with regard to the stability of public highways.	Yes.
The proposed basement will significantly increase the differential depth of foundations relative to neighbouring properties.	The development will result in the extension of the foundation depth of the basement relative to neighbouring properties.	No – see below for further details.
The site is in an area known to be at risk from surface water flooding.	There is a potential risk of surface water following the construction.	No – see below for further details.

6.2 Outstanding Risks and Issues

The significant impacts which require further information have been described in detailed below in order to assess the likelihood of them occurring and the scope for reasonable engineering mitigation.

a. The site is within 100m of a watercourse, well (used / disused) or potential spring line & the lowest point of the proposed excavation is close to, or lower than, the mean water level in any local pond or spring line

As noted, there are no watercourses in the vicinity of the site.

The site is within a densely developed urban area, with a number of barriers to overland flow created by the existing residential development (i.e. the building footprint and the walls around the perimeter of the site).

Current information suggests that Fitzjohns Avenue marks the route of the River Tyburn, a former watercourse that has become lost through culverting and urban development of the catchment.

Assuming the watercourse exists in the area within a culverted section, this would flow southwards following the slope along Fitzjohns Avenue towards the River Thames. In an extreme flood event, the highway provides an open - and largely unobstructed - flow route.

The proposed basement development is located to the rear (west) side of the existing property and would be outside the extent of any such flow route. As such, no overland pathways to or from this feature exist across the site.

b. The proposed basement will significantly increase the differential depth of foundations relative to neighbouring properties.

The excavation and construction of the basement at the site has the potential to cause some movements in the surrounding ground if not properly managed. However, it is understood that ground movements and/or instability will be managed through the proper design and construction of mitigation measures during the works. This will require close collaboration with the appointed contractor's temporary works coordinator.

The Party Wall Act (1996) will apply to this development because neighbouring houses lie within a defined space around the proposed building works. The party wall process should be followed and adhered to during this development.

A ground movement assessment was carried out at the site by Applied Geotechnical Engineering under the instruction of Site Analytical Services Limited (Report Reference P4143). The report is provided as Appendix B to this report and concludes that given good workmanship, the basement to No 81 Avenue Road may be constructed without imposing more than 'very slight' damage on the adjoining properties.

A monitoring plan should be set out at design stage and should include a monitoring strategy, instrumentation and monitoring plans and action plans. Trigger levels on movements will need to be defined. Precise levelling or reflective survey targets should be installed at the garden walls and neighbouring buildings. Monitoring should take place in advance of the proposed works as a base-line survey, during the works and for a period following the completion of the works, to understand the long term effects.

c. The site is in an area known to be at risk from surface water flooding.

Although the modelling of the site by the Environment Agency shows a 'Low' risk of flooding for No. 81, Avenue Road was flooded during the 2002 event which indicates Avenue Road as a hotspot zone for surface water flooding.

In applying the Exception Test and assessing the risk associated with surface water and sewer flooding the following is considered:

- The proposed basement construction does not change the impermeable proportion at the site (this remains essentially the same). As such, the basement will not have an adverse impact on the site's surface water run-off.

- At the time of writing this report, the drainage details had not been finalised; however it is our understanding that the drainage details will incorporate a pumping device to protect the property from sewer flooding.

The proposed development will not increase flood risk at the site or the surrounding area. Also since the development is on already developed land, it will not adversely impact the Council's sustainability objectives.

7.0 BIA CONCLUSIONS

1. At the time of reporting of September 2016, it is proposed to demolish the existing property and construct a new 3 storey dwelling with a single storey basement beneath the footprint of the property extending into the front and rear garden.

   The proposed basement dig level is understood to be up to 6.95m below the garden level (39.45mOD).

2. Conditions at the site were investigated by Site Analytical Services Limited in July and August 2016 (SAS Report Reference 16/25552-1). The boreholes revealed ground conditions that were generally consistent with the geological records and known history of the area and comprised up to 1.60m thickness of Made Ground, underlain by superficial Head Deposits with the London Clay Formation at depth.

3. As proven from the site investigation, the Bedrock geology underlying the site (solid permeable formations) has been classified as Unproductive Strata; rock layers or drift deposits with low permeability that have negligible significance for water supply or river base flow.

4. Groundwater levels in the immediate vicinity of the property have been recorded below the level of the proposed basement and therefore the impact on the groundwater is likely to be minimal.

5. A monitoring plan will be set out at design stage and will include a monitoring strategy, instrumentation and monitoring plans and action plans.

6. The excavation and construction of the basement at the site has the potential to cause some movements in the surrounding ground if not properly managed. However, it is understood that ground movements and/or instability will be managed through the proper design and construction of mitigation measures during the works.

p.p. SITE ANALYTICAL SERVICES LIMITED

A Garnham BSc (Hons) MSc FGS
Senior Engineer

T P Murray MSc BSc (Hons) FGS
Geotechnical Engineer

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9.0 APPENDIX A – GROUND INVESTIGATION FACTUAL REPORT

Site Analytical Services Ltd.

Site Investigations, Analytical & Environmental Chemists, Laboratory Testing Services.

Units 14 + 15, River Road Business Park,
33 River Road, Barking, Essex IG11 0EA

Directors: J. S. Warren, M.R.S.C., P. C. Warren, J. I. Pattinson, BSc (Hons). MSc
Consultants: G. Evans, BSc., M.Sc., P.G. Dip., FGS., MIEnvSc. A. J. Kingston, BSc C.Eng. MIMM
F. J. Gibbs, F.I.B.M.S. F.I.F.S.T., F.R.S.H. K. J. Blanchette

Tel: 0208 594 8134
Fax: 0208 594 8072
E-Mail: services@siteanalytical.co.uk

Your Ref: Our Ref: **Ref: 16/25552-1**
September 2016

81 AVENUE ROAD,

LONDON, NW8 6HR

FACTUAL REPORT ON A GROUND INVESTIGATION

Prepared for

Elliott Wood Partnership LLP

Acting on behalf of

Mr B K Mirchandani

CONTENTS

1.0 INTRODUCTION

1.1 Outline and Limitations of Report

At the request of Mr B. K. Mirchandani, a ground investigation was carried out in connection with a proposed residential basement development at the above site. A Phase 1 Preliminary Risk Assessment (Desk Study) is presented under separate cover in Site Analytical Services Limited Report Reference 16/25552.

The information was required for the design and construction of foundations and infrastructure for the proposed development at the existing site which includes demolition of the existing property and construction a new 3 storey dwelling with a single storey basement beneath the footprint of the property extending into the front and rear garden.

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 SITE DETAILS

(National Grid Reference: TQ 270 837)

2.1 Site Location

81 Avenue Road is a residential property, located on the western side of Avenue Road, Camden at approximate postcode NW8 6HR. The residential dwelling has two levels of accommodation; ground and first floor. The residential property comprises a front driveway, a single storey garage and a rear garden, which contains an outdoor swimming pool. The site covers an approximate area of 0.15 Hectares with the general area being under the authority of the Camden Council.

The site is located on the western side of Avenue Road with residential properties to the north-west and south-east and roadways to the north-east, south-west.

2.2 Geology

The 1:50000 Geological Survey of Great Britain (England and Wales) covering the area (Sheet 256, 'North London', Solid and Drift Edition) indicates the site to be underlain the London Clay Formation at depth.

2.3 Previous Investigations

A Phase 1 Preliminary Risk Assessment (PRA) (SAS Report Ref: 16/25552 dated September 2016) has been undertaken across the site by Site Analytical Services Limited.

3.0 SCOPE OF WORK

3.1 Site Works

The proposed scope of works was agreed by the client prior to the commencement of the investigations. To achieve this, the following works were undertaken:-

- The drilling of two rotary percussive boreholes to 15.00m below ground level (Boreholes 1 and 2).
- The drilling of one continuous flight auger borehole to 15.00m below ground level (Borehole 3).
- Sampling and in-situ testing as appropriate to the ground conditions encountered in the boreholes.
- Laboratory testing to determine the engineering properties of the soils encountered in the exploratory holes.
- Factual reporting on the results of the investigation.

3.2 Ground Conditions

The locations of the exploratory holes are shown on the site sketch plan, Figure 1.

The boreholes revealed ground conditions that were consistent with the geological records and known history of the area and comprised Made Ground up to 1.60m in thickness resting on superficial head deposits with the London Clay Formation at depth.

These ground conditions are summarised in the following table. For detailed information on the ground conditions encountered in the boreholes, reference should be made to the exploratory hole records presented in Appendix A.

Strata	Depth to top of strata (mbgl)	Level to top of strata (mOD)	Depth to base of strata (mbgl)	Level to base of strata (mOD)	Description
Made Ground	0.00	46.43 to 46.35	0.60 to 1.60	45.83 to 44.80	Stone slabs and concrete over silty sandy clay containing brick and concrete fragments.
Superficial Head Deposits	0.60 to 1.60	45.83 to 44.80	3.20 to 3.50	43.20 to 42.93	Firm silty sandy gravelly clay.
London Clay Formation	3.20 to 3.50	43.20 to 42.93	15.00 (base of boreholes)	31.43 to 31.35	Stiff becoming very stiff silty sandy clay with gypsum crystals.

Table A: Summary of Ground Conditions in Exploratory Holes

3.3 Groundwater

Groundwater was not encountered in the boreholes and the soils remained essentially dry throughout.

It must be noted that the speed of excavation is such that there may well be insufficient time for further light seepages of groundwater to enter the borehole and hence be detected, particularly within more cohesive soils.

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

Groundwater was not subsequently encountered within the monitoring standpipes within Boreholes 1 and 2, but was encountered at a depth of 2.30mbgl (44.13mOD) within the standpipe in BH3 after a period of approximately three weeks.

It should be noted that the comments on groundwater conditions are based on observations made at the time of the investigation (July and August 2016) and that changes in the groundwater level could occur due to seasonal effects and also changes in drainage conditions.

4.0 IN-SITU TESTING AND LABORATORY TESTS

4.1 Standard Penetration Tests

The results of the Standard Penetration Tests carried out in the natural soils are shown on the exploratory hole records in Appendix A.

4.2 Undrained Triaxial Compression Test Results

Undrained Triaxial Compression tests were carried out on eight undisturbed 100mm diameter samples taken from within Boreholes 1 and 2.

The results of the tests are given within Table 1, contained in Appendix B

4.3 Hand Vane Tests

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 high strength in accordance with BS 5930 (2015).

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

4.4 Classification Tests

Atterberg Limit tests were conducted on four samples taken at depth in Boreholes 1, 2 and 3 and showed the samples tested to fall into Classes CI and CH according to the British Soil Classification System.

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

4.5 Sulphate and pH Analyses

The results of the sulphate and pH analyses made on four soil samples are presented on Table 3, contained in Appendix B.

p.p. SITE ANALYTICAL SERVICES LIMITED

T P Murray MSc BSc (Hons) FGS
Geotechnical Engineer

5.0 REFERENCES

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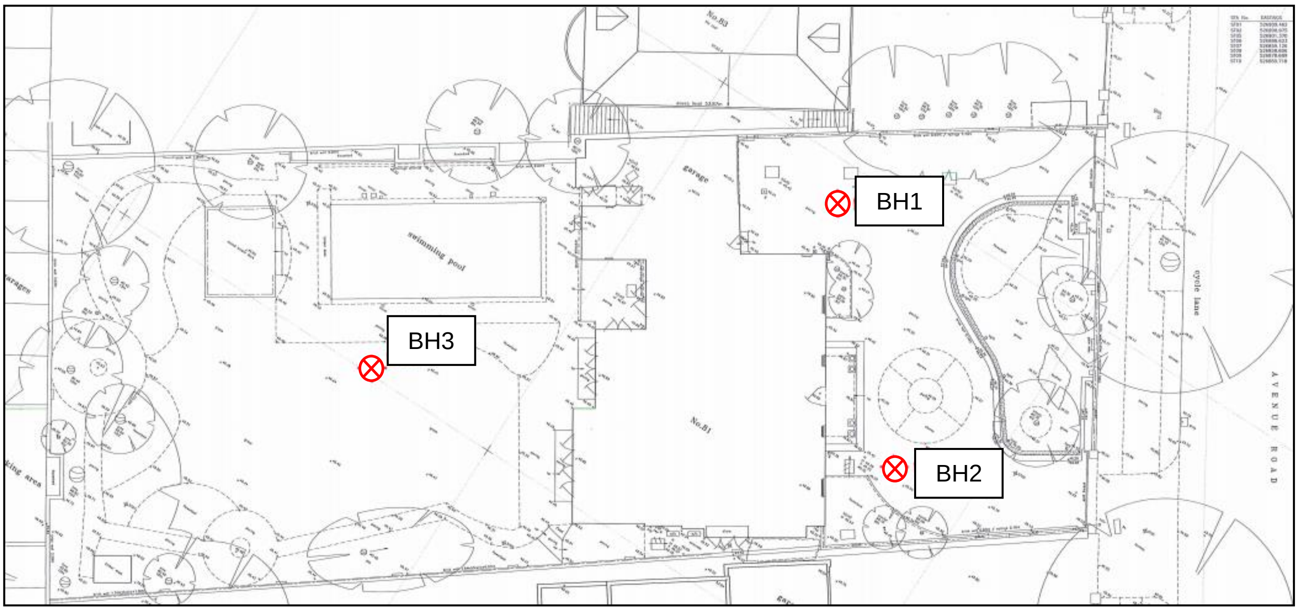
Site Analytical Services Ltd.			REF: 16/25552
LOCATION:	81 Avenue Road, London, NW8 6HR		FIG: 1
TITLE:	Site Sketch Plan	DATE: Sept 2016	SCALE: NTS



A P P E N D I X `A’

Borehole / Trial Pit Logs

Site Analytical Services Ltd.

Site	Borehole Number
81 AVENUE ROAD, LONDON, NW8 6HR	BH1

Boring Method	Casing Diameter	Ground Level (mOD)	Client	Job Number
ROTARTY PERCUSSIVE	128mm cased to 0.00m	46.40	B. K. MIRCHANDI	1625552
	Location	**Dates**	**Engineer**	**Sheet**
	TQ270837	26/07/2016-26/08/2016	ELLIOTT WOOD PARTNERSHIP LTD.	1/2

Depth (m)	Sample / Tests	Casing Depth (m)	Water Depth (m)	Field Records
0.25	D1			
0.50	D2			
0.75	D3			
1.00-1.45 1.00	SPT(C) N=12 D4		DRY	2,3/3,3,3,3
1.75	D5			
2.00-2.45	U1			50 blows
2.75	D6			
3.00-3.45 3.00	SPT(C) N=16 D7		DRY	3,3/3,4,4,5
3.75	D8			
4.00-4.45	U2			70 blows
4.75	D9			
5.00-5.45 5.00	SPT N=32 D10		DRY	7,8/7,8,8,9
6.00	D11			
6.50-6.95	U3			110 blows
7.50	D12			
8.00-8.45 8.00	SPT N=39 D13		DRY	8,8/9,10,10,10
9.00	D14			
9.50-9.95	U4			120 blows

Level (mOD)	Depth (m) (Thickness)	Description
46.36	0.04	MADE GROUND: Stone paving slab.
46.30	0.10	MADE GROUND: Cement.
46.20	0.20 (0.20)	MADE GROUND: Reinforced concrete.
46.00	0.40 (0.20)	MADE GROUND: Grey concrete with fragments of brick rubble.
45.80	0.60 (1.00)	MADE GROUND: Slightly pink sand and gravel with type 1 fill.
		MADE GROUND: Light brown mottled silty sandy clay containing occassional fragments of brick and concrete rubble.
44.80	1.60 (0.40)	Firm brown mottled silty sandy CLAY.
44.40	2.00 (1.20)	Firm brown gravelly CLAY. Gravels are fine to coarse grained sub-angular to sub-rounded flint.
43.20	3.20 (5.30)	Firm becoming stiff then very stiff slightly silty sandy CLAY. Claystones present at 8.30m depth.
37.90	8.50 (1.50)	Very stiff dark grey blue silty sandy CLAY, containing occassional gypsum crystals.

