4.5.5 Results

The Gas Screening Value is calculated as follows:

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

On-site monitoring has shown emissions of methane in air of 0.0% and carbon dioxide in air of up to 2.5% recorded during the monitoring visits. The maximum borehole flow rate was 0.0 l/h.

As such the Gas Screening Value for methane at site is 0.0 l/h and the Gas Screening Value for carbon dioxide at site is also 0.0 l/h. As such the worst case value for the site would be less than 0.01 litres of gas per hour.

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

These results equate to a Characteristic Situation 1, which requires no special precautions at site.

Employing the NHBC `traffic light' characterisation system, the site would be classified as Green in accordance with CIRIA Report C665. Table 8.7 using the Gas Screening Value for methane and carbon dioxide and as such gas prevention measures would not be considered necessary for the site.

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

5.0 FOUNDATION DESIGN

5.1 General

It is proposed to construct a new single storey basement to approximately 3.30m below ground level beneath the existing property at 36 Heath Drive, London, NW3 7SD, together with 5 light wells, rear and side extensions at ground and first floor, roof re-modelling and internal refurbishments. Exact details of the finalised structure, layout and loadings were not available at the time of preparation of this report, although foundation loads are expected to moderate and of the order 100-150kN/m^2, whilst ground slab loadings are expected to be of the order of 10-15kN/m^2.

5.2 Conventional Spread Foundations

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

Based on the ground and groundwater conditions encountered in the boreholes, it should be possible to support the proposed new development on conventional basement raft foundations taken down below the made ground and any weak superficial soils and placed in the stiff weathered London Clay deposits encountered at a depth of about 1.10m below existing ground level.

Such foundations placed within natural soils may be designed to allowable net bearing pressures of the order of 200kN/m^2 at 2.00m depth increasing linearly to about 250kN/m^2 at 3.00m depth in order to allow for a factor of safety of about three against general shear failure.

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

In addition, foundations may need to be taken deeper should they be within the zones of influence of either existing or recently felled trees and any proposed tree planting. The depth of foundation required to avoid the zone likely to be affected by the root systems of trees is shown in the recommendations given in NHBC Standards, Chapter 4.2, April 2003, "Building near Trees" and it is considered that this document is relevant in this situation.

5.3 Piled Foundations

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

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

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

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

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

5.4 Retaining Walls

It is proposal to construct a new basement at the site together with five light wells at the rear and side extensions at ground and first floor, roof re-modelling and internal refurbishments. Exact details of the structure, layout and loadings were not available at the time of preparation of this report.

The results of the investigation indicated that made ground occurs to a depth of up to 1.40m below existing ground level. This is followed by stiff becoming very stiff clay deposits down to a depth of at least 12.0m below ground level. The general groundwater level beneath the site lies at a depth of about 2.0m below existing ground level.

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

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

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

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

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

Founding Material	Depth to top (m)	Description	Critical Angle of Shearing Resistance (°) (Φ'_{crit})[1]	Coefficient active pressure (Ka)	Coefficient passive resistance (Kp)
London Clay	1.40 to 1.60	Stiff becoming very stiff silty CLAY	21	0.45	2.2

Table A. Summary of design parameters for proposed basement foundation

Notes:

1. Calculated using guidance from BS8002

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

The main phase of uplift or heave will come immediately following the excavation of the basement when the greatest elastic rebound of the soil (caused by the loss of the overburden pressure) will occur. Heave can be reduced by proceeding with the excavation in stages and observing and recording any movement that occurs over a set period of time. It may therefore be advantageous to delay the construction until an adequate proportion of the uplift has occurred. Once this monitoring period has elapsed and a suitably qualified engineer is confident that the majority of uplift has occurred, basement construction can commence.

Should a basement raft foundation be adopted, then there is also a potential for some total and differential settlement and consequently the foundation should be constructed on a 300mm thick proof rolled layer of gap graded granular fill and be of a sufficient stiffness to be capable of allowing for a minimum of two linear metres loss of support. Any service entry and exit points should be designed to accommodate settlement by the use of sealed flexible joints.

These processes and other ways of dealing with ground movements are described at length in BS8004 (British Standard Code of Practice for Foundations).

5.5 Basement Floor Slab

Due to the potential for swelling within the natural cohesive soils it is recommended that the ground slabs should be designed as being fully suspended.

5.6 Excavations

Shallow excavations for foundations and services are likely to require nominal side support in the short term and groundwater is unlikely to be encountered in significant quantities once any accumulated surface water within the made ground has been removed. Deeper and longer excavations below approximately 1.5m below existing ground level will require close side support and some inflows of groundwater are likely to be encountered.

The results of the in-situ permeability test indicated the apparent permeability of the materials at the site to be of the order of 2.4×10^{-7} m/sec, assuming that the cohesive soils are effectively impermeable This value lies approximately midway in the range of published data for fissured and weathered clays and / or silty sands and is classed as very low to low permeability material of poor drainage characteristics.

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

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

5.7 Chemical Attack on Buried Concrete

The results show the natural soil samples to have water soluble sulphate contents of up to 1.41g/litre associated with slightly acidic to 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 unless precautions are taken. The final design of buried concrete according to Tables C1 and C2 of BRE Special Digest 1:2005 should be in accordance with Class DS-2 conditions.

In addition, segregations of gypsum were noted within the London Clay and scattered small gypsum crystals were also noted at depth. Consequently, it is considered that any buried concrete at depth may be attacked by such sulphates in solution and that it would be prudent to design any such deep buried concrete in accordance with full Class DS-2 conditions.

p.p. SITE ANALYTICAL SERVICES LIMITED

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Geotechnical Engineer

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Senior Geotechnical Engineer

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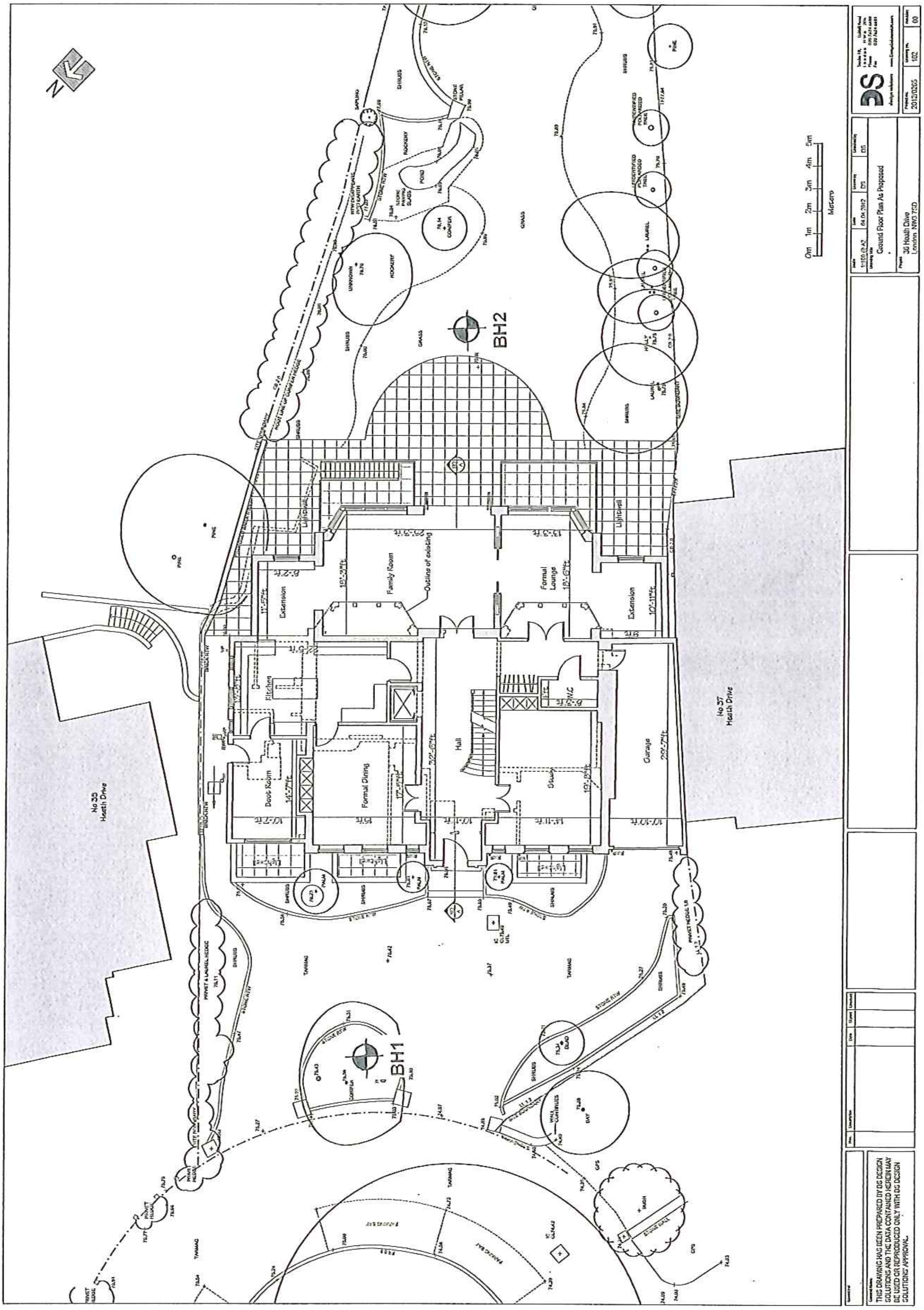
Site Analytical Services Ltd.

LOCATION: 36 Heath Drive, Hampstead, London NW3 7SD		REF:	12/19442
		FIG:	1
TITLE: Site Sketch Plan	DATE: June 2012	SCALE:	NTS



APPENDIX `A'

Borehole / Trial Pit Logs

Site Analytical Services Ltd.

Site	Borehole Number
36 HEATH DRIVE, HAMPSTEAD, LONDON, NW3 7SD	BH1

Boring Method	Casing Diameter	Ground Level (mOD)	Client	Job Number
CONTINUOUS FLIGHT AUGER	100mm cased to 0.00m		MR T SANJAY WADHWANI	1219442
	Location	**Dates**	**Engineer**	**Sheet**
	TQ 255 855	26/06/2012	MARTIN REDSTON ASSOCIATES	1/2

Depth (m)	Sample / Tests	Casing Depth (m)	Water Depth (m)	Field Records	Level (mOD)	Depth (m) (Thickness)	Description	Legend	Water
						(0.20) 0.20	MADE GROUND - flower bed of topsoil		
0.25	D1					(0.40) 0.60	MADE GROUND - brick and concrete rubble with some grey brown clayey silty sand		
0.50	D2								
0.75	D3					(0.80) 1.40	MADE GROUND - stiff to very stiff dark brown sandy silty clay, fine gravel, ashes and brick fragments		
1.00 1.00-1.30	D4 M1 121/300								
1.50 1.50	D5 V1 140+					(6.10) 7.50	Stiff becoming stiff to very stiff brown and mottled orange brown and veined blue grey silty CLAY with occasional partings of orange brown silty fine sand and occasional small gypsum crystals		
2.00 2.00	D6 V2 116								
2.50 2.50	V3 129 D7								
3.00 3.00	D8 V4 138								
3.50 3.50	D9 V5 140+								
4.00 4.00	V6 140+ D10								
4.50 4.50	D11 V7 140+								
5.00 5.00	D12 V8 140+								
6.00 6.00	D13 V9 140+								
7.00 7.00	D14 V10 140+								
						(2.50) 10.00	Stiff to very stiff dark grey brown fissured silty CLAY with occasional partings of light brown silty fine sand and scattered small gypsum crystals		
8.00 8.00	D15 V11 140+								
9.00 9.00	V12 140+ D16								

Remarks
V = Vane Test - Result in kPa
M = Mackintosh Probe - Blows/Penetration (mm)
D = Disturbed Sample
Groundwater was not encountered during boring

Scale (approx)	Logged By
1:50	JIP

Figure No. 1219442.BH1

Site Analytical Services Ltd.

