TRAIN AND KEMP
ENGINEERING BUSINESS SOLUTIONS

BASEMENT IMPACT ASSESSMENT

for

40 FROGNAL LANE

LONDON NW3 6PP

by

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CONTENTS

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01	07.09.15	Incorporating C Geologist’s input
01A	11.09.15	Appendices shuffled to suit splitting report into 3 parts for transmittal
03	11.10.16	Responding to BIA Audit, May 2016, Appendix 2, Query Tracker Ground water monitoring expanded in 3.4 Ground movement and damage assessment added in 4.4 and Appendix 2. Ground movement monitoring addressed in 4.5
04	18.01.17	Responding to BIA Audit November 2016 with 2.5 Juxtaposition of Foundations 4.5 Deflexion of Basement Walls & Damage to Existing Buildings 4.6 Assessment expanded
05	03.03.17	4.5 Excavation propped Schematic Cross Section and Levels added to Appendix 5
06	21.04.17	4.4 and 4.5 rewritten and pile and excavation calculations added to Appendix 2

This report has been prepared by Train and Kemp (Consulting Engineers) LLP using reasonable skill and care in accordance with the instructions of its client. No liability is extended to other parties.

The lead author is a Chartered Engineer. The Hydrogeology section has been co-authored by Clive Carpenter as a Chartered Geologist.

Norman Train C.Eng, FIStructE, FICE

1. INTRODUCTION

This Basement Impact Assessment, BIA, has been prepared in support of a planning application for the refurbishment of 40 Frognal Lane NW3 6PP, which includes a basement extension and swimming pool.

A Structural Stability Report was produced in 2011; this has been superseded by this BIA.

The BIA has been prepared in accordance with LB of Camden CPG4, Basements and Lightwells, July 2015. It takes cognisance of the Camden Geological, Hydrogeological and Hydrological Study, CGHHS. In Section 5, Surface Flow and Flooding, it also makes reference to Camden Flood Report, 2003, and Surface Water Management Plan, 2013.

The BIA has been prepared by Norman Train, a Chartered Engineer, and in accordance with CPG4, Section 3 on Groundwater Flow, the hydrogeology has been reviewed and amended by Clive Carpenter, a Chartered Geologist, with his letter dated 07.09.15 in Appendix 3. Clive was sent a copy of Revision 0 and his comments and additional text have been incorporated in the text of Section 3.

Construction Techniques are presented in Appendix 1; Ground Movement in Appendix 2; scheme drawings in Appendix 3 and the site investigations in Appendix 4.

1.1 BIA Stages

A Stage 1 Screening utilising the questions in CPG4 is given in the respective sections. Additional questions to the CPG4 screening are:

Stability10B		Excavation below the water table and need for dewatering
F6A	Surface Water Flooding	relating to Surface Water Management Plan Local Flood Risk Zones

The screening has been used to define the Stage 2 Scope of the Assessment.

As part of the Stage 3 site investigations and study:

- A visual inspection was made of Nos 38 and 40 Frognal Lane.
- London Underground was contacted about the alignment to the Northern Line Tunnels.
- AP Geotechnics completed a Site Investigation in September 2011, which is included in Appendix 4 together with further sets of standpipe readings taken on 23.01.14 and 21.04.15 from monitoring boreholes located within the landplot of the adjacent property No.38 Frognal Lane.

The Stage 4 Impact Assessment of the scheme is presented in Sections 3 to 6.

2. EXISTING SITE AND PROPOSED DEVELOPMENT

For the purposes of this assessment, Frognal Lane is taken to the north and Frognal to the east. This means Chesterford Gardens is also to the north.

2.1 Site

40 Frognal Lane is a Grade 2 star detached three storey house with a semi basement, raised ground floor and first floor with a pitched roof over.

The site is located on the crest of the slope above the steeper sections of Frognal Lane with an access shared with Nos 42, 42a and 44, which are all detached houses set in their own large gardens.

The site is a trapezium with a 20m frontage to Frognal Lane widening out to 30m along the rear southern boundary with No 42a; the site is 45m front to back. The house is 18m, front to back, located along the eastern boundary with the access road with the garden to the west. A private access has been constructed at the western end of the road frontage with a separate garage block.

As can be seen in the OS location plan in Appendix 3, the nearest buildings to No 40 are:
- No 38 Frognal Lane which is 20m to the west of the house down Frognal Lane.
- No 42 Frognal Lane which is 25m to the southeast of the house
- No 42a Frognal Lane which is 8m to the south
- No 44 Frognal Lane which is 15m to the east of the house.

Beyond the cluster of houses on the shared access are the rear gardens to the houses along Frognal to the east and Langland Gardens to the west.

London Underground Tunnels
The Northern Line Edgware Branch is located to the east of the site.

2.2 Topography and Levels

The site is on the slope down from Hampstead village to the Finchley Road. The natural gradient is generally from northeast to southwest with localised variations and plateaux. The eastern end of Frognal Lane, between the shared access and the junction with Frognal, is at +94m, but to the west the road drops with a gradient of 1 in 10 towards the junction with Langland Gardens.

There is a slight fall across the site from the northern end of the shared access to a low point in the southwest corner which is 2m lower at +92m. The garden to the west is lower than the shared access.

Along Frognal Lane, the natural slope has been cut into terraces and No 38 is 3m below No 40 at the back of pavement, but this reduces to 1m at the southern end of the garden. There is a formal retaining wall to the front half of this level change with sloping beds and banking to the rear. The retaining wall is 2m high at the front northern corner of the pool.

No 38 Frognal Lane is at +91m; it does not have a basement.

2.3 Proposed Scheme

As shown on drawing P23A in Appendix 3, it is proposed to construct a basement swimming pool beneath the garden to the west of the existing house. The pool will be 22.5m long by 8m wide. Because of the fall of the ground and the need to have a sustainable thickness of top soil cover above the pool, the pool excavating will be a maximum of 5m below the lower ground floor at the deep end; this is shown on sections AA and CC on drawings P24 and P26A respectively. A staircase link will lead down from the lower ground floor to the swimming pool at its southern end.

This means that:
i. the pool surround is 3.5m to the west of the main house at an excavation level of +88.5m
ii. the pool is 6m to the west of the main house at an excavated level of +86.5m.

Whilst the main house is within 8m of No 42a, the staircase link to the pool is 12m from No 42a.

The pool is at an angle to No 38 with the front and rear corners of the flank wall to being 6m and 8m respectively from the pool. The boundary retaining wall is 1m less.

As shown on P26A, the pool has two covered sky lights at the northern end and a clerestory light along part of the western elevation.

2.4 Basement and Foundations

Swimming Pool
The swimming pool will be constructed as a reinforced concrete box with propping to the walls during the excavation phase.

Foundations
The pool is in the garden, outside the house, and given that:

i. it will extend below the water table;
ii. it will extend into the underlying London Clay;
iii. it is deeper than the foundations to No 38

A secant piled wall will be used as a coffer dam construction for the pool. As shown on Section AA, the link staircase extends back into the existing house and where this is above the water table, shallow concrete foundations will be used with the lower ground floor of the house being underpinned locally using traditional concrete pins.

Construction techniques are discussed in Appendix 1.

Buoyancy will have to be taken into account with the design of the piles to the pool.

2.5 Juxtaposition of Foundations

As illustrated in the Schematic Site Cross Section and Levels in Appendix 5, the various levels are:

	Current	Proposed	Comment
No 40 Upper Grd Fl	+95		
No 40 Driveway	+94		Same as shared access
No 40 Garden	+92 to +94	+94	Presently the ground drops away to west
No 40 Lower Grd Fl	+92		Assumed that foundations are at same depth
Pool Surround		+89	Excavation depth +88.5
Pool Deep End		+87	Excavation depth +86.5
No 38 Grd Floor	+91		Same as forecourt to front of No 38
No 38 Foundation	+90		

The pool is to be constructed on the side of a hill. Currently the garden to the west of the house falls away to the western boundary. In order to achieve the 1m soil cover to the lid of the pool. this will be made up to the same level as the shared driveway. Beyond the pool, the existing levels to the west will be maintained.

The pool structure steps with the pool surround up the slope and the pool itself down the slope having excavation depths of 3.5m to 4.5m. It should be noted that these excavation depths are different to the backfill depths to the completed concrete box which will be nearer 6.5m. See schematic site cross section in Appendix 4. Whilst the headline level difference between the pool deep end [+87] and the garden over [+94] is 7m, the retained height will be less than 5m in that the garden cannot be reinstated prior to the pool lid being completed.

- The pool surround is 3.5m below No 40 lower ground floor at a horizontal clearance of 3.5m
- The pool deep end is 3.5m below No 38 foundations at 6m to 8m horizontal clearance.
- The western boundary retaining wall is at the same level as No 38 ground floor; the pool deep end is 4.5m below this at 5m to 7m horizontal clearance.

2.6 Trees

As shown on P21A, at the southern end of the garden is a 7m magnolia tree which is 3.5m from the staircase link. There are also two mature trees along the western boundary which are 13m and 15m high and 8m from the pool structure. Along Frognal Lane are two trees along the kerbline which are 11m from the pool.

3 GROUNDWATER FLOW

3.1 Stage 1 Screening

GW1 CGHH Fig 4 shows the site is founded on the Claygate Member overlying London Clay, the latter outcropping further to the south to the west side of Lindfield Gardens. As shown on CGHH Fig 8, Aquifer Designation Map, the Claygate Member is a Secondary A Aquifer.

GW1A Water will tend to collect at the base of Claygate Member perching above the impervious London Clay. Given the anticipated moderately low permeability of this silty sandy clay, the Claygate Member is expected to contain water all year round.

GW1B Water collecting at the base of the Claygate Member will generate water issues (surface seepages) where its contact with the underlying London Clay reaches ground surface.

GW2 CGHH Fig 11, Watercourses, shows that two tributaries of the River Westbourne start in Frognal and Langland Gardens to the southeast and southwest of the site. Both tributaries are over 100m from the site and are at a lower altitude. There is a further tributary 500-600m to the north of Oak Hill Way. There are no reported springs in the area.

GW3 CGHH Fig 14, Hampstead Heath Surface Water Catchment, shows that the Hampstead Ponds catchment is 0.75km to the north of the site.

GW4 Whilst the pool extends out beneath the garden, this will be reinstated with soft landscaping and there is no change in the hard landscaping. See drawings in Appendix 3.

GW5 The current surface water drains will be maintained and with no significant changes to the impervious area, the net discharge to ground will be similar.