Remarks
D= Disturbed Sample
C= Standard Penetration Test- Cone
S= Standrad Penetration Test
Groundwater was not encountered during excavation
Excavating from 0.00m to 1.00m for 1 hour.

Scale (approx)	Logged By
1:50	MH

Figure No. 1625552.BH1

Site Analytical Services Ltd.

Site	Borehole Number
81 AVENUE ROAD, LONDON, NW8 6HR	BH1

Boring Method	Casing Diameter	Ground Level (mOD)	Client	Job Number
ROTARTY PERCUSSIVE	128mm cased to 0.00m	46.40	B. K. MIRCHANDI	1625552
	Location	**Dates**	**Engineer**	**Sheet**
	TQ270837	26/07/2016	ELLIOTT WOOD PARTNERSHIP LTD.	2/2

Depth (m)	Sample / Tests	Casing Depth (m)	Water Depth (m)	Field Records	Level (mOD)	Depth (m) (Thickness)	Description	Legend	Water
					36.40	10.00	Very stiff becoming hard dark grey blue silty sandy CLAY, containing occassional gypsum crystals.		
10.50	D15								
11.00-11.45	SPT N=50		DRY	10,12/12,12,12,14					
11.00	D16								
12.00	D17								
12.50-12.95	U5			230 blows		(5.00)			
13.75	D18								
14.55-15.00	SPT N=80		DRY	16,18/20,20,20,20					
14.55	D19								
				26/07/2016:DRY	31.40	15.00	Complete at 15.00m		

Remarks
D= Disturbed Sample
C= Standard Penetration Test- Cone
S= Standrad Penetration Test
Groundwater was not encountered during excavation

Scale (approx)	Logged By
1:50	MH

Figure No. 1625552.BH1

Site Analytical Services Ltd.

Site	Borehole Number
81 AVENUE ROAD, LONDON, NW8 6HR	BH1

Installation Type	Dimensions		Client	Job Number
Single Installation	Internal Diameter of Tube [A] = 50 mm Diameter of Filter Zone = 128 mm		B. K. MIRCHANDI	1625552
	Location	**Ground Level (mOD)**	**Engineer**	**Sheet**
	TQ270837	46.40	ELLIOTT WOOD PARTNERSHIP LTD.	1/1

Legend	Water	Instr (A) (B)	Level (mOD)	Depth (m)	Description
					Bentonite Seal
			45.40	1.00	
					Slotted Standpipe
			40.40	6.00	
					Bentonite Seal
			39.40	7.00	
					General Backfill
			31.40	15.00	

Groundwater Strikes During Drilling

Date	Time	Depth Struck (m)	Casing Depth (m)	Inflow Rate	Readings				Depth Sealed (m)
					5 min	10 min	15 min	20 min	

Groundwater Observations During Drilling

Date	Start of Shift					End of Shift				
	Time	Depth Hole (m)	Casing Depth (m)	Water Depth (m)	Water Level (mOD)	Time	Depth Hole (m)	Casing Depth (m)	Water Depth (m)	Water Level (mOD)
26/07/16							15.00		DRY	

Instrument Groundwater Observations

Inst. [A] Type : **Inst. [B] Type :** Slotted Standpipe

Date	Instrument [A]			Instrument [B]			Remarks
	Time	Depth (m)	Level (mOD)	Time	Depth (m)	Level (mOD)	

Remarks

Site Analytical Services Ltd.

Site	Borehole Number
81 AVENUE ROAD, LONDON, NW8 6HR	BH2

Boring Method	Casing Diameter	Ground Level (mOD)	Client	Job Number
ROTARTY PERCUSSIVE	128mm cased to 0.00m	46.35	B. K. MIRCHANDI	1625552
	Location	**Dates**	**Engineer**	**Sheet**
	TQ270837	27/07/2016	ELLIOTT WOOD PARTNERSHIP LTD.	1/2

Depth (m)	Sample / Tests	Casing Depth (m)	Water Depth (m)	Field Records	Level (mOD)	Depth (m) (Thickness)	Description
					46.31	0.04	MADE GROUND: Stone paving slab.
					46.25	0.10	MADE GROUND: Cement.
0.25	D1				46.15	0.20 (0.30)	MADE GROUND: Reinforced Concrete.
0.50	D2				45.85	0.50	
0.75	D3					(0.70)	MADE GROUND: Grey sandy gravelly concrete crush containing frequent fragments of concrete rubble.
1.00-1.45 1.00	SPT(C) N=12 D4		DRY	1,2/3,3,3,3			
					45.15	1.20	MADE GROUND: Brown molttled clay containing occassional fragments of brick and concrete rubble.
						(0.70)	Firm brown mottled silty sandy CLAY.
1.75	D5						
					44.45	1.90	Firm brown mottled very silty sandy gravelly CLAY. Gravels are fine to corase grained, sub-angular to sub-rounded flint.
2.00-2.45 2.00	SPT(C) N=15 D6		DRY	3,4/3,4,4,4			
						(1.50)	
2.75	D7						
3.00	D8						
					42.95	3.40	Firm becoming stiff then very stiff brown mottled silty sandy CLAY.
3.75	D9						
4.00-4.45	U1			80 blows			
4.75	D10					(2.80)	
5.00-5.45 5.00	SPT N=34 D11		DRY	7,7/8,8,9,9			
6.00	D12						
					40.15	6.20	Very stiff brown silty sandy CLAY.
6.50-6.95	U2			100 blows			
7.50	D13					(2.50)	
8.00-8.45 8.00	SPT N=40 D14		DRY	9,9/10,10,10,10			
							Claystones present at 5.90m depth.
					37.65	8.70	Very stiff dark grey blue silty sandy CLAY, containing occassional gypsum crystals.
9.00	D15						
						(1.30)	
9.50-9.95	U3			130 blows			

Remarks
D= Disturbed Sample
C= Standard Penetration Test- Cone
S= Standrad Penetration Test
Groundwater was not encountered during excavation
Excavating from 0.00m to 1.00m for 1 hour.

Scale (approx)	Logged By
1:50	MH

Figure No. 1625552.BH2

Site Analytical Services Ltd.

Site	Borehole Number
81 AVENUE ROAD, LONDON, NW8 6HR	BH2

Boring Method	Casing Diameter	Ground Level (mOD)	Client	Job Number
ROTARTY PERCUSSIVE	128mm cased to 0.00m	46.35	B. K. MIRCHANDI	1625552
	Location	**Dates**	**Engineer**	**Sheet**
	TQ270837	27/07/2016	ELLIOTT WOOD PARTNERSHIP LTD.	2/2

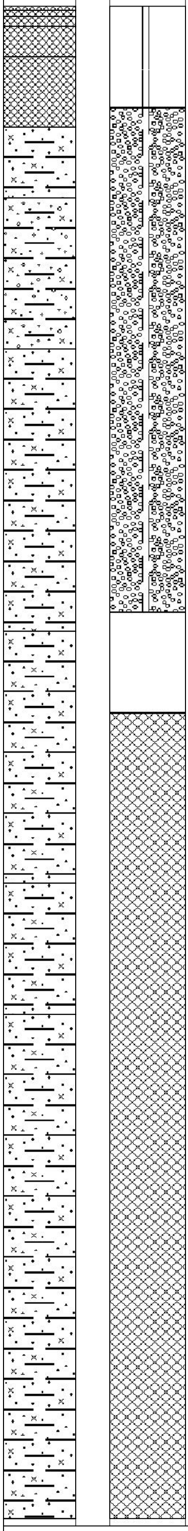
Depth (m)	Sample / Tests	Casing Depth (m)	Water Depth (m)	Field Records	Level (mOD)	Depth (m) (Thickness)	Description	Legend	Water
					36.35	10.00	Very stiff becoming hard dark grey blue silty sandy CLAY, containing occassional gypsum crystals.		
10.50	D16								
11.00-11.45 11.00	SPT N=52 D17		DRY	11,12/12,12,14,14					
12.00	D18								
12.50-12.95	U4			220 blows		(5.00)			
13.75	D19								
14.55-15.00 14.55	SPT N=86 D20		DRY	20,20/20,22,22,22					
				27/07/2016:DRY	31.35	15.00	Complete at 15.00m		

Remarks	Scale (approx)	Logged By
	1:50	MH
	Figure No. 1625552.BH2	

Site Analytical Services Ltd.

Site	Borehole Number
81 AVENUE ROAD, LONDON, NW8 6HR	BH2

Installation Type	Dimensions		Client	Job Number
Single Installation	Internal Diameter of Tube [A] = 50 mm Diameter of Filter Zone = 128 mm		B. K. MIRCHANDI	1625552
	Location	**Ground Level (mOD)**	**Engineer**	**Sheet**
	TQ270837	46.35	ELLIOTT WOOD PARTNERSHIP LTD.	1/1



Legend	Water	Instr (A)	Level (mOD)	Depth (m)	Description
			45.35	1.00	Bentonite Seal
			40.35	6.00	Slotted Standpipe
			39.35	7.00	Bentonite Seal
			31.35	15.00	General Backfill

Groundwater Strikes During Drilling

Date	Time	Depth Struck (m)	Casing Depth (m)	Inflow Rate	Readings 5 min	10 min	15 min	20 min	Depth Sealed (m)

Groundwater Observations During Drilling

Date	Start of Shift Time	Depth Hole (m)	Casing Depth (m)	Water Depth (m)	Water Level (mOD)	End of Shift Time	Depth Hole (m)	Casing Depth (m)	Water Depth (m)	Water Level (mOD)
27/07/16							15.00		DRY	

Instrument Groundwater Observations

Inst. [A] Type : Slotted Standpipe

Date	Instrument [A] Time	Depth (m)	Level (mOD)	Remarks

Remarks

Site Analytical Services Ltd.

Site	81 AVENUE ROAD, LONDON, NW8 6HR			**Borehole Number**	BH3
Boring Method	CONTINUOUS FLIGHT AUGER	**Casing Diameter** 100mm cased to 0.00m	**Ground Level (mOD)** 46.43	**Client** B. K. MIRCHANDI	**Job Number** 1625552
		Location TQ270837	**Dates** 26/07/2016	**Engineer** ELLIOTT WOOD PARTNERSHIP LTD.	**Sheet** 1/2

Depth (m)	Sample / Tests	Casing Depth (m)	Water Depth (m)	Field Records	Level (mOD)	Depth (m) (Thickness)	Description	Legend	Water
0.25	D1					(0.60)	MADE GROUND: Grass surface iver dark brown slightly sandy slightly gravelly clay, containing occassional fragments of brick and concrete rubble.		
0.50	D2				45.83	0.60	Stiff brown silty CLAY.		
0.75	D3								
1.00	D4					(0.80)			
1.00	V1 84								
					45.03	1.40	Stiff light brown silty CLAY.		
1.50	D5								
1.50	V2 81								
2.00	D6					(1.60)			
2.00	V3 100								
2.50	D7								
2.50	V4 130+								
3.00	D8				43.43	3.00	Stiff brown silty slightly gravelly CLAY. Gravels are fine grained, sub-angular to sub-rounded flint.		
3.00	V5 130+					(0.50)			
3.50	D9				42.93	3.50	Stiff brown silty sandy CLAY.		
3.50	V6 130+								
4.00	D10								
4.00	V7 130+								
4.50	D11								
4.50	V8 130+								
5.00	D12								
5.00	V9 130+					(3.50)			
6.00	D13								
6.00	V10 130+								
7.00	D14				39.43	7.00	Stiif dark grey blue silty sandy CLAY, containing occassional gypsum crystals.		
7.00	V11 130+								
8.00	D15								
8.00	V12 130+					(3.00)			
9.00	D16								
9.00	V13 130+								

Remarks
D= Disturbed Sample
V= Vane Test- Result in kPa
Groundwater was not encountered during boring
Excavating from 0.00m to 1.00m for 1 hour.

Scale (approx) 1:50 | **Logged By** MH

Figure No. 1625552.BH3

Site Analytical Services Ltd.

Site	Borehole Number
81 AVENUE ROAD, LONDON, NW8 6HR	BH3

Boring Method	Casing Diameter	Ground Level (mOD)	Client	Job Number
CONTINUOUS FLIGHT AUGER	100mm cased to 0.00m	46.43	B. K. MIRCHANDI	1625552
	Location	**Dates**	**Engineer**	**Sheet**
	TQ270837	26/07/2016	ELLIOTT WOOD PARTNERSHIP LTD.	2/2

Depth (m)	Sample / Tests	Casing Depth (m)	Water Depth (m)	Field Records	Level (mOD)	Depth (m) (Thickness)	Description	Legend	Water
					36.43	10.00	Stiif dark grey blue silty sandy CLAY, containing occassional gypsum crystals.		
10.00	D17								
10.00	V14 130+								
11.00	D18								
11.00	V15 130+								
12.00	D19								
12.00	V16 130+								
						(5.00)			
13.00	D20								
13.00	V17 130+								
14.00	D21								
14.00	V18 130+								
					31.43	15.00	Complete at 15.00m		
15.00	D22			26/07/2016:DRY					
15.00	V19 130+								

Remarks	Scale (approx)	Logged By
	1:50	MH
	Figure No.	
	1625552.BH3	

Site Analytical Services Ltd.

Site	Borehole Number
81 AVENUE ROAD, LONDON, NW8 6HR	BH3

Installation Type	Dimensions		Client	Job Number
Single Installation	Internal Diameter of Tube [A] = 50 mm Diameter of Filter Zone = 100 mm		B. K. MIRCHANDI	1625552
	Location	**Ground Level (mOD)**	**Engineer**	**Sheet**
	TQ270837	46.43	ELLIOTT WOOD PARTNERSHIP LTD.	1/1

Legend	Water	Instr (A)	Level (mOD)	Depth (m)	Description
			45.43	1.00	Bentonite Seal
			40.43	6.00	Slotted Standpipe
			39.43	7.00	Bentonite Seal
			31.43	15.00	General Backfill

Groundwater Strikes During Drilling

Date	Time	Depth Struck (m)	Casing Depth (m)	Inflow Rate	Readings 5 min	10 min	15 min	20 min	Depth Sealed (m)

Groundwater Observations During Drilling

Date	Start of Shift Time	Depth Hole (m)	Casing Depth (m)	Water Depth (m)	Water Level (mOD)	End of Shift Time	Depth Hole (m)	Casing Depth (m)	Water Depth (m)	Water Level (mOD)
26/07/16							15.00		DRY	

Instrument Groundwater Observations

Inst. [A] Type : Slotted Standpipe

Date	Instrument [A] Time	Depth (m)	Level (mOD)	Remarks

Remarks

Site Analytical Services Ltd.

Standard Penetration Test Results

Site : 81 AVENUE ROAD, LONDON, NW8 6HR

Client : B. K. MIRCHANDI

Engineer : ELLIOTT WOOD PARTNERSHIP LTD.

