WITANHURST CONSTRUCTION MANAGEMENT LTD

41 HIGHGATE WEST HILL LONDON

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INTERPRETATIVE GEOTECHNICAL REPORT

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1. INTRODUCTION

1.1.1 This report has been prepared for Witanhurst Construction Management Ltd ('WCML') in connection with the proposed redevelopment at No 41 Highgate West Hill, London N6 6LR known as 'Witanhurst' (see Figure 1 for site location).

1.1.2 The redevelopment includes the construction of a 10m deep basement directly below the courtyard area in front of (i.e to the east of) the existing house, a three storey Grade II listed mansion with existing part single-basement. The new basement will measure approximately 35m by 70m in plan as shown in Figure 2. Michael Barclay Partnership (MBP) are the Structural Engineers.

1.1.3 Following a recent ground investigation carried out on site by Ground Engineering Ltd (GEL), Geotechnical Consulting Group Ltd ('GCG') have been instructed by WCML to prepare an interpretative geotechnical report. The report is to include a summary of the ground conditions, geotechnical properties of the soil strata and engineering advice on the design of the basement retaining walls, the basement slab and the basement internal bearing piles.

1.1.4 Environmental contamination issues and gas generation are beyond the scope of this report.

1.1.5 This report has been prepared for WCML taking into account particular instructions and requirements. It is not intended for, and should not be relied upon by, any third party and no responsibility is undertaken to any third party.

1.1.6 For the purpose of descriptions provided in this report, the existing house is assumed to face due east.

2. EXISTING INFORMATION

2.1 The property

2.1.1 The existing house is an imposing L-shaped, three storey mansion house with part single-basement set in approximately 5.5 acres. The property is situated in an elevated position on the west side of Highgate Hill (Figure 1). The existing basement is approximately 2.5m - 3m deep, but does not extend to the very front of the building at all points along the facade as shown on Figures 2, 4 and 5.

2.1.2 The existing foundations are corbelled brick over mass concrete footings founded at approximately 1.3m below existing basement level (i.e around 4m below existing ground level), as determined in a separate trial pit investigation.

2.1.3 Historical information indicates that the house was built to its current shape in 1913-1920 incorporating a property known as Parkfield, built in the early 18th Century. The house is surrounded by a large garden area that extends downslope for about 100m to the rear (i.e to the west), and a courtyard which extends for about 60m upwards to the front (i.e to the east) of the house to a gatehouse. Aerial photos of the property are shown in Figure 3.

2.1.4 The front facade of the existing house is generally in good condition, with only minor cracking locally.

2.2 Proposed Development

2.2.1 It is proposed to demolish the north-eastern wing of the existing house and to construct a 10m deep basement beneath the front courtyard to the east and to the north, as shown cross-hatched in plan on Figure 2. The depth of the basement reduces to only 5m depth in the small northern area beyond the solid black line in Figure 2. The courtyard is to be re-instated above the new basement at the end of the project. Preliminary sections AA and FF are shown in Figures 4 and 5 with levels.

2.2.2 It is likely that the top slab will be installed at around 1 - 2m depth below existing ground level. There is also to be a swimming pool installed in the western half of the basement and so there will be no intermediate floor to provide support to the basement retaining wall on the side where the house sits.

2.2.3 The site is bounded by Highgate West Hill to the south, by the gatehouse and private properties to the east, by the existing house to the west and a sunken garden area to the north.

2.2.4 There is a substantial existing masonry retaining wall running along the southern boundary, between the site and Highgate West Hill below, and this wall has a significant existing outward tilt. It is likely that this wall will require to be rebuilt.

2.2.5 There is also a substantial masonry retaining wall running along the north-eastern boundary, between the site and the neighbouring gardens above. There is a large copper beech tree at the upper level in the neighbour's garden, near the back of this retaining wall.

2.2.6 The location of services around the site has not been checked and is beyond the scope of this report. The extent of services needs to be established.

2.3 Topography

2.3.1 The existing house sits on a locally flat terraced area at about +125mOD. The ground falls away steeply behind the property to the west at around 1 in 7 down towards Hamptstead Heath. Directly in front of the house (i.e to the east) where the new basement is to be situated the courtyard area is approximately level at about +125mOD, but then rises gently at around 1 in 40 up to the gatehouse which lies at around +127mOD.

2.4 Geological Maps

2.4.1 The geology of the area is shown on the British Geological Survey 1:50,000 Sheet No 256 "North London" and on 1:10,560 (6 inch map) Sheet No TQ28NE (Figure 6). These maps show that the site is underlain by Bagshot Sand, overlying the Claygate Beds and the London Clay at depth.

2.4.2 The information from the geological maps suggests that the Bagshot Sand is around 5m in thickness and that the Claygate Beds are around 25m-30m in thickness at the location of the site. It is expected that these two strata will dominate the works on site and that the London Clay will not have any significant influence owing to its greater depth.

2.4.3 Typically the Bagshot Sand is composed of horizontally bedded sands with occasional thin gravel beds and lenses of silt and clay, representing deposition in a shallow water marine environment. It is normal to find more clayey material at the base of the Bagshot Sand together with some gravelly material and ferruginous nodules.

2.4.4 The Claygate Beds typically comprise a sequence of horizontally bedded fine-grained sands, silts and firm to stiff clays. The upper part of the Claygate Beds is usually very sandy.

2.5 Previous Ground Investigation

2.5.1 A ground investigation was undertaken by Albury SI Ltd in 1999 [Ref 1] in the locations shown on Figure 7. The investigation included one deep shell & auger borehole ('BH1') to 20m depth and six shallow window sampler boreholes to 5m depth, plus associated laboratory testing.