GW6 CGHH Fig 13, Hampstead Heath Map, shows that the nearest water feature is the Whitestone Pond, 0.75km to the north of the site, at a higher elevation, in a different catchment and on overlying strata and hence too remote to affect the site.

3.2 Stage 2 Scoping

- The site is founded on a Secondary A Aquifer
- The basement may extend through any perched water table in the base of Claygate Member on top of the London Clay.

Whilst the site is more than 100m from a watercourse, given the uncertainty in the precise positon of the spring points of small tributaries, this has been included in the impact assessment in Section 3.4

3.3 Stage 3 Study and Site Investigation

<u>Study</u>
The Claygate Member is the youngest part of the London Clay Formation forming the transition between the more plastic homogeneous clay in the lower parts of the Formation and the sands of the overlying Bagshot Formation. The Claygate Member consists of laminated clays with lenses of fine sands. The sand percentage increases towards the top of the unit.

In hydrogeological terms, the Claygate Member has sufficient sand content to give hydraulic continuity and to allow slow groundwater flow to pass through it, although its permeability varies considerably depending on the thickness and lateral extent of the interbedded sand lenses. The base of the Claygate Member often has water that collects on the top of the underlying impervious London Clay; where the London Clay reaches ground surface, this can result as a spring or seepage line. This may be seasonal in nature depending on the size of the catchment contributing water to the seepages.

The junction between the Claygate Member and London Clay is shown on CGHH Fig 4 to pass along Lindfield Gardens, across Langland Gardens and Frognal, to the south of Frognal Close, at an elevation of approximately 82m AOD. This is coincident with the start of the River Westbourne tributary shown on CGHH Fig 11 (100m southwest of the site), within a shallow valley. A second tributary commences beneath University College School (150m southeast of the site), again on the Claygate Member/London Clay contact, again at an elevation of approximately 82m AOD, again in a shallow valley feature.

The location of these two tributaries, suggests the site is located near a groundwater divide. Hence the area of the catchment contributing to the tributary commencing on Langland Gardens is relatively small.

CGHH Fig 11 shows that there are five feeder tributaries to the southwest of Hampstead extending for a width of some 1.5km and their catchment is to the south of the River Fleet. The catchment area for the Langland Gardens and Frognal tributaries is likely to be approximately 40 hectares (800m wide, 500m long). Assuming a typical average recharge into the Claygate Member of no more than 250mm/yr, this would yield an annual groundwater contribution to the tributary of 100,000m^3/yr, which equates on average to 3 l/s.

Whilst it is unknown whether these tributaries flow year round or just in winter months, clearly a flow of typically 3l/s is fairly minimal, especially if dispersed along a wide seepage horizon.

<u>Site Investigations</u>
A site investigation was completed in the garden to No 40 Frognal Lane in July 2011. This comprised two boreholes to 18m in depth with a standpipe installed in one of these (BH1).

In December 2013, two window samplers were sunk in the No 38 forecourt to a depth of 6.0m and standpipes installed in both of these (WS1 and WS2).

Data from both of these investigations are also included in Appendix 4, together with a location plan of the boreholes.

40 Frognal Lane
The two boreholes were at a level of +93.2m OD. Made ground was found to a depth of 0.9 to 1.3m with silty clay [Claygate Member] to a depth of 5.1 to 5.6m, overlying brown clay [weathered London Clay] with occasional pockets of sand to 7.8 to 8.6m, overlying stiff fissured dark grey clay [unweathered London Clay]. The Claygate Member is some 4.2-4.3m thick beneath the garden and its interface with the weathered London Clay is at approximately +87.56m at BH1.

A standpipe installed in BH1 recorded water at a depth of 3.9m and 2.9m in August and September 2011 respectively. Unfortunately the standpipe seal was not deep enough to fully seal out the overlying Made Ground (just below which a water strike was observed). It is therefore not possible to determine whether the two water levels relate to slow filling from leakage from the Made Ground or from deeper seepages (at 5.50m bgl) near the base of the Claygate Member. It is also unclear whether these water levels (the latest is the shallowest) are recording a continuing rise in groundwater levels yet to achieve a static water level or a seasonal increase in the static water level.

A rising head permeability test conducted on the Claygate Member in BH1 indicated that the Coefficient of Permeability $k = 4.2 \times 10^{-7}$ m/s. However the casing depth of 1.00mbgl was insufficient to seal out the Made Ground and it seems likely this permeability test is at least partially reflecting the probable higher permeability of the saturated base of the Made Ground. Permeability values of 10^{-7}m/s are known to have been reported elsewhere for these strata.

Two water samples were taken from BH1 and were tested for dissolved sulphate. These contained 180 mg/l for the water sample at 1.40m bgl and 240 mg/l sulphate for the sample at 5.0m bgl. These relatively high concentrations of sulphate indicate slow groundwater flow, as they demonstrate long residency times of the water in contact with sulphate bearing strata.

38 Frognal Lane

The investigation established that there is up to 0.9m of Made Ground along the western boundary overlying the Claygate Member. The top of the London Cay was not found and is therefore lower than +85m OD.

This is consistent with the geological maps showing the base of the Claygate Member to be at a ground elevation of approximately +82m to +83m within 100 to 200m west of the site.

Groundwater Level Monitoring

The groundwater readings were taken on two subsequent occasions and the results are tabulated below. There is a consistency across the three standpipes, with water levels all between approximately 1-3m bgl.

Standpipe Readings

Location	Grd Level OD	05.08.11		20.09.11		18.12.13		23.01.14		21.04.15	
		Depth BGL	OD	Depth BGL	OD	Depth BGL	OD	Depth BGL	OD	Depth BGL	OD
BH1 [No 40]	+93.2m	3.9m	+89.3m	2.9m	+90.3m					Grass re-turfed, standpipe lost	
WS2 [East]	+90.9m					2.0m	+88.9m	0.8m	+90.1m	1.6m	+89.3m
WS1 [West]	+90.8m					2.8m	+88.0m	1.5m	+89.3m	2.3m	+88.5m
Difference across site							0.9m		0.8m		0.8m
Difference BH1 to WS2			BH1 Av=	+89.8m			0.9m		-0.3m		0.5m

When the standpipes were dipped in April 2015, the grass to No 40 had been re-turfed and the BH1 standpipe was lost.

The two trends that can be discerned are:

i. There are seasonal variations to the depth of the phreatic surface; and
ii. Based on WS1 and WS2 and an interpolation of BH1, the phreatic surface has an approximate gradient which is half that of the topography.

Seasonal Variation

The difference in water levels between January 2014 and April 2015 for both window samplers is a lowering of 0.8m. This confirms groundwater flow to be occurring and groundwater to leave the site after a period of rainfall recharge. Early 2014 was one of the wettest periods on record.

It is less certain whether the rise between December 2013 to January 2014 relates to rainfall recharge or groundwater level re-stabilisation after drilling, or a combination of both. January-February 2014 is known however to have been a period of extremely high groundwater levels in southern England, and therefore can be considered to be close to the highest elevation likely to be observed on site

Phreatic Surface Gradient

As shown on 10998/SI/01 in Appendix 4, the latest groundwater level readings for each of the three standpipes (Sept 2011 for BH1, and Apr 2015 for WS1 and WS2), allows the groundwater levels to be triangulated as a first approximation of the phreatic surface and direction of flow; albeit for water levels taken in different years at different times of year. These show the groundwater flow direction to be towards the west-north-west, indicating the groundwater flow beneath the site passes beneath No 38 and Frognal Lane.

The calculated hydraulic gradient is 5×10^{-2} (1 in 20). This is very steep for a hydraulic gradient and confirms both the low permeability of the strata as well as the perching of the water body on the underlying London Clay.

The Sept 2011 groundwater level may well not reflect the final rest groundwater level in BH1 nor the seasonal high. Assuming the Apr 2015 equivalent groundwater level at BH1 is at a greater altitude would increase the hydraulic gradient and re-orientate the groundwater flow direction slightly further northwards. The lack of an Apr 2015 groundwater level at BH1 is not substantive however in understanding groundwater flow beneath the site.

Groundwater Throughflow Estimation
The groundwater throughflow beneath the site can be calculated using Darcy's Law Q = A x k x i, , where

A = the cross-sectional area of the section being considered,
k= hydraulic conductivity = 4×10^{-7} m/s
i = hydraulic gradient = 5×10^{-2}

The Claygate Member thickness is 4m. The width of the site is 45m and that of the pool perpendicular to the groundwater flow direction is 24m. The existing house extends 1m into the Claygate Member for a width of 18m. This gives the following estimations of groundwater flow

Whole Site:	$Q = 4 \times 45 \times 4 \times 10^{-7} \times 5 \times 10^{-2} =$	3.6×10^{-3} l/s.
House:	$Q = 1 \times 18 \times 4 \times 10^{-7} \times 5 \times 10^{-2} =$	3.6×10^{-4} l/s
Pool:	$Q = 4 \times 24 \times 4 \times 10^{-7} \times 5 \times 10^{-2} =$	1.9×10^{-3} l/s

A flow of 2×10^{-3} l/s is less than 200 l/day or $0.2m^3$ daily average; this is very little water.

3.4 Stage 4 Impact Assessment

<u>Ground Water Abstractions</u>
Given the low permeability and restricted saturated thickness, the Claygate Member in this part of Hampstead is not considered a viable continuous source of water supply.

There are no records of any licenced groundwater abstractions in the vicinity of No 40.

<u>Foundations within the Water Table</u>
Groundwater levels across the site are typically +90.0m at the northern end of the proposed pool area, rising to 90.3m at the southern end of the proposed pool area, but can be expected to rise a further 0.8m higher than this, based upon groundwater responses observed at No.38.

The pool excavation is at + 86.0m and pool surround is at + 88.0m. Both of these are below the anticipated water table.

Appropriate construction techniques are discussed in Appendix 1. These will be exclusion (cut-off) techniques (given the surface of the underlying weathered London Clay is estimated to be at 87.5m), combined with internal temporary dewatering (pumping) techniques as necessary.

<u>Groundwater Flow Obstructions</u>
The affected throughflow from the pool construction is less than $0.2m^3$ average daily. This will be diverted around the outside of the pool sides to re-converge downstream. The water balance of the groundwater catchments feeding the river tributaries will not alter due to the pool construction, as the waters presently passing beneath the site will not be discharged from the catchment. The flow will remain in the same catchment and the tributaries base flow will remain the same.