Job Number 1625552

Sheet 1 / 1

Borehole Number	Base of Borehole (m)	End of Seating Drive (m)	End of Test Drive (m)	Test Type	Seating Blows per 75mm		Blows for each 75mm penetration				Result	Comments
					1	2	1	2	3	4		
BH1	1.00	1.15	1.45	CPT	2	3	3	3	3	3	N=12	
BH1	3.00	3.15	3.45	CPT	3	3	3	4	4	5	N=16	
BH1	5.00	5.15	5.45	SPT	7	8	7	8	8	9	N=32	
BH1	8.00	8.15	8.45	SPT	8	8	9	10	10	10	N=39	
BH1	11.00	11.15	11.45	SPT	10	12	12	12	12	14	N=50	
BH1	14.55	14.70	15.00	SPT	16	18	20	20	20	20	N=80	
BH2	1.00	1.15	1.45	CPT	1	2	3	3	3	3	N=12	
BH2	2.00	2.15	2.45	CPT	3	4	3	4	4	4	N=15	
BH2	5.00	5.15	5.45	SPT	7	7	8	8	9	9	N=34	
BH2	8.00	8.15	8.45	SPT	9	9	10	10	10	10	N=40	
BH2	11.00	11.15	11.45	SPT	11	12	12	12	14	14	N=52	
BH2	14.55	14.70	15.00	SPT	20	20	20	22	22	22	N=86	

A P P E N D I X `B'

Laboratory Test Data

Ref: 16/25552-1

UNDRAINED TRIAXIAL COMPRESSION TEST

LOCATION 81 Avenue Road, London, NW8 6HR

BH/TP No.	MOISTURE CONTENT %	BULK DENSITY Mg/m^3	LATERAL PRESSURE kN/m^2	COMPRESSIVE STRENGTH kN/m^2	COHESION kN/m^2	ANGLE OF SHEARING RESISTANCE degrees	DEPTH m
BH1	26	2.00	50	188	94		2.25
	26	2.01	80	190	95		4.25
	27	1.96	130	245	122		6.75
	27	2.00	190	226	113		9.75
	28	2.03	250	266	133		12.75
BH2	27	2.02	80	215	108		4.25
	24	2.00	130	248	124		6.75
	27	2.03	250	269	135		12.75

Ref: 16/25552-1

PLASTICITY INDEX &
MOISTURE CONTENT
DETERMINATIONS

LOCATION 81 Avenue Road, London, NW8 6HR

BH/TP No.	Depth m	Natural Moisture %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Passing 425 µm %	Class
BH1	3.75	26	57	22	35	100	CH
BH2	3.75	22	48	22	26	100	CI
BH3	3.50	26	59	23	36	100	CH
	4.00	26	58	25	33	100	CH

Ref: 16/25552-1

SULPHATE & pH DETERMINATIONS

LOCATION 81 Avenue Road, London, NW8 6HR

BH/TP No.	DEPTH BELOW GL m	SOIL SULPHATES AS SO_4 TOTAL %	SOIL SULPHATES AS SO_4 WATER SOL g/l	WATER SULPHATES AS SO_4 g/l	pH	CLASS	SOIL - 2mm %
BH1	6.00		2.37		6.2	DS-3	100
	12.00		0.93		6.5	DS-2	100
BH2	8.00		2.39		6.4	DS-3	100
BH3	10.00		1.11		6.5	DS-2	100

Classification – Tables C1 and C2 : BRE Special Digest 1 : 2005

Ref: 16/25552-1

GROUND WATER MONITORING

LOCATION 81 Avenue Road, London, NW8 6HR

GROUNDWATER MONITORING RECORD			
Date	**Weather Conditions**	**Ground Conditions**	**Temperature (°C)**
16/08/2014	Cloudy with sunny spells	Dry	20.7
Monitoring Point Location	**Depth to water (mBGL)**		**Depth to Base of well (mBGL)**
BH1	Dry		6.00
BH2	Dry		6.01
BH3	2.3		6.00

Table 4

10.0 APPENDIX B – GROUND MOVEMENT ASSESSMENT

applied geotechnical ENGINEERING

REPORT CONTROL SHEET

81 Avenue Road, London NW8 6HR

Damage Category Assessment

Project number P4143

Report number **P4143/01**
Revision number
Issue number **01**

Archive number

This document has been produced for and on behalf of
Applied Geotechnical Engineering

Made by:	M G Brice	Associate
Checked by:	N A Smith	Director

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01	Sept 2016	Site Analytical Services Ltd	1 x pdf
01	Sept 2016	AGE	1 x pdf

1.0 Introduction

In connection with the proposal to redevelop No 81 Avenue Road, London NW8 6HR, including demolition of the existing structure followed by the excavation of a basement and reconstruction of a house above, Applied Geotechnical Engineering Ltd (AGE) has been instructed by Site Analytical Services Ltd (SAS), on behalf of their client, to provide information on the effect of basement construction on the neighbouring properties. The addresses of these properties are understood to be Nos 79 and 83 Avenue Road and, potentially, properties to the rear understood to be part of the Queensmead development. Our understanding of the relative positions of these buildings is shown in Figure 1.

In the text below locations described as 'right', 'left' or 'rear' of the site are as viewed from Avenue Road, unless otherwise stated.

The structural engineer for the project is ElliottWood (EW). A plan and section of the proposed basement of the property are given in Figure 2.

Topographical information is available, and is understood to be related to OS datum. The general ground level in the vicinity of the proposed excavation is 46.4mOD. It is understood that new basement construction will involve excavation to a general level between approximately 4.3m and 5.6m below existing ground (to between 40.8 and 42.1mOD), with a deeper local excavation to approximately 7.0m depth (to 39.4mOD) for a swimming pool basin. Excavation will be carried out within bored pile walls. It is further understood that pile wall design has not yet been undertaken, therefore, for the purposes of this assessment only, pile depth will be taken as 1.4 x adjacent excavation depth.

It is not known whether the neighbouring buildings have basements, though it is noted there is a passageway to the left of No 83 at a level of 44.3mOD, approximately 2.1m below general ground level suggesting a sub-structure of some form is present beneath this house. It will be assumed that a basement is not present beneath No 79, this is potentially conservative.

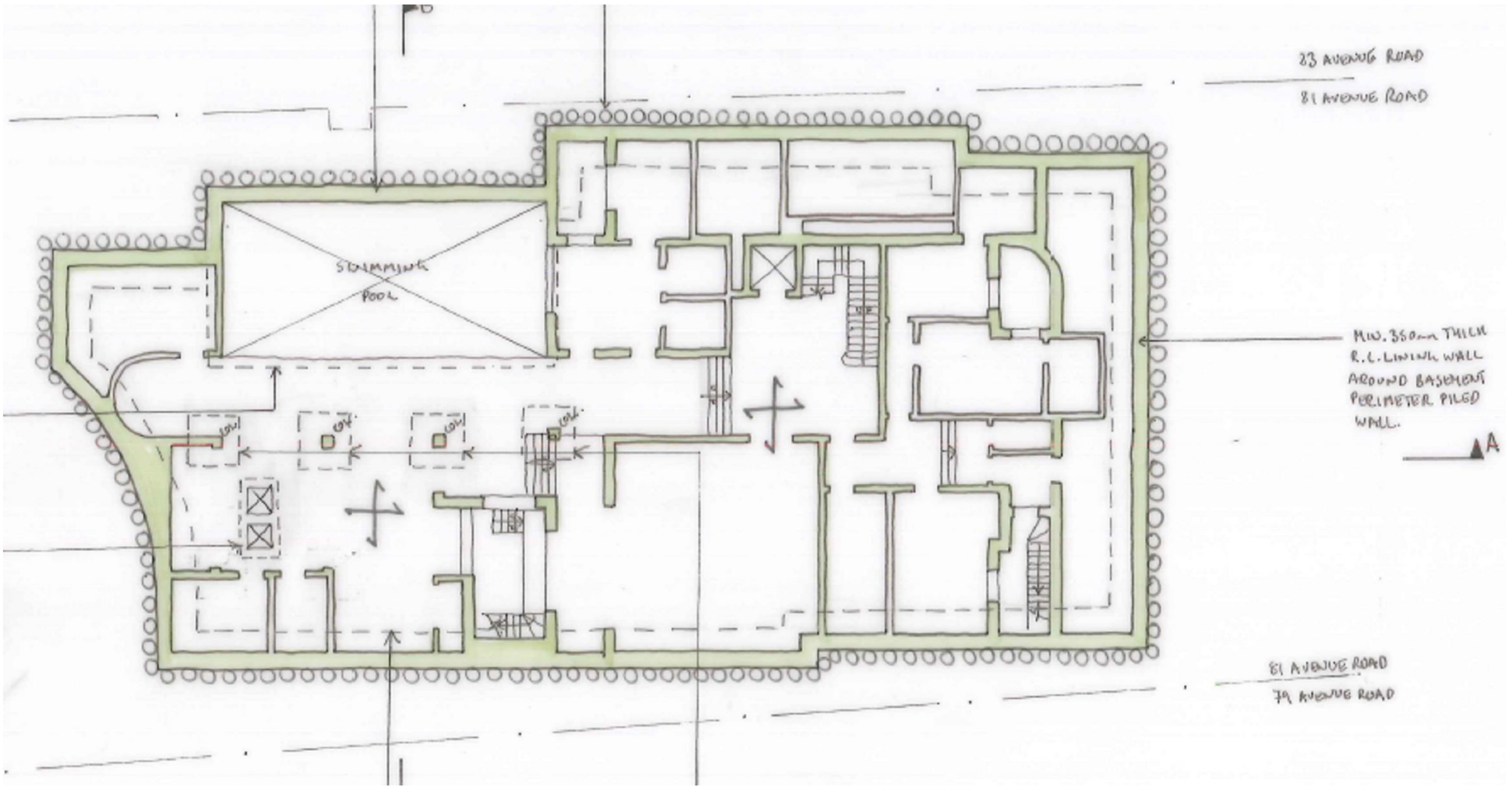
It is required that a predicted damage category assessment be made on the neighbouring properties.

2.0 Information Provided

The following relevant information has been provided for use in this assessment:-

i) SAS Borehole logs dated 26+27/7/2016.
ii) Existing and proposed building loads (file 2150623 drg 160728 loadings for GMA.pdf).
iii) Wolff Architects drawing 1510-EX-00 – topographic survey.
iv) eHRW Drawing 1474-SK-006(P1).
v) EW Drawings 2150623/S90P1, S100P1, S200P1 and S201P1.
vi) Drawing files 1510-FE-100-Existing and –PL-WORKING_160704.
vii) Email correspondence SAS-AGE dated 9/8/16 to 19/9/16.

Figure 1 – Site Context



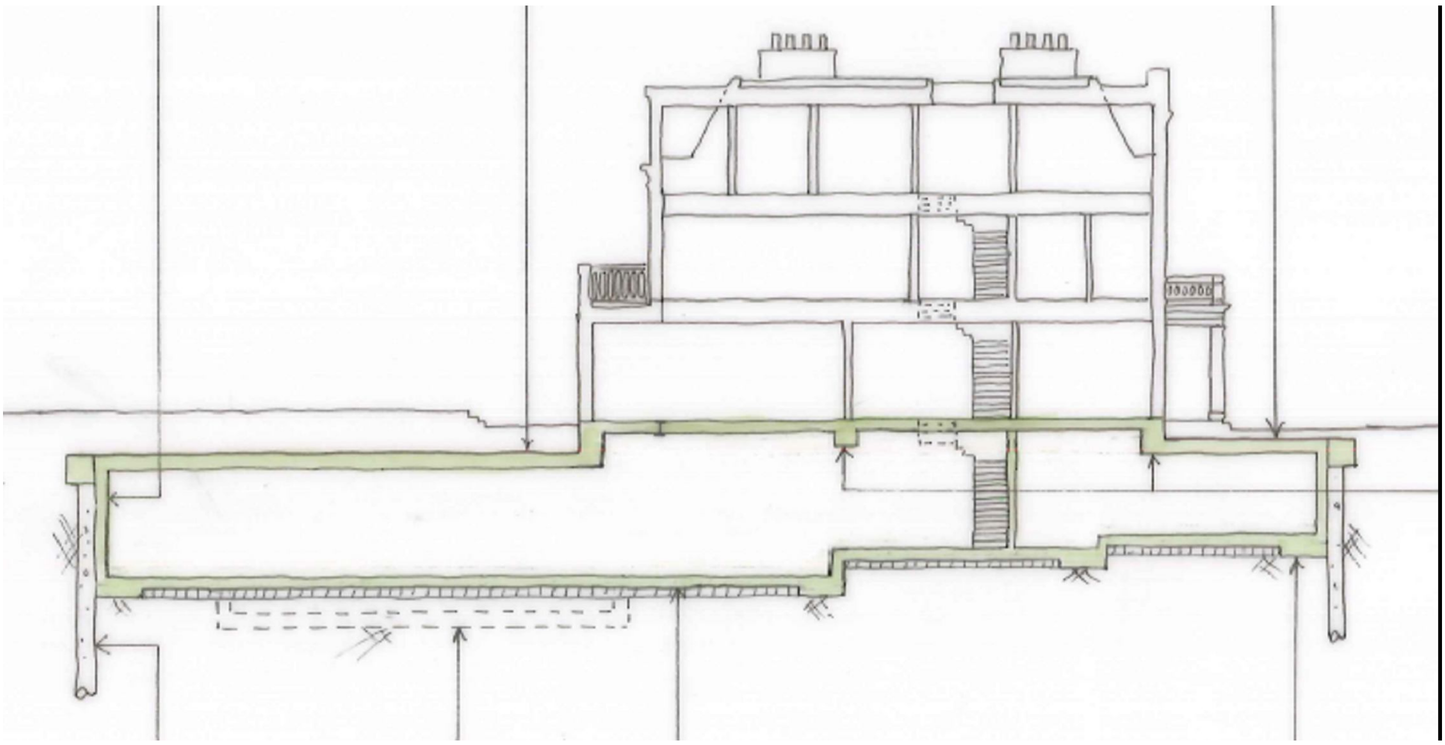


Figure 2 –Proposed Basement Plan and Section
(Extracts from EW sketches 2150623/S.90 and S.200)

3.0 Anticipated Ground Conditions

The external ground level adjacent to the property has been taken as approximately 46.4mOD.

The published geological map (BGS 1:50 000 sheet 256: North London) indicates the site to lie on the London Clay (silty clay). On a developed site such as this Made Ground is also anticipated.

On the basis of the published mapping, the base of the London Clay is anticipated to lie at approximately –23mOD (approximately 70m depth bgl).

A ground investigation was undertaken at the site in July 2016 (Item 'i' in Section 2 above). This comprised two rotary percussion boreholes to 15m depth in the front garden (BH1+2) and a single continuous-flight auger borehole, also to 15m depth (BH3) in the rear garden.

The commencement levels of the boreholes are all estimated to be at approximately 46.4mOD. They confirmed the presence of between 1.2 and 1.6m of Made Ground at the front of the house, and 0.6m of Made Ground at the rear. In all boreholes this was underlain by a firm or stiff gravelly clay, interpreted here as Head, to between 3.2 and 3.5m depth. Beneath this, London Clay was encountered to the base of the boreholes.

For the purposes of this report the existing foundations of No 81 Avenue Road are taken to bear onto the Head deposit at 1.5m depth (44.9mOD). The foundations of No 79 are taken to lie at a similar level, but No 83 is taken to lie deeper, at 44.0mOD on account of the low-lying passageway to the left of that house. Small variations from these assumptions have no significant effect on the outcome of the analysis.

Groundwater was not encountered during the siteworks. A standpipe was installed in each of the three boreholes with a response zone from 1m to 6m bgl in each case. Subsequent monitoring, approximately 3 weeks after installation, showed the standpipes in boreholes BH1 and BH2 to be dry, while a water level of 2.3mbgl was recorded in BH3. It is suspected that the water recorded in BH3 may be due to surface water inflow. Irrespective of this uncertainty, it is considered unlikely that significant quantities of groundwater will enter the excavation.

On the basis of the above, the soil sequence in the area of the proposed basement is taken to be:-

Ground Level 46.4mOD
Base of Made Ground 1.5mbgl (44.9mOD)
Base of Head 3.4mbgl (43.0mOD)
Base of London Clay ~69.4m bgl (-23mOD).

The Made Ground lies above excavation depth, and is assumed to lie above the founding level of neighbouring buildings. It will therefore not influence ground movements significantly, and will not be considered in detail.

Standard penetration tests (SPT) were carried out in BHs1+2.