Site	Borehole Number
36 HEATH DRIVE, HAMPSTEAD, LONDON, NW3 7SD	BH1

Boring Method	Casing Diameter	Ground Level (mOD)	Client	Job Number
CONTINUOUS FLIGHT AUGER	100mm cased to 0.00m		MR T SANJAY WADHWANI	1219442
	Location	**Dates**	**Engineer**	**Sheet**
	TQ 255 855	26/06/2012	MARTIN REDSTON ASSOCIATES	2/2

Depth (m)	Sample / Tests	Casing Depth (m)	Water Depth (m)	Field Records	Level (mOD)	Depth (m) (Thickness)	Description	Legend	Water
10.00	D17						Stiff to very stiff becoming very stiff dark grey brown fissured silty CLAY with occasional partings of light brown silty fine sand and scattered small gypsum crystals		
10.00	V13 140+								
						(2.00)			
11.00	D18								
11.00	V14 140+								
12.00	D19					12.00			
12.00	V15 140+			26/06/2012:DRY					
							Complete at 12.00m		

Remarks	Scale (approx)	Logged By
	1:50	JIP
	Figure No. 1219442.BH1	

Site Analytical Services Ltd.

Site	Borehole Number
36 HEATH DRIVE, HAMPSTEAD, LONDON, NW3 7SD	BH1

Installation Type	Dimensions	Client	Job Number
MONITORING STANDPIPE	Internal Diameter of Tube [A] = 50 mm Diameter of Filter Zone = 100 mm	MR T SANJAY WADHWANI	1219442

Location	Ground Level (mOD)	Engineer	Sheet
TQ 255 855		MARTIN REDSTON ASSOCIATES	1/1

Legend	Water	Instr (A)	Level (mOD)	Depth (m)	Description
					Bentonite Seal
				1.00	Slotted Standpipe
				5.00	Bentonite Seal
				6.00	General Backfill
				12.00	

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					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)
26/06/12				DRY			12.00		DRY	

Instrument Groundwater Observations

Inst. [A] Type : SINGLE STANDPIPE

Date	Instrument [A]			Remarks
	Time	Depth (m)	Level (mOD)	
10/07/12		2.11		Gas and permeabilty readings taken
20/07/12		2.36		Gas readings taken
26/07/12		2.44		Gas readings taken

Remarks
Lockable cover set in concrete
Gas valve fitted

Site Analytical Services Ltd.

Site	Borehole Number
36 HEATH DRIVE, HAMPSTEAD, LONDON, NW3 7SD	BH2

Boring Method	Casing Diameter	Ground Level (mOD)	Client	Job Number
CONTINUOUS FLIGHT AUGER	100mm cased to 0.00m		MR T SANJAY WADHWANI	1219442
	Location	**Dates**	**Engineer**	**Sheet**
	TQ 255 855	26/06/2012	MARTIN REDSTON ASSOCIATES	1/2

Depth (m)	Sample / Tests	Casing Depth (m)	Water Depth (m)	Field Records	Level (mOD)	Depth (m) (Thickness)	Description	Legend	Water
						(0.25) 0.25	MADE GROUND - grass over topsoil		
0.25	D1						Stiff mottled brown, orange brown and grey sandy silty CLAY with some pockets and partings of orange brown silty fine sand and scattered fine to medium flint gravel		
0.50	D2								
0.75	D3								
1.00	D4					(1.35)			
1.00	V1 92								
1.50	D5					1.60			
1.50	V2 104						Stiff becoming stiff to very stiff brown and mottled orange brown and veined blue grey silty CLAY with occasional partings of orange brown silty fine sand and occasional small gypsum crystals		
2.00	D6								
2.00	V3 114								
2.50	V4 124								
2.50	D7								
3.00	D8								
3.00	V5 136								
3.50	D9								
3.50	V6 140					(4.40)			
4.00	V7 140+								
4.00	D10								
4.50	D11								
4.50	V8 140+								
5.00	D12								
5.00	V9 140+								
						6.00			
6.00	D13						Stiff to very stiff dark grey brown fissured silty CLAY with occasional partings of light brown silty fine sand and scattered small gypsum crystals		
6.00	V10 140+								
7.00	D14								
7.00	V11 140+								
8.00	D15					(4.00)			
8.00	V12 140+								
9.00	V13 140+								
9.00	D16								
						10.00			

Remarks
V = Vane Test - Result in kPa
D = Disturbed Sample

Scale (approx)	Logged By
1:50	JIP

Figure No.
1219442.BH2

Site Analytical Services Ltd.

Site 36 HEATH DRIVE, HAMPSTEAD, LONDON, NW3 7SD				**Borehole Number** BH2
Boring Method CONTINUOUS FLIGHT AUGER	**Casing Diameter** 100mm cased to 0.00m	**Ground Level (mOD)**	**Client** MR T SANJAY WADHWANI	**Job Number** 1219442
	Location TQ 255 855	**Dates** 26/06/2012	**Engineer** MARTIN REDSTON ASSOCIATES	**Sheet** 2/2

Depth (m)	Sample / Tests	Casing Depth (m)	Water Depth (m)	Field Records	Level (mOD)	Depth (m) (Thickness)	Description	Legend	Water
10.00 10.00	D17 V14 140+						Stiff to very stiff becoming very stiff dark grey brown fissured silty CLAY with occasional partings of light brown silty fine sand, occasional small claystone nodules and scattered small gypsum crystals		
11.00 11.00	D18 V15 140+			SEEPAGE(1) at 11.00m.		(2.00)			∇1
12.00 12.00	D19 V16 140+			26/06/2012:DRY		12.00			
							Complete at 12.00m		

Remarks	**Scale (approx)** 1:50	**Logged By** JIP
	Figure No. 1219442.BH2	

Site Analytical Services Ltd.

Site	36 HEATH DRIVE, HAMPSTEAD, LONDON, NW3 7SD
Borehole Number	BH2
Installation Type	MONITORING STANDPIPE
Dimensions	Internal Diameter of Tube [A] = 50 mm Diameter of Filter Zone = 100 mm
Client	MR T SANJAY WADHWANI
Job Number	1219442
Location	TQ 255 855
Ground Level (mOD)	
Engineer	MARTIN REDSTON ASSOCIATES
Sheet	1/1

Legend	Water	Instr (A)	Level (mOD)	Depth (m)	Description
					Bentonite Seal
				1.00	
					Slotted Standpipe
				5.00	
					Bentonite Seal
				6.00	
					General Backfill
	∇1				
				12.00	

Groundwater Strikes During Drilling

Date	Time	Depth Struck (m)	Casing Depth (m)	Inflow Rate	Readings 5 min	Readings 10 min	Readings 15 min	Readings 20 min	Depth Sealed (m)
26/06/12		11.00	0.00	SEEPAGE					

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)
26/06/12				DRY			12.00		DRY	

Instrument Groundwater Observations

Inst. [A] Type : SINGLE STANDPIPE

Date	Instrument [A] Time	Depth (m)	Level (mOD)	Remarks
10/07/12		-		No access
20/07/12		1.88		Gas readings taken
26/07/12		2.01		Gas readings taken

Remarks
Lockable cover set in concrete
Gas valve fitted

APPENDIX `B'

In-situ, Laboratory Test and Gas Monitoring Data

Site Analytical Services Ltd.

Ref: 12/19442

PLASTICITY INDEX & MOISTURE CONTENT DETERMINATIONS

LOCATION 36 Heath Drive, London, NW3 7SD

BH/TP No.	Depth m	Natural Moisture %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Passing 425 µm %	Class
BH1	1.50	32	72	24	48	98	CV
	2.50	31	68	24	44	99	CH
	3.50	32	67	24	43	100	CH
BH2	1.00	32	72	24	48	95	CV
	2.00	29	74	27	47	100	CV
	3.00	28	63	23	40	100	CH

Site Analytical Services Ltd.

Ref: 12/19442

SULPHATE & pH DETERMINATIONS

LOCATION 36 Heath Drive, London, NW3 7SD

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	2.00		0.50		6.0	DS-2	100
BH2	1.50		1.41		5.8	DS-2	98

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

Ref: 12/19442

GAS MONITORING

LOCATION 36 Heath Drive, London, NW3 7SD

MONITORING DATE 10th July 2012

BOREHOLE REF:		**BH1**	**BH2**
Methane	(%)	0.0	-
Carbon Dioxide	(%)	2.5	-
Oxygen	(%)	17.5	-
Hydrogen Sulphide	(p.p.m.)	0	-
Carbon Monoxide	(p.p.m.)	0	-
Atmospheric Pressure	(mb)	1005	-
Water Level	(m.bgl)	2.11	-
Oxygen in Air	(%)	21.0	-
Flow	(l/hour)	0.0	-

N.B. Methane Lower Explosive Limit - 5% Gas in Air

Note: No access to location BH2

Table 3

Ref: 12/19442

GAS MONITORING

LOCATION	36 Heath Drive, London, NW3 7SD
MONITORING DATE	20th July 2012

BOREHOLE REF:		**BH1**	**BH2**
Methane	(%)	0.0	0.0
Carbon Dioxide	(%)	2.1	1.4
Oxygen	(%)	17.2	18.8
Hydrogen Sulphide	(p.p.m.)	0	0
Carbon Monoxide	(p.p.m.)	0	0
Atmospheric Pressure	(mb)	1005	1005
Water Level	(m.bgl)	2.36	1.88
Oxygen in Air	(%)	21.0	21.0
Flow	(l/hour)	0.0	0.0

N.B. Methane Lower Explosive Limit - 5% Gas in Air

Table 3a

Ref: 12/19442

GAS MONITORING

LOCATION	36 Heath Drive, London, NW3 7SD
MONITORING DATE	26th July 2012

BOREHOLE REF:		**BH1**	**BH2**
Methane	(%)	0.0	0.0
Carbon Dioxide	(%)	2.2	0.4
Oxygen	(%)	17.7	20.5
Hydrogen Sulphide	(p.p.m.)	0	0
Carbon Monoxide	(p.p.m.)	0	0
Atmospheric Pressure	(mb)	1009	1009
Water Level	(m.bgl)	2.44	2.01
Oxygen in Air	(%)	20.8	20.8
Flow	(l/hour)	0.0	0.0

N.B. Methane Lower Explosive Limit - 5% Gas in Air

Table 3b

Ref: 12/19442

LOCATION 36 Heath Drive, London, NW3 7SD

RISING HEAD PERMEABILITY TEST – BOREHOLES

Borehole Number	BH1	
Initial Groundwater Depth	2.11	m
Length of Test Section	4.00	m
Diameter of Borehole	0.10	m
Test Duration	60	mins
Depth of Water at Commencement of Test	4.04	m
Permeability	2.4×10^{-7}	m/sec

Time Elapsed (mins)	Depth of Water Below top of Casing (m)	Time Elapsed (mins)	Depth of Water Below top of Casing (m)
0.00	4.04	10.00	3.65
1.00	3.96	15.00	3.51
2.00	3.92	20.00	3.40
3.00	3.87	30.00	3.21
4.00	3.83	40.00	3.12
5.00	3.80	50.00	3.05
7.50	3.71	60.00	2.99

Remarks

Intake Factor (F) = 0.275

$$\text{Permeability } (k) = \frac{A}{F(t_2-t_1)} \times \log_e \frac{H_1}{H_2}$$

A = Cross Sectional Area of Borehole
H_1 = Head of Water at Time t_1
H_2 = Head of Water at Time t_2

Appendix B. Ground Movement Assessment

REPORT CONTROL SHEET

36 Heath Drive, London

Damage Category Assessment

Project number P4120

Report number **P4120**
Revision number
Issue number **01**

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1.0 Introduction

In connection with the proposal to redevelop No 36 Heath Drive, London, NW3 7SD, involving the construction of a single-level basement, 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 those properties are Nos 35 and 37 Heath Drive, which lie to the left and right of the site, respectively. The relative locations of these properties are shown below in Figure 1.