The proposed pool will impact on the level of the groundwater table locally, raising the free surface on the upstream side and depressing it on the downstream side. The magnitude of the groundwater level changes due to construction of the impermeable basement pool is difficult to quantify. Experience in similar strata suggests these changes will be of the order of 0.2 -0.8m. On the upstream side, the pool may impact on the existing lower ground floor to No 40 and this may need tanking. Conversely on the downstream side, any lowering of the phreatic surface should have ceased before No 38.

Impervious Area
The pool is set down below the garden and will have 1.0m soil cover to its lid; consequently the pool will not have any impact on the impervious area, with infiltrating rainfall passing through the soil zone into the surrounding Made Ground and from there into the underlying Claygate Member.

SUDs techniques will be adopted to minimise and where possible reduce the rate of discharge to the existing drainage network.

Mitigation of Groundwater Flow Impacts
The impermeable box around the pool will impede the small groundwater flow (0.004 l/s) presently passing beneath the site. This will result in groundwater levels rising up-gradient of the site and lowering down-gradient of the site. The distance over which this may occur is difficult to predict without numerical modelling, but previous experience is that these will be local to the basement. The consequences of the groundwater level alterations may be to induce localised ground settlement.

Accordingly it is proposed to place a coarse granular drain outside of the construction phase cut-offs and completed impermeable box, to route groundwater around the basement/pool, and back into the surrounding strata.

Ongoing Monitoring
Historically standpipe readings were taken on five occasions. These give a reasonable indication of the seasonal range of the groundwater table level. Taking a precautionary approach to the groundwater flow truncation and mitigation, an informed assessment has been made of the flows.

The initial standpipes have been destroyed and it is not possible to extend the current monitoring regime.

It is recognised however that the quality of the data needs to be improved and it is suggested that further monitoring is made a pre-construction planning condition with the need to submit say a further three data sets.

3.5 Hydrogeology Summary

- There are no records of any water abstractions from the Claygate Member in the vicinity of the site;
- The pool will extend into the groundwater table and the construction techniques will have to take cognisance of this;
- The pool will have minimal effect on the regional ground water regime particularly with respect to the catchment of nearby watercourses;
- A high permeability drainage material will be placed outside of the cut-offs and constructed impermeable basement, to route groundwater flows from the up-gradient side to the down-gradient side, thereby maintaining pre-development groundwater flows and groundwater levels around the site and preventing local alterations to groundwater levels.
- It is not possible to extend the previous groundwater monitoring and it is suggested that further monitoring is included as a pre-commencement planning condtion.

4 GROUND STABILITY

4.1 Stage 1 Screening

Stab 1. CGHH Fig 16, Slope Angle Map, shows that the existing slopes on this part of Hampstead are less than 7^{o} [1 in 8]. The site location plan in Appendix 3 shows that Frognal Lane has a gradient of 1 in 10 to the west in front of the No 38.

Stab 2. There will be no significant remodelling of the slopes

Stab 3. There are no steep slopes in the vicinity and 1:25,000 OS Explorer Map shows that the 5m contours in this part of Hampstead are spaced at a minimum of 50m giving a gradient of 1 in 10

Stab 4. The site is a reasonably level plateau and 1:25,000 OS Explorer Map shows that the 5m contours in this part of Hampstead are spaced at a minimum of 50m giving a gradient of 1 in 10

Stab 5. London Clay does not outcrop on the site; it is over 5m deep.

Stab 6. Mature trees along the southern and western boundaries will remain. No trees will be removed.

Stab 7. No signs of movement in the original house.

Stab 8. CGHH Fig 11, Watercourses, shows that the nearest watercourse is the head of a tributary of the River Westbourne in Langland Gardens which is over 100m from the site.

Stab 9. CGHH Fig 16 does not record any worked ground to this part of Hampstead.

Stab 10A CGHH Fig 8 shows the site is founded on a Secondary A Aquifer

Stab 10B Water will collect to the base of the Claygate Member and particularly under wet winter conditions this will rise above the proposed pool and its surround.

Stab 11. The site is 0.75km to the south of Whitestone Pond as the nearest Hampstead pond

Stab 12. The proposed pool is more than 10m away from Frognal Lane.

Stab 13. The pool is a minimum of 6m from No 38 Frognal Lane. The pool excavation is at +86.5m and so is 3.5m below the foundations to No 38. The excavation to the pool surround is 3.5m from No 40 and is 3.5m deeper.

Stab 14. The Northern Line tunnels are beneath Hampstead High Street to the east of the site.

4.2 Stage 2 Scoping

- The site is on a Secondary A aquifer
- The pool excavation is a minimum of 6m from No 38 Frognal Lane and is 3.5m deeper.
- The pool surround excavation is a minimum of 3.5m from No 40 Frognal Lane and is 3.5m deeper.

4.3 Stage 3 Study and Site Investigation

Survey of Houses
There were no signs of any significant movement in No 40; the category of cracking to the Burland Scale would be Class 0.

There were no signs of any significant movement in No 38 the category of cracking to the Burland Scale would be Class 1.

Land Stability
Both the site walkover and the records in the CGHHS show that there are no problems of land stability in the vicinity of the site. There were no signs of any slippage to the slope and the mature trees are vertical.

Site Investigation
Details of the soil profile established during the site investigations are given in Appendix 3. There is 1.3m of made ground overlying the Claygate Member which extends to at least 5.1 in depth overlying weathered London Clay. The Claygate Member has a firm consistency. The house is currently founded in the Claygate Member. The pool will be founded in the London Clay.

4.4 Ground Rebound and Movement

There are two types of ground movement associated with basement excavations:

i. Rebound due to removal of the soil mass
ii. Settlement and inward deflexion of the basement walls due to:
    a. the insertion of the piles themselves
    b. The removal of the soil mass in front of the wall.

Estimation of settlement and inward deflexion of the walls is in accordance with CIRIA C580, Section 2.5.2. Ref 2.

<u>Rebound</u>
The removal of the soil within the excavation will reduce the load on its base and the ground will recover or 'rebound'. This is assessed in Appendix 2 assuming Boussinesq elastic stress distribution with the application of Newmark Charts.

The following four cases have been assessed;

i.	the eastern wall to the pool enclosure:	11mm maximum in centre and 5.5mm at corners
ii.	the western flank wall to No 40:	4mm maximum in centre and 2mm at corner
iii.	the western wall to the pool	14mm maximum in centre and 7mm at corners
iv.	the eastern flank wall to No 38	4mm at western corner and 6mm at eastern corner

The boundary retaining wall is nearer to the pool than No 38 flank wall and its rebound will be greater. Interpolating between the pool west wall and No 38 would suggest that this will be between 5mm and 6mm.

<u>Insertion of Piles</u>
As C580 table 2.2 a secant piled wall will generate:
0.08% horizontal movement beside the pile reducing linearly over 1.5 times the pile depth
0.05% vertical movement beside the pile reducing linearly over 2 times the pile depth.

With excavations of 3.5m, the piles will be of the order of 10m deep. The movements associated with such piles are given in Appendix 2.

<u>Removal of Soil Mass in front of the wall</u>

Horizontal Strain
In accordance with C580, table 2.4, the ground surface movements will be:

Unpropped:	0.4% of the excavation depth at the face of the wall. As C580 Fig 2.11, this reduces linearly to zero at over four times the excavation depth. This is equivalent to 0.1% horizontal strain.
Propped:	0.15% of the excavation depth at the face of the wall. As C580 Fig 2.11, this reduces linearly to zero at over four times the excavation depth. This is equivalent to 0.04% horizontal strain.

The excavation will be propped.

Vertical Settlement
In accordance with CIRIA C580, fig 2.11, the vertical settlement for a propped excavation will be:

0.05% at the wall
0.08% at one excavation depth
0.04% at two excavation depths
0.0% at three excavation depths

<u>Combined Movements and Strains</u>
The pile and excavation strains are reported on different bases and these need to convert to actual movements so the accumulative value can be calculated.

As Appendix 2, the sections considered are the face of the building nearest the excavation and multiples of the excavation depth back across the building. The vertical displacement Δ is the difference between the average slope and the specific displacement of a section across the building. This has been taken as a third of the way across Nos 38 and 40. The Horizontal Strain is taken across the whole building.

With the boundary wall, the nearest and furthest corners are considered.

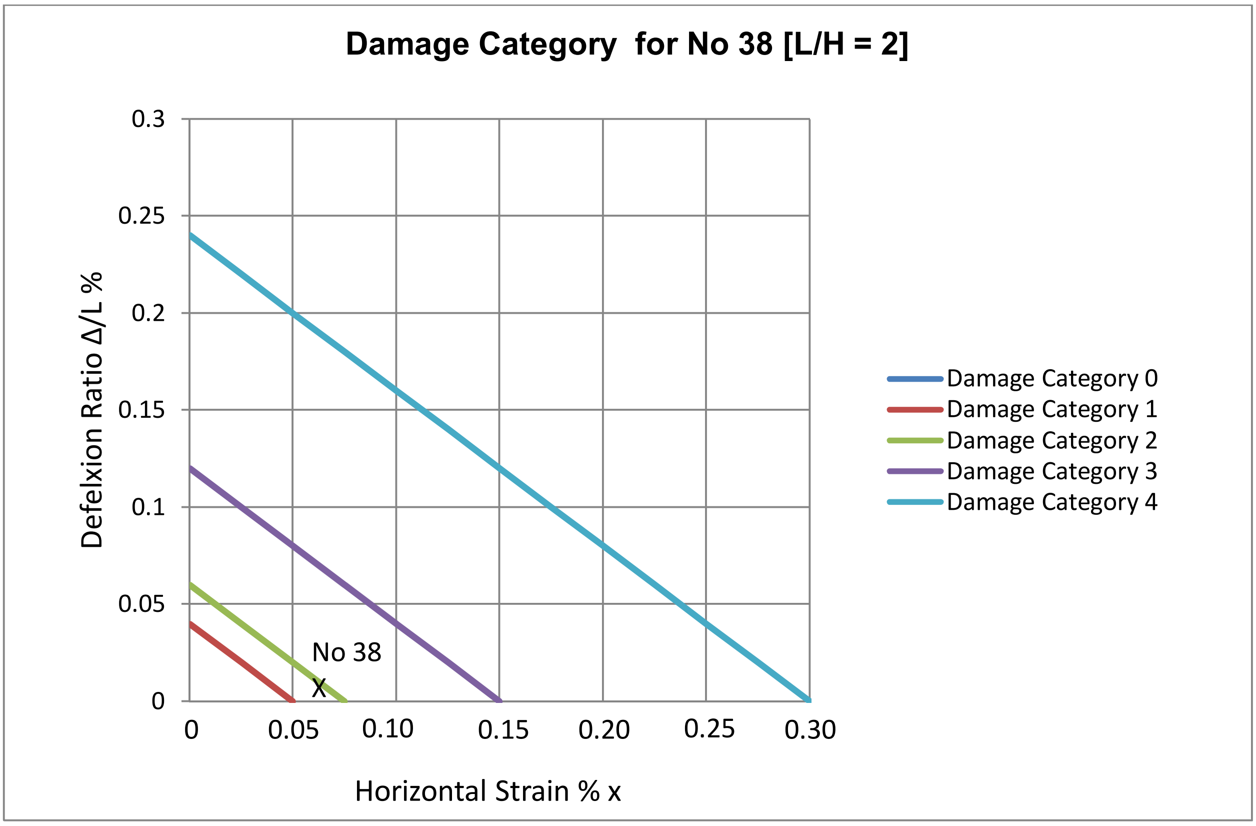
4.5 Damage to Existing Structures.