2.5.2 The relevant borehole logs are shown faintly on the geological section in Figure 9 (the section is viewed in an easterly direction). The borehole logs generally indicate Made Ground overlying a shallow band of sandy clay to a maximum depth of 4.8m, underlain by clayey silty sand with seams of clay.

2.5.3 The relative density of the granular material was assessed by SPT blowcount in BH1 (results plotted in Figure 11) and the strength of the cohesive material was assessed by hand shear vane and by a limited number of undrained 38mm triaxial tests in the laboratory (Figure 16). The results of all the fieldwork and laboratory testing of soil samples are included on Figures 11 to 16 and are discussed more fully in Section 5 below.

2.5.4 No water strikes were noted on any of the Albury borehole logs. A standpipe was installed in BH1 at 6m depth on completion. No readings are provided on the borehole log, but the standpipe is understood to have been dry.

3. GROUND INVESTIGATION 2009

3.1 Description of Field Work

3.1.1 A more detailed ground investigation was carried out this year by Ground Engineering Ltd in January 2009 [Ref 2], comprising three shell & auger boreholes to a maximum depth of 25m and three machine dug trial pits to 4.7m depth. These were generally located along the eastern edge and at the northern end of the new basement as summarised on Figure 8.

3.1.2 The three boreholes went to 25m, 16.5m and 20m depth respectively. BH2 was abandoned at 16.5m depth due to continued ingress of ground water and 'blowing sand' conditions. The borehole logs are summarised on the geological section shown on Figure 9 in bold.

3.1.3 Standard Penetration Tests (SPTs) were carried out in each of the boreholes (results plotted in Figure 11) and samples were taken for laboratory testing.

3.1.4 On completion of the field work, standpipe piezometers were installed in each of the boreholes at depths of between 14m and 15m. Pressure transducers (known as 'divers') were also installed in the standpipes in order to take continuous water level readings and falling head tests were also undertaken.

3.1.5 The three trial pits were dug by machine to between 4.0 and 4.7m depth. All of the pits were found to be stable and dry and samples were again taken for laboratory testing.

3.1.6 A separate trial pit investigation was undertaken from inside the existing basement and around its external walls, in order to determine the nature of the existing foundations and their depth. The founding level of the neighbouring retaining wall to the north-east was also determined. The existing foundations along the front facade of the house were found to be corbelled brick to 1.55m below ground level at the south-east corner, and corbelled brick to approximately 2.5m below ground level mid-way along the east elevation with mass concrete likely continuing below this depth.

3.2 Laboratory Testing

3.2.1 The following laboratory testing was carried out on soil samples taken from the boreholes and the trial pits:

- Density tests, to indicate the bulk unit weight of the soil - see Figure 12
- Particle Size Distribution tests including sedimentation tests for finer grained material, to indicate the predominant soil type - see Figure 13
- Atterberg Limit tests including moisture content, to indicate the plasticity and state of the soil - see Figure 14 and 15
- Unconsolidated Undrained Triaxial tests BS1377-7:1990 (8), to provide the undrained strength of the cohesive material - see Figure 16
- Consolidated Drained Triaxial tests BS 1377-8:1990 (8), to provide drained strength of the cohesive material - see Figure 17

- Small Shear Box tests BS 1377-7:1990 (4) on remoulded samples, to provide the drained strength of the granular material - see Figure 17
- Oedometer unloading tests BS 1377-5:1990, to provide information on the compressibility of the soil and its likely swelling behaviour on unloading - see Figure 18
- Sulphate tests and pH tests, to determine the aggressivity of the soil and groundwater to buried concrete.

4. GROUND CONDITIONS

4.1 Description of Strata Encountered

Figure 9 shows a geological section drawn north to south through the site (see Figure 8 for approximate location). The section summarises the borehole logs from both ground investigations (Albury SI in 1999 shown in faint, and GEL in 2009 in bold). The latter are considered to be more representative of the likely ground conditions at the new basement location. The general succession of soil strata was found to be Made Ground overlying Bagshot Sand, overlying the Claygate Beds as described in more detail below.

Made Ground

4.1.1 The Made Ground was found to vary in thickness and in nature over the proposed basement area. Along the eastern edge of the basement in the front courtyard area the Made Ground was found to be relatively shallow, varying between 0.35m and 0.6m in depth and containing tarmac, hard clay, gravel, cobbles and brick. At the northern and southern ends of the new basement (i.e in the garden areas) the Made Ground was found to be deeper, varying between 1.5m to 2.0m in depth and containing topsoil, sandy clay, gravel, ash and brick.

4.1.2 Along the western edge of the new basement the depth and nature of the Made Ground will be controlled by the existing house's basement construction and foundations. In two recent trial pits the Made Ground was found to extend to at least 2.5m depth directly in front of the house, and generally to comprise a sandy clay with pebbles.

Bagshot Sand

4.1.3 The Bagshot Sand is considered to be represented by the first two sub-strata shown on the geological section on Figure 9:

Sub-stratum 1: Medium dense brown SAND and GRAVEL, locally clayey (extending to a maximum depth of 2.9m in GEL's boreholes).

Sub-stratum 2: A band of firm to stiff brown and grey silty sandy CLAY (extending to a maximum depth of 4.4m in GEL's boreholes).

4.1.4 Sub-stratum 2 is not described as a clay in GEL's trial pit logs. Instead it is described as a very clayey SAND, with occasional partings of sandy clay and rare ferruginous concretions and nodules.

Claygate Beds

4.1.5 The Claygate Beds are considered to be represented by the third and fourth sub-strata shown on the geological section on Figure 9:

Sub-stratum 3: Medium dense brown and grey SAND containing occasional clay seams and bands locally (extending to a maximum depth of 16.1m in GEL's boreholes).