Right, left and rear are as viewed from the front of the property on Heath Drive.

The structure of No 37 lies at approximately 4° to that of No 36, for the purposes of this analysis the structures have been treated as parallel. This has no significant effect on the analysis.

The structural engineer for the project is Martin Redston Associates (MRA). A plan of the proposed basement of the property is given below in Figure 2.

Site levels have been provided. It is not stated that these are to Ordnance Datum, but available OS data suggest they may be; for the purposes of the current study the supplied levels are taken as relative to OD.

The site slopes upward gently from front to rear, the respective elevations of the front and rear gardens being approximately 75.5mOD and 76mOD. For the purposes of the current study the external ground level is taken as 76.0mOD. Beyond the limits of the site the ground level is believed to fall from left to right; the ground floor of No 35 being set slightly higher than that of No 36, while No 37 appears to be set approximately 900mm lower.

The existing house is on 3 levels, the upper level being within the roof structure. The neighbouring houses are of a similar height. The three properties are effectively detached, though single-storey additions to Nos 36+37 do appear to be very closely located, if not connected, along the property boundary.

The existing building does not have a basement. The proposed basement is to be constructed beneath, and extending outside the footprint of, the existing building, following support of the main structure. It will be excavated within bored-pile walls along the sides, and open cut along the front and rear, with the remaining existing walls being supported on underpins and temporary propping.

It is understood that the pile walls have yet to be designed, and in the absence of definitive information, and for the purposes of this analysis only, the pile wall depth will be taken as 1.4 x adjacent dig depth, calculated from existing ground level, or the estimated depth required to carry the imposed vertical load, whichever is greater.

It is understood that the construction of the basement to No 36 will involve excavation from an existing internal floor level of approximately 76.1mOD to a general level of approximately 72.1mOD (4m depth below existing ground floor level).

It is not clear whether No 35 Heath Drive has a basement, but it is understood that No 37 does not. For the purposes of the current study it is conservative to assume that neither of the neighbouring buildings have basement structures.

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

2.0 Information Provided

The following relevant information has been provided for use in these calculations:-

i) SAS Borehole logs dated June 2012, and laboratory test results.

ii) MRA Drawing 12.302/D-01 and D-02 (Proposed and existing loads).

iii) Devilfish Design Ltd drawings D032.00-.02, .05, .10-.12, .19-.22, .25, .30 - .32.

iv) Email correspondence SAS-AGE dated 3/11/15 to 7/12/15.

No 35

No 36 (Front)

No 37

Figure 1 - Location

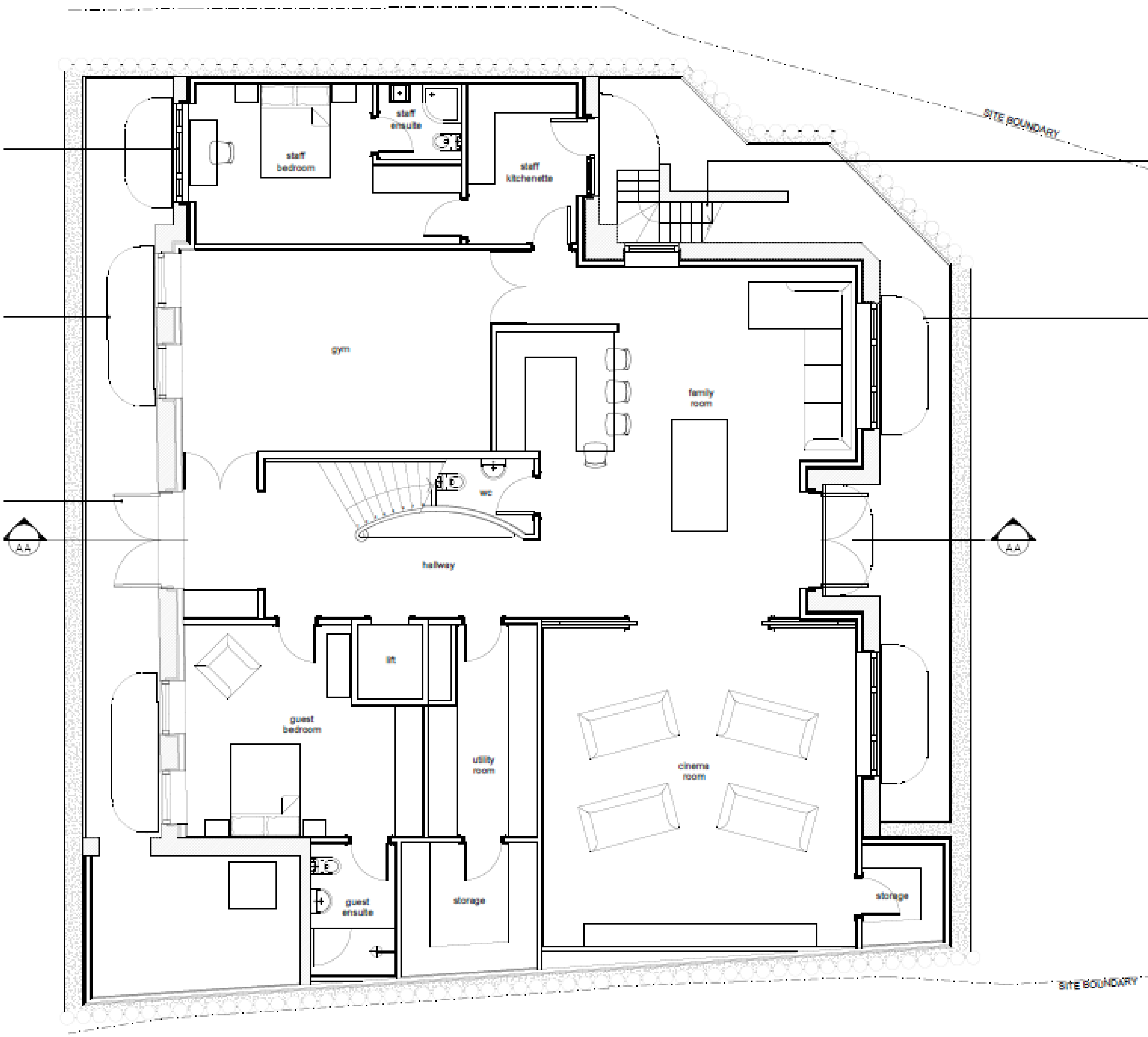


Figure 2 –Proposed Basement Plan (extract of Devilfish Design drawing D032.19)

3.0 Anticipated Ground Conditions

The average existing ground level in the area of the proposed basement is taken to be at approximately 76mOD.

The published geological map (BGS 1:50 000 sheet 256: North London) indicates the site to lie on the London Clay (silty clay), with the propensity for the formation of Head. The edge of the overlying Claygate Beds outcrop is indicated to lie very close to the site.

On the basis of the published mapping the base of the London Clay is anticipated to lie at approximately –18mOD. This translates to approximately 94m depth.

A ground investigation was undertaken at the site in June 2012 (Item ‘i’ in Section 2 above). This comprised 2No continuous-flight auger boreholes: BH1 to 12m depth in the front driveway of No 36, and BH2 to 12m depth in the rear garden. Commencement levels for the two boreholes can be interpreted as 75.3mOD and 76.0mOD respectively.

The boreholes confirmed 1.4m depth of Made Ground at the front of the house, overlying London Clay. At the rear, Made Ground was not encountered, but the London Clay was overlain by 1.6m of material considered to be Head deposits.

The natural soils (Head and London Clay) were described as ‘stiff’, passing down to ‘very stiff’ with depth, in both locations.

For the purposes of this report, the ground level in the area of the basement is taken to be 76.0mOD, and the top of the London Clay is taken to lie at 74.4mOD.

For the purposes of the current study the foundation depth of the existing house is taken to be 1.6m (74.4mOD). No 35 is taken to be founded at a similar level, while No 37 is taken to be founded at 73.4mOD.

No groundwater was encountered in BH1 during drilling, but a seepage was struck in BH2 at 11m depth. Water-monitoring standpipes were installed in both boreholes, with response zones from 1m to 5m depth in both cases. Subsequent monitoring on 26 July 2012 (4 weeks after installation) indicated water levels of 2.44m bgl and 2.01mbgl in Bhs 1+2 respectively. These were the last in a sequence of 3 readings over the previous weeks, and showed gradually falling water levels. It is considered possible that the standpipes had been filled for permeability testing and may not have reached equilibrium; further monitoring is recommended if the instruments are still accessible.

On the basis of the above, and for the purposes of this analysis only, the soil sequence at the proposed basement site is taken to be:-

Ground Level (general):- 76.0mOD
Top of London Clay:- 74.4mOD (1.6mbgl)
Base of London Clay:- -18mOD.

The Made Ground lies above proposed excavation depth, therefore it does not influence ground movements in response to the proposed works and will not be considered in detail.

No determinations of bulk clay strength were carried out in the London Clay. On the basis of previous experience and published data the following undrained strength profiles have been adopted for the analysis:-

From the top of London Clay at 74.4mOD to 20m depth (54.4mOD):-
$Su = 45 + 7z_1$ (kPa).

Where z_1 is the depth below the top of the London Clay.

From 20m depth (54.4mOD) to the base of the London Clay at –18mOD:-
$Su = 185 + 3.5z_2$ (kPa)

Where z_2 is the depth below 54.4mOD.

The use of a bilinear profile reduces the possibility of excessive strengths and stiffnesses being predicted at depth.

4.0 Loads

Existing and proposed basement loads have been provided by the engineer (Item ‘ii’ in Section 2 above). It is understood that the existing loads will persist through the basement excavation stage, transferred by propping to the excavation level.

Excavation from existing ground 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 BS5975:2008 and current good practice. Inadequate propping of walls 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.

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 :-
From the top of London Clay at 74.4mOD to 20m depth (54.4mOD):-
$E_{uo} = 69.8 + 10.85z_1$ (MPa)
$E'_{o} = 55.8 + 8.7z_1$ (MPa)

Where z_1 is the depth below the top of the London Clay.