Damage occurs when the tension strains in the building fabric exceed a critical value and cracks form. A limiting strain, ε_{lim}, can be defined for different sizes of cracks, or damage classifications.

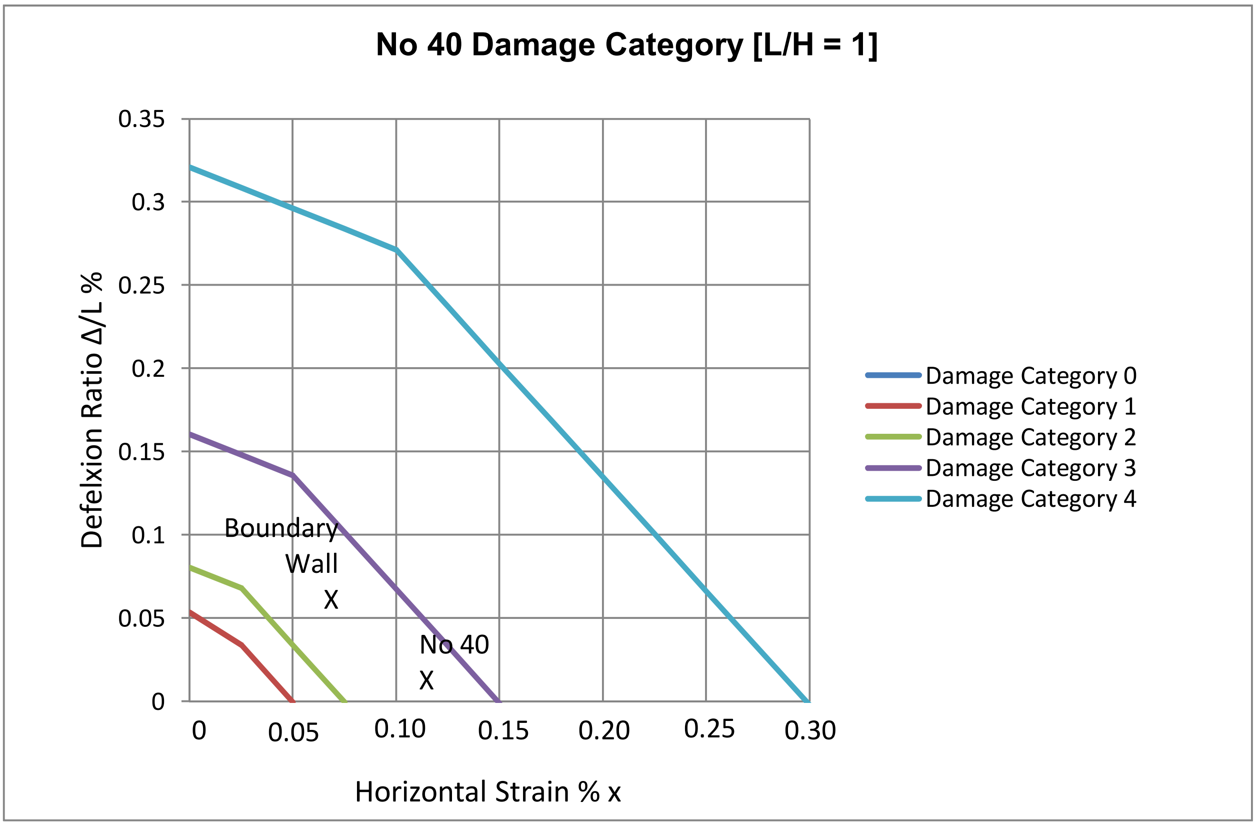
The two key tension strains are vertical distortions and horizontal tension strain from the settlement/rebound wave. The façade of the building can be considered as a large deep beam with the bending and diagonal strains within it depend on its proportions, i.e. ratio of the Length/Height, L/H. On tall narrow buildings, with L/H below unity, diagonal cracking predominates whereas on long terraces, bending predominates.

Strain Envelopes can be defined as proportions of ε_{lim} for various L/H ratios. This is of limited value and it is more useful in practice to develop envelopes of different damage categories for a given façade proportion.

The data from Appendix 2 is presented on the graphs for the two buildings



No 38 is of Damage Category 1



No 40 is of Damage Category 2. No 40 will be redecorated and repaired as part of the works. Redecoration and repair of No 38 can be addressed through the party wall award arrangements.

No 38 & 40 Mitigation.
CPG4 Para 3.30, requires mitigation measures where the risk of damage is Category 1 or higher. The mitigation is that the pool excavation be propped.

Nos 38/40 Boundary Wall.
The strain models assume that the building is radial to the excavation; they are not applicable when the structure is tangential to the excavation.

In Appendix 2, the horizontal strains across the width of the wall have been calculated at both pool corners. ; At 2m difference in clearance, the strains are the same; there is not the same change in strain as with a radially orientated structure. For illustrative purposes only, the calculated strains for the wall have been plotted on L/D=1 graph and are Damage Category 2.

The horizontal movement of the nearest and furthest corners is 7.8 and 6.4mm respectively. On a 22m pool, this gives a horizontal strain of 1.4mm/22m = 6 x 10^{-5}. The vertical movement is 6.4mm and 5.3mm respectively. Considering the transition back to zero movement beyond the pool, Δ will some be of the order of 5mm on a distance of say 30m, giving a Deflexion Ratio of 5mm/30m = 1.6 x 10^{-4}. This would be Damage Classification 0.

4.6 Stage 4 Impact Assessment

Condition of the Houses
No 40 is robust and free from any significant distortions or cracks; it will be able to accommodate the construction of the proposed staircase link, but tanking may be required along the western side to guard against any upstream water table rises with the construction of the pool.

No 38 is also robust and free from any significant distortions or cracks;

Slope Gradients and Land Stability
There is no evidence of any slope instability.

Trees
The mature trees along the western boundary are being maintained and whilst there may be some marginal impact on their root bowls, this will not affect the stability of the ground.

Groundwater
Water table levels and groundwater flow are considered in 3.4. Under winter conditions, the water table will rise above the level of both the pool and surround and these should be designed for hydrostatic and buoyancy loadings.

Depending on the timing of the construction works, excavation beneath the water table will need to be considered. There are established techniques for such construction and these are discussed further in Appendix 1.

Whilst spring lines tend to cause local stability problems, these are too remote to be of concern.

Stability of No 38 Frognal Lane
The swimming pool will be excavated to +86.5m OD. Assuming that the foundations to No 38 extend 0.9m below ground level to a depth of +90m, the pool will be 3.5m lower at an average clearance of 7m.

As Section 4.5, the movement associated with a propped secant piled wall will be Burland Category 1. The use of props is the mitigation measure, reducing the anticipated damage to acceptable levels. Sensibly as part of the Party Wall Notice, a trial pit should be taken down beside the foundations to No 38 to verify their precise depth and a schedule of condition will be required.

Stability of No 40 Frognal Lane
As Section 4.5, the movement associated with a propped secant piled wall will be Burland Category 2. The use of props is the mitigation measure, reducing the anticipated damage to acceptable levels. Since the construction of the pool is being undertaken by the owner of No 40, the effects of the works on the house will be monitored.

Stability of Nos 38 and 40 Boundary Wall
The boundary wall is nearer to the pool than either of the buildings and will be subjected to greater movement. The wall is tangential to the zone of influence and whilst the orthodox application of ground strains and deflexion ratios are not applicable, such an assessment is still Burland Category 2, reducing to Category 0 if considered along its own length.

CPG4 Para 30 relates to internal damage and the inconvenience in repairing and redecorating neighbouring properties and homes. Any damage to the boundary wall will be superficial and can be repaired without compromising the integrity of the wall. There is no need for any mitigation.

Ground Movement Monitoring
It is not clear how far the elastic rebound discussed in Section 4.4 has been incorporated in the field data on which C580 is based. As Appendix 2, the predicted maximum vertical settlements will be of the order of 7mm for No 40 and 5mm for No 38. These figures can hence be considered an upper bound value with the actual figures tending to be less. Given these magnitudes, a formal ground movement monitoring regime is neither required nor justified.

Apart from a general requirement to “monitor” in CPG4 Para 3.17, there is no specific requirement for ground movement monitoring to be considered in a BIA. Monitoring is included in Para 3.37, relating to Section 106 Basement Construction Plans, and Para 4.3 with Construction Management Plans, but both of these are post planning.

The actual building to Nos 38 and 40 will be better at monitoring movement and this can, and should, be left to the detailed design stage. Any fine cracks that develop can be addressed with the normal party wall award procedures.

4.7 Stability Summary

- The pool should be designed for buoyancy loads with excavation being undertaken beneath the water table.
- Propped secant piles will be used to ensure that the impact of the pool on Nos 38 or 40 Frognal Lane is minimised.
- The depth of No 38 foundations is should be investigated as part of the Party Wall Notices to establish whether these need to be included in the pile design
- The need for movement monitoring should be considered during detailed design.

5 SURFACE FLOW AND FLOODING

5.1 Stage 1 Screening

Flood 1. The site is not within Hampstead Ponds Catchment

Flood 2. There will not be any material changes in surface water flows to Frognal Lane. The forecourt will remain as a gravel surface to give a permeable surface with the roofs discharging to the surface water drains as at present.

Flood 3. There is no increase in the impervious area.

Flood 4. There will be no changes in flow rate onto neighbouring land

Flood 5. There will be no changes in quality of water discharge

Flood 6. Floods in Camden [2003] records that Frognal Gardens flooded in 1975 and Chesterford Gardens in 2002.