Sub-stratum 4: Firm to stiff grey silty sandy CLAY (extending to the maximum depth of the boreholes i.e 25m at least).

A different classification to the above is adopted in GEL's final SI Report, where SS1 is designated to be Bagshot Sand, SS2 and SS3 are designated to be Claygate Beds and SS4 is designated to be London Clay. But this is a difference in nomenclature only and does not affect engineering performance.

4.2 Ground water

4.2.1 There is unlikely to be a uniform water table across the site. Ground water was struck at different levels in each of the boreholes. It is likely that there are a series of local perched water tables.

4.2.2 The highest water strike during drilling was recorded in Borehole 1, where water was struck at 9.0m depth. This subsequently rose to 7.1m depth at a medium rate of inflow. Other water strikes were encountered at 15.5m (rising at a medium rate to 14.0m) in BH1, at 13.2m (rising slowly to 10.9m) in BH2, and at 17.8m (seepage only) in BH3.

4.2.3 The log for Borehole 1 states that on the second day of drilling the water level within the borehole was observed to be 5.6m below ground level. However this is not supported by observations recorded in the other boreholes, or during the subsequent standpipe monitoring. It is thought more likely to be the result of adding water during drilling.

4.2.4 Standpipe piezometers were installed in all three boreholes (at 14m depth in BHs 1 and 3, and at 15m depth in BH 2) with full depth response zones. Water level readings were taken automatically at 4 hour intervals throughout April 2009 using down-the-hole pressure transducers at the base of the boreholes called 'divers'. These have indicated much lower water levels than those reported during drilling. The following maximum water levels have been recorded in each standpipe piezometer to date:

- BH1 10.9m below ground level
- BH2 10.1m below ground level
- BH3 9.35m below ground level

Taking into account ground level at the three borehole locations, the three standpipes currently indicate an existing fairly uniform water table at approximately +115mOD.

4.2.5 A standpipe was also installed by Albury in their Borehole 1 in 1999, however this was only installed to 6m depth and was found to be dry.

4.2.6 Falling head tests have been carried out in each of the three boreholes by Ground Engineering in March 2009. In each case water was added to the borehole and then allowed to restabilise. The speed at which equilibrium was reached is a measure of the mass permeability of the surrounding soil. The following mass soil permeabilities were deduced from these tests. These permeabilities are all typical of predominantly fine sandy material, rather than clayey material and are likely to represent horizontal flow within sub-stratum 3.

- BH1 $k = 5 \times 10^{-5}$ m/s (typical of a fine sand)
- BH2 $k = 2 \times 10^{-6}$ m/s (typical of a silty fine sand)
- BH3 $k = 3 \times 10^{-6}$ m/s (typical of a silty fine sand)

4.2.7 Sub-strata 2 and 4 (which are predominantly clayey) are likely to have lower permeabilities than the values given above. These sub-strata will act as a barrier to downward flow of ground water and are consequently likely to have periodic perched water tables at their surface. Clay bands within sub-stratum 3 may also act as local barriers to the downward flow of ground water, causing some additional perching. Overall, horizontal permeability will be higher than vertical permeability in all the sub-strata.

4.3 Ground Model

4.3.1 The following general ground model is proposed for design purposes, as summarised on Figure 10:

		Top	Bottom
Made Ground		+125.0	+124.0
Bagshot Sand	*(sub-stratum 1)*	+124.0	+122.5
	(sub-stratum 2)	+122.5	+121.0
Claygate Beds	*(sub-stratum 3)*	+121.0	+110.0
	(sub-stratum 4)	+110.0	+100.0
Ground water table 1		+122.5	
Ground water table 2		+118.0*	

4.3.2 Recommended geotechnical design properties for each sub-stratum are given in the next section.

4.3.3 Figure 10 illustrates the concept of two ground water tables. The principal ground water table is assumed to lie at 7m below ground level (i.e +118.0mOD), which is a reasonably conservative assessment of the water strikes recorded during drilling and the subsequent standpipe piezometer monitoring exercise (and is also mindful of the local clay band at 7.3m depth in BH1 and the relatively high moisture contents recorded in sub-stratum 3 below this level). In addition there is likely to be a local perched water table at the surface of the clayey sub-stratum 2 (i.e +122.5mOD). The latter is conservatively assumed to be hydrostatic through sub-stratum 2 but not to act below this.

4.3.4 * A separate hydrological report has determined that damming action caused by the new basement obstructing the natural flow of groundwater in a westerly downslope direction could cause ground water to rise by up to 1m on the uphill side. It is consequently recommended that Ground water table 2 is assumed to lie at +119.0mOD along the *eastern* edge of the new basement.

5. GEOTECHNICAL PROPERTIES OF MATERIALS FOR DESIGN

5.1 General

5.1.1 The results of all the fieldwork and laboratory testing of soil samples from both ground investigations are summarised on Figures 11 to 18.

5.1.2 For the principally sandy sub-strata (SS1 and SS3), SPT values, laboratory grading tests and shear box tests are most relevant. For the principally clayey sub-strata (SS2 and SS4), the laboratory grading and sedimentation tests, Atterberg Limits, unit weight, triaxial tests and oedometer tests are most relevant. Appropriate values to adopt for design in each of the soil sub-strata are discussed below and summarised in Tables 1 to 3.

5.2 Made Ground

5.2.1 Only one SPT was undertaken in the Made Ground (giving a blowcount value of 11) and one shear vane test, which gave an undrained strength of 65kPa. No samples were tested from the Made Ground other than for sulphates. The Made Ground was found to be variable in both thickness and content. In the 2009 boreholes in the courtyard area it was described as 'hard'.