From 20m depth (54.4mOD) to the base of the London Clay at –18mOD:-
$E_{uo} = 287 + 5.4z_2$ (MPa)
$E'_{o} = 230 + 4.3z_2$ (MPa)

Where z_2 is the depth below 54.4mOD.

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 piles
iii) Vertical and horizontal movement due to deflection of piles, following removal of support from in front of the piles 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. Therefore in this report short-term movements are calculated using the CIRIA data, and subsequent long-term movement is calculated using PDISP.

The CIRIA plots do not describe the ground movement outside open cut excavations (as proposed at the front and rear of the property). No buildings that are likely to be damaged by the excavations lie to the front and rear, therefore the ground movement in these areas is not considered in the following analysis.

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 BS5975 and BS8002.

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 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 35 Heath Drive, Rear Wall.

The front wall of No 35 is further-removed from the proposed excavation than the rear wall, and therefore can be taken to suffer a lesser degree of damage, by inspection.

5.4.1 Vertical Movement

Profiles of short- and long-term vertical ground movement along the rear wall of No 35 Heath Drive have been calculated, as described above, and plotted in Figure 4.

The wall is taken to be approximately 10.4m long and approximately 6m high, above ground level. It lies in the position shown on the plan in Figure 4. The wall lies oblique to the co-ordinate axes, but locations along the wall are adequately described by the X-co-ordinate.

The analysis indicates a maximum overall tilt of approximately 3.7mm over the 10.4m length of the wall. This equates to a whole-wall gradient of less than 1 in 2800. 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 1.2mm over the 10.4m 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 3 (Ref 2 Figure 6) a horizontal strain/limiting tensile strain ratio of 0.84 is obtained, indicating that a horizontal strain of 0.063% is acceptable for a 'very slight' category of damage. This analysis does not take account of the stiffness of the wall; the result is therefore 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 7 is predicted to be 0.064%, extending approximately 8.4m from the excavation, and therefore affecting the proximal 3.6m or so of the rear wall of No 35. This level of horizontal strain is greater than the 0.063% 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 0.063% to 0.138%. However, the analysis does not take into account the stiffness of the wall in the horizontal or vertical directions, and is conservative in this respect.

It is therefore considered that the predicted level of damage to this wall can be taken as 'very slight'.

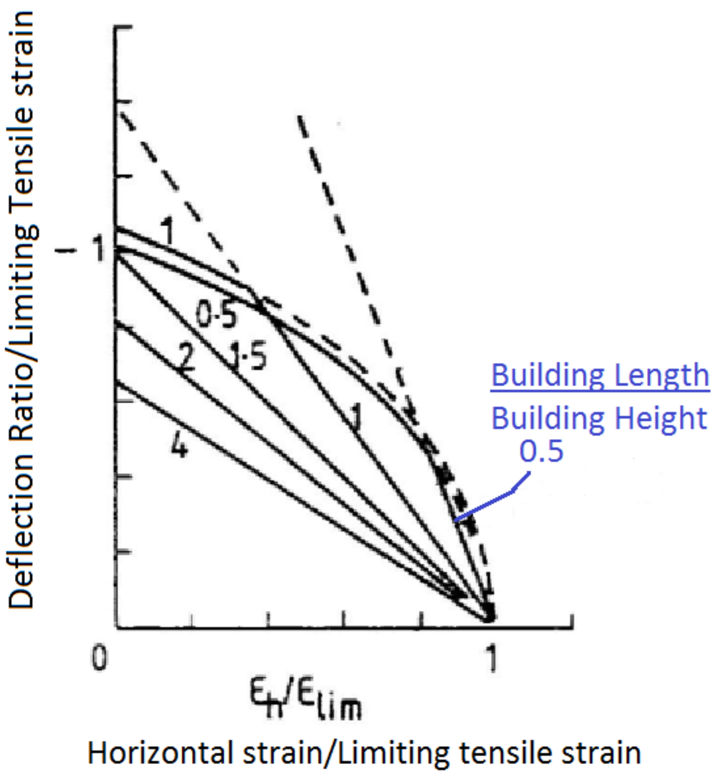


Figure 3 (from Ref 2)

5.5 Predicted movement – No 35 Heath Drive, main right flank wall.

5.5.1 Vertical Movement

Profiles of short- and long-term vertical ground movement along the right flank wall of No 35 Heath Drive have been calculated and plotted in Figure 5.

This wall is taken to be 11.6m long and 6m high, above ground. It lies in the position shown on the plan in Figure 5. The wall lies oblique to the co-ordinate axes, but locations along the wall are adequately described by the Y-co-ordinate.

The analysis indicates a maximum overall tilt of approximately 1.5mm over the 11.6m length of the wall. This equates to a whole-wall gradient of less than 1 in 7700. 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 1.1mm over the 11.6m 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 0.13. By reference to Figure 3 (Ref 2 Figure 6) a horizontal strain/limiting tensile strain ratio of 0.86 is obtained, indicating that a horizontal strain of 0.064% is acceptable for a 'very slight' category of damage. This analysis does not take account of the stiffness of the wall; the result is therefore conservative in this respect.

5.5.2 Horizontal Movement

From Section 5.3 above, the greatest average horizontal ground strain adjacent to the excavation at No 36 is predicted to be 0.064%. Due to the orientation of this flank wall, the anticipated horizontal strain along the plane of the wall can be expected to be significantly less than 0.064%, and therefore significantly less than the limit for 'very slight' damage calculated above.

The predicted damage category for this wall, is therefore 'very slight' or less, as defined in Ref 2.

5.6 Predicted movement – No 35 Heath Drive, minor right flank wall.

Profiles of short- and long-term vertical ground movement along the right flank wall of the single-storey extension to No 35 Heath Drive have been calculated and plotted in Figure 6.

The wall is taken to be 6.1m long and 3m high, above ground level. It lies in the position shown on the plan in Figure 6. The wall lies oblique to the co-ordinate axes, but locations along the wall are adequately described by the Y-co-ordinate.

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

The maximum wall distortion (Delta – as defined by Burland, Ref 2) is 0.5mm over the 6.1m length of the wall. The potential for significant horizontal ground strain along the line of this wall, resulting from the excavation at No 36, is very slight by inspection, therefore the vertical distortion of the wall can be taken as negligible by inspection.

The predicted damage category for this wall, is therefore 'very slight' or less, as defined in Ref 2.

5.7 Predicted movement – 37 Heath Drive, extension flank wall.

The general ground level at No 37 is understood to be approximately 900mm lower than the adjacent land at No 36.

The status of the flank wall of the single-storey extension on the left side of No 37 is not known; it appears to lie very close to the property boundary but underpinning is not proposed here so it has been assumed that the wall is separate from, but very close to, the No 36 building.

The structure of No 37 lies at approximately 4º to that of No 36, for the purposes of this analysis the structures have been treated as parallel, this has no significant effect on the analysis.

5.7.1 Vertical Movement

Profiles of short- and long-term vertical ground movement along the left flank wall of the extension to No 37 Heath Drive have been calculated, as described above, and plotted in Figure 7.

The wall is taken to be 14.4m long and 3m high, above ground. It lies in the location shown on the plan in Figure 7.

The analysis indicates a maximum overall tilt of approximately 0.5mm over the 14.4m length of the wall. This is negligible.

The maximum wall distortion (Delta – as defined by Burland, Ref 2) is 0.5mm over a 10.8m 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 0.06 By reference to Figure 3 (Ref 2 Figure 6) a horizontal strain/limiting tensile strain ratio of 0.92 is obtained, indicating that a horizontal strain of 0.069% is acceptable for a 'very slight' category of damage. This analysis does not take account of the stiffness of the wall; the result is therefore conservative in this respect.

5.7.2 Lateral movement.

Due to the nature of the proposed works no significant horizontal strain along the line of this wall is anticipated as a result of the excavation. However the piling works are noted to be very close to the line of this wall and therefore care must be taken during construction, and a hit-one miss-two (or more) pile construction sequence is recommended.

With the adoption of suitable precautions, the predicted damage category for this wall can therefore be taken as 'very slight' or less, as defined in Ref 2.

5.8 Predicted movement – 37 Heath Drive, main left flank wall.

The general ground level at No 37 is understood to be approximately 900mm lower than the adjacent land at No 36.

The structure of No 37 lies at approximately 4º to that of No 36, for the purposes of this analysis the structures have been treated as parallel, this has no significant effect on the analysis.

Profiles of short- and long-term vertical ground movement along the main left flank wall of No 37 Heath Drive have been calculated, as described above, and plotted in Figure 8.

The wall is taken to be 14.9m long and 6m high, above ground. It lies in the location shown on the plan in Figure 8.

The analysis indicates a maximum overall tilt of approximately 0.5mm over the 14.9m length of the wall. This is negligible.

The maximum wall distortion (Delta – as defined by Burland, Ref 2) is 0.5mm over the 14.9m length of the wall. This is a lesser distortion than that predicted for the extension flank wall in Section 5.7 above, and the main wall is more robust. Therefore, by inspection the predicted damage category for this wall can be taken as 'very slight' or less, as defined in Ref 2.

5.9 Predicted movement – No 37 Heath Drive, Front Wall.

The general ground level at No 37 is understood to be approximately 900mm lower than the adjacent land at No 36.

The structure of No 37 lies at approximately 4º to that of No 36, for the purposes of this analysis the structures have been treated as parallel, this has no significant effect on the analysis.

5.9.1 Vertical Movement

Profiles of short- and long-term vertical ground movement along the front wall of No 37 Heath Drive have been calculated, as described above, and plotted in Figure 9.

The wall is taken to be approximately 7.7m long and approximately 6m high, above ground level. It lies in the position shown on the plan in Figure 9.

The analysis indicates a maximum overall tilt of approximately 3.4mm over the 7.7m length of the wall. This equates to a whole-wall gradient of less than 1 in 2200. 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 0.4mm over the 7.7m 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 0.07. By reference to Figure 3 (Ref 2 Figure 6) a horizontal strain/limiting tensile strain ratio of 0.94 is obtained, indicating that a horizontal strain of 0.071% is acceptable for a ‘very slight’ category of damage. This analysis does not take account of the stiffness of the wall; the result is therefore 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 7 is predicted to be 0.064%. This is less than the 0.071% limit for very slight damage calculated above, therefore the predicted level of damage to this wall can be taken as ‘very slight’ or less, as defined in Ref 2.

5.10 Predicted movement – No 37 Heath Drive, Rear Wall.

The general ground level at No 37 is understood to be approximately 900mm lower than the adjacent land at No 36.

The structure of No 37 lies at approximately 4º to that of No 36, for the purposes of this analysis the structures have been treated as parallel, this has no significant effect on the analysis.

5.10.1 Vertical Movement

Profiles of short- and long-term vertical ground movement along the rear wall of No 37 Heath Drive have been calculated, as described above, and plotted in Figure 10.

The wall is taken to be approximately 15m long and approximately 6m high, above ground level. It lies in the position shown on the plan in Figure 10.

The analysis indicates a maximum overall tilt of approximately 2.9mm over the 15m length of the wall. This equates to a whole-wall gradient of less than 1 in 5000. 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 1.5mm over the 15m 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 0.13. By reference to Figure 3

(Ref 2 Figure 6) a horizontal strain/limiting tensile strain ratio of 0.83 is obtained, indicating that a horizontal strain of 0.062% is acceptable for a 'very slight' category of damage. This analysis does not take account of the stiffness of the wall; the result is therefore conservative in this respect.