CGHH Fig 15, Flood Map, also shows that Frognal flooded in 2002, but this is not listed in Floods in Camden

Flood 6A The Surface Water Management Plan, 2013, Fig 3.1 shows that LFRZ 3015, Frognal is located to the east of the site

5.2 Stage 2 Scoping

- Possible issue with the site located to the side of a Local Flood Risk Zone.

5.3 Stage 3 Study and Site Investigation

<u>Frognal LFRZ 3015</u>
The Surface Water Management Plan 2013 defines a Local Flood Risk Zone, LFRZ 3015 to the east and north of the site encompassing Frognal, Frognal Gardens and Windmill Hill. The concern is surface water flooding and sewer capacity problems which caused water to collect behind the railway cutting to the south. This may have been partly resolved by the Sumatra Road Scheme implemented by Thames Water. This scheme provides underground storage of storm water which is held before delayed discharge into the combined sewer network.

5.4 Stage 4 Impact Assessment

Frognal LFRZ 3015
Frognal Lane is adjacent to Frognal LFRZ.

The flooding records point to the higher ground to the north, as represented by Frognal Gardens and Chesterford Gardens, as being liable to flood. Any surface water flowing down Chesterford Gardens will be below the site.

There may be some side flow from Frognal along Frognal Lane, but this will continue down the slope of Frognal Lane, rather than collecting at the shared access to No 40.

The pool is more than 10m from the road and will be in a waterproof box. As good detailing, the two sky lights to the pool should be taken up as water resilient construction for a freeboard of 0.3m above the garden level. This will ensure that any surface flooding does not impact on the pool. The clerestory light is just above the level difference with the western boundary to No 38 and the levels of the surrounding area can be landscaped to ensure that water is not trapped at this location.

5.5 Flooding Summary

- Whilst the site is adjacent to a Local Flood Risk Zone, there are no indications that any surface water flows along the road frontage would impact on the pool.

6 ADDITIONAL IMPACT ASSESSMENTS

6.1 Sustainability Amenity and Landscape

The application meets all the relevant requirements of current policy and no untoward impact is expected in these categories.

6.2 Lightwells

There are two skylights within the garden and a clerestory window along the western side. These are not visible from the road and have minimal impact.

6.3 Tree Protection

The magnolia tree and the two trees along the western boundary are to remain. The magnolia tree is 3m clear of the pool and the northern of the western boundary trees is 5m clear.

Tree protection measures will be included to protect the canopy of the trees and their roots.

6.4 Third Party Considerations and Impact on Neighbours

The rear gardens to the surrounding properties act as a buffer and there will be no significant impact on these.

6.5 Cumulative Impacts

The environmental setting is such that the impacts of the proposed scheme are minimal and as such there is no cumulative impact.

7 SUMMARY

The site is found on the Claygate Member with the pool extending down into the London Clay below. The pool will be constructed as a secant piled box in a garden remote from the adjacent buildings and will be propped prior to excavation. The propping will ensure that there is minimal movement in the surrounding ground and in particular that the foundations to No 38 Frognal Lane are not affected.

The groundwater throughflow obstructed by the pool has a daily average of less than 0.2m^3/day, and is too small and too remote to impact on the catchments areas or flow in the river tributaries down slope of the site.

The pool will affect the groundwater table immediately around it. A high permeability drainage material will be placed outside of the cut-offs and constructed impermeable basement, to route groundwater flows from the up-gradient side to the down-gradient side, thereby maintaining pre-development groundwater flows and groundwater levels around the site and preventing local alterations to groundwater levels

The basement box should be designed for hydrostatic and buoyancy loadings. The excavation of the proposed pool will use established techniques; during wet winter months, techniques dealing with construction beneath the water table will be required. Depending on the level of the foundations to No 38, these may generate a loading case on the pool perimeter piles and a trial pit will be required to verify whether the foundations are so shallow that such a design loading occurs.

There are no problems of ground stability or surface water flooding and this means that there are no concerns with the environmental setting of the site. Sensibly Nos 38 and 40 should be used to monitor movement, but there is no need for a ground movement monitoring regime.

There is nothing in this BIA to suggest that the construction of the proposed basement and swimming pool will have a detrimental impact on the site, neighbouring sites or natural environment.

8 REFERENCES

Ref 1: CIRIA SP200; Building response to Tunnelling: Case Studies from construction of the Jubilee Line Extension, London. 2001

Ref 2: CIRIA C580; Embedded Retaining Walls – Guidance for economic design

Appendix 1 Construction Techniques

Construction below the Water Table

Construction below the water table is a common civil engineering requirement and there are a number of techniques available to achieve this. The two common methods are:

i. Exclusion using a cofferdam solution;
ii. Groundwater lowering using pumps.

These methods are not exclusive and a hybrid of both is often used. It needs to be recognised that de-watering by pumping can lead to ground settlement.

The cross section of the pool is 23m, extending below the Claygate Member into the London Clay.

Whist the water table will extend up to 3m above the London Clay, the anticipated groundwater throughflow is so small that this can easily be accommodated by pumping.

With the piles founded in the underlying London Clay as a cut off, a solution would be to install sumps on the inside of the excavation with pumps used to control the seepage through the joints between the contiguous piles. The pumps would be controlling water seeping into the excavation and would not be used to lower the water table. As discussed in 3.4, a further phase of ground water monitoring is required and in the absence of any assessment of the anticipated seepage, secant piling has been adopted.

The excavation would be taken down within the secant piles to below the base of the Claygate Member and the upper sections of the liner wall completed to give an impervious construction with a cut off into the London Clay. To minimise the impact of the obstruction to the ground water throughflow, a granular drainage layer will be installed around the outside of the piled wall once the upper liner wall is completed.

Sensibly the works should be programmed in the drier summer months when the water table is lower anyway.

A construction variation could be to construct a secant piled wall which would more expensive but which would only require minimal pumping and would allow the liner wall to be constructed as a single element.

Appendix 2 Ground Movement Calculations

40 Frognal Lane, NW3 6PP

One Dimensional Basement Rebound Calculations

Revision 01 October 2016

1. Excavation

1.1 Plan Dimensions

1.1.1 The pool enclosure is 21m long by 8m wide, with the pool on the western side. The enclosure is parallel with No 40 and 4m from the main house.

1.1.2 No 38 Frognal Lane is at a slight angle to the pool and only extends back a third of the pool length. The rear corner of No 38 is 8m from the side of the pool. The front corner of No 38 is 6m from the corner of the pool.

1.2 Relative Levels

1.2.1 Taking the western garden as a maximum ground level at +93.5m, the pool surround is 5m deep at +88.5m with the pool a further 2m at +86.5m.

1.2.2 The house has a lower ground floor which is +92m; the foundations are likely to be +91.5m. This means the pool surround excavation is 3m below the foundations and the deep end is 5m below

1.2.3 No 38 is at +91m and its foundations are at +90m. Taking an average depth to the excavation of the pool and surround of 87m, this means the pool itself is 3m below No 38.

1.3 Construction

1.3.1 The pool will be constructed with a perimeter piled wall which will be constructed before any excavation is undertaken.

2. Soil Data

2.1 Investigation Data

2.1.1 As AP Geotechnics Report, 2011, two boreholes were sunk as part of the investigations establishing that the site is founded on London Clay.

2.1.2 The soil profile is Claygate Members to 5m overlying London Clay which was proved to a depth of 20m.

2.2 Strength and Stiffness Parameters

2.2.1 The soil data has been taken from AP Geotechnics Site Investigation. The four triaxial results between 6m and 10m were in the range 120kN/m^2 and 140kN/m^2, with the value at 12.5m being 110kN/m^2.

2.2.2 The SPTs increased in value with depth with an average value at 17.5m of 39. After Stroud, this would give a C_u =5.5 x SPT value = 214kN/m^2.

2.2.3 The undrained strength profile in the rebound calculations has been taken as:

5m depth: C_u = 100kN/m^2
20m depth C_u = 200kN/m^2

2.2.4 The undrained stiffness of London Clay ranges typically from 40MN/m^2 to over 160MN/m^2 and suggested correlations between undrained cohesion, C_u, and undrained Youngs Modulus, E_u, range between E_u = 400C_u to 600 C_u

2.2.5 A value of E_u = 425 C_u has been adopted in the current analysis of the rebound. [CIRIA C580, Appendix 2]

3. Analysis

3.1 Analysis

3.1.1 The analysis is based on Boussinesq elastic stress distribution utilising Newmark Charts to give the stress below the corner of a rectangle. Two metre thick slices were considered with the reduction in pressure on each slice after the excavation of the basement. Using the E_u for each slice, the change in strain can be obtained and from this the change in thickness for each slice.

3.1.2 The four locations considered are:

i. The centre of the pool enclosure eastern wall.
ii. The corner of the western wall to the house which aligns with the centre of the pool excavation.
iii. The centre of the pool western wall with the deeper excavation
iv. The front corner of No 38 eastern wall which aligns with the corner of the pool at 6m clearance. The difference in level has been modelled by taking adjusting the backfill coefficient in proportion to the relative depth between the over dig and backfill.

3.1.3 No allowance has been made for the reinforcing effect of the perimeter piles, which will reduce the heave through adhesion.

3.2 Analysis Cut Off

3.2.1 In theory the Boussinesq elastic stress distribution, extends to infinity. In practice the Newmark curves used to find the coefficients stop when the depth is more than 10 times the narrowest rectangle dimension.

3.2.2 The analyses were stopped when either:

i. The change in stress due to the excavation was less than 1/200 of the overburden stress;
ii. The heave was less than 0.1mm on a 2m slice.

3.3 Drained Parameters

3.3.1 Drained heave results would be higher than the undrained values, but would take a greater time to achieve the necessary pore water pressure equilibrium. Given that the basement box construction will commence as soon as the excavation is complete the net reduction in overburden will be reduced and any drained results would relate to the completed building.

4. Results

4.1 Centre of Pool Enclosure Eastern Wall

4.1.1 Based on a 5m excavation, the anticipated rebound is 11mm in the middle of the wall, reducing to 5mm to 6mm at the corners.

4.2 Corner of Western Flank Wall to No 40 Frognal Lane

4.2.1 The anticipated rebound is 4mm in the middle of the pool, which is close to the front corner of the flank wall. This will reduce to 2mm at the rear corner of the pool which is close to the rear corner of the house.

4.3 Centre of Pool Western Wall

4.3.1 Based on a 7m excavation with a 2m backfill for the pool surround, the anticipated rebound is 14mm in the middle of the wall, reducing to 7mm at the corners.