In the Head and London Clay the results of the SPT tests can be correlated with the bulk undrained shear strength of clay deposits using the method of Stroud (Ref 1). Based upon the available plasticity results (average PI= 33%, range 26%-36%, 4 results, all in London Clay) an f_1 coefficient of 4.5 has been adopted. Lower plasticity results may be anticipated in the Head material, which would indicate a higher f_1 value, and higher undrained strength, but such an adjustment has not been adopted; this is potentially conservative.

The resulting strength values are plotted in Figure 3 below. Laboratory undrained shear strength determinations are also available from the ground investigation, for the London Clay, and these results are also plotted in Figure 3. No relevant additional information is available from the BGS archive.

A strength profile based on the data presented in Figure 3, and previous experience, has been adopted. The profile is taken to be linear through the Head deposits and into the upper part of the London Clay. The profile is described by:-

$Su = 60 + 7.5z_1$ (kPa) to 24.9mOD (20m below the top of the Head/London Clay), then

$Su = 210 + 3.7z_2$ (kPa) from 24.9mOD to –23mOD (base of the London Clay).

Where z_1 is the depth in metres below the top of the Head, and z_2 is the depth in metres below 24.9mOD.

The use of this bilinear profile reduces any tendency for prediction of excessive strength (and stiffness) at depth. This profile is considered to be conservative.

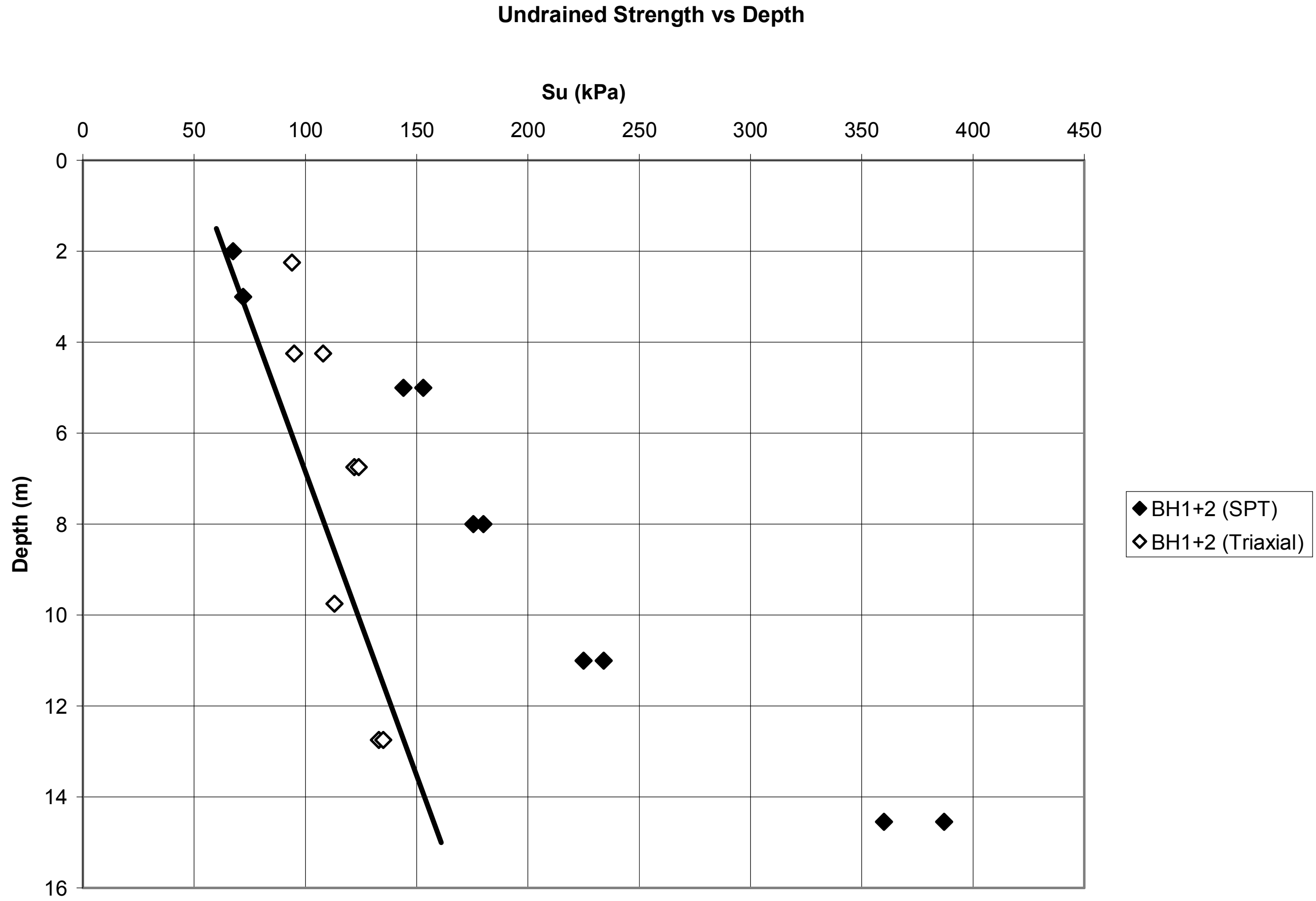


Figure 3 – Undrained Strength vs Depth

4.0 Loads

The proposed basement loads have been provided by the engineer (Item 'v' in Section 2 above).

Excavation from existing basement level to the new basement formation level will yield a significant load reduction; a bulk unit weight of 20kN/m^3 has been adopted for the calculation of this unload.

5.0 Estimated movement

5.1 Temporary support to the basement walls.

It is assumed within the following calculations that the basement perimeter retaining walls will be stiffly and safely propped at all stages of construction in line with relevant national standards and current good practice. Inadequate propping is likely to result in increased ground movements, and therefore increased damage to adjacent properties, as well as increased risk of injury to personnel.

It is generally recommended that consideration be given to the preloading of temporary basement wall props, and to the monitoring of prop loads during critical stages of excavation.

5.2 Soil stiffness values

An equivalent-elastic analysis has been carried out using the program PDisp. The program takes no account of structural (building) stiffness.

The soil stiffness parameters are as given below.

In the absence of reliable data the stiffness of the Head has been taken to be similar to that of the London Clay. The Head will be excavated within the basement footprint therefore this approximation is not considered to be critical to the analysis.

The London Clay has been treated as a non-linear material. The small-strain stiffness is taken as 80% of the small-strain stiffness calculated from recent high quality data (Bond Street Station). These data yielded E_{uo} = 1940Su, therefore for the purposes of the current analysis take:-

$E_{uo} = 1550 \times Su$; (Poisson's ratio = 0.5)
$E'_o = 1240 \times Su$; (Poisson's ratio = 0.2)

Yielding:-

$E_{uo} = 93 + 11.6\, z_1$ MPa to 24.9mOD, then
$E_{uo} = 325 + 5.8\, z_2$ MPa from 24.9mOD to -23mOD.

$E'_o = 74.4 + 9.3\, z_1$ MPa to 24.9mOD, then
$E'_o = 260 + 4.6 z_2$ MPa from 24.9mOD to –23mOD.

Where z_1 is the depth in metres below the top of the Head deposit, and z_2 is the depth in metres below 24.9mOD.

A non-linear degradation curve relating stiffness to strain, based on published data for the London Clay, has been used.

5.3 Causes of ground movement outside the excavation

The analysis considers three causes of ground movement outside the excavation, these are:-
i) Vertical ground movement due to vertical changes in load resulting from building works and excavation
ii) Vertical and horizontal movement due to installation of pile walls.
iii) Vertical and horizontal movement due to deflection of pile walls, following removal of support from in front of the wall by excavation.

The first of these causes is investigated using equivalent-elastic analysis in the program PDISP. The second and third are based upon case-history data presented in Figures 2.8, 2.9 and 2.11 in CIRIA C580 (Ref 3). These data relate to installation in stiff clays. It is currently understood that the plots presented by CIRIA in the above figures include short-term movement arising from cause 'i' above, but would not necessarily include unloading due to demolition. Therefore in this report short-term movements are calculated using the CIRIA data combined with an assessment of demolition heave calculated in PDISP; the subsequent long-term movement is calculated using PDISP.

The CIRIA plots relate vertical and horizontal ground movement to the depth of the wall installed (for Cause 'ii' above), or to the depth of excavation within that wall (for Cause 'iii' above) as appropriate. Data relating to the secant bored pile wall case history in Ref 3 Figure 2.8 are considered to be unreliable and have been ignored. In addition, data relating to counterfort diaphragm walls have not been taken into account in this analysis.

The CIRIA data indicate that:-

a) Adjacent to the pile wall, vertical ground settlement resulting from wall installation can be taken to equal 0.04% of wall depth, reducing linearly to zero at a distance of 2 x wall depth from the wall (Ref 3, Figures 2.8b and 2.9b).

b) Adjacent to the pile wall, vertical ground settlement resulting from wall deflection can be taken to equal 0.04% of excavation depth, increasing to 0.08% of excavation depth at a distance of 0.6 x excavation depth from the wall, then reducing approximately linearly to zero at a distance of 3 x excavation depth from the wall. (Ref 3, Figure 2.11b).

c) Adjacent to the pile wall, horizontal ground movement resulting from wall installation can be taken to equal 0.04% of wall depth, reducing linearly to zero at a distance of 1.5 x wall depth from the wall (Ref 3, Figures 2.8a and 2.9a).

d) Adjacent to the pile wall, horizontal ground movement resulting from wall deflection can be taken to equal 0.15% of excavation depth, reducing linearly to zero at a distance of 4 x dig depth from the wall. (Ref 3, Figure 2.11a).

The above trends rely on good workmanship and stiffly-propped, stiff walls. Temporary support of excavations should be designed to relevant national standards and current good practice.

It will be noted that the horizontal ground movements described in 'c' and 'd' above will tend to yield consistent average ground strains; these are (0.04%/1.5 =) 0.0267% average horizontal ground strain resulting from wall installation, and (0.15%/4 =) 0.0375% average horizontal ground strain resulting from yielding of the wall due to basement excavation within. There is therefore a consistent prediction, following wall installation and basement excavation, of a total of 0.064% average total horizontal ground strain within a distance of 1.5 x wall depth from the excavation, reducing, at greater distance, to 0.0375% horizontal ground strain, out to a distance of 4 x excavation depth from the excavation. These results are used in the following sections.

CIRIA C580 is used to predict the ground movement under plane-strain conditions. Near the corners of the excavation, plane-strain conditions are unlikely to develop and the buttressing effect around these corners has been taken into account in calculating the predicted (reduced) vertical ground movements, using the method of Fuentes and Devriendt (Ref 4). This method has not been sufficiently verified for the case of horizontal ground movements, and therefore is not taken into account rigorously in that part of the analysis, however the tendency for horizontal ground movement to be reduced at excavation corners is noted where appropriate.

Note that, in all the plots of vertical movement, settlement is taken as positive and heave as negative. The CIRIA data are understood to relate to movement at, or close to, ground level.

The analysis assumes that excavation is carried out reasonably uniformly across the footprint of the basement. If this is not the case, and there are temporary substantial variations in the excavation depth, then more severe short-term wall distortions may arise than are predicted here.

5.4 Predicted movement – No 79 Avenue Road, front wall.

5.4.1 Vertical Movement

Profiles of short- and long-term vertical ground movement along the front wall of No 79 Avenue Road have been calculated and plotted in Figure 5.

The wall is taken to be approximately 14.6m long and approximately 6m high, above ground level. It lies in the position shown on the plan in Figure 5.

The analysis indicates a maximum overall tilt of 2.6mm along the length of this wall. This equates to a whole-wall gradient of less than 1 in 5600. This is less than the 1:400 gradient recognised as requiring remedial action.

The maximum predicted wall distortion (Delta – as defined by Burland, Ref 2) is 1.6mm within the length of the wall. The limit on tensile strain for ‘very slight’ damage is 0.075% (Ref 2) therefore the ratio of deflection ratio to limiting tensile strain is 0.15. By reference to Figure 4 (Ref 2 Figure 6) the horizontal strain/limiting tensile strain ratio is found to be 0.81, indicating that a maximum horizontal strain of 0.061% is acceptable for a ‘very slight’ category of damage. The analysis does not take into account the stiffness of the wall, and is conservative in this respect.

5.4.2 Lateral movement.

From Section 5.3 above, the greatest average horizontal ground strain adjacent to the proposed excavation at No 79 is predicted to be 0.064%. This is greater than the 0.061% limit for very slight damage calculated above, indicating that damage may lie at the lower end of the ‘slight’ category, which in this case extends from 0.061% to 0.136%. However this magnitude of horizontal ground strain is predicted to extend less than 10m from the excavation (at X=0m) and therefore affects only the first 1m length or so of the front wall of No 79. The remainder of the wall is predicted to be subject to a horizontal strain of 0.0375%. Additionally the horizontal stiffness of the wall has not been taken into account. The predicted level of damage to this wall is therefore considered to be 'very slight'.

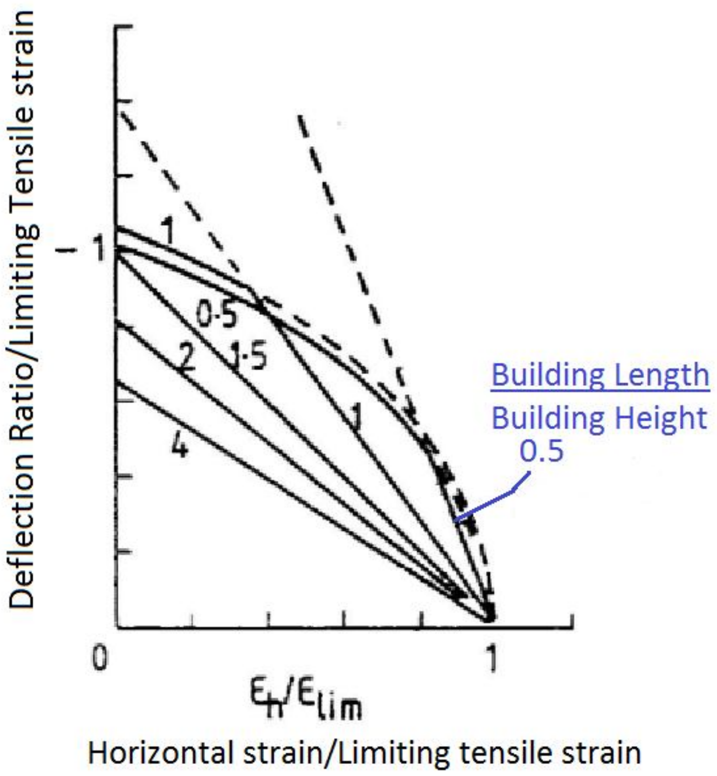


Figure 4 (from Ref 2)

5.5 Predicted movement – No 79 Avenue Road, rear wall.

5.5.1 Vertical Movement

Profiles of short- and long-term vertical ground movement along the rear wall of No 79 Avenue Road have been calculated and plotted in Figure 6.

The wall is taken to be approximately 15.1m long and approximately 6m high. It lies in the position shown on the plan in Figure 6.

The analysis indicates a maximum overall tilt of approximately 2.8mm over the length of the wall. This equates to a whole-wall gradient of less than 1 in 5300. This is less than the 1:400 gradient recognised as requiring remedial action.

The maximum predicted wall distortion (Delta – as defined by Burland, Ref 2) is found to be 1.8mm over the length of the wall. The limit on tensile strain for 'very slight' damage is 0.075% (Ref 2) therefore the proportion of deflection ratio to limiting tensile strain is 0.16. By reference to Figure 4 (Ref 2 Figure 6) a horizontal strain/limiting tensile strain ratio of 0.80 is obtained, indicating that a horizontal strain of 0.06% is acceptable for a 'very slight' category of damage. This analysis does not take account of the stiffness of the wall, and is conservative in this respect.