5.10.2 Lateral movement.

From Section 5.3 above, the greatest average horizontal ground strain adjacent to the proposed excavation at No 7 is predicted to be 0.064%. This level of strain extends approximately 4.4m from the excavation, and therefore affects the proximal 3m or so of the rear wall. This level of horizontal strain is greater than the 0.062% 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 0.062% to 0.137%. However, the analysis does not take into account the stiffness of the wall in the horizontal or vertical directions, and is conservative in this respect.

It is therefore considered that the predicted level of damage to this wall can be taken as 'very slight'.

5.11 Predicted damage summary

On the basis of the above, the level of damage to Nos 35 and 37 Heath Drive is predicted to be 'very slight' or less, as defined in Ref 2.

This conclusion assumes a high standard of workmanship and adequate propping of the basement excavation. In particular the proximity of the proposed right-side pile wall to the flank wall of the side-extension to No 37, requires particular care in piling and selection of piling method and sequence.

A plot of the calculated short-term settlement contours is presented in Figure 11 below.

6.0 Groundwater

It is proposed to excavate to a maximum depth of approximately 4m below existing ground level, through a layer of Made Ground into a thick deposit of London Clay. Groundwater was encountered at 11m depth during the ground investigation, and groundwater was recorded in subsequent monitoring of piezometers. Nevertheless, the geology of the site is such that persistent significant groundwater flows are not expected within or across the site.

There is no 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 proposed basement to No 36 Heath Drive can be constructed without imposing more than 'very slight' damage on the adjoining properties at Nos 35 and 37.

The development is not likely to affect the local groundwater regime.

References:

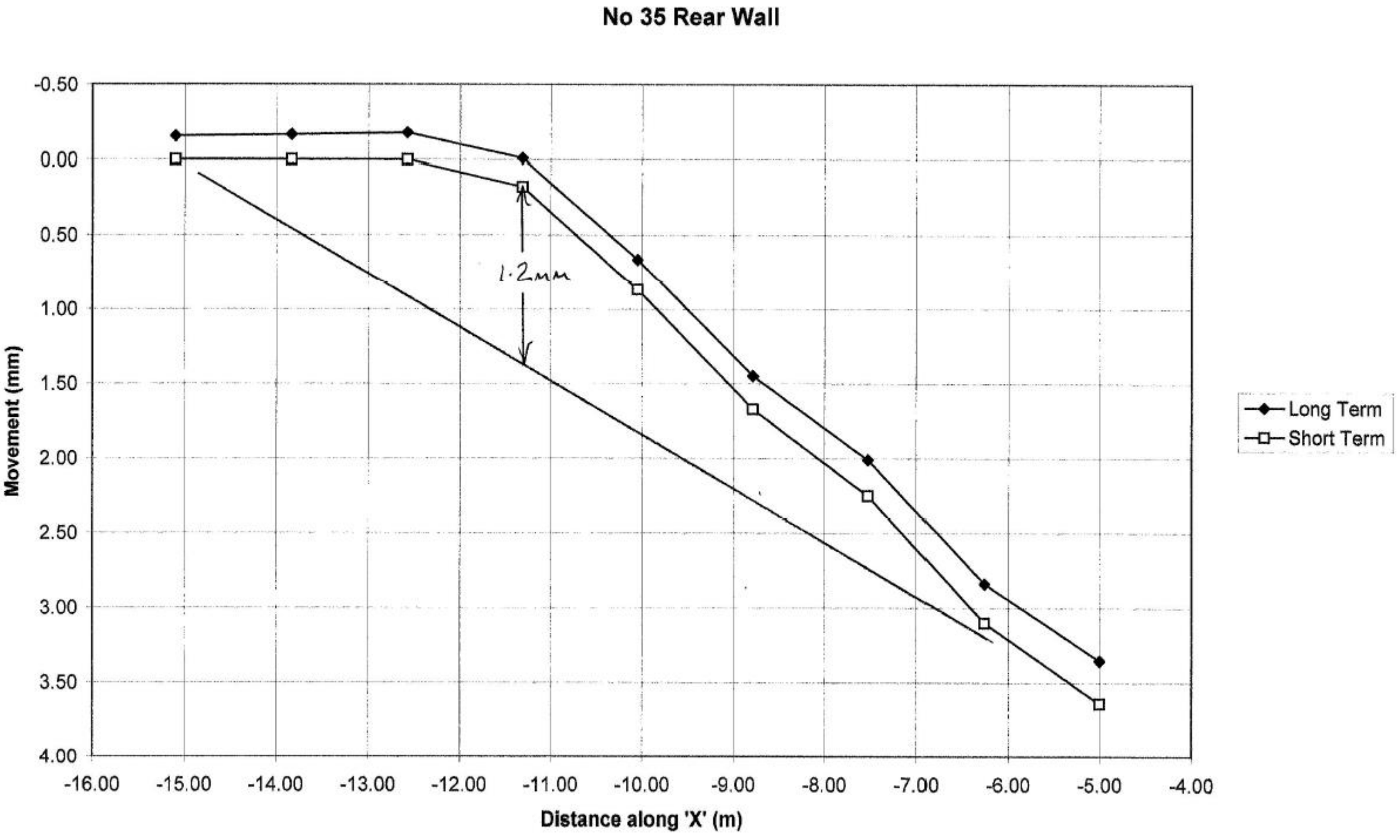
1 Stroud M A (1989) 'The standard penetration test – its application and interpretation'. In 'Penetration testing in the UK', Thomas Telford pub.

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

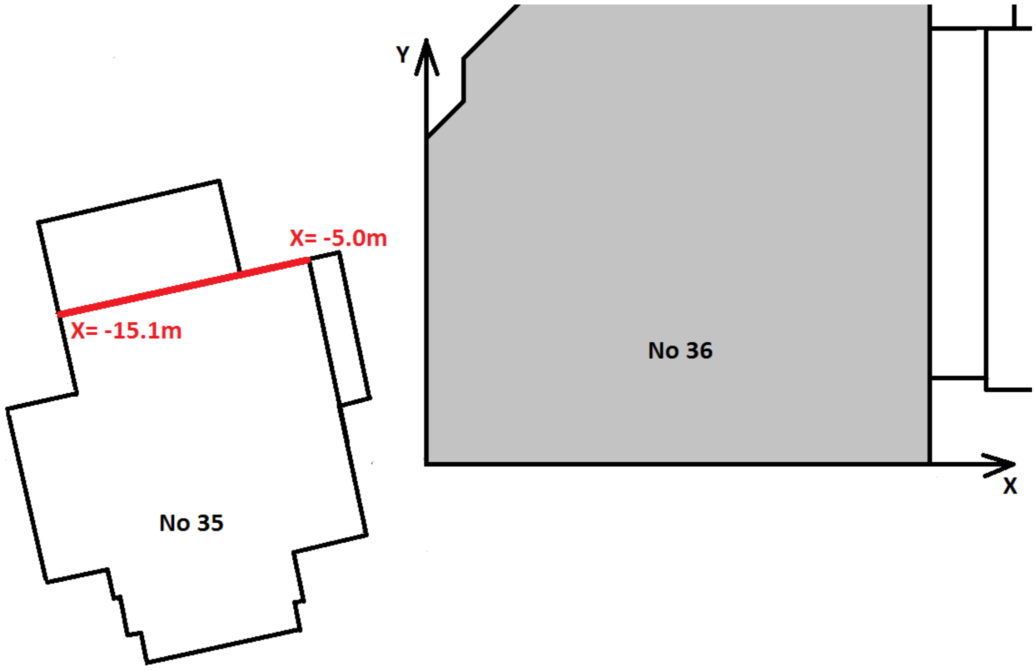
3 Gaba A R, Simpson B, Powrie W, Beadman D R (2003) Embedded retaining walls - guidance for economic design, CIRIA Report C580, London. ISBN: 978-0-86017-580-3.

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 4-11 follow below)

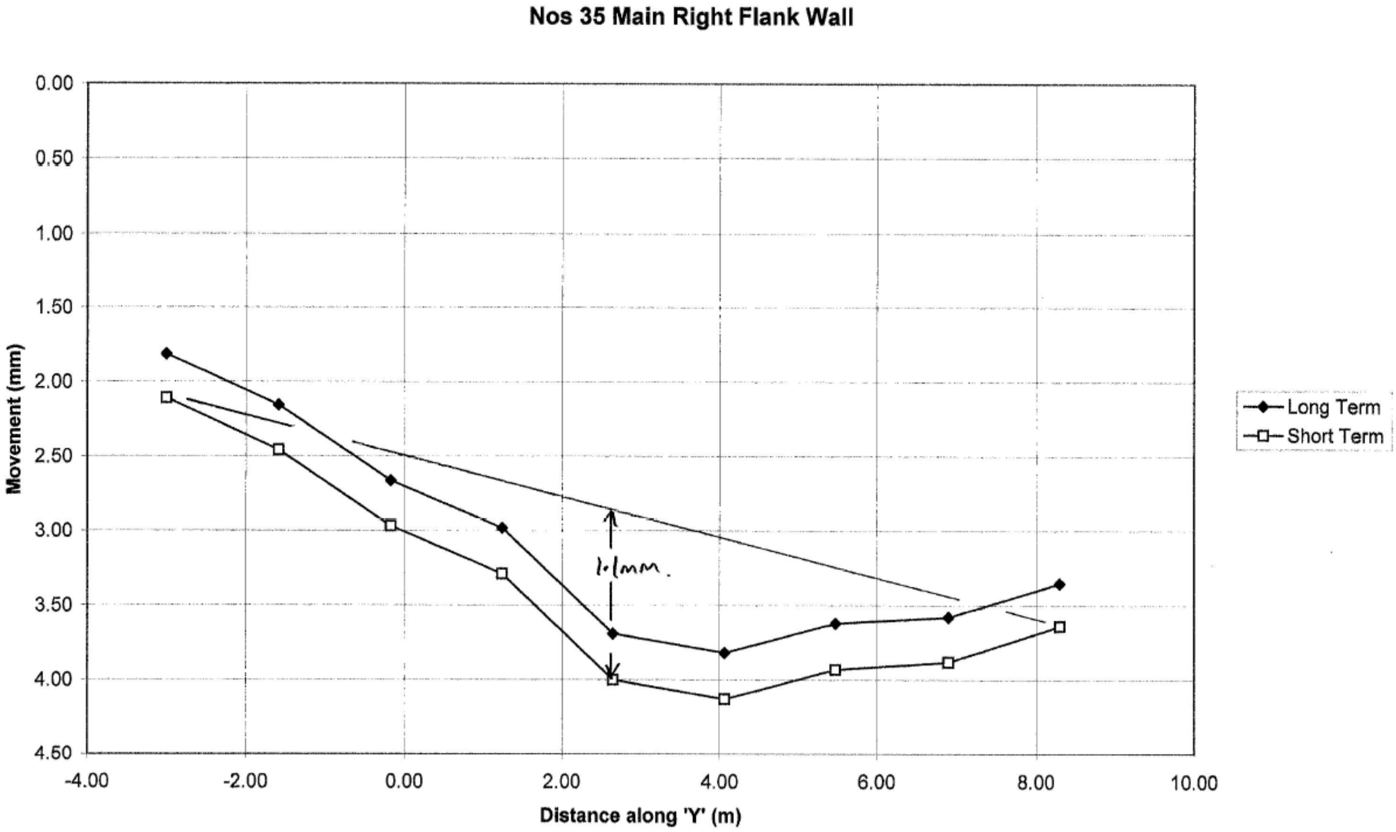


No 35 Rear Wall
Movement (mm)
-0.50
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
-16.00
-15.00
-14.00
-13.00
-12.00
-11.00
-10.00
-9.00
-8.00
-7.00
-6.00
-5.00
-4.00
Distance along 'X' (m)
1.2mm
Long Term
Short Term