4.4 No 38 Frognal Lane Eastern Flank Wall

4.4.1 The anticipated rebound is less than 4mm at the front corner of No 38. The difference in levels between Nos 38 and 40 gardens is taken into account in generating the Newmark Coefficients. The rear corner will be towards the centre of the pool, but given that the clearance increases, the two parameters will tend to cancel each other out; the back corner of the flank wall could have a rebound of say 5mm.

14/10/2016 | Job No: 10998

40 Frognal Lane NW3
Rebound to Pool Enclosure Eastern Wall with Pool Excavation

Excavation	21	m	8	m	Soil Density kN/m^3	18		Strength Profile		
								Depth		C_u
					Basement Excavation BGL	5.0	m	5.0	m	100 kN/m^2
Newmark Rectangles	11	m x	8	m x 5m	Overburden removed	-90	kN/m^2	20.0	m	200 kN/m^2

	Depth BGL	Depth below Base	Av below Base	Newm'k Rect'gle Coeffs	Coeff Total 2xCoeff	Overburden Orig	Overburden Re-moved	After Excav-ation	Layer Thick m	C_u kN/m^2	E_u= 425C_u MN/m^2	Rebound mm	Rebound Acc
Existing Grd	0												
Pool Side	5.0	0				90.0				100			
			1	0.250	0.500	108.0	-45.0	63.0	2.00	107	45.33	-2.0	-10.6
	7	2.0											
			3	0.243	0.486	144.0	-43.7	100.3	2.00	120	51.00	-1.7	-8.6
	9	4.0											
			5	0.226	0.452	180.0	-40.7	139.3	2.00	133	56.67	-1.4	-6.9
	11	6.0											
			7	0.211	0.422	216.0	-38.0	178.0	2.00	147	62.33	-1.2	-5.5
	13	8.0											
			9	0.189	0.378	252.0	-34.0	218.0	2.00	160	68.00	-1.0	-4.3
	15	10.0											
			11	0.145	0.290	288.0	-26.1	261.9	2.00	173	73.67	-0.7	-3.3
	17	12.0											
			13	0.128	0.256	324.0	-23.0	301.0	2.00	187	79.33	-0.6	-2.5
	19	14.0											
			15	0.110	0.220	360.0	-19.8	340.2	2.00	200	85.00	-0.5	-2.0
	21	16.0											
			17	0.093	0.186	396.0	-16.7	379.3	2.00	200	85.00	-0.4	-1.5
	23	18.0											
			19	0.080	0.160	432.0	-14.4	417.6	2.00	200	85.00	-0.3	-1.1
	25	20.0											
			21	0.068	0.136	468.0	-12.2	455.8	2.00	200	85.00	-0.3	-0.8
	27	22.0											
			23	0.060	0.120	504.0	-10.8	493.2	2.00	200	85.00	-0.3	-0.5
	29	24.0											
			25	0.053	0.106	540.0	-9.5	530.5	2.00	200	85.00	-0.2	-0.2
	31	26.0											
			27	0.046	0.092	576.0	-8.3	567.7	2.00	200	85.00	-0.2	-0.2
	33	28.0											
			29	0.041	0.082	612.0	-7.4	604.6	2.00	200	85.00	-0.2	-0.2
	35	30.0											
			31	0.037	0.074	648.0	-6.7	641.3	2.00	200	85.00	-0.2	-0.2
	37	32.0											

Notes
1. London Clay Strength Profile from AP Geothechnics Report
2. Eu/Cu= 425 after Ho (1991), as reported in CIRIA C580

14/10/2016 Job No: 10998

40 Frognal Lane NW3
Rebound to House Western Wall
with Pool Excavation

Excavation	21	m	8	m	Soil Density kN/m^3	18		Strength Profile		
Newmark Rectangles								Depth	C_u	
Over Dig	11	m x	12	m	Basement excavation BGL:	5.0	m	5.0 m	100	kN/m^2
Back Fill	11	m x	4	m x 5m deep	Overburden removed	-90	kN/m^2	20.0 m	200	kN/m^2

	Depth BGL	Depth below Base	Av below Base	Newmark Coefficients Over Dig	Back Fill	Coeff Total [2 x Diff]	Overburden Orig	Re-moved	After Excav-ation	Layer Thick m	C_u kN/m^2	E_u= 425C_u MN/m^2	Rebound mm	Acc
Exist Grd	0													
Pool Side	5.0	0					90.0				100			
			1	0.250	0.248	0.004	108.0	-0.4	107.6	2.00	107	45.33	0.0	-3.8
	7	2.0												
			3	0.246	0.223	0.046	144.0	-4.1	139.9	2.00	120	51.00	-0.2	-3.8
	9	4.0												
			5	0.237	0.182	0.110	180.0	-9.9	170.1	2.00	133	56.67	-0.3	-3.6
	11	6.0												
			7	0.220	0.145	0.150	216.0	-13.5	202.5	2.00	147	62.33	-0.4	-3.2
	13	8.0												
			9	0.211	0.113	0.196	252.0	-17.6	234.4	2.00	160	68.00	-0.5	-2.8
	15	10.0												
			11	0.177	0.090	0.174	288.0	-15.7	272.3	2.00	173	73.67	-0.4	-2.3
	17	12.0												
			13	0.155	0.075	0.160	324.0	-14.4	309.6	2.00	187	79.33	-0.4	-1.9
	19	14.0												
			15	0.136	0.063	0.146	360.0	-13.1	346.9	2.00	200	85.00	-0.3	-1.5
	21	16.0												
			17	0.120	0.053	0.134	396.0	-12.1	383.9	2.00	200	85.00	-0.3	-1.2
	23	18.0												
			19	0.106	0.043	0.126	432.0	-11.3	420.7	2.00	200	85.00	-0.3	-0.9
	25	20.0												
			21	0.093	0.037	0.112	468.0	-10.1	457.9	2.00	200	85.00	-0.2	-0.6
	27	22.0												
			23	0.083	0.031	0.104	504.0	-9.4	494.6	2.00	200	85.00	-0.2	-0.4
	29	24.0												
			25	0.071	0.027	0.088	540.0	-7.9	532.1	2.00	200	85.00	-0.2	-0.2
	31	26.0												

Notes
1. London Clay Strength Profile from AP Geotechnics Report
2. Eu/Cu= 425 after Ho (1991), as reported in CIRIA C580

14/10/2016 | Job No: 10998

40 Frognal Lane NW3
Rebound to Pool Western Wall
with Pool Excavation

Excavation	18	m	5	m			Soil Density kN/m^3	18		Strength Profile				
Newmark Rectangles										Depth		C_u		
Pool	9	m x	8	m x	7	m deep	Basement excavation BGL:	7.0	m	5.0	m	100	kN/m^2	
Pool Side	9	m x	3	m x	2	m thick	Overburden removed	-126	kN/m^2	20.0	m	200	kN/m^2	

	BGL	D'pth below Pool	Av below Pool	N'm'k Coeff Pool Dig	D'pth below Side	Av below Side	N'm'k Coeff Side Fill	Coeff Total [See Footer]	Overburden Orig	Overburden Re-moved	After Excav-ation	Layer Thick m	C_u kN/m^2	E_u= 425C_u MN/m^2	Rebound mm	Rebound Acc
EGL	0															
Pool	5.0				0				90.0				100			
Side						1	0.250	-0.143	108.0	18.0	126.0	0.00	100	42.50	0.0	-13.4
Pool	7	0			2											
Base			1	0.250		3	0.223	0.373	144.0	-46.9	97.1	2.00	107	45.33	-2.1	-13.4
	9	2			4											
			3	0.246		5	0.182	0.388	180.0	-48.9	131.1	2.00	120	51.00	-1.9	-11.3
	11	4			6											
			5	0.237		7	0.145	0.391	216.0	-49.3	166.7	2.00	133	56.67	-1.7	-9.4
	13	6			8											
			7	0.220		9	0.113	0.375	252.0	-47.3	204.7	2.00	147	62.33	-1.5	-7.6
	15	8			10											
			9	0.211		11	0.090	0.371	288.0	-46.7	241.3	2.00	160	68.00	-1.4	-6.1
	17	10			12											
			11	0.177		13	0.075	0.311	324.0	-39.2	284.8	2.00	173	73.67	-1.1	-4.7
	19	12			14											
			13	0.155		15	0.063	0.274	360.0	-34.5	325.5	2.00	200	85.00	-0.8	-3.7
	21	14			16											
			15	0.136		17	0.053	0.242	396.0	-30.5	365.5	2.00	200	85.00	-0.7	-2.9
	23	16			18											
			17	0.120		19	0.043	0.215	432.0	-27.1	404.9	2.00	200	85.00	-0.6	-2.1
	25	18			20											
			19	0.106		21	0.037	0.191	468.0	-24.0	444.0	2.00	200	85.00	-0.6	-1.5
	27	20			22											
			21	0.093		23	0.031	0.168	504.0	-21.2	482.8	2.00	200	85.00	-0.5	-0.9
	29	22			24											
			23	0.083		25	0.027	0.151	540.0	-19.0	521.0	2.00	200	85.00	-0.4	-0.4
	31	24			26											

Notes
1. Strength Profile from AP Geotechnics Report
2. Eu/Cu= 425 after Ho (1991), as reported in CIRIA C580
3. Level Differences: Pool Side backfilled 2m Pool excavation 7m = 2/7=29% Coefficient Total is 2(Coeff Pool-0.29Coeff Side)

40 Frognal Lane NW3
Rebound to No 38 Eastern Flank Wall
with Pool Excavation

Excavation	19	m	8	m		Soil Density kN/m^3	18		Strength Profile		
Newmark Rectangles									Depth	C_u	
Over dig	9	m x	15	m x	6 m deep	Basement excavation BGL:	7.0	m	5.0 m	100	kN/m^2
Backfill	9	m x	7	m x	4 m thick	Overburden removed	-126	kN/m^2	20.0 m	200	kN/m^2