5.2.2 Based on the material descriptions the average bulk unit weight of the Made Ground may be assumed to be $20kN/m^3$ and the angle of friction, ϕ, taken to be 30°. Other recommended design parameters for this material may be found in Table 1 below.

Table 1 Recommended design parameters for Made Ground

	Unit weight γ (kN/m^3)	SPT (N)	Undrained strength c_u (kPa)	ϕ	c' (kPa)	K_o	Drained Young's Modulus E_h' (MPa)
Made Ground	20	N = 12 + z	40	30°	-	0.5	2N

where z is the depth below ground level in metres and N is the SPT blowcount.

5.3 Bagshot Sand

Sub-stratum No 1

5.3.1 Laboratory grading tests for this material obtained by sieving are shown in Figure 13. The material is shown to be primarily a sand and gravel. No Atterberg Limit tests were undertaken in this material but it may be assumed that this sub-stratum has negligible plasticity. The clay content is likely to be very low, less than 5%.

5.3.2 SPT blowcounts in this material were found to vary from 13 to 72 as shown on Figure 11, indicating that the sand and gravel is medium dense to dense. For design purposes the design line of N = 12 + z may be adopted, where z is the depth below ground level in metres and N is the SPT blowcount value.

5.3.3 Other recommended design parameters for this material are given in Table 2.

Sub-stratum No 2

5.3.4 This material is described in GEL's borehole logs as a firm silty sandy clay with bands of sand, but is described in GEL's machine dug trial pits as a very clayey sand. Gradings for the sub-stratum obtained by sieving and sedimentation tests are shown in Figure 13 which show clay contents in the range 13 to 26%. This is sufficiently high to suggest that the material will behave principally as a clay. Atterberg Limits are plotted in Figures 14 and 15, which indicate a clay material of intermediate plasticity.

5.3.5 Moisture contents are shown on Figure 14 and these indicate that the clay is at or around its natural moisture content. The clay does not appear to be in a significantly desiccated state due to the presence of the trees.

5.3.6 Figure 12 shows that the bulk density ranged from 1.99 to 2.06Mg/m^3 (the one data point lying outside this range is thought to be a rogue). The bulk unit weight of this material may be taken as 20kN/m^3 for design.

5.3.7 Figure 16 shows that undrained shear strength (measured both by hand vane and in triaxial tests) ranged from 60 to 160kPa. A conservative value of $c_u = (60 + 5z)$ kPa should be taken for design, where z is the depth below ground level.

5.3.8 Figure 17 shows the results of the consolidated drained triaxial tests carried out in this clayey sub-strata. This graph plots peak shear stress against vertical effective stress at the depth of each sample. Each point represents a suite of three tests carried out on U38 sub-samples consolidated to three different cell pressures. The gradients of the mini-lines drawn through each point correspond to the individual ϕ, c' values reported by GEL. There is considerable variation in GEL's reported ϕ, c' values, but it will be seen from Figure 17 that the grey dashed line of $\phi = 28°$, c' = 15kPa provides a moderately conservative fit to all the clay data over the relevant stress range and is consequently recommended for design.

5.3.9 Other recommended design parameters for this sub-stratum are given in Table 2.

Table 2 Recommended design parameters for Bagshot Sand

	Unit weight γ (kN/m^3)	SPT (N)	Undrained strength c_u (kPa)	ϕ	c' (kPa)	K_o	Drained Young's Modulus E_h' (MPa)
sub-stratum 1	20	N = 12 + z	-	36°	-	1.0	2.5N
sub-stratum 2	20	-	$c_u = 60 + 5z$	28°	15	1.0	600 c_u

5.4 Claygate Beds

Sub-stratum No 3

5.4.1 The material in this sub-stratum is described in GEL's borehole logs as a medium dense sand, with thin clay seams and occasional bands of firm to stiff clay. This is the most significant sub-stratum in terms of the basement retaining wall design.

5.4.2 Gradings for the sub-stratum obtained by sieving and sedimentation tests are shown in Figure 13 which indicate a significant fines content (i.e clay plus silt content). It is likely that the deposit contains a significant degree of 'fabric'. Discrete thin seams of clay and silt are likely to be interbedded in a repetitive sequence with thicker bands of sand. The deposit is consequently likely to have a relatively high permeability horizontally, as suggested by the falling head tests in the boreholes. A 300mm thick discrete band of stiff clay was also found within this sub-stratum in BH1 at 7.3m depth.

5.4.3 Clay contents were found to lie in the range 5 to 21%, only slightly lower than for the 'clay' sub-strata above and below. Atterberg Limits are plotted in Figures 14 and 15, which taken at face value indicate a silt or clay-like material of low to intermediate plasticity. However the latter is likely to be misleading due to mixing of the discrete layers of sand, silt and clay in the laboratory. It is more likely that the sub-stratum will behave primarily as a sand in its natural undisturbed state.

5.4.4 Moisture contents are shown on Figure 14. These are relatively high from about 7.5m downwards, possibly indicating the location of the water table (see para 4.3.3 above). These moisture content determinations were carried out some 3 months after the fieldwork, however, and may not be entirely reliable.

5.4.5 Figure 11 shows that SPT values throughout the sub-stratum ranged from as low as 10 at the surface to 46. Good agreement was found between the four deep boreholes across the site. A zone of denser material was found in three of the boreholes within the depth range 7m to 11m. The grey dashed line shown on Figure 11 may be assumed for design, given by the equation $N = 12 + z$ where z is measured from ground level.

5.4.6 Undrained shear strength and bulk density were not assessed for this sub-stratum, owing to its principally cohesionless nature.