5.5.2 Lateral movement.

From Section 5.3 above, the greatest average horizontal ground strain adjacent to the proposed excavation at No 79 is predicted to be 0.064%. This is greater than the 0.06% limit for very slight damage calculated above, indicating that damage may lie at the lower end of the 'slight' category, which in this case extends from 0.06% to 0.135%. However, as for the front wall above, this magnitude of strain is predicted to extend less than 10m from the excavation and therefore affects only the first 2m length or so of the rear wall, the remainder is subject to a horizontal ground strain of 0.0375%. In addition the analysis does not take into account the horizontal stiffness of the wall. The predicted level of damage to this wall is therefore considered to be 'very slight'.

5.6 Predicted movement – No 79 Avenue Road, right flank wall.

5.6.1 Vertical movement

Profiles of short- and long-term vertical ground movement along the right flank wall of No 79 Avenue Road have been calculated and plotted in Figure 7.

This wall is taken to be approximately 10.7m long and approximately 6m high above ground level. It lies in the position shown in Figure 7.

The analysis indicates a maximum overall tilt of approximately 0.5mm over the length of the wall. This is negligible.

The maximum predicted wall distortion (Delta – as defined by Burland, Ref 2) is also negligible, indicating that a horizontal strain of 0.075% is acceptable for a 'very slight' category of damage.

5.6.2 Lateral movement.

Due to the position of this wall in relation to the proposed works the horizontal ground strain along the plane of the wall is predicted to be negligible.

The predicted level of damage to this wall can therefore be taken as 'very slight' or less.

5.7 Predicted movement – No 79 Avenue Road, wall to rear of garage.

5.7.1 Vertical Movement

Profiles of short- and long-term vertical ground movement along the wall to the rear of the garage at No 79 Avenue Road have been calculated and plotted in Figure 8.

The wall is taken to be approximately 6.5m long and approximately 6m high. It lies in the position shown on the plan in Figure 8.

The analysis indicates a maximum overall tilt of approximately 3mm over the length of the wall. This equates to a whole-wall gradient of less than 1 in 2100. This is less than the 1:400 gradient recognised as requiring remedial action.

The maximum predicted wall distortion (Delta – as defined by Burland, Ref 2) is negligible, indicating that a horizontal strain of 0.075% is acceptable for a 'very slight' category of damage.

5.7.2 Lateral movement.

From Section 5.3 above, the greatest average horizontal ground strain adjacent to the proposed excavation at No 79 is predicted to be 0.064%. This is less than the 0.075% limit for very slight damage calculated above. The predicted level of damage to this wall can therefore be taken as 'very slight' or less.

5.8 Predicted movement – No 83 Avenue Road, front wall.

5.8.1 Vertical Movement

Profiles of short- and long-term vertical ground movement along the front wall of No 83 Avenue Road have been calculated and plotted in Figure 9.

The wall is taken to be approximately 15.4m long and approximately 7m high. It lies in the position shown on the plan in Figure 9.

The analysis indicates a maximum overall tilt of approximately 4.3mm over the length of the wall. This equates to a whole-wall gradient of less than 1 in 3500. This is less than the 1:400 gradient recognised as requiring remedial action.

The maximum predicted wall distortion (Delta – as defined by Burland, Ref 2) is 1.8mm over the length of the wall. The limit on tensile strain for 'very slight' damage is 0.075% (Ref 2) therefore the ratio of deflection ratio to limiting tensile strain is 0.16. By reference to Figure 4 (Ref 2 Figure 6) the horizontal strain/limiting tensile strain ratio is found to be 0.80, indicating that a minimum horizontal strain of 0.060% is acceptable for a 'very slight' category of damage. The analysis does not take into account the stiffness of the wall, and is conservative in this respect.

5.8.2 Lateral movement.

From Section 5.3 above, the greatest average horizontal ground strain adjacent to the proposed excavation at No 79 is predicted to be 0.064%. This is greater than the 0.060% limit for very slight damage calculated above, indicating that damage may lie at the lower end of the 'slight' category, which in this case extends from 0.060% to 0.135%. However this magnitude of horizontal ground strain is predicted to extend less than 10m from the excavation (at X=21.3m) and therefore affects only the first 8m length or so of the front wall of No 83 (to X=31m or so on Figure 9). The remainder of the wall is predicted to be subject to a horizontal strain of 0.0375%. The major part of the vertical wall distortion lies beyond the zone of maximum horizontal ground strain. Additionally, the horizontal stiffness of the wall has not been taken into account.

The predicted level of damage to this wall is therefore considered to be 'very slight'.

5.9 Predicted movement – No 83 Avenue Road, rear wall.

5.9.1 Vertical Movement

Profiles of short- and long-term vertical ground movement along the rear wall of No 83 Avenue Road have been calculated and plotted in Figure 10.

This wall is taken to be approximately 15.4m long and approximately 7m high. It lies in the position shown on the plan in Figure 10.

The analysis indicates a maximum overall tilt of approximately 4.1mm over the length of the wall. This equates to a whole-wall gradient of less than 1 in 3700. This is less than the 1:400 gradient recognised as requiring remedial action.

The maximum predicted wall distortion (Delta – as defined by Burland, Ref 2) is 2mm over the length of the wall. The limit on tensile strain for 'very slight' damage is 0.075% (Ref 2) therefore the ratio of deflection ratio to limiting tensile strain is 0.17. By reference to Figure 4 (Ref 2 Figure 6) the horizontal strain/limiting tensile strain ratio is found to be 0.78, indicating that a minimum horizontal strain of 0.059% is acceptable for a 'very slight' category of damage. The analysis does not take into account the stiffness of the wall, and is conservative in this respect.

5.9.2 Lateral movement.

From Section 5.3 above, the greatest average horizontal ground strain adjacent to the proposed excavation at No 79 is predicted to be 0.064%. This is greater than the 0.059% limit for very slight damage calculated above, indicating that damage may lie at the lower end of the 'slight' category, which in this case extends from 0.059% to 0.134%. However this magnitude of horizontal ground strain is predicted to extend less than 10m from the excavation (at X=21.3m) and therefore affects only the first 8m length or so of the front wall of No 83 (to X=31m or so on Figure 9). The remainder of the wall is predicted to be subject to a horizontal strain of 0.0375%. The major part of the vertical wall distortion lies beyond the zone of maximum horizontal ground strain. Additionally, the horizontal stiffness of the wall has not been taken into account.

The predicted level of damage to this wall is therefore considered to be 'very slight'.

5.10 Predicted movement – No 83 Avenue Road, left flank wall.

5.10.1 Vertical movement

Profiles of short- and long-term vertical ground movement along the left flank wall of No 83 Avenue Road have been calculated and plotted in Figure 11.

This wall is taken to be approximately 15.4m long and approximately 7m high. It lies in the position shown in Figure 7.

The analysis indicates a maximum overall tilt of approximately 0.8mm over the length of the wall. This is negligible.

The maximum predicted wall distortion (Delta – as defined by Burland, Ref 2) is also negligible, indicating that a horizontal strain of 0.075% is acceptable for a 'very slight' category of damage.

5.10.2 Lateral movement.

Due to the position of this wall in relation to the proposed works the horizontal ground strain along the plane of the wall is predicted to be negligible.

The predicted level of damage to this wall can therefore be taken as 'very slight' or less.

5.11 Predicted movement – Garages associated with Queensmead development, rear wall (as viewed from Queensmead).

It is not clear whether damage to the garages is of significance in the current context. They have been included here for completeness.

5.11.1 Vertical Movement

Profiles of short- and long-term vertical ground movement along the rear wall of the garages associated with the Queensmead development have been calculated and plotted in Figure 12.

This wall is taken to be continuous over a long distance; a 60m length of the wall has been considered. The wall is taken to be approximately 3m high. It lies in the position shown on the plan in Figure 12.

The analysis indicates negligible overall tilt over the length of the wall.

The maximum predicted wall distortion (Delta – as defined by Burland, Ref 2) is 4mm over a 45m length of the wall. The limit on tensile strain for 'very slight' damage is 0.075% (Ref 2) therefore the ratio of deflection ratio to limiting tensile strain is 0.12. By reference to Figure 4 (Ref 2 Figure 6) the horizontal strain/limiting tensile strain ratio can be estimated to be approximately 0.76, indicating that a minimum horizontal strain of 0.057% is acceptable for a 'very slight' category of damage.

5.11.2 Lateral movement.

Due to the position of this wall in relation to the proposed works the horizontal ground strain along the plane of the wall is predicted to be negligible.

The predicted level of damage to this wall can therefore be taken as 'very slight' or less.

5.12 Predicted movement – Garages associated with Queensmead development, dividing walls.

It is not clear whether damage to the garages is of significance in the current context. They have been included here for completeness.

5.12.1 Vertical Movement

Profiles of short- and long-term vertical ground movement along the dividing walls between the garages associated with the Queensmead development have been calculated and plotted in Figure 13.

The walls are taken to be approximately 5.9m long and 3m high. The particular wall under consideration lies in the position shown on the plan in Figure 13.

The analysis indicates a maximum overall tilt of approximately 3.1mm over the length of the wall. This equates to a whole-wall gradient of less than 1 in 1800. This is less than the 1:400 gradient recognised as requiring remedial action.

The maximum predicted wall distortion (Delta – as defined by Burland, Ref 2) is negligible, indicating that a minimum horizontal strain of 0.075% is acceptable for a 'very slight' category of damage.

5.12.2 Lateral movement.

From Section 5.3 above, the greatest average horizontal ground strain adjacent to the proposed excavation at No 79 is predicted to be 0.064%. This is likely to be an over-estimate of the strain at the position of the garage walls, as only a short length of the walls lies within the zone of maximum horizontal ground strain. Nevertheless this is less than the 0.075% limit on horizontal strain allowable for very slight damage, calculated above.

The predicted level of damage to this wall can therefore be taken as 'very slight' or less.

5.13 Predicted damage summary

On the basis of the above, the level of damage to Nos 79 and 83 Avenue Road, and to the garages associated with the Queensmead development, is predicted to be 'very slight' or less, as defined in Ref 2. By inspection, the predicted level of damage to the Queensmead building itself is also very slight or less, as this lies more distant from the proposed excavation than do the garages.

This conclusion assumes a high standard of workmanship and adequate propping of the basement excavation in line with appropriate national standards and current best practice.

A plot of the calculated short-term settlement contours is presented in Figure 14 below. The figure shows a maximum of approximately 2mm predicted short-term settlement to the Avenue Road trafficked road pavement. This is likely to be significantly less than the seasonal movements associated with the mature trees that grow along the road, and given the typically flexible nature of such pavements these movements are unlikely to be noticeable.

6.0 Groundwater

It is proposed to excavate to a maximum general depth of approximately 5.6m through approximately 1.5m of Made Ground and clayey Head deposit into a thick deposit of London Clay. Groundwater was not encountered during the ground investigation. A standpipe was installed in each of the three boreholes, with a response zone from 1m to 6m bgl in each case. Subsequent monitoring, approximately 3 weeks after installation, showed two of the standpipes to be dry, while the other (BH3) recorded a water level at 2.3m bgl; it is suspected this water may have accumulated by surface water inflow. Significant water inflows to the proposed excavation during construction are considered unlikely.

On the basis of the ground investigation it is considered that there is limited potential for significant groundwater flow within the proposed basement depth, and therefore the development will not affect the local groundwater regime.

7.0 Conclusions and Recommendations

From the above, it is concluded that, given good workmanship, the basement to No 81 Avenue Road may be constructed without imposing more than 'very slight' damage on the adjoining properties.

The development is not likely to disrupt any existing local groundwater flows.

References:

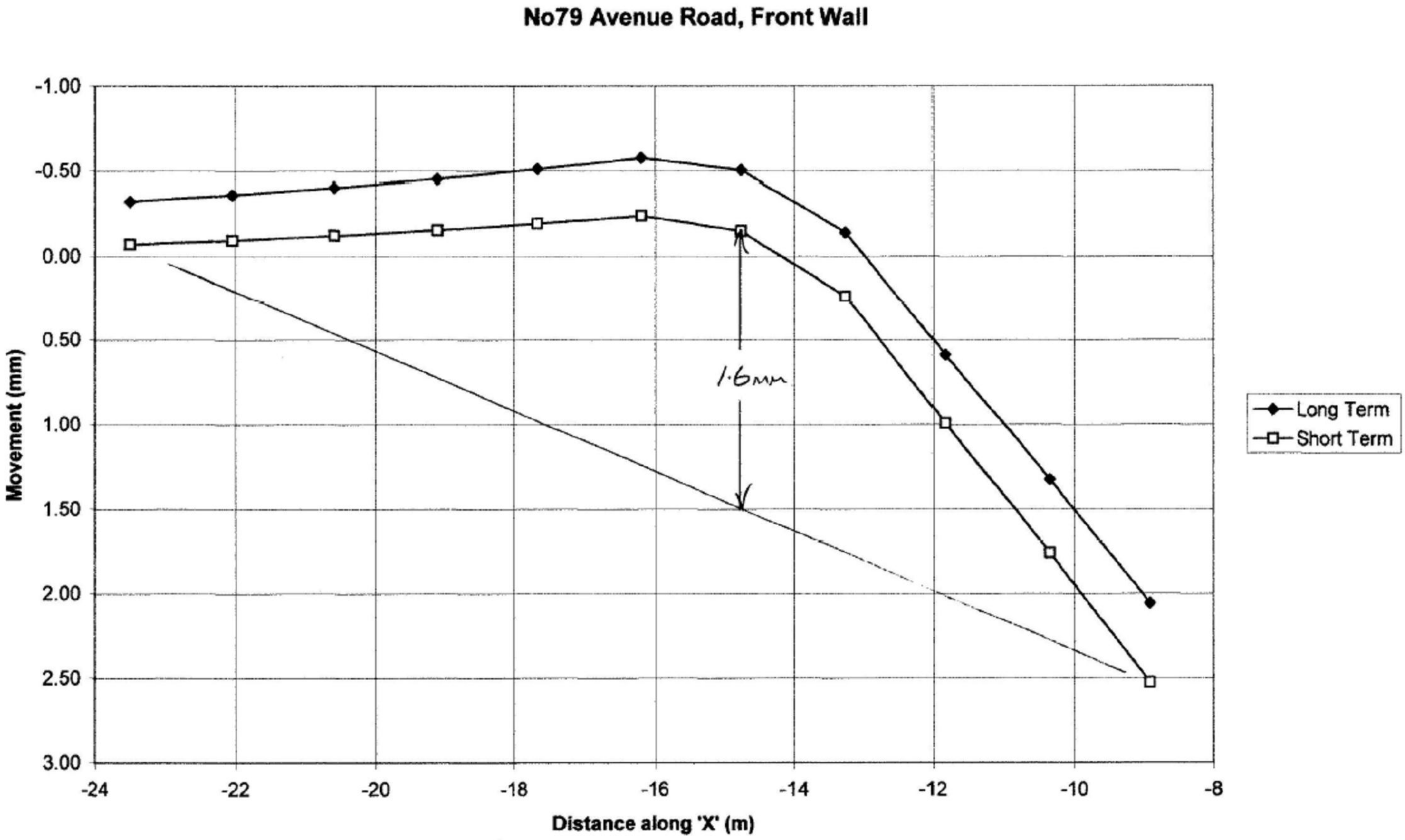
1 Stroud M A (1989) 'The standard penetration test – its application and interpretation'. In 'Penetration testing in the UK', Thomas Telford pub.

2 Burland JB (1997). 'Assessment of risk of damage to buildings due to tunnelling and excavation'. In 'Earthquake Geotechnical engineering' Ishihara (Ed). Balkema pub.