	BGL	D'pth below Pool	Av below Pool	N'm'k Coeff Pool Over Dig	D'pth below Side	Av below Side	N'm'k Coeff Garder Back Fill	Coeff Total [See Footer]	Overburden Orig	Re-moved	After Excav-ation	Layer Thick m	C_u kN/m^2	E_u= 425C_u MN/m^2	Rebound mm	Acc
EGL	0															
													113			
Garder	6.0	-1			0				108.0							
			-0.5			0.5	0.250	-0.167	117.0	21.0	138.0	1.00	110	46.75	0.4	
Pool	7	0			1											
Base			1	0.250		2	0.247	0.085	144.0	-10.8	133.2	2.00	120	51.00	-0.4	-3.4
	9	2			3											
			3	0.246		4	0.228	0.094	180.0	-11.8	168.2	2.00	133	56.67	-0.4	-3.0
	11	4			5											
			5	0.235		6	0.195	0.105	216.0	-13.2	202.8	2.00	147	62.33	-0.4	-2.6
	13	6			7											
			7	0.207		8	0.170	0.094	252.0	-11.8	240.2	2.00	160	68.00	-0.3	-2.2
	15	8			9											
			9	0.193		10	0.141	0.099	288.0	-12.5	275.5	2.00	173	73.67	-0.3	-1.8
	17	10			11											
			11	0.167		12	0.119	0.088	324.0	-11.0	313.0	2.00	187	79.33	-0.3	-1.5
	19	12			13											
			13	0.148		14	0.096	0.084	360.0	-10.6	349.4	2.00	200	85.00	-0.2	-1.2
	21	14			15											
			15	0.132		16	0.082	0.077	396.0	-9.7	386.3	2.00	200	85.00	-0.2	-0.9
	23	16			17											
			17	0.114		18	0.064	0.071	432.0	-9.0	423.0	2.00	200	85.00	-0.2	-0.7
	25	18			19											
			19	0.102		20	0.057	0.064	468.0	-8.1	459.9	2.00	200	85.00	-0.2	-0.5
	27	20			21											
			21	0.087		22	0.050	0.054	504.0	-6.8	497.2	2.00	200	85.00	-0.2	-0.3
	29	22			23											
			23	0.080		24	0.043	0.051	540.0	-6.5	533.5	2.00	200	85.00	-0.2	-0.2
	31	24			25											

Notes
1. Strength Profile from AP Geotechnics Report
2. Eu/Cu= 425 after Ho (1991), as reported in CIRIA C580
3. Difference in levels: No 38 Garden 4m backfill on 6m av pool excavation= 4/6=75% Coefficient Total = Coeff Pool-0.75xCoeff Garden

40 Frognal Lane NW3 6PP
Ground Movement

Pile Type: Secant Depth 10 m
Horizontal Strain 0.08 % at pile reducing to 0.04 % at 0.5 pile length and linearly to zero over a further 1 pile length
Vertical Strain 0.05 % at pile reducing linearly to zero over 2 pile lengths

Excavation Propped
Horizontal Strain 0.2 % at wall reducing linearly to zero over 4 excavated depths
Vertical Strain 0.1 % at one excavation depth reducing linearly to zero over next 2 excavated depths

Building	Pool	Dept Diff m	Location	Clearance m	% of Pile	Horizontal Movement Excavate ε %	Horizontal Movement Excavate mm	Horizontal Movement Pile ε %	Horizontal Movement Pile mm	Horizontal Movement Total mm	Av Diff mm	Av Dist m	Av ε %	Vertical Movement Excavate ε %	Vertical Movement Excavate mm	Vertical Movement Pile ε %	Vertical Movement Pile mm	Vertical Movement Total mm	Diff mm	Dist m
No 40	Edge	3.5	Building Face	3.5	35%	0.113	3.9	0.078	7.8	11.7				0.080	2.8	0.041	4.1	7.0		
																			2.3	3.5
L/H =	1		3.5 m back	7	70%	0.075	2.6	0.032	3.2	5.8				0.040	1.4	0.033	3.3	4.7		
											11.3	10.5	0.11						2.3	3.5
			7 m back	10.5	105%	0.038	1.3	0.018	1.8	3.1				0.000	0.0	0.024	2.4	2.4		
																			0.9	3.5
			11 m back	14	140%	0.000	0.0	0.004	0.4	0.4				0.000	0.0	0.015	1.5	1.5		
																		Σ =	5.5	10.5

Displacements
Vertical Difference: Building Face to 3.5 m back 2.3 mm
Av slope = ΣDiff/ΣDist = 0.05 % Av Difference on 3.5 m = 1.8 mm
Δ = 0.5 mm
Δ/ΣDist= 0.004 %

Building	Pool	Dept Diff m	Location	Clearance m	% of Pile	Horizontal Movement Excavate ε %	Horizontal Movement Excavate mm	Horizontal Movement Pile ε %	Horizontal Movement Pile mm	Horizontal Movement Total mm	Av Diff mm	Av Dist m	Av ε %	Vertical Movement Excavate ε %	Vertical Movement Excavate mm	Vertical Movement Pile ε %	Vertical Movement Pile mm	Vertical Movement Total mm	Diff mm	Dist m
No 38	Deep End	3.5	Building Face	7	70%	0.075	2.6	0.032	3.2	5.8				0.040	1.4	0.033	3.3	4.7		
																			2.3	3.5
L/H =	2		3.5 m back	10.5	105%	0.038	1.3	0.018	1.8	3.1				0.000	0.0	0.024	2.4	2.4		
											5.8	10.5	0.06						0.9	3.5
			7 m back	14	140%	0.000	0.0	0.004	0.4	0.4				0.000	0.0	0.015	1.5	1.5		
																			0.9	3.5
			11 m back	17.5	175%	0.000	0.0	0.000	0.0	0.0				0.000	0.0	0.006	0.6	0.6		
																		Σ =	4.1	10.5

Displacements
Vertical Difference: Building Face to 3.5 m back 2.3 mm
Av slope = ΣDiff/ΣDist = 0.04 % Av Difference on 3.5 m = 1.4 mm
Δ = 0.9 mm
Δ/ΣDist= 0.009 %

40 Frognal Lane NW3 6PP
Ground Movement

Building	Pool	Dept Diff m	Location	Clearance m	% of Pile	Horizontal Movement Excavate ε %	Horizontal Movement Excavate mm	Horizontal Movement Pile ε %	Horizontal Movement Pile mm	Horizontal Movement Total mm	Av Diff mm	Av Dist m	Av ε %	Vertical Movement Excavate ε %	Vertical Movement Excavate mm	Vertical Movement Pile ε %	Vertical Movement Pile mm	Vertical Movement Total mm	Diff mm	Dist m	ε %
Boundary	Deep		Nearest Corner	5 m																	
Wall	End	4.5	Face of Wall	5	50%	0.108	3.8	0.040	4.0	7.8				0.076	2.6	0.038	3.8	6.4			
L/H =	N/A										0.5	0.7	0.07						0.4	0.7	0.06
			0.7 m back	5.7	57%	0.103	3.6	0.037	3.7	7.3				0.069	2.4	0.036	3.6	6.0			
			Furthest Corner	7 m																	
			Face of Wall	7	70%	0.092	3.2	0.032	3.2	6.4				0.058	2.0	0.033	3.3	5.3			
											0.5	0.7	0.07						0.4	0.7	0.06
			0.7 m back	7.7	77%	0.086	3.0	0.029	2.9	5.9				0.052	1.8	0.031	3.1	4.9			

Appendix 3 Chartered Geologist’s Review and Endorsement

GWP consultants

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Mr N Train
Train and Kemp (Consulting Engineers) LLP
10 Kennington Park Place
London
SE11 4AS

GWP Report No: 150902

Our ref: nt040915 final.docx
Your ref:

07 September 2015

Dear Mr Train

40 Frognal Lane Basement Impact Assessment

Please accept this letter as my formal endorsement of the section on addressing Groundwater Flow matters, contained in the report "Basement Impact Assessment for the basement extension of 40 Frognal Lane, London NW3 6PP".

For the avoidance of doubt to third parties, I have co-authored the Groundwater Flow components of the BIA, having reviewed and amended an earlier draft version (v.0) prepared by Train and Kemp.

My qualifications to undertake this work are as follows:

- BSc (Hons) in Geology Class 2.i,
- MSc in Hydrogeology and Groundwater Resources;
- Fellow of the Geological Society of London (FGS);
- Chartered Geologist (C.Geol);
- European Geologist (EurGeol)
- Chartered Member of the Chartered Institute of Water and Environmental Management (MCIWEM C.WEM).

I am a practising Hydrogeologist and have more than 25 years of experience in groundwater matters.

I have reviewed the public domain geological and topographic maps as well as the site specific ground investigation data for the site. This information confirms the site to be located in a shallow valley depression on the Claygate Member of the London Clay, a moderately low permeability deposit, generally considered to be capable of supporting slow groundwater flow.

It is possible the Claygate Member provides a groundwater baseflow contribution to the tributaries of the River Westbourne, reported to commence some 100m west and 150m east of the site, both at locations with an elevation of 82-83mAOD and close to the reported contact of the Claygate Member and the underlying London Clay.

The 2 No. cable percussion boreholes on the development site and 2 No. window samplers at the adjacent property describe encountering up to 1.30m thickness of Made Ground overlying silty clay to a depth of 5m, underlain by stiff brown fissured clay with occasional sand to a depth of 8m. The Made Ground is described as shingle, brick rubble with some clay. The unweathered London Clay is identifiable at approximately 8.0m depth.

Water strikes were observed in all boreholes during drilling. These were typically near the base of the Made Ground, but also observed near the base of the root zone (approximately 2-3m depth) in the underlying strata, and at greater depth (5-6m) immediately above the stiff brown clay.

A Rising Head Test was undertaken on one borehole, giving an estimated permeability of 4.18 x 10^{-7} m/s. This permeability value is a little high for the Claygate Member, and might be due to the contribution of the connected Made Ground, but I am aware of permeabilities of 10^{-7} m/s reported for this strata type elsewhere.

Standpipes were installed in 3 of the boreholes, although for 2 boreholes the elevations of the seals continued to allow hydraulic connection from the overlying Made Ground. Monitoring of water levels over the following two weeks of each investigation (Aug-Sep 2011 and Dec 2013-Jan 2014) as well as more recently (Apr 2015) show water levels typically 1-3m depth (approximately 89.5 – 90.5mAOD) across the development site.

My interpretation of the above ground investigation and subsequent monitoring is that the Claygate Member is partially saturated and does contain groundwater, albeit within a relatively low permeability stratum. A steep hydraulic gradient (approximately 5 x 10^{-2}) exists across the site, indicating groundwater flow occurs to the west-north-west, and confirming both limited permeability and perching of water on the underlying London Clay.

These groundwater flows are however very small and very slow. Using the above hydraulic gradient and permeability, and assuming a saturated thickness of 4m and cross-sectional width of 25m, the flows can be estimated at 0.002 l/s (*i.e.* 0.18 m^3/d). This quantity of flow is best described as seepage.