5.4.7 Figure 17 shows the results of two drained shear box tests on this material, both corresponding to samples from 5m depth. As mentioned above, Figure 17 plots peak shear stress against vertical effective stress corresponding to the depth of each sample. Each point represents a suite of three tests carried out on sub-samples tested at three different normal effective stresses.

5.4.8 The gradients of the mini-lines drawn through each point on Figure 17 correspond to the individual ϕ, c' values reported by GEL. The angles of friction determined in each case were in excess of 40°. Taking into account the clay content and plasticity of the sub-stratum generally however, it is recommended that a lower value of $\phi = 36°$, $c' = 0$ be adopted for design.

5.4.9 Other recommended design parameters for this sub-stratum are given in Table 3.

Sub-stratum No 4

5.4.10 This substratum will govern the design of the main piles and is also likely to dominate heave calculations beneath the basement slab in the long term. The material in this sub-stratum is described in GEL's borehole logs as a firm becoming stiff silty sandy clay, but is described in Albury's BH1 as a medium dense silty sand with seams of clay.

5.4.11 Gradings obtained by sieving and sedimentation tests are shown in Figure 13, which show clay contents in the range 14 to 43% indicating that the material will behave principally as a clay. Atterberg Limits are plotted in Figures 14 and 15, which indicate a clay material of high plasticity. Moisture contents are shown on Figure 14 to decrease from 33% to 21% with depth.

5.4.12 Figure 12 shows that the bulk density ranged from 1.92 to 2.08Mg/m^3 (the one data point lying outside this range is not thought to be representative). The bulk unit weight of this material may be taken as 20kN/m^3 for design.

5.4.13 Figure 16 shows that undrained shear strength measured in unconsolidated triaxial tests ranged from 119 to 210kPa. A design line of $c_u = (60 + 5z)$ kPa may be assumed, where z is depth below ground level in metres.

5.4.14 Figure 11 shows that SPT blowcounts ranged from 21 to 33. The trend line of $N = 12 + z$ may be adopted for this sub-stratum.

5.4.15 Figure 17 shows the results of two consolidated drained triaxial tests performed on samples from this sub-stratum. This graph plots peak shear stress against vertical effective stress at the depth of each sample. Each point represents a suite of three tests carried out on U38 sub-samples consolidated to three different cell pressures. The gradients of the mini-lines drawn through each point correspond to the individual ϕ, c' values reported by GEL.

5.4.16 The triaxial samples were split, dried and photographed after testing. The clay was found to be interlaminated with thin seams of sand and silt. This fabric will tend to increase the speed of swelling of the deposit after unloading, due to the shortened drainage paths.

5.4.17 As shown on Figure 17 there is a significant difference in the sets of ϕ, c' values from these two sets of tests. This is likely to be due to differences in clay content. The sample from BH3 had a clay content of 43% compared with the sample from BH1 which had a clay content of only 20%. Figure 17 shows that the grey dashed line of $\phi = 28°$, $c' = 15$kPa provides a reasonable fit to the data however over the relevant stress range and is consequently recommended for design.

5.4.18 Two oedometer swelling tests were carried out on samples from this sub-stratum. The samples were reconsolidated to their estimated in situ stress and then unloaded in stages approximately corresponding to the basement excavation, in order to determine their swelling potential. The results are shown plotted in Figure 18.

5.4.19 The difference in behaviour between the two samples is again attributed to their significantly different gradings. The sample from BH3 had a clay content of 43% while the sample from BH1 had a clay content of only 15%. The latter also had a much lower swelling pressure than the latter (1kPa compared to 98kPa). The gradients of the two swelling lines are similar however and the trend line whose equation is given by $e = 0.87 - 0.03 \log_{10}\sigma'_v$ may be used for design.

5.4.20 The coefficient of consolidation (c_v) was calculated by GEL to be 2.7m^2/year in both tests. It is likely however that the soil will drain considerably faster than this during unloading owing to the presence of thin horizontal sand laminae.

5.4.21 Other recommended design parameters for this sub-stratum are given in Table 3.

Table 3 Recommended design parameters for Claygate Beds

	Unit weight γ (kN/m^3)	SPT (N)	Undrained strength c_u (kPa)	ϕ	c' (kPa)	K_o	Drained Young's Modulus E_h' (MPa)
sub-stratum 3	20	N = 12 + z	-	36°	-	1.0	2.5N
sub-stratum 4	20	-	c_u = 60 + 5z	28°	15	1.0	600 c_u

5.5 Chemistry

5.5.1 Sulfate and pH tests were carried out on selected water and soil samples from the boreholes in both ground investigations, in order to determine the aggressivity of the soil to buried concrete. In all cases the values were insignificant and the site consequently falls into Design Sulfate Class DS-1 according to BRE Special Digest 1 (which is the lowest class possible).

6. RECOMMENDED DESIGN APPROACH AND METHODOLOGY

6.1 General

6.1.1 The key features of the proposed new basement development are as follows:

a) The basement is situated directly in front of the existing Grade II listed house (Figures 2, 4 and 5). Ground movements must be kept to a minimum in order to avoid damage being caused to the front facade.

b) The basement is relatively deep at 10m, especially when the additional surcharge from the existing house to the west and the existing masonry retaining wall to the east are taken into account.

c) The floor to ceiling height within the western half of the basement is relatively large, due to no intermediate floor level above the swimming pool.

d) The basement has no superstructure loading above it and so buoyancy is a potential issue.

e) The basement extends to the edges of the site on its eastern and southern sides where there are existing 3m high masonry retaining walls, one stepping up the other stepping down.

6.1.2 It is understood that the top slab will be installed at around 1 - 2m depth below existing ground level. The courtyard is to be re-instated above the top slab at the end of the project.

6.1.3 The depth of the basement reduces to only 5m depth in the small northern area beyond the solid black line in Figure 2.