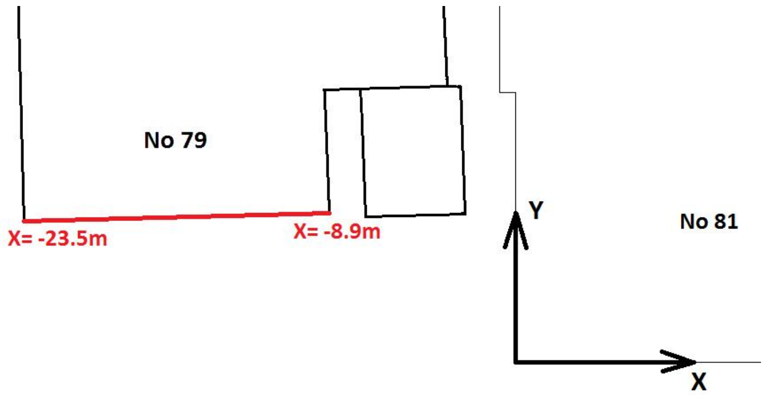
3 Gaba A R, Simpson B, Powrie W, Beadman D R (2003) Embedded retaining walls - guidance for economic design, CIRIA Report C580, London. ISBN: 978-0-86017-580-3.

4 Fuentes, R. and Devriendt, M (2010). 'Ground movements around the corners of excavations: empirical calculation method'. Journal of geotechnical and geoenvironmental engineering, ASCE v136 Issue 10 pp1414-1424.

(Figures 5 - 14 follow below)

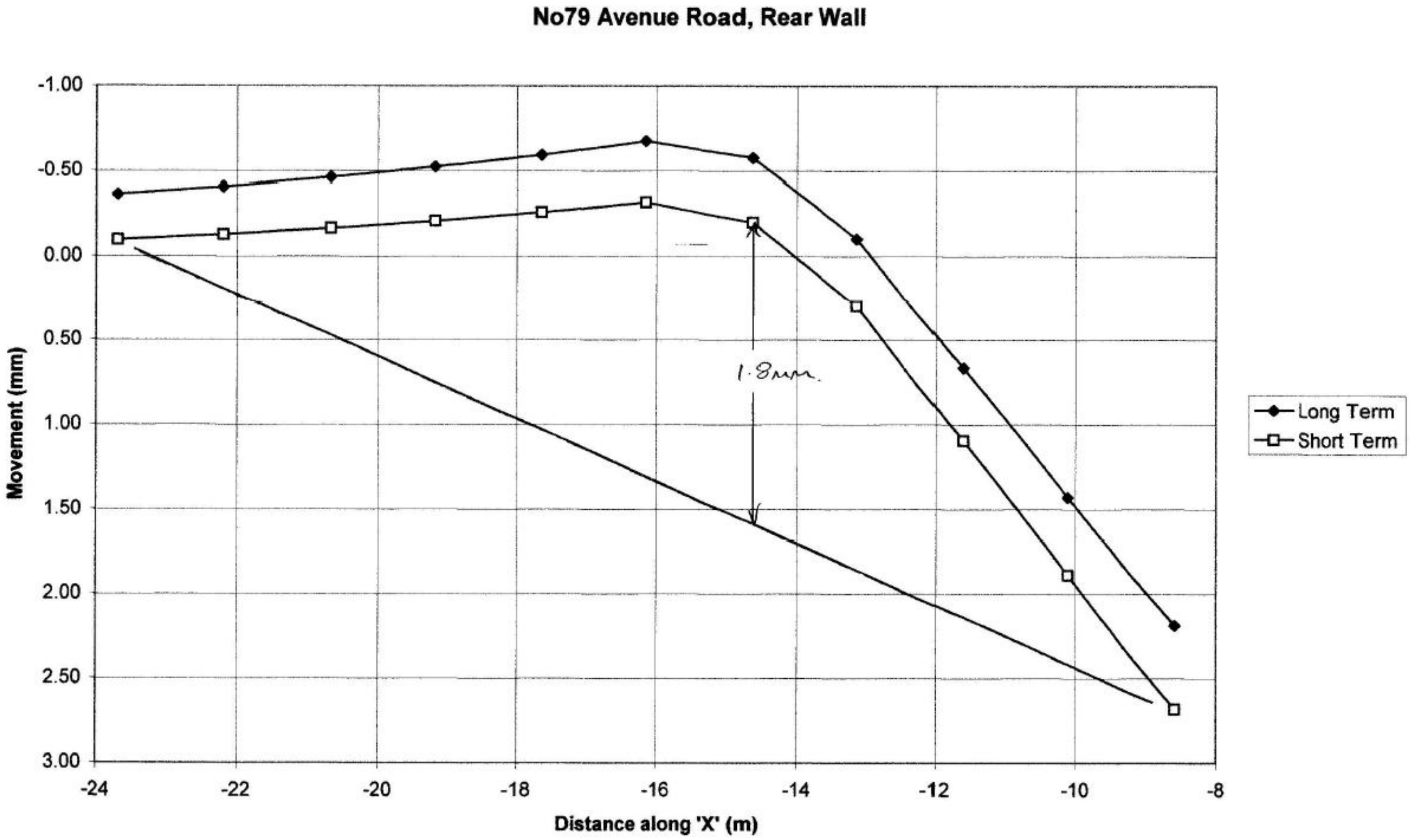


No79 Avenue Road, Front Wall
Movement (mm)
Distance along 'X' (m)
Long Term
Short Term
1·6mm
-1.00
-0.50
0.00
0.50
1.00
1.50
2.00
2.50
3.00
-24
-22
-20
-18
-16
-14
-12
-10
-8

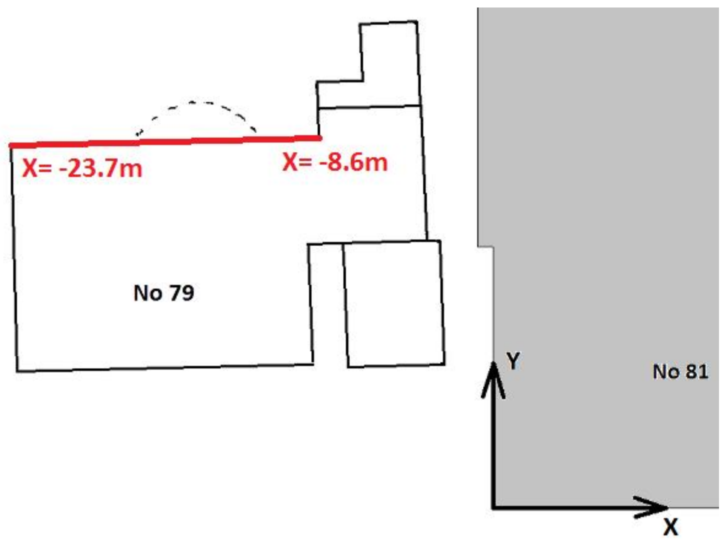


No 79
No 81
X= -23.5m
X= -8.9m
Y
X

Figure 5



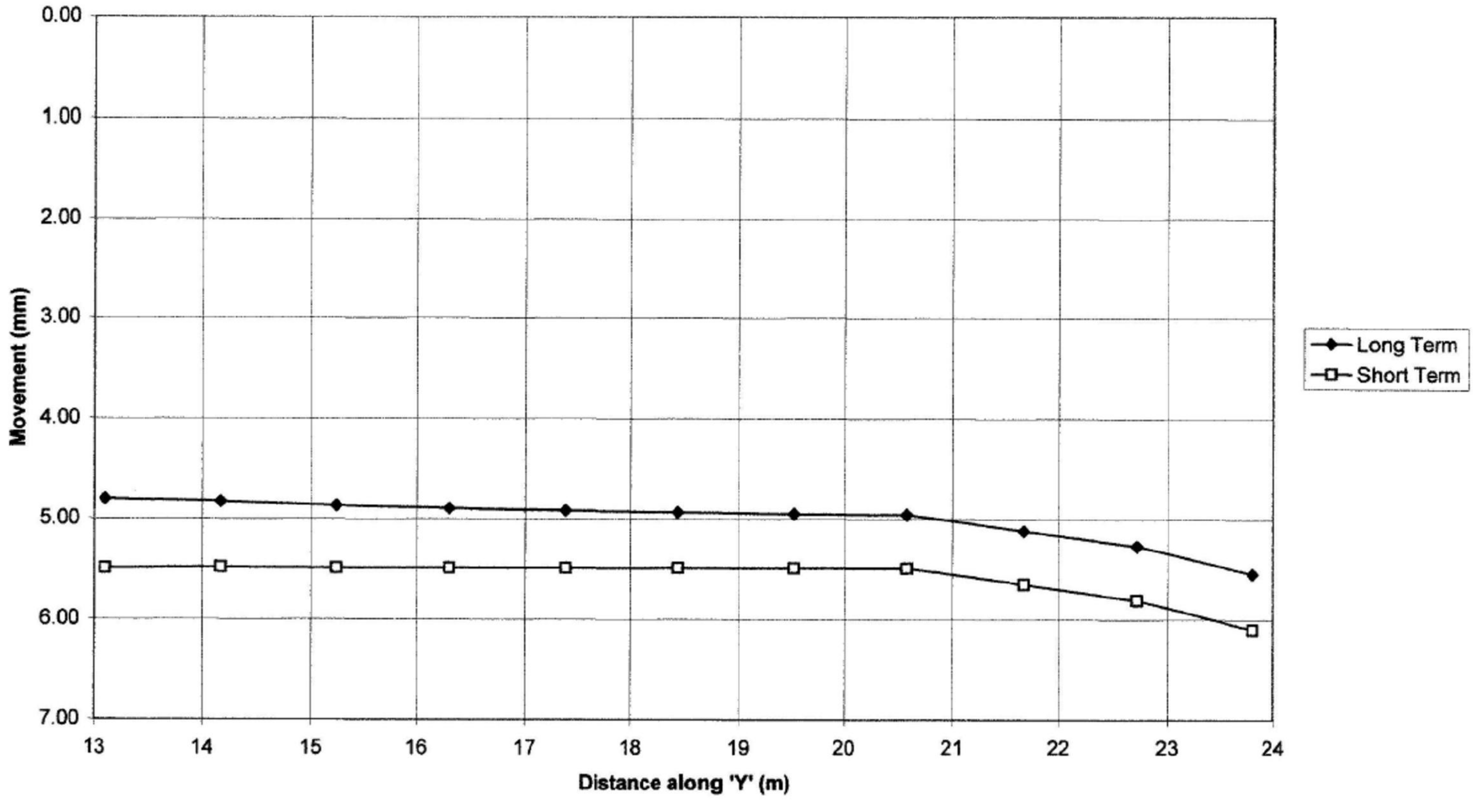
No79 Avenue Road, Rear Wall
-1.00
-0.50
0.00
0.50
1.00
1.50
2.00
2.50
3.00
Movement (mm)
-24
-22
-20
-18
-16
-14
-12
-10
-8
Distance along 'X' (m)
1·8mm.
Long Term
Short Term



X= -23.7m
X= -8.6m
No 79
No 81
Y
X

Figure 6

No79 Avenue Road, Right Flank Wall



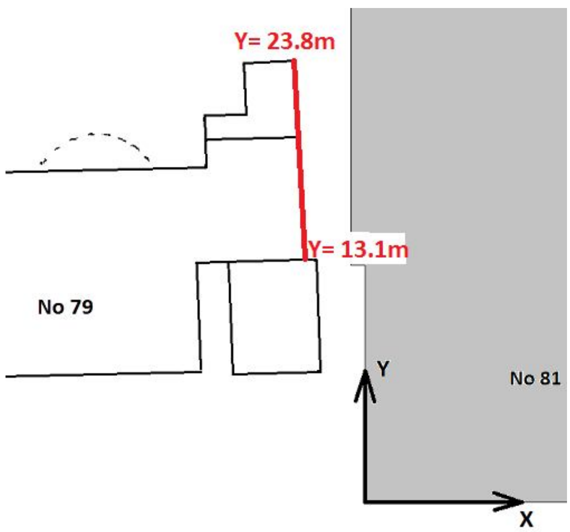
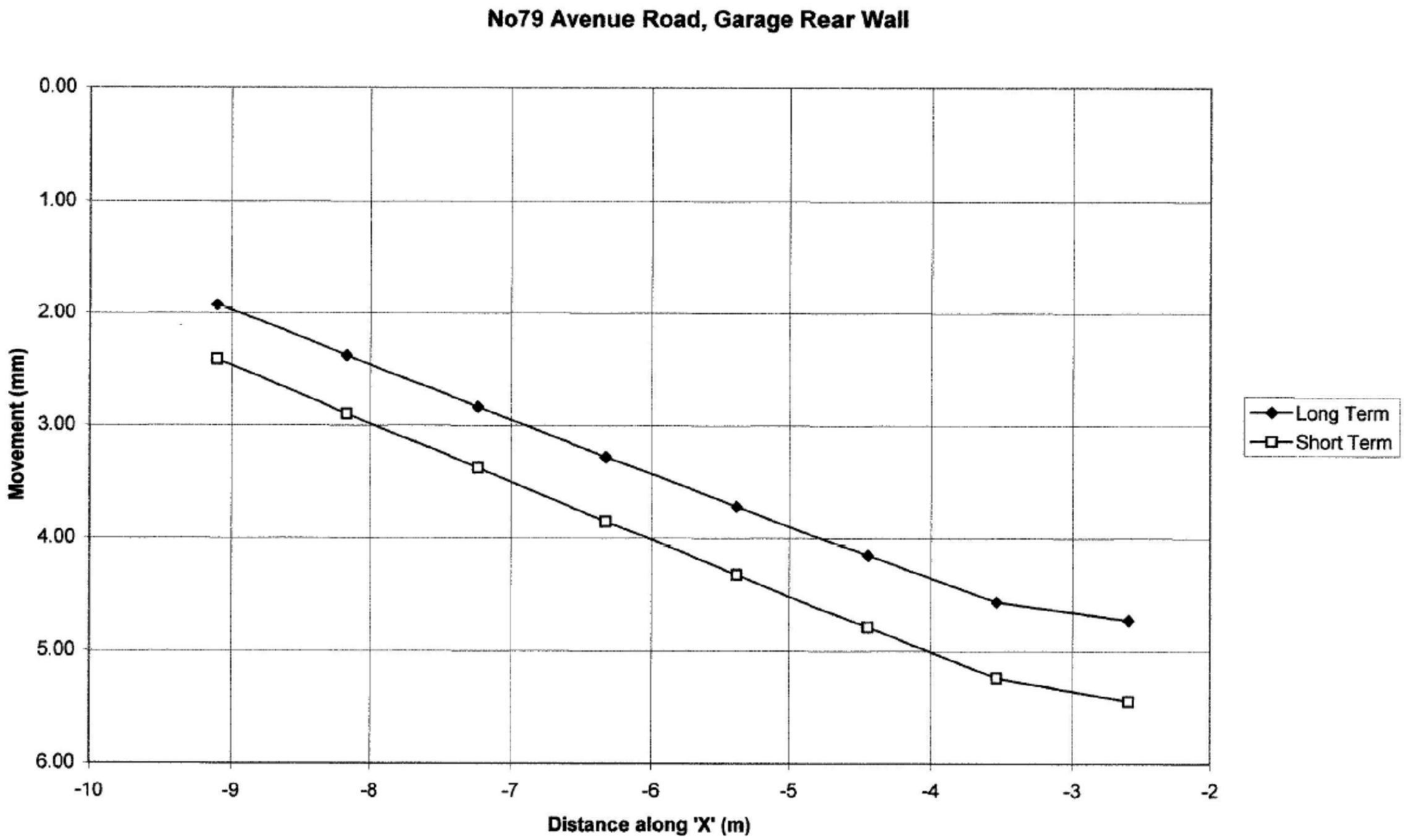
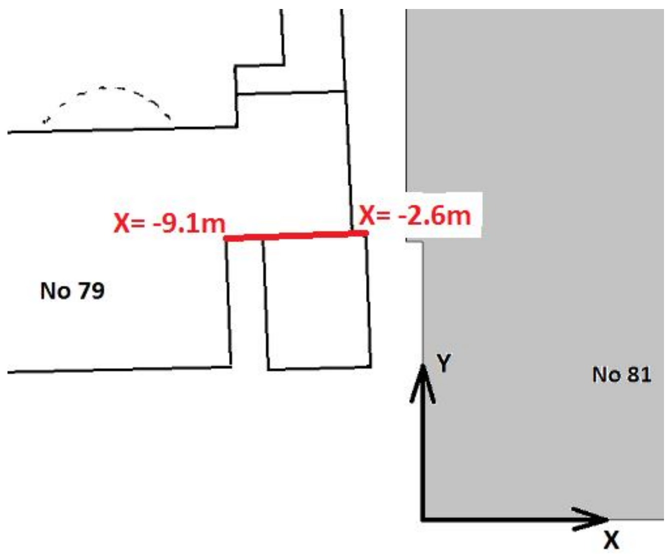