The extent to which these seepages are ephemeral or year round is unclear. The groundwater monitoring clearly shows evidence of rainfall recharge replenishing the strata, and the strata do not appear to become entirely dry during the summer months.

It is understood the proposed basement pool extension will consist of a concrete box, approximately 5m wide by 25m long, with an excavation depth of up to 5.0m (88.17mAOD). This excavation will therefore pass through the Made Ground and into the Claygate Member beneath, and at its deepest may be below the top of the weathered London Clay (87.57mAOD) in places. With groundwater levels measured on the site ranging between 89.5-90.5mAOD, the basement will be partially below the groundwater table.

It is understood the construction methods of the basement are likely to include sheet piling, coffer dams and/or concrete piles to ensure the stability of the ground given the close proximity to properties on either side of the proposed excavation. These impermeable structures, as well as the constructed basement concrete box, are likely to act as barriers to the groundwater flowing within the shallowest 5.0m of strata, including therefore the Claygate Member and Made Ground.

Therefore, there is the potential to reduce groundwater flow across the site resulting in groundwater level rise on the up-gradient (east) side of the basement and lower groundwater levels on the down gradient (west) side.

It is understood the construction design includes for the placement of a coarse granular fill material outside of the cut-offs/construction box, to a depth of 0.5m below the monitored summer groundwater levels. The effect of this drainage medium will be to intercept existing groundwater flows and route them around the structure, allowing them to re-infiltrate into the strata on the down-gradient side. This is considered to be an adequate mitigation measure to prevent groundwater level and flow alteration both locally, and more regionally to maintain any baseflows to local water courses.

The Basement Impact Assessment does, in my opinion, correctly characterise the site geology and hydrogeology, identify the potential for minor impacts, the scale of these impacts and how these can be mitigated in the construction methods and basement design. It is my opinion that the groundwater sections of the Basement Impact Assessment are fit-for-purpose.

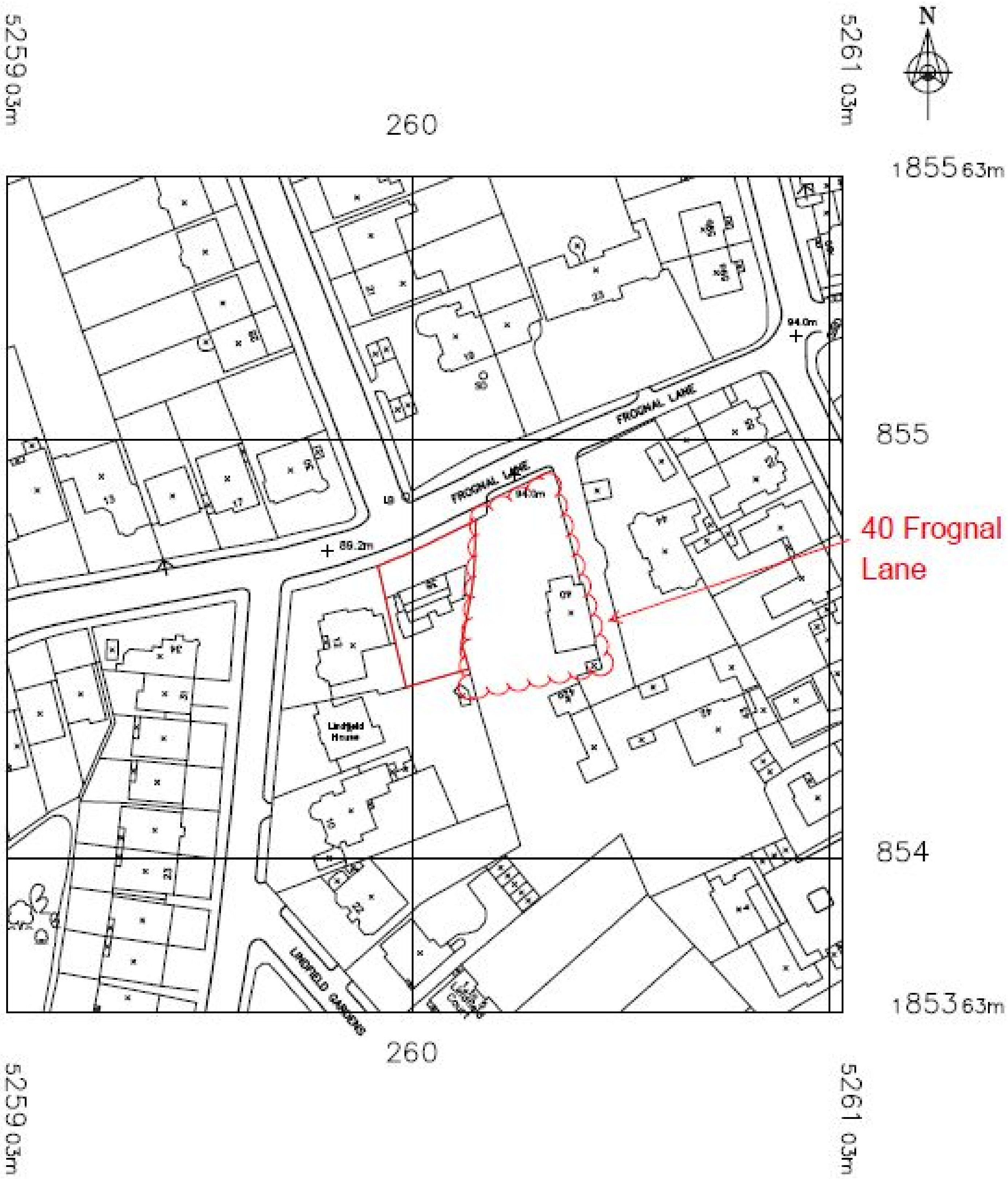
I would recommend re-commencing groundwater level monitoring in the 3 No. standpipes over the next three months (at monthly intervals) to further inform the depth to which granular material should be placed outside of the cut-off box. This will ensure groundwater flow re-routing from the up hydraulic gradient to the down gradient side.

Yours sincerely

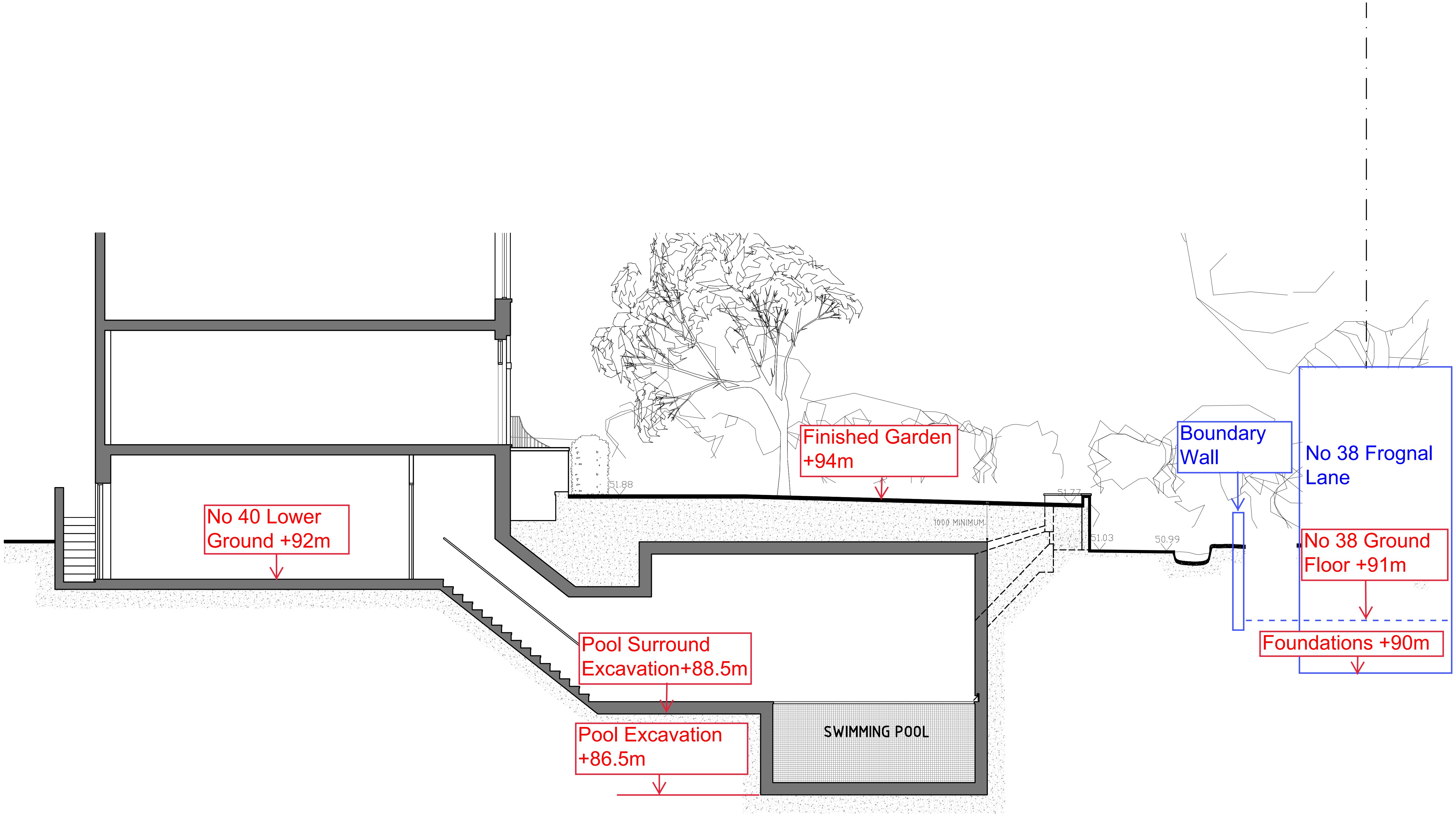
Clive Carpenter
BSc (Hons) MSc FGS CGeol MICWEM EurGeol AMAE

Chief Hydrogeologist and Equity Partner

Appendix 4 Site Location Plan and Basement Drawings



OS Extract
Scale 1:1250



Conceptual Site Model and Levels

AMENDMENTS	
A	SOIL DEPTH INCREASED TO 1m 05-11 NRLT
B	
C	
D	
E	
F	
G	
H	
J	
K	
L	
M	
N	

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CONTRACT
40 FROGNAL LANE NW3

DRAWING
PROPOSED SECTION AA

SCALE 1:50	DATE 06-09
DRAWN BY HB	CHECKED NRLT

DRAWING No.
0820 P 24 A