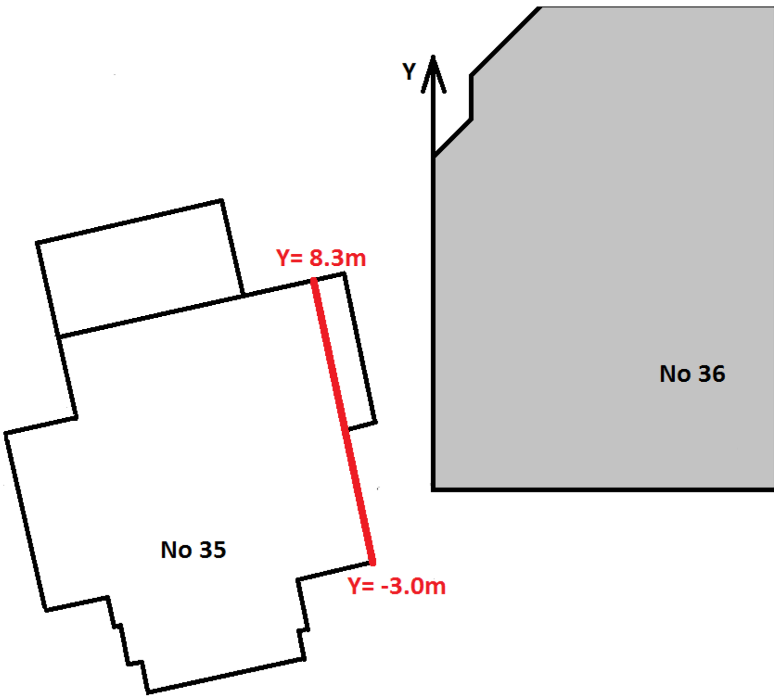


Y
X
X= -5.0m
X= -15.1m
No 35
No 36

Figure 4

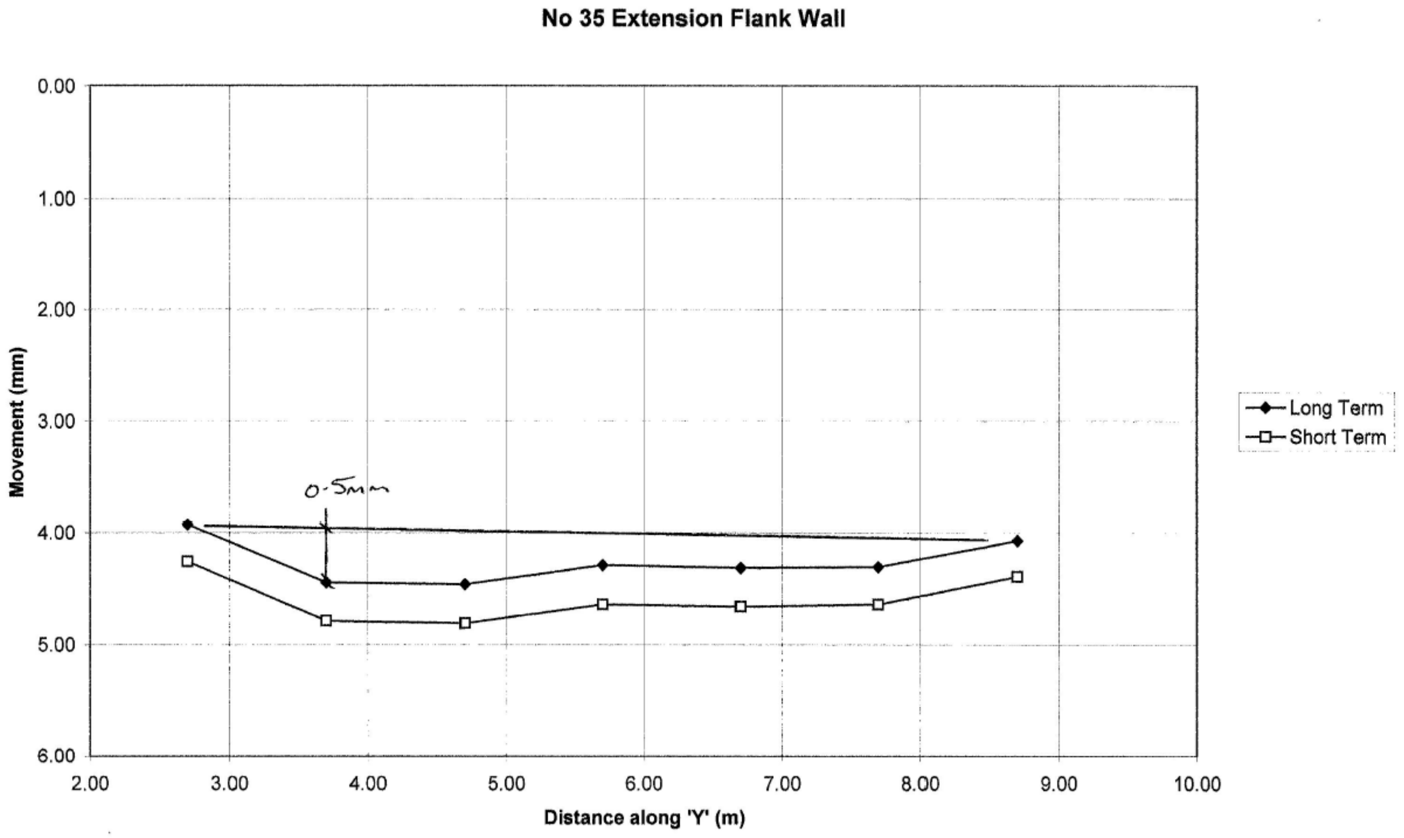


Nos 35 Main Right Flank Wall
Movement (mm)
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
-4.00
-2.00
0.00
2.00
4.00
6.00
8.00
10.00
Distance along 'Y' (m)
Long Term
Short Term
1.1mm.

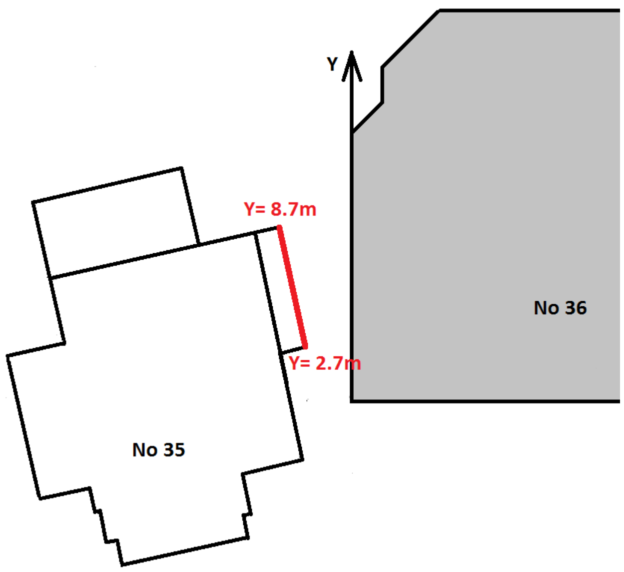


Y
Y= 8.3m
Y= -3.0m
No 35
No 36

Figure 5

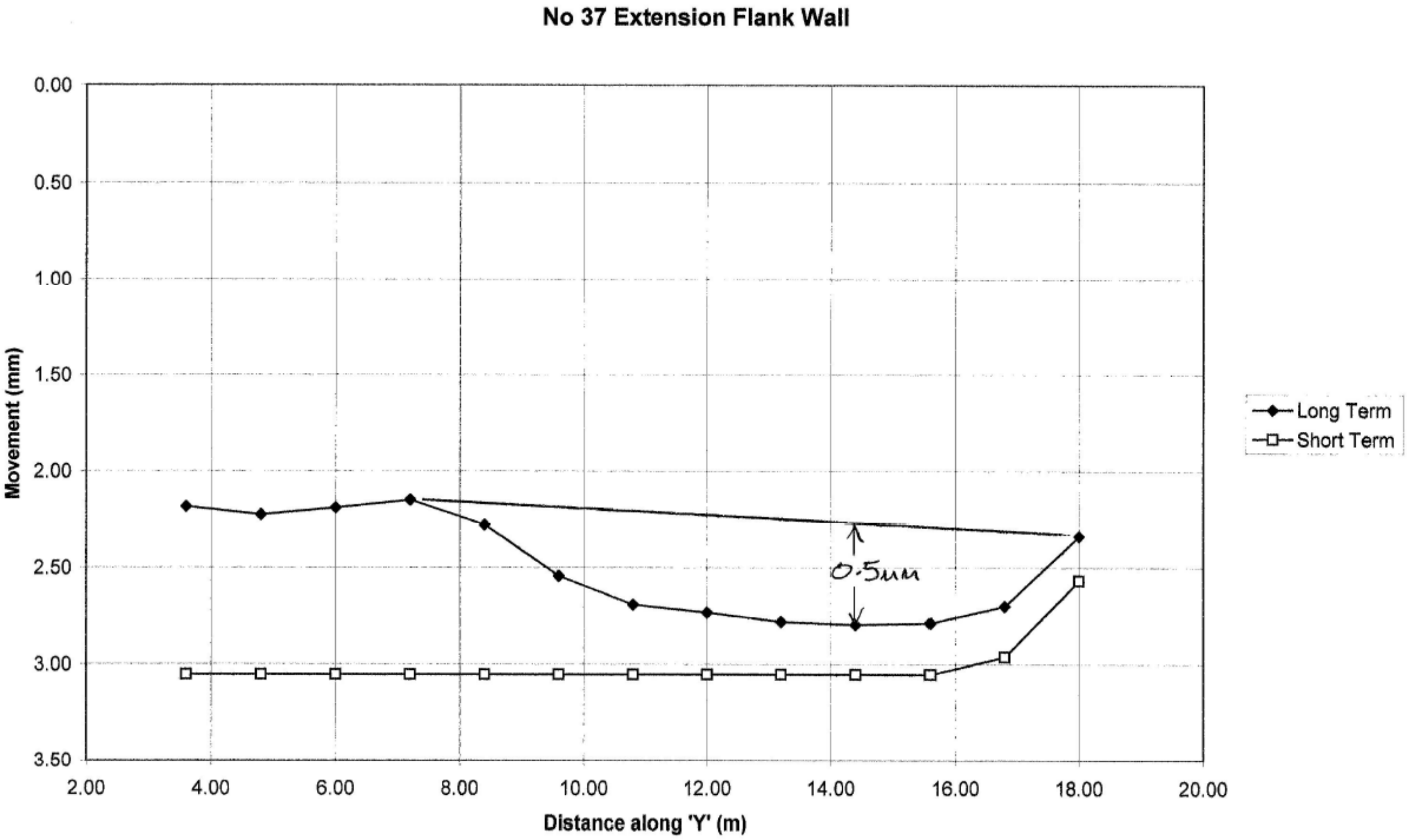


No 35 Extension Flank Wall
Movement (mm)
0.00
1.00
2.00
3.00
4.00
5.00
6.00
0·5mm
2.00
3.00
4.00
5.00
6.00
7.00
8.00
9.00
10.00
Distance along 'Y' (m)
Long Term
Short Term