6.1.4 The north-eastern wing of the existing house and garage building are to be demolished, thus alternative means of support for the existing masonry boundary retaining wall to the east must be provided where current support is to be removed.

6.1.5 The existing masonry retaining wall running along the southern boundary, between the site and Highgate West Hill below, has a significant existing outward tilt and may need to be reconstructed.

6.2 Methods of construction

6.2.1 Items (a) and (b) above strongly favour the use of "top-down" construction, since permanent propping at the top of the basement retaining wall would then exist from the outset and ground movements around the new basement may be minimised.

6.2.2 In view of (b) and (c) and the need to minimise ground movements, a relatively substantial retaining wall section will be required which favours the use of medium to large diameter secant piles. Given the high water table and the need to present a barrier to the free flow of water into the site, a contiguous piled wall is not recommended.

6.2.3 The absence of any substantial vertical loading from the superstructure on the basement wall means that it may be possible to adopt a hard-soft secant pile solution, but only if a structural lining wall is constructed afterward on the inside face of the piled wall. The latter will provide durability, reinforcement and watertightness in the long term. In hard-soft walls the female piles are formed using a cement bentonite slurry or weak concrete and are unreinforced, increasing the speed of construction. The female piles will need to go to the same depth as the male piles in order to prevent a shortened water path existing beneath the toe of the wall during basement excavation.

6.2.4 Secant piles may either be installed by CFA rig or by traditional segmentally-cased bored pile techniques. The latter is a slower operation but can offer a higher degree of risk-free excavation in granular soils near existing foundations and can also insure a better interlock at depth than CFA piles.

6.2.5 Sheet piles may also be considered as an alternative to a secant piled wall, but would need to be installed by jacking or by hydraulic press in order to avoid subjecting the existing house to excessive vibrations. The zone of dense sand found in three of the boreholes within the depth range 7m to 11m (where SPTs of up to 46 were recorded) may inhibit installation by jacking techniques even with local jetting, and specialist advice should be sought on this point. It will not be possible to extract and reuse the piles if top-down construction is employed. Water tightness is not so assured for a sheet pile wall as for a secant pile wall below dig level. The logistics of storing sheet piles on site should also be considered.

6.2.6 Wall installation close to the foundations of the existing house will carry a risk of ground movement to the existing facade, whatever the method employed. Typically secant pile installation will cause ground movements of up to 0.05% of the wall depth, hence around

8 - 9mm. If the line of the basement wall can be moved away from the existing house by a couple of metres or so then this risk will reduce significantly to approximately 4 - 5mm.

6.2.7 Additional settlement of the existing facade is likely to occur during excavation of the basement due to minor wall deflections. For top-down construction this settlement is likely to amount to around 0.1% the effective excavation depth, H, depending on the stiffness of the propping, the flexibility of the wall adopted and the standard of construction. This would equate to around 8mm. Movements are likely to die away exponentially with distance from the excavation, becoming negligible beyond 2H to 3H (say 20m).

6.2.8 Settlement of the facade caused by wall installation and wall deflection during excavation will be offset to an extent by heave resulting from net unloading of the ground due to the basement excavation. It is estimated that up to 8mm heave could occur beneath the front facade in the long term.

6.2.9 Ground movement and the risk of damage to the existing facade may be kept to a minimum if the house foundations are stabilised in advance by underpinning and monitored during excavation.

6.2.10 A mid-height temporary prop will be required to provide adequate temporary support to the basement wall as excavation continues down to basement slab level.

6.2.11 Designing the basement slab as a ground bearing raft is unlikely to be viable due to buoyancy limitations. The basement does not benefit from a superstructure acting above it and is consequently unlikely to have sufficient self-weight.

6.2.12 It is recommended instead that the basement slab is designed as a suspended slab supported on bearing piles. The slab should be constructed on a compressible void filler material to prevent soil swelling pressures acting on the underside of the slab, and the piles should be designed to act in tension to resist water uplift pressure. There would be no need for under slab drainage in such a scheme.

6.2.13 If a suspended basement slab is adopted, then there would be no need for sophisticated soil-structure interaction analysis. The latter would only be required if the basement slab were to be designed as an unpiled or a pile-assisted raft.

6.2.14 The ‘tension’ piles may be used in the short term as compression piles to support the weight of the basement top slab during top-down construction. The piles should be installed from top slab level but only concreted to basement slab level. The columns may then be installed as plunge columns into the wet concrete and revealed during excavation. These piles will need to be cased.

6.3 Design of Basement Retaining Walls

6.3.1 The following moderately conservative peak soil strength parameters are recommended for retaining wall design, in terms of effective stress:

Stratum	γ	ϕ	c' (kPa)	K_{ah}	K_{ph}
Made Ground	20	30°	0	0.34	-
Bagshot Sand (SS1)	20	36°	0	0.27	5.0
Bagshot Sand (SS2)	20	28°	15	0.37	3.2
Claygate Beds (SS3)	20	36°	0	0.27	5.0
Claygate Beds (SS4)	20	28°	15	0.37	3.2

6.3.2 Recommended strata levels for general design have already been set out in Section 4.3 above ("Ground Model") and are also summarised in Figure 10. The assumption of 1m of Made Ground in Figure 10 will need to be increased for the wall section alongside the existing house foundations, where a greater depth of Made Ground is likely to exist (in the range 2.5m to 4m depending on location).

6.3.3 It is recommended that the retaining wall is designed according to the procedure set out in CIRIA C580 *Embedded retaining walls - guidance for economic design (2003)*. Design Approach A should be adopted, based on the moderately conservative strength parameters given above. Drained conditions should be assumed on both sides of the wall, even during construction.