Figure 7

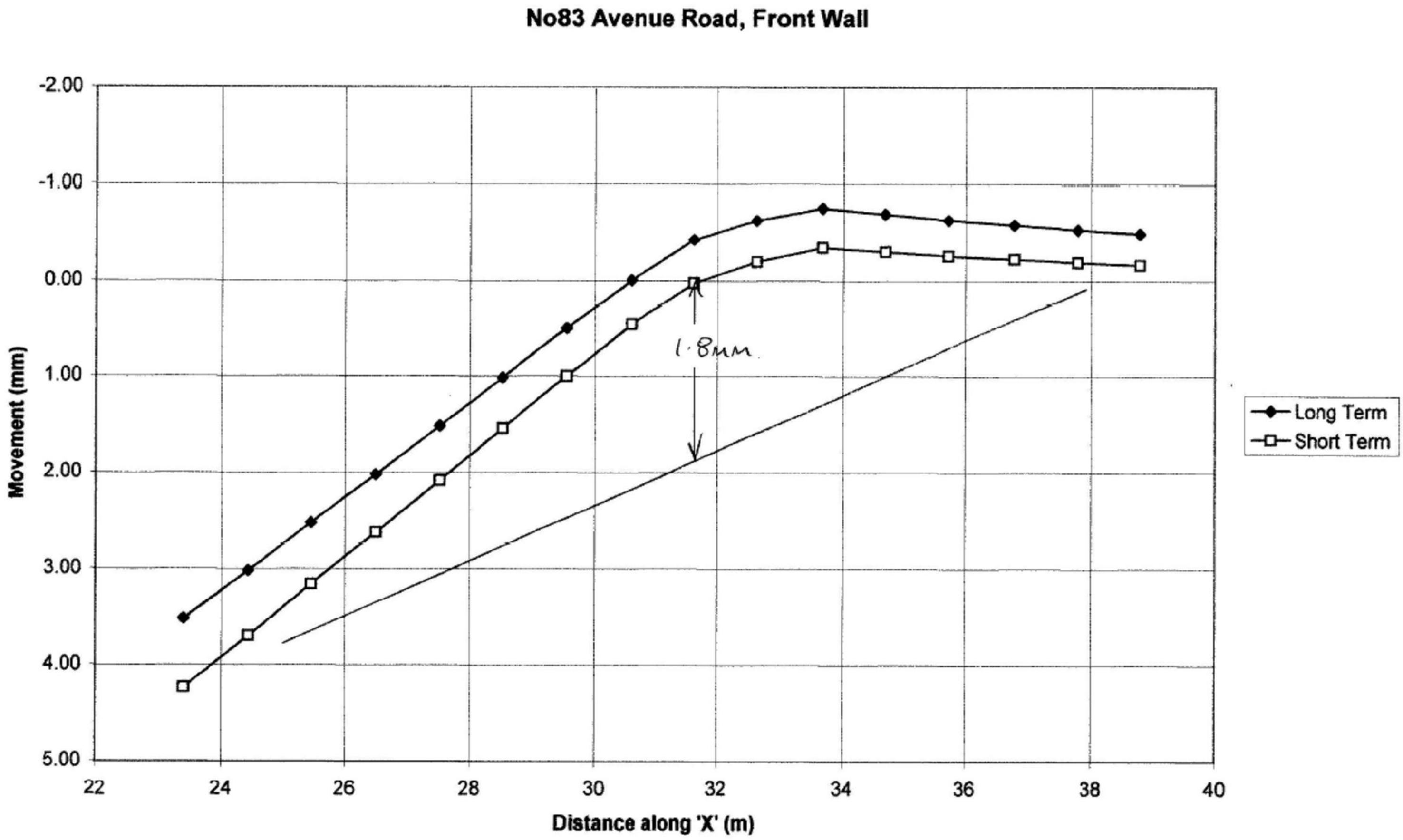


No79 Avenue Road, Garage Rear Wall
Movement (mm)
0.00
1.00
2.00
3.00
4.00
5.00
6.00
-10
-9
-8
-7
-6
-5
-4
-3
-2
Distance along 'X' (m)
Long Term
Short Term



X= -9.1m
X= -2.6m
No 79
No 81
Y
X

Figure 8


No83 Avenue Road, Front Wall
Movement (mm)
Distance along 'X' (m)
-2.00
-1.00
0.00
1.00
2.00
3.00
4.00
5.00
22
24
26
28
30
32
34
36
38
40
1·8mm
Long Term
Short Term

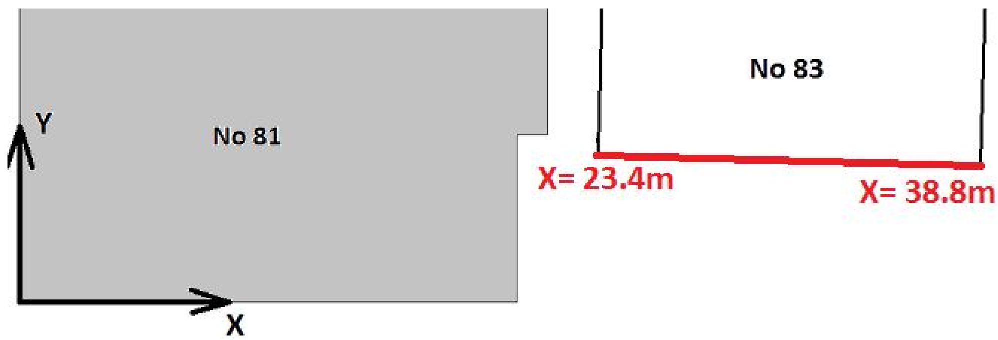
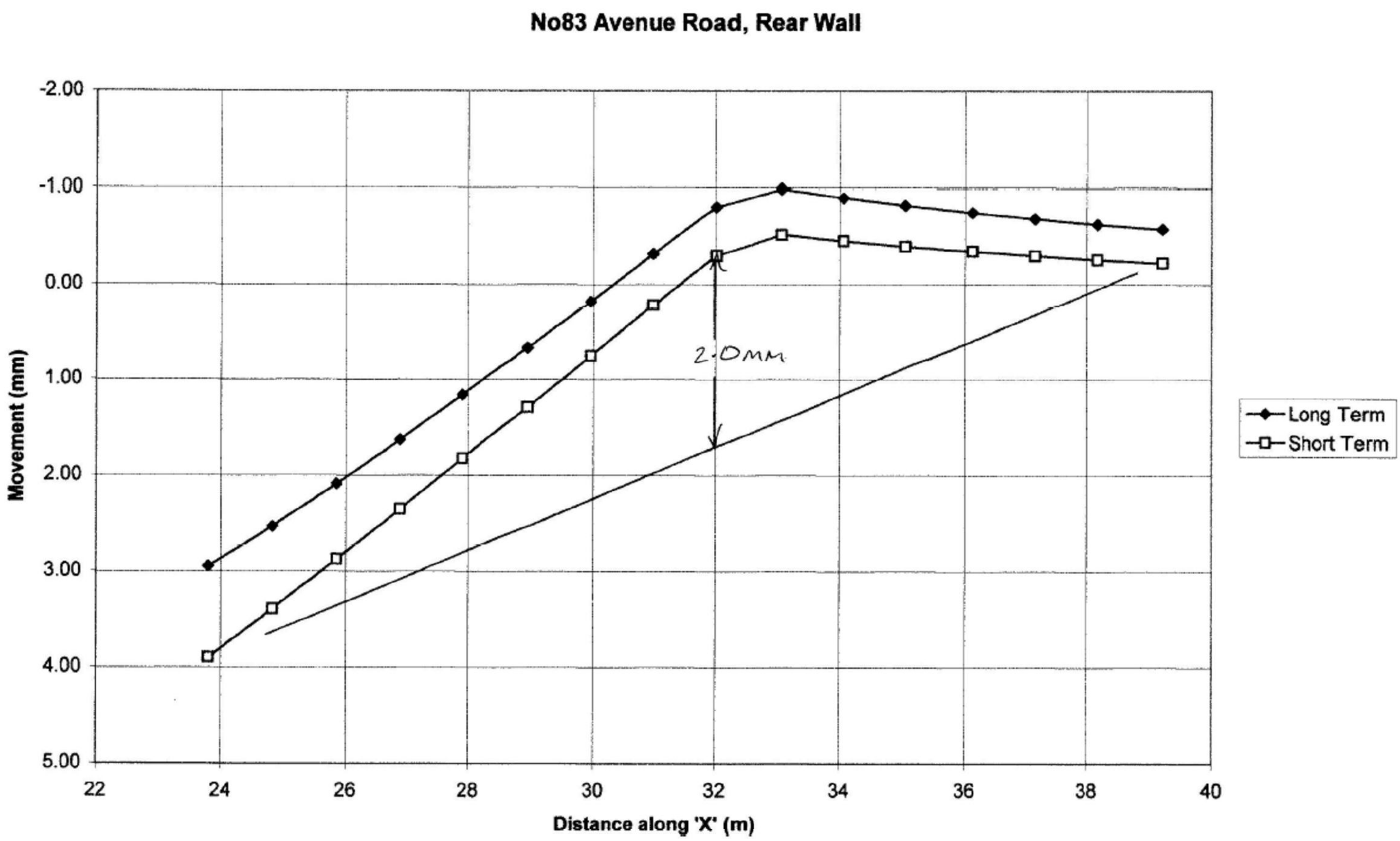
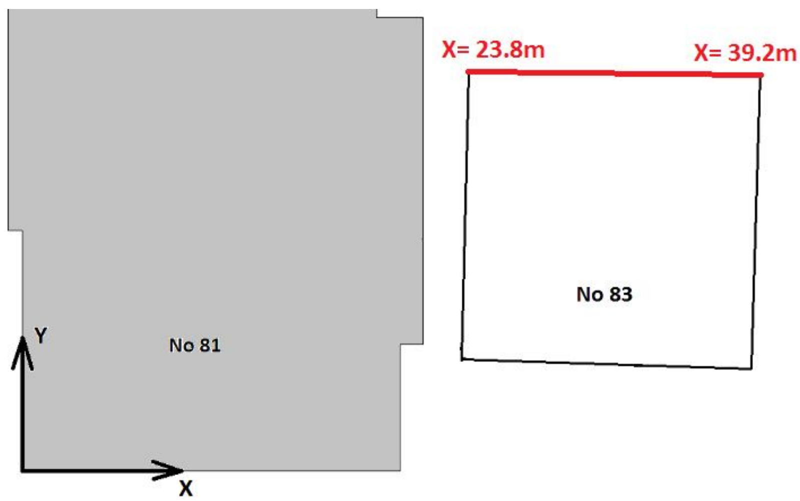

No 81
No 83
Y
X
X= 23.4m
X= 38.8m

Figure 9

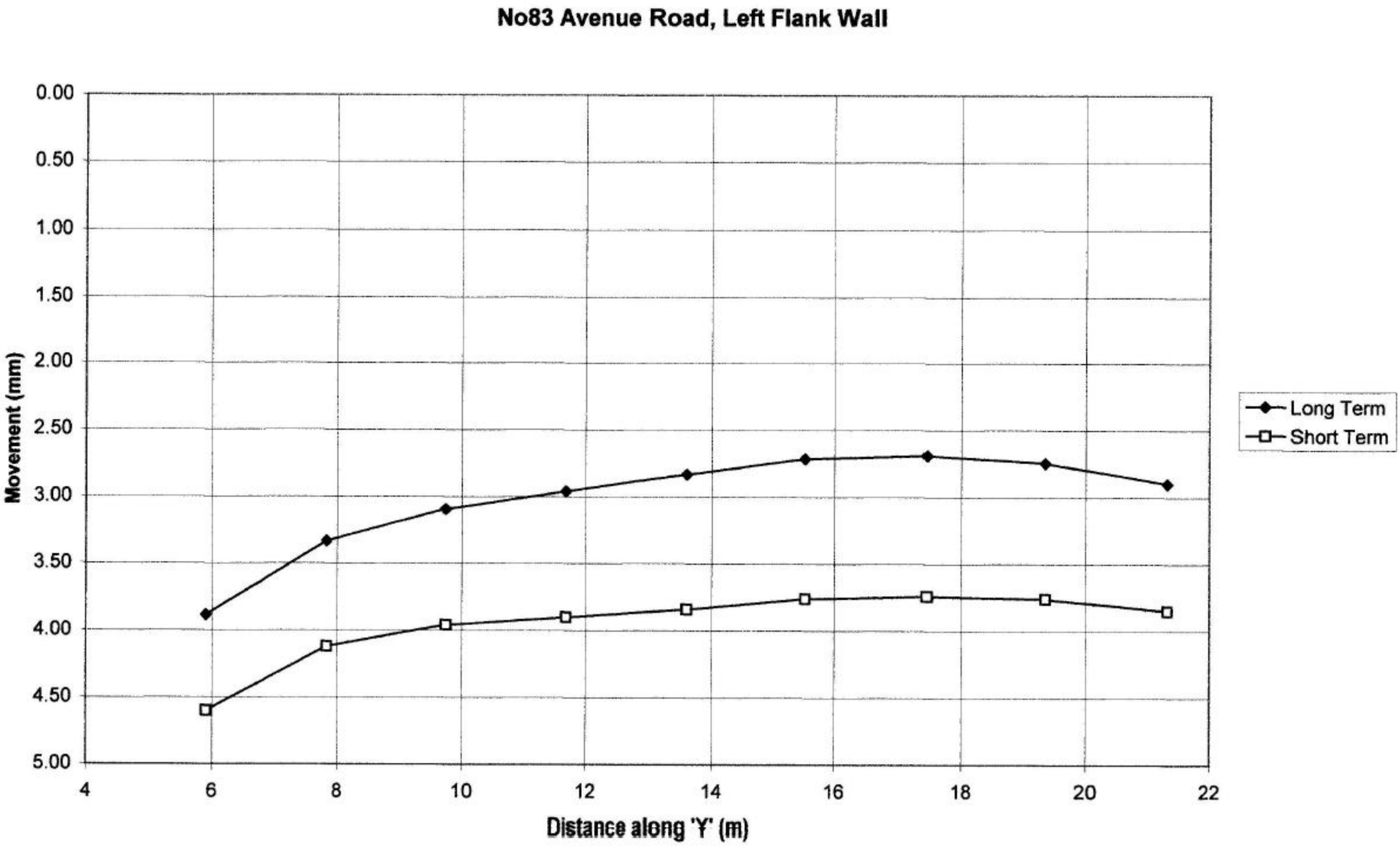


No83 Avenue Road, Rear Wall
-2.00
-1.00
0.00
1.00
2.00
3.00
4.00
5.00
Movement (mm)
22
24
26
28
30
32
34
36
38
40
Distance along 'X' (m)
2.0mm
Long Term
Short Term