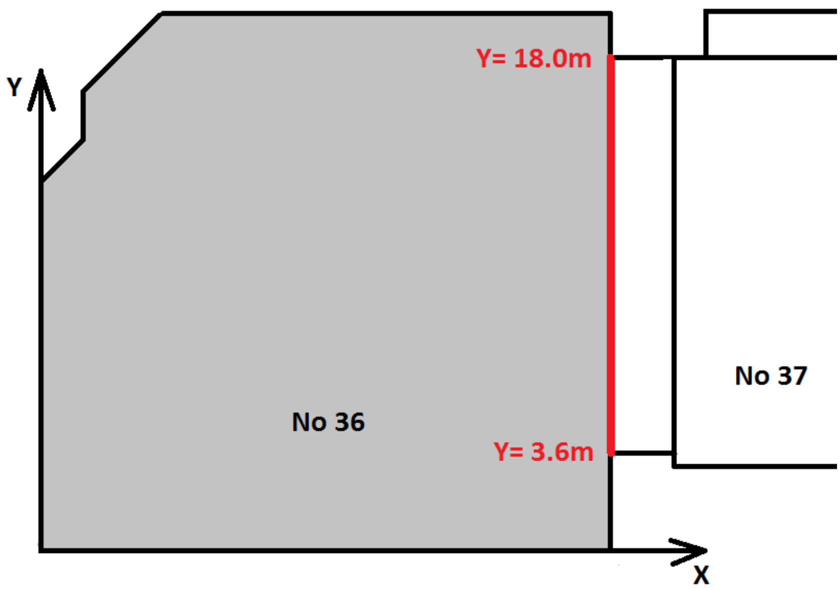


Y
Y= 8.7m
Y= 2.7m
No 36
No 35

Figure 6

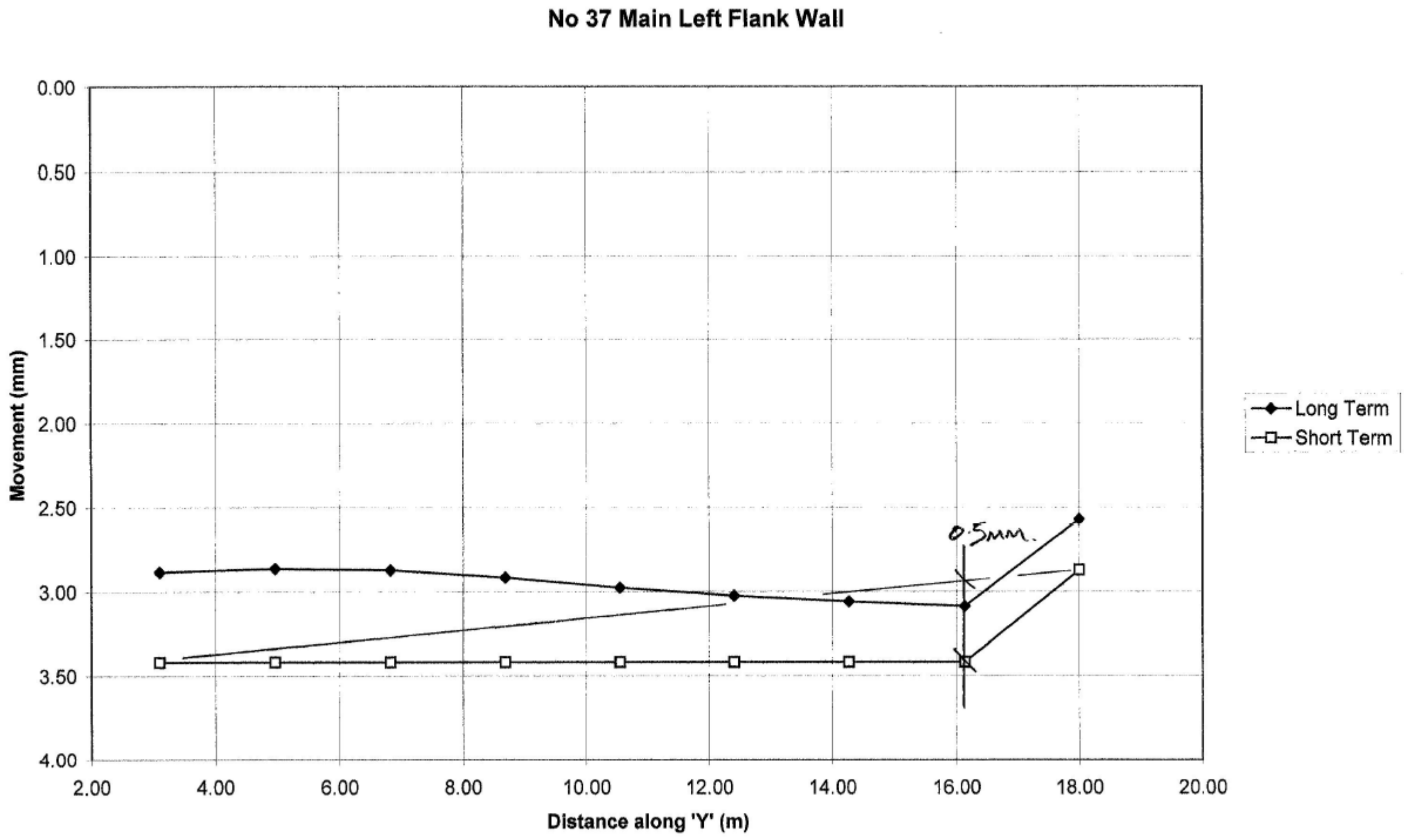


No 37 Extension Flank Wall
Movement (mm)
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
2.00
4.00
6.00
8.00
10.00
12.00
14.00
16.00
18.00
20.00
Distance along 'Y' (m)
0.5mm
Long Term
Short Term

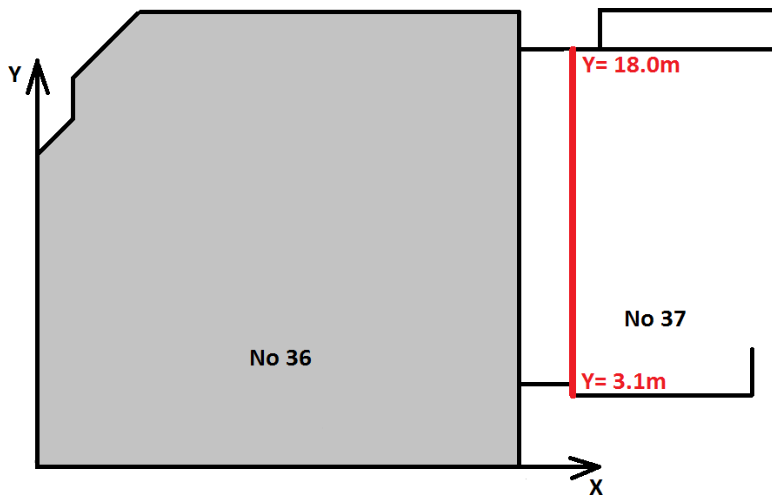


Y
Y= 18.0m
No 37
No 36
Y= 3.6m
X

Figure 7

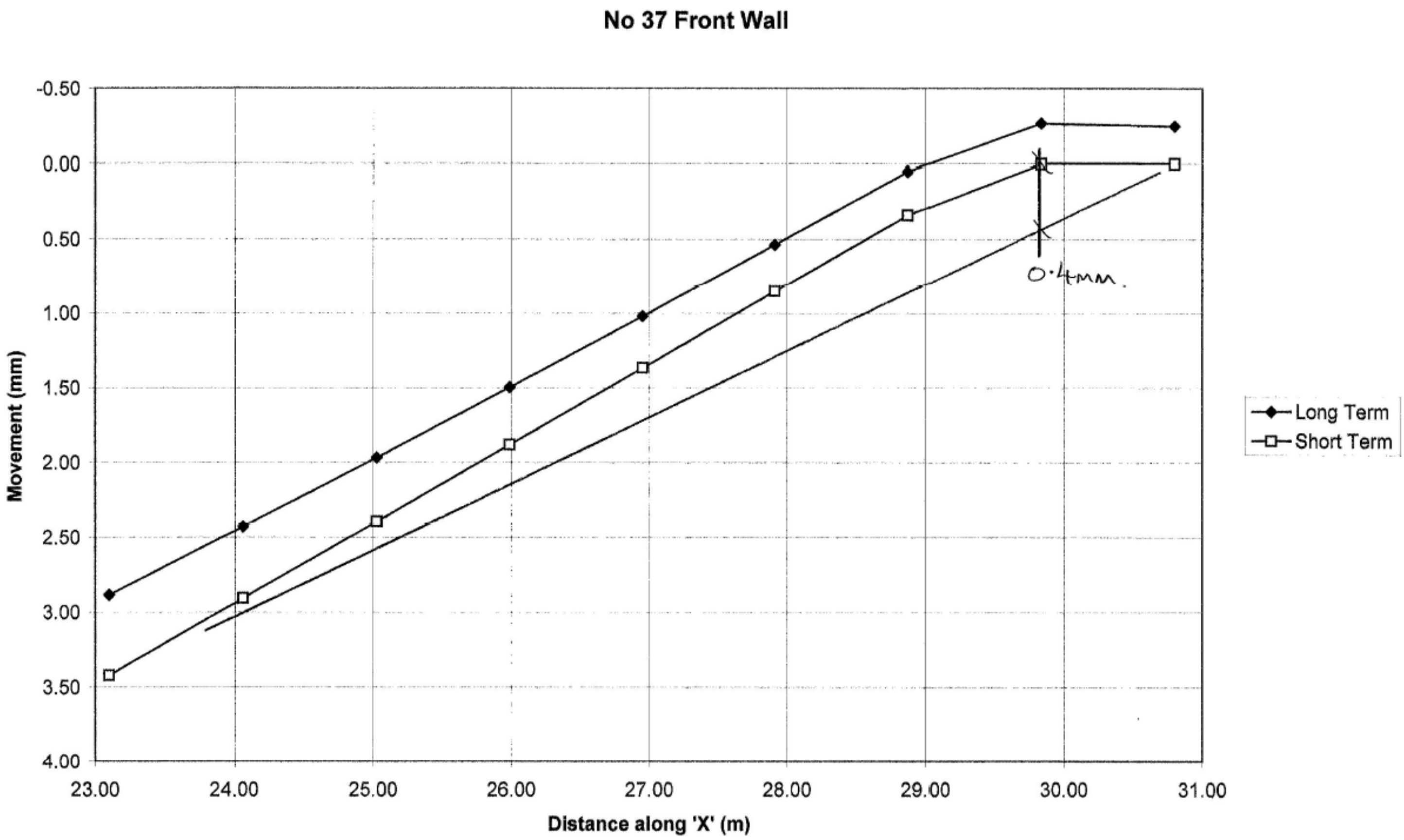


No 37 Main Left Flank Wall
Movement (mm)
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
2.00
4.00
6.00
8.00
10.00
12.00
14.00
16.00
18.00
20.00
Distance along 'Y' (m)
Long Term
Short Term
0.5mm.