6.3.4 Steady state groundwater flow should be assumed around the toe of the wall during construction before the basement slab is cast (hydrostatic conditions may be assumed in the long term after construction of the basement slab). The principal ground water table should be assumed to lie at +118.0mOD, except for the eastern boundary where +119.0mOD should be assumed.

6.3.5 The values of ϕ and c' given in the above table are unfactored, but the values of K_{ah} and K_{ph} quoted are *design* values (i.e they are based on the quoted soil strength parameters divided by 1.2). They are also based on a wall friction of 0.67 ϕ as recommended in CIRIA C580. The subscript '$_h$' denotes that the values of K_{ah} and K_{ph} represent the horizontal component of earth pressure.

6.3.6 Hydraulic cut-off and uplift should also be checked.

6.3.7 There are at least four separate wall design cases to be considered around the perimeter of the basement, owing to the varying conditions around the site. These are:

1) the standard case (full height basement, ground conditions as per the ground model plus a nominal surcharge, say 10kPa)
2) alongside the existing house facade (full height basement, relatively deep made ground plus high surcharge due to the existing building loads, and no intermediate prop in the long term)
3) alongside the existing 3m high masonry retaining wall on east boundary (full height basement, ground conditions as per ground model, but relatively high surcharge due to wall, plus 1m higher water table)
4) northern section (half-height basement only)

6.3.8 Preliminary retaining wall calculations have been carried out for the full height basement. It has been assumed that top down construction will be employed, a permanent top slab will be installed at around 1m depth below ground level, an intermediate prop will be installed at around 5.5m (estimated value only) below ground level and the total depth of excavation will be around 11m. The latter makes an approximate allowance for the thickness of the basement slab, a void filler and some accidental excavation.

6.3.9 The required wall lengths were found to lie in the range 17m to 18m (the former for case 1, the latter for cases 2 and 3) measured from existing ground level. Maximum factored bending moments (corresponding to the situation of maximum excavation, before the basement slab is cast) were found to lie in the range 600kNm/m to 830kNm/m respectively (these values have already been factored by 1.5). A 750mm diameter hard-soft secant pile wall would be capable of meeting the lower end of this range but 880mm diameter piles may be required for the higher end of the range.

6.3.10 In the long term, after construction of the basement slab, the maximum bending moment in the wall will depend on whether the basement wall is supported at its mid-height by an intermediate floor level, or not. Where the wall is supported at its mid-height, the long term maximum bending moment may be expected to be significantly lower than the values quoted above which apply to the temporary condition during construction. Where the wall is *not* supported at its mid-height in the long term, maximum bending moments are likely to be approximately similar to the values given above for the temporary condition.

6.3.11 According to the preliminary calculations, the wall is likely to penetrate some metres into the clayey sub-stratum ('SS4') of the Claygate Beds, forming an effective hydraulic cut-off to the natural flow of ground-water laterally across the site.

6.3.12 A separate hydrological report has determined that the effect of damming the natural ground-water flow regime locally is likely to raise ground water levels by up to 1m on the eastern side of the basement. This effect has already been taken into account in the advice provided above.

6.4 Design of Basement Slab

6.4.1 A compressible void filler material should be incorporated beneath the basement slab to prevent soil swelling pressure acting on the underside of the slab.

6.4.2 The basement slab will need to be designed to withstand the sum of the crushing load of the compressible filler, plus water uplift pressure less the self-weight of the slab. An external water table level at +119mOD should be assumed for design purposes.

6.4.3 The water table level of +119mOD represents the highest level thought likely ever to arise during the lifetime of the basement and is what the basement slab should be designed to resist incorporating standard load factors and partial factors contained in the structural codes. The designer should also check however that a residual factor of safety in excess of unity will exist in the extremely unlikely event that the water table rises to ground level.

6.4.4 The thickness of the compressible filler material needs to be sufficient to accommodate the maximum likely differential heave of the soil beneath the slab after construction. Preliminary calculations based on the oedometer swelling tests indicate that the latter is unlikely to exceed 50mm when confined by the crushing load of the compressible filler.

6.5 Design of Basement Internal Bearing Piles

6.5.1 The piles will initially act as compression piles supporting the weight of the structure during top-down construction.

6.5.2 After construction of the basement slab, when the structure has become water tight, the piles may continue to act in compression or go into tension, depending on whether the water uplift force exceeds the structural loading acting on the piles. As already mentioned, an external water table level at +119mOD should be considered when calculating the uplift pressure acting on the underside of the basement slab.

6.5.3 If 'A' is the slab area associated with an individual pile, then the required working capacity of the pile Q_w may be assessed by summing the following actions:
(a) the column load acting on the pile
(b) the self weight of the slab (plus appropriate live load) over area A
(c) the self weight of the pile
(d) the water uplift pressure acting over area A
(e) the failure load of the compressible void filler material acting over area A

where (a), (b) and (c) act downwards and (d) and (e) act upwards.

6.5.4 For the design of the piles in tension the live loading in (b) should be ignored and (d) should be based on a water table at +119mOD. For the design of the piles in compression (c) should be ignored, (d) should be based on a water table at +115mOD if this gives a more onerous condition than +119mOD, and (e) should be taken as the self-weight of the slab only. In the short term during construction only (a) applies.

6.5.5 A number of design cases will clearly need to be considered in order to arrive at the most onerous value of required working pile capacity, given that the load on each pile (both in magnitude and direction) is likely to vary during construction and in the long term. The most onerous case should then be used to determine the required pile length and reinforcement.