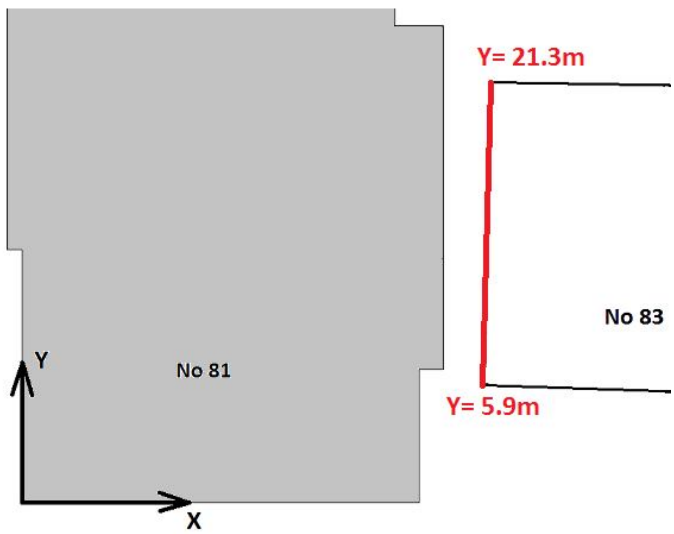


X= 23.8m
X= 39.2m
No 83
No 81
Y
X

Figure 10

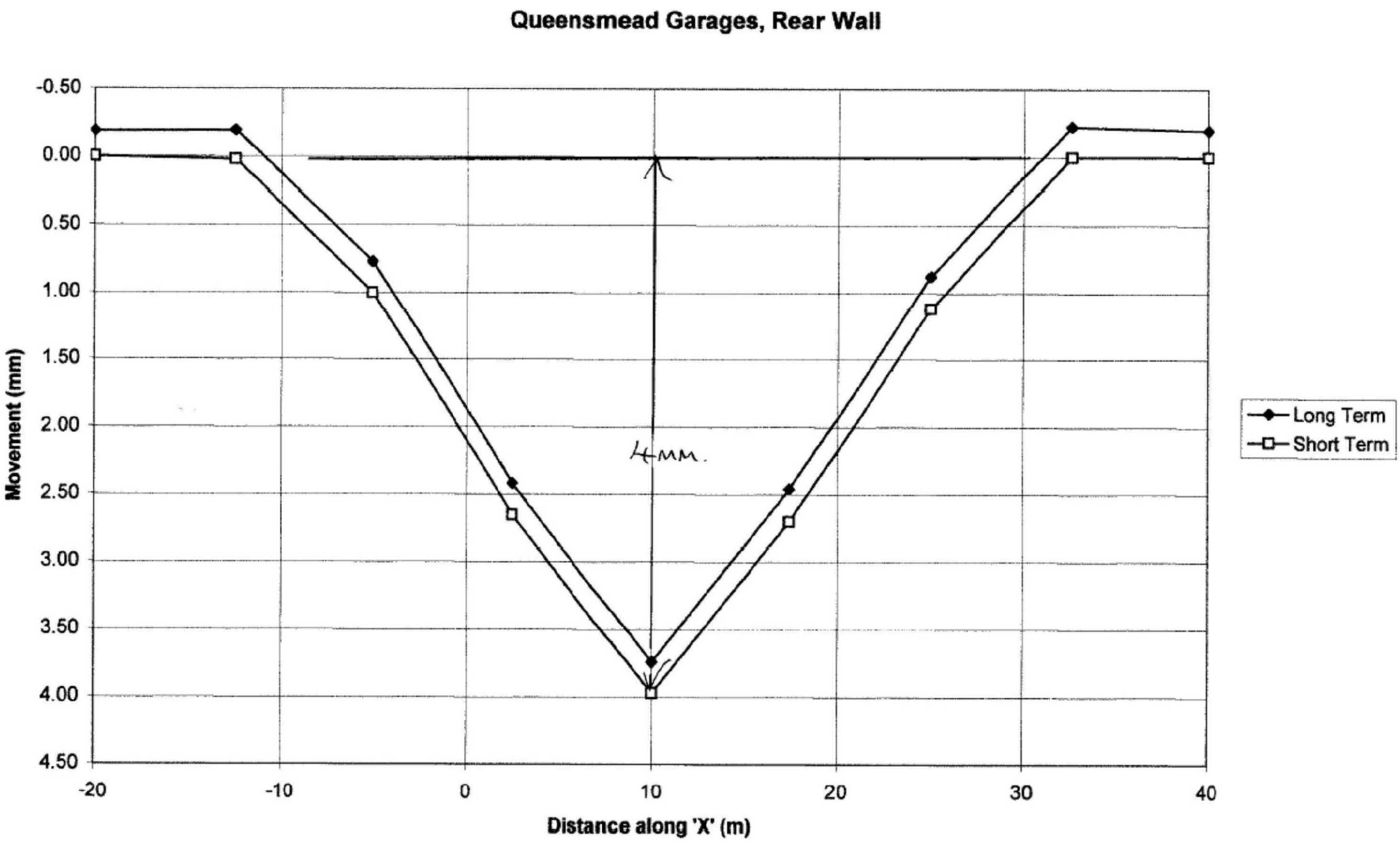


No83 Avenue Road, Left Flank Wall
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
5.00
Movement (mm)
4
6
8
10
12
14
16
18
20
22
Distance along 'Y' (m)
Long Term
Short Term

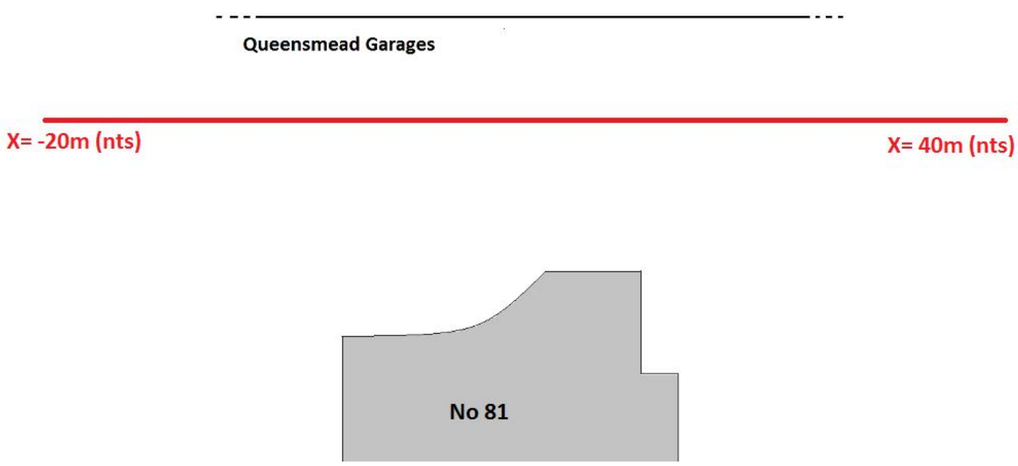


Y= 21.3m
No 83
Y
No 81
Y= 5.9m
X

Figure 11

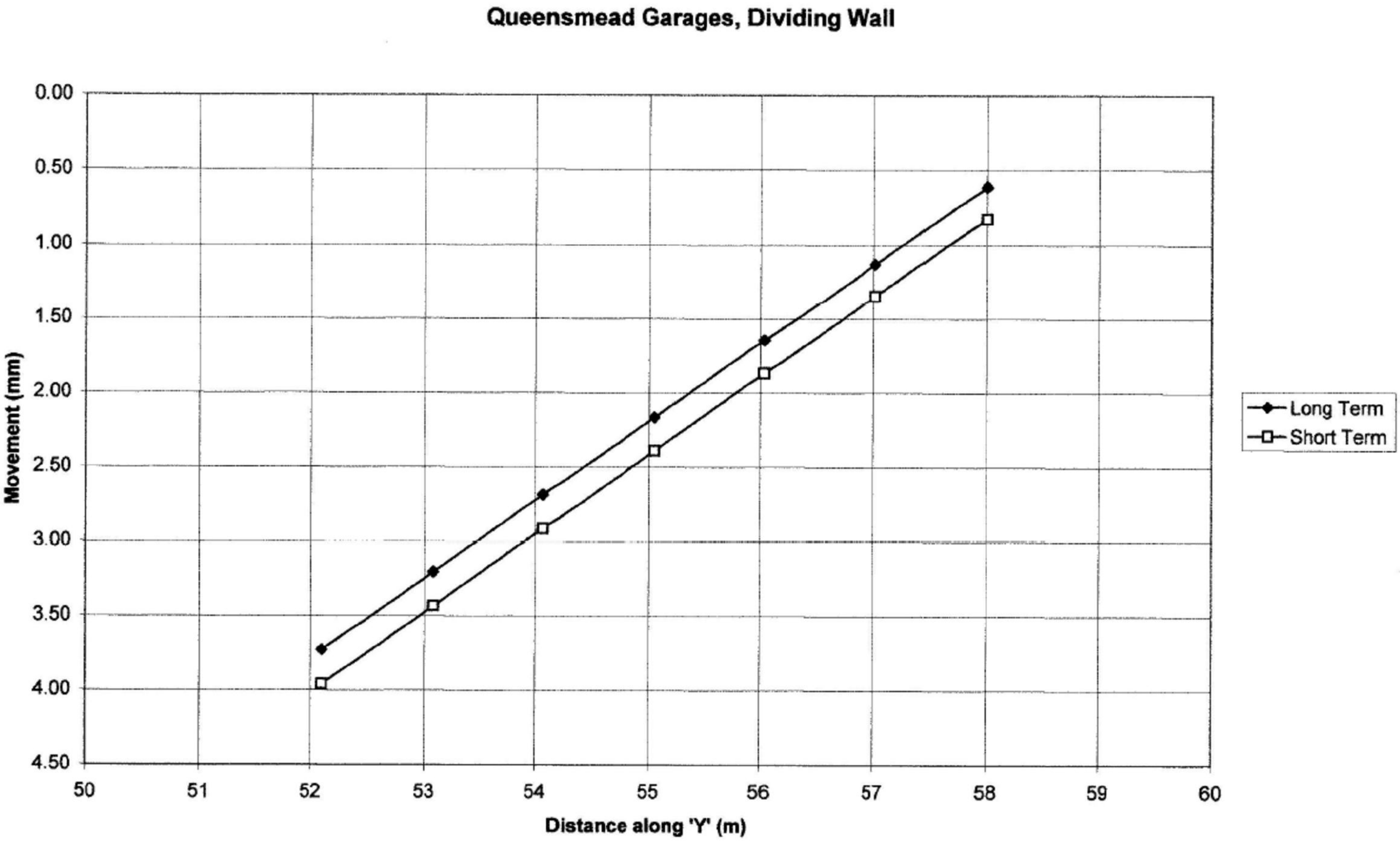


Queensmead Garages, Rear Wall
Movement (mm)
-0.50
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
-20
-10
0
10
20
30
40
Distance along 'X' (m)
Long Term
Short Term
4mm.

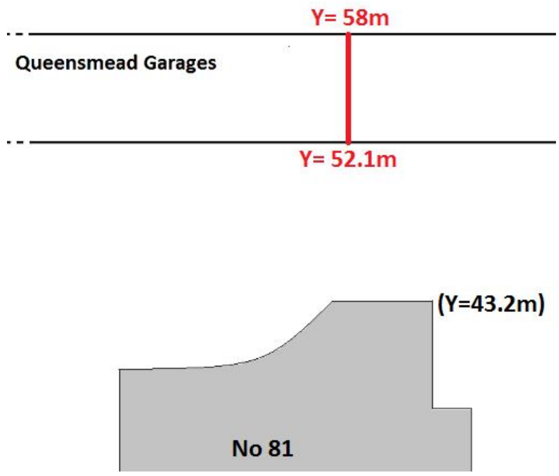


Queensmead Garages
X= -20m (nts)
X= 40m (nts)
No 81

Figure 12



Queensmead Garages, Dividing Wall
Movement (mm)
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
50
51
52
53
54
55
56
57
58
59
60
Distance along 'Y' (m)
Long Term
Short Term



Y= 58m
Queensmead Garages
Y= 52.1m
(Y=43.2m)
No 81

Figure 13

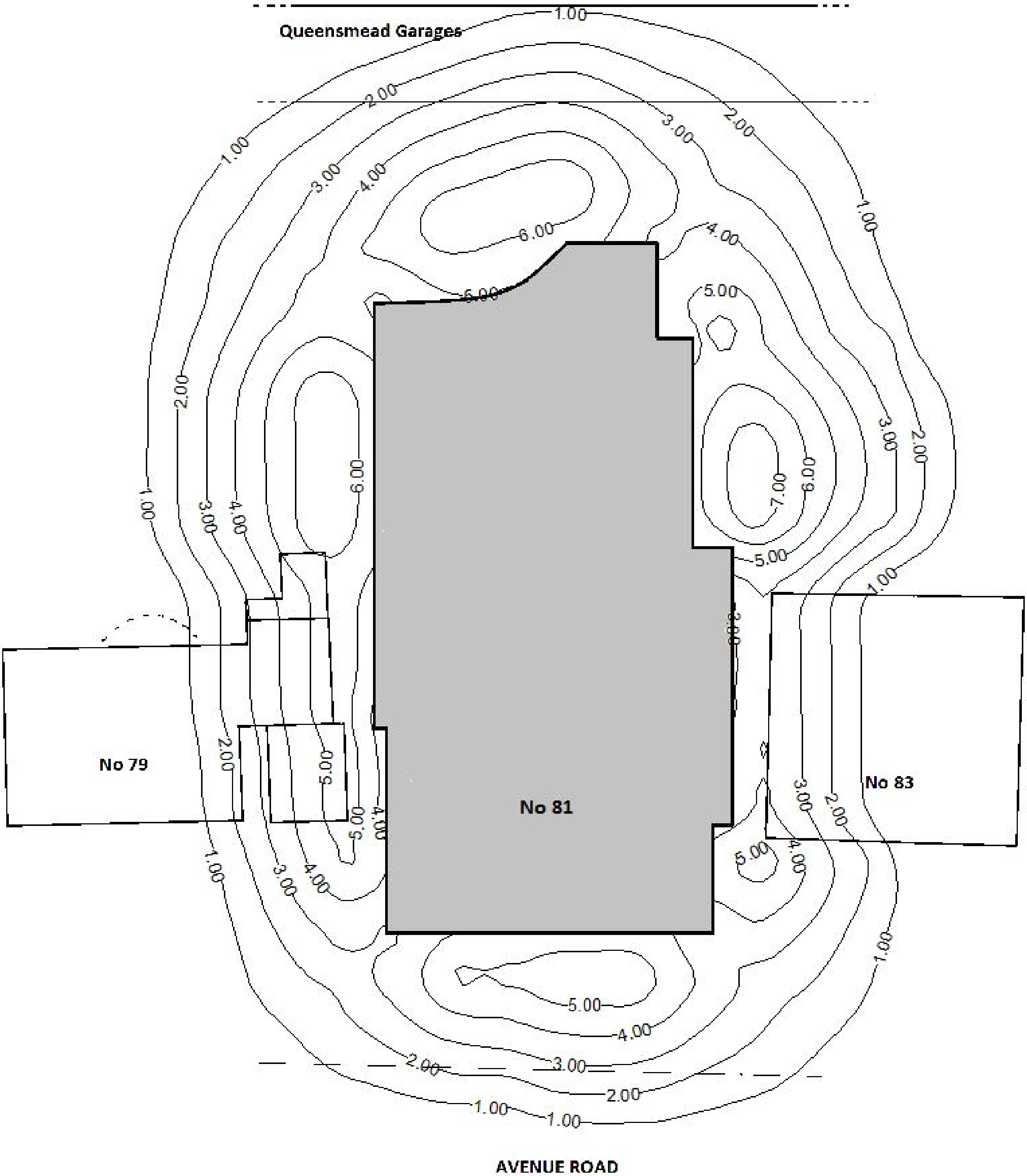


Figure 14
(Short-term ground settlement contours)