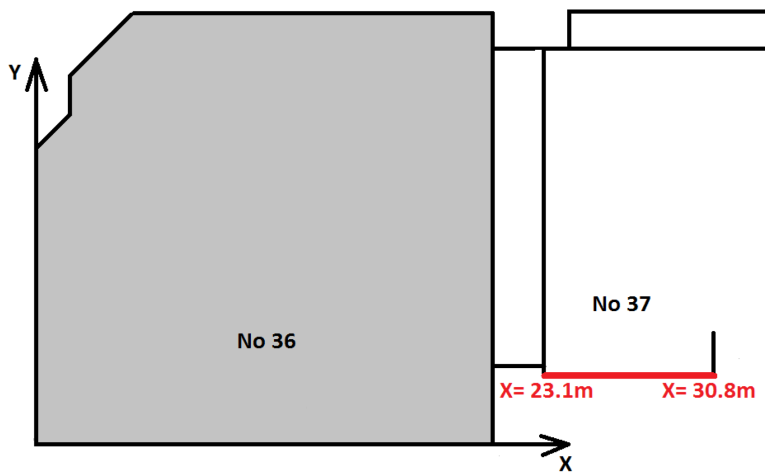


Y
Y= 18.0m
No 37
No 36
Y= 3.1m
X

Figure 8

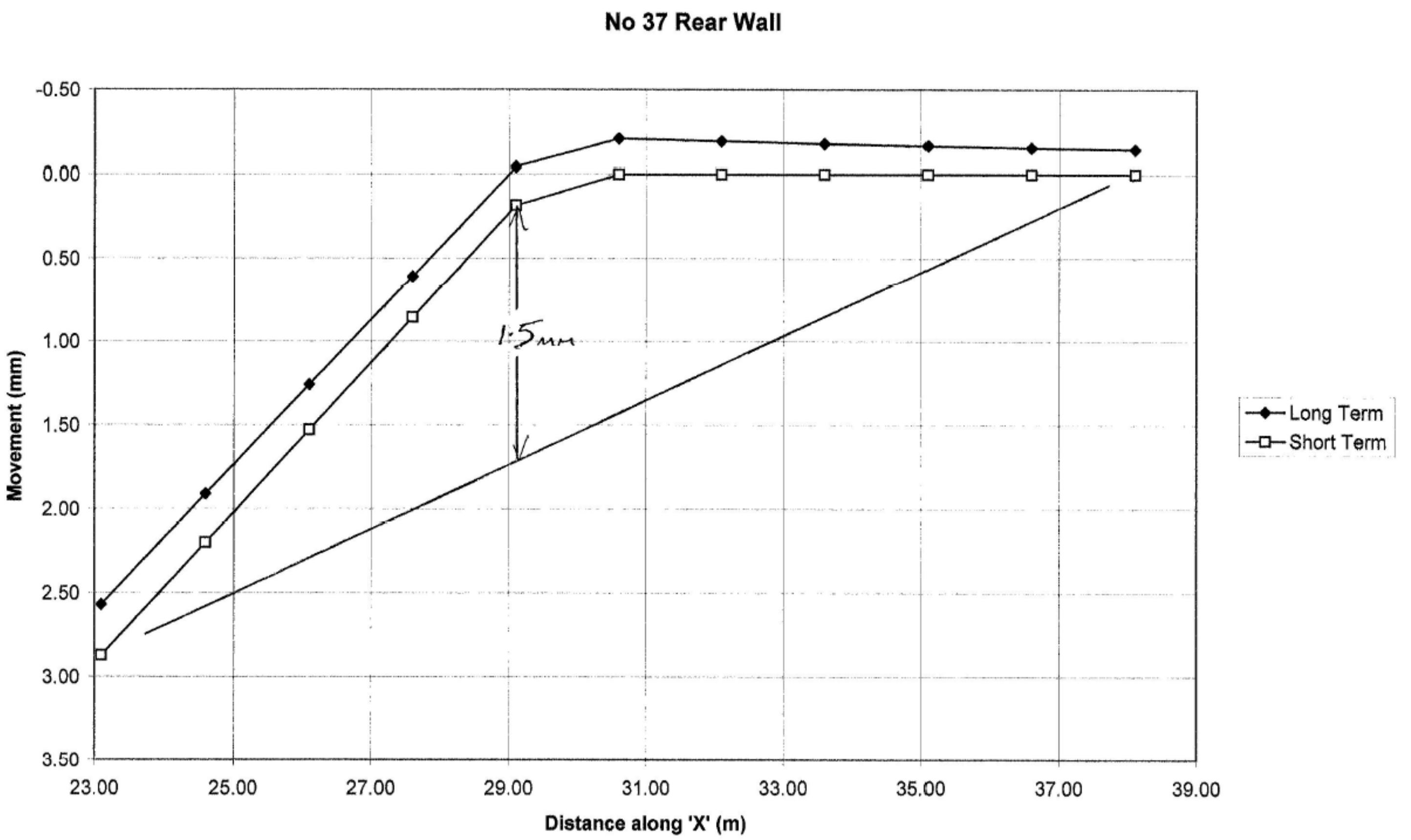


No 37 Front Wall
Movement (mm)
-0.50
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
23.00
24.00
25.00
26.00
27.00
28.00
29.00
30.00
31.00
Distance along 'X' (m)
0·4mm.
Long Term
Short Term

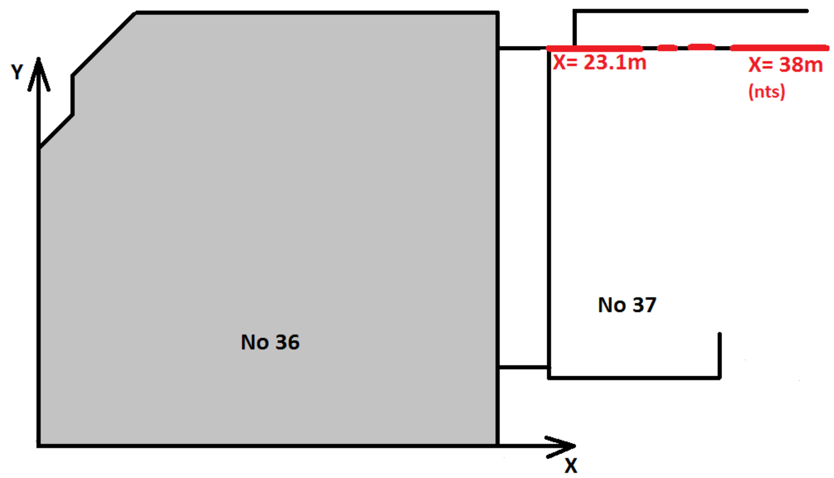


Y
No 36
No 37
X= 23.1m
X= 30.8m
X

Figure 9



No 37 Rear Wall
Movement (mm)
-0.50
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
23.00
25.00
27.00
29.00
31.00
33.00
35.00
37.00
39.00
Distance along 'X' (m)
1.5mm
Long Term
Short Term



Y
X
No 36
No 37
X= 23.1m
X= 38m
(nts)

Figure 10

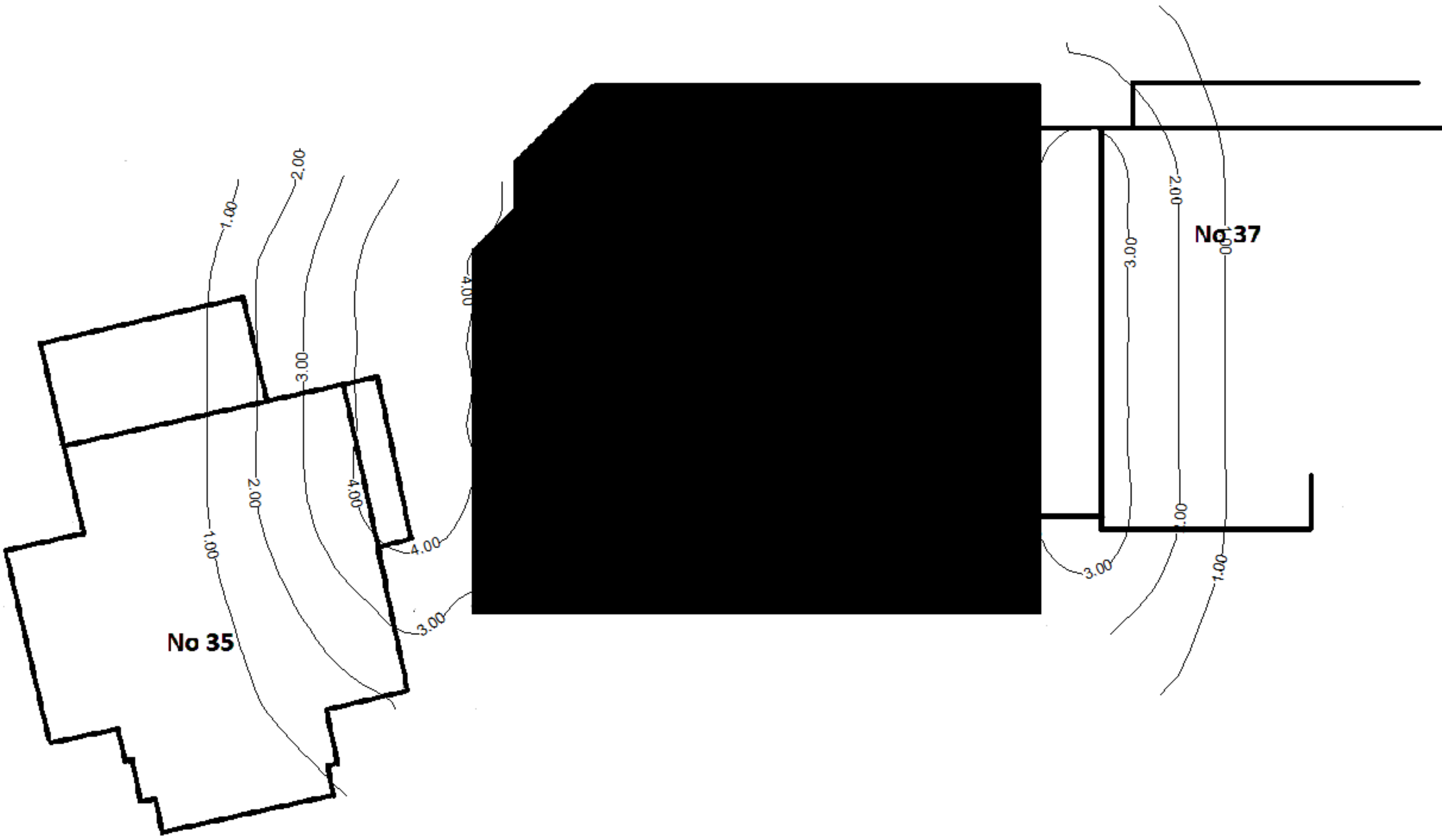


Figure 11
(Short-term ground settlement contours - settlement not estimated at open-cut faces)