6.5.6 The value of Q_w derived above should be used to obtain the required ultimate pile capacity Q_{ult} , according to the expression $Q_{ult} / Q_w = F$, where F is the required factor of safety (recommended value 3.0). The value of Q_{ult} may be calculated approximately from the following:

Compression Piles

$$Q_{ult} \quad = \quad Q_s \quad + \quad Q_b$$

Tension Piles

$Q_{ult} = Q_s$

where:

Q_s	=	π D L.q_s
Q_b	=	9 π $D^2/4$. c_{ub}
L	=	pile penetration below excavation level
D	=	pile diameter
q_s	=	$\alpha\ c_{u\ ave}$ in clay
	=	K tanδ. $\sigma'_{v\ ave}$ in sand
α	=	adhesion factor (0.6)
$c_{u\ ave}$	=	average value of c_u along pile shaft (see Table 3)
c_{ub}	=	value of c_u at base of pile (see Table 3)
$\sigma'_{v\ ave}$	=	average vertical effective stress acting along pile shaft (assume 80% of value which existed before excavation)
K	=	0.7
δ	=	ϕ (36° from Table 3)

6.5.7 Preliminary pile design charts are provided in Figure 19 based on the above formulae. When column loads and pile spacings are known and pile working loads have been assessed, approximate required pile lengths may be estimated from these charts. The pile lengths given in Figure 19 refer to the total bored length from Ground Level.

6.5.8 The charts incorporate a factor of safety (F) equal to 3.0, as it is assumed that no preliminary pile test will be carried out. It is likely to be impractical to undertake meaningful pile tests when the piles are to be concreted to the underside of the basement slab level only.

6.5.9 The designer should also check that the tension piles possess a residual factor of safety in excess of unity in the extremely unlikely event that the water table rises to ground level.

7. SUMMARY AND CONCLUSIONS

1 A new basement measuring approximately 70m by 35m in plan and up to 10m deep is to be constructed at Witanhurst, directly in front of the existing house (Figures 2, 4 and 5).

2 Two ground investigations have been carried out on the site, one in 1999 and one in 2009 (Refs 1 and 2). These have been summarised in plan on Figures 7 and 8 and in section on Figure 9.

3 The general succession of strata revealed by both GI's has been found to be Made Ground, overlying Bagshot Sand overlying the Claygate Beds.

4 The Bagshot Sand has been found to consist of two sub-strata (referred to as SS1 and SS2). The upper sub-stratum (SS1) is principally a sand and gravel material and the lower (SS2) is principally a sandy clay.

5 The Claygate Beds have also been found to consist of two sub-strata (referred to as SS3 and SS4). The upper sub-stratum (SS3) is principally a sand containing clay seams locally, and the lower sub-stratum (SS4) is principally a sandy clay. Horizontal permeability is likely to be much greater than vertical permeability due to horizontal laminations.

6 Borehole log descriptions and results of laboratory and field testing in each strata have been summarised and discussed. A ground model has been proposed giving recommended design levels for each of the sub-strata (the Made Ground, SS1, SS2, SS3 and SS4) in Figure 10. Recommended geotechnical parameters for design have been provided in Tables 1, 2 and 3.

7 Based on evidence from water strikes in the boreholes, subsequent monitoring in standpipe piezometers, and an examination of the soil particle size gradings, a design water table level of +118mOD* has been proposed in Figure 10 for the purposes of basement retaining wall design. A locally perched water table at the surface of SS2 at +122.5mOD should also be considered.

8 *A slightly higher design water table level of +119mOD should be assumed along the eastern side of the new basement, to take account of possible damming action of the natural flow of ground water caused by the basement. The level of +119mOD should also be used for water uplift calculations beneath the basement slab.

9 Buried concrete should be designed for Design Sulfate Class DS-1 according to BRE Special Digest 1.

10 Possible methods of basement construction have been discussed. Given the need to keep ground movements to a minimum, the most appropriate design solution is considered to be top down construction using a secant hard-soft bored pile retaining wall, with a permanent lining wall subsequently constructed on the inside face for durability, reinforcement and watertightness.

11 It is recommended that the retaining wall is designed according to the procedure set out in CIRIA C580 *Embedded retaining walls - guidance for economic design (2003),* adopting Design Approach A. Drained conditions should be assumed on both sides of the wall, even during construction.

12 A number of separate wall design cases have been identified around the basement and preliminary calculations for the design of the retaining wall have been carried out. The maximum required wall length measured from existing ground level was found to lie in the range 17m to 18m.

13 Maximum factored wall bending moments were found to lie in the range 600kNm/m to 830kNm/m (these values have already been factored by 1.5). A 750mm diameter hard-soft

secant pile wall is likely to be capable of meeting the lower end of this range but 880mm diameter piles may be required for the higher end of the range.

14 It is recommended that the basement slab is suspended on bored piles. The piles should be installed before the start of the main excavation, and part-concreted up to basement slab level only. Plunge columns may be inserted into the wet concrete in order to support the top slab.

15 A compressible void filler is recommended beneath the basement slab. The design requirements of the basement slab with regard to withstanding water uplift have been discussed.

16 The main bearing piles should be designed to act in compression during excavation and to act in tension in the permanent works. Relevant pile design formulae have been provided, together with a preliminary design chart in Figure 19. The latter is based on a factor of safety of 3.0 since it is considered unlikely that a pile test will be practical, given the low cut-off level of the piles.

8. REFERENCES

1 Albury SI Ltd *Report on a Geotechnical Investigation at 41 Highgate West Hill,* Final Report No 99/4169/KJC (July 1999).

2 Ground Engineering *Site Investigation Report, Witanhurst, 41 Highgate West Hill, London N6* , Report Reference No C11681 (May 2009).

3 CIRIA C580 *Embedded retaining walls - guidance for economic design,* London (2003).

4 BRE Special Digest 1 *Concrete in Aggressive Ground,* Garston, Watford (2005).