REPORT CONTROL SHEET

51 Gower Street, London WC1E 6HJ

BIA Land Stability (incl Damage Category Assessment)

Project number **P4142**

Report number **P4142/01Rev1**
Revision number
Issue number **01**

Archive number

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

Made by:	M G Brice	Associate UK Registered Ground Engineering Adviser
Checked by:	N A Smith	Director UK Registered Ground Engineering Adviser

Revisions:

Revision No	**Details**	Date
01	Minor text amendments	2/9/16

Distribution:

Issue No	Date	Distribution	No of copies
01	Sept 2016	Croft Structural Engineers	1 x pdf
01	Sept 2016	AGE	1 x pdf

1.0 Introduction

In connection with the proposal to redevelop the detached garage building behind No 51 Gower Street, London WC1E 6HJ, involving demolition of the existing building, excavation of a single-level basement and construction of a dwelling, Applied Geotechnical Engineering Ltd (AGE) has been instructed by Croft Structural Engineers (CSE) on behalf of their client, to provide contributions to the Screening, Scoping and Impact Assessment phases of a Basement Impact Assessment (BIA) relating to Land Stability, including a Burland Damage Category Assessment of the neighbouring properties. The addresses of these properties are No 53 Gower Street, and Nos 1-12 Ridgemont Gardens to the right and left of the site respectively, and Nos 34-40 Ridgemont St. to the front. No 51 Gower Street is understood to be under the same ownership as the garage, for completeness it will be included here.

The site lies to the rear of No 51 Gower Street (as viewed from Gower St), with a frontage and access off Chenies Street. Right, left and rear are as viewed from the front of the site on Chenies St, unless noted otherwise below.

Our understanding of the relative positions of these buildings is shown in Figure 1.

The structural engineer for the project is Croft Structural Engineers. A plan and section of the proposed basement of the property are given below in Figure 2.

The general exterior ground level local to the site is understood to be approximately 27.0mOD. The pavement level at the front of the site is 26.7mOD.

It is understood that basement construction will be carried out from an existing garage floor level of approximately 27.1mOD, within underpin walls, to an excavated level of approximately 23.0mOD (4m below general exterior ground level).

The buildings on Ridgemont Gardens have a lower ground floor, which appears to be set approximately 1.5m below street level. Nos 51+53 Gower Street also have Lower Ground floors and it is understood that these are approximately 3m deep. It is not known whether these buildings have deeper basements. Nos 34-40 Ridgemont Street do not appear to have lower ground floors, and it has been assumed that none are present. This is potentially conservative.

2.0 Information Provided

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

i) CSE Basement Impact Assessment Ref 150214 dated 28 April 2015 (incl GI Report).
ii) H Fraser Consulting BIA: Groundwater report Ref 30076R1 dated 27 April 2015.
iii) CSE Drawings 150214 SL-10,
iv) Charles Brice Architects Drawings 358/SY/01+02 dated November 2000.
v) FT Architects Drawings 231_00_01 and 231_14_01 annotated with existing and proposed loads (by CSE), and Drawings 231_00_01-03 and 100, and 231_14_01-05.
vi) AGE request for information marked up by CSE.
vii) Email correspondence CSE-AGE dated 23/6/16 to 2/9/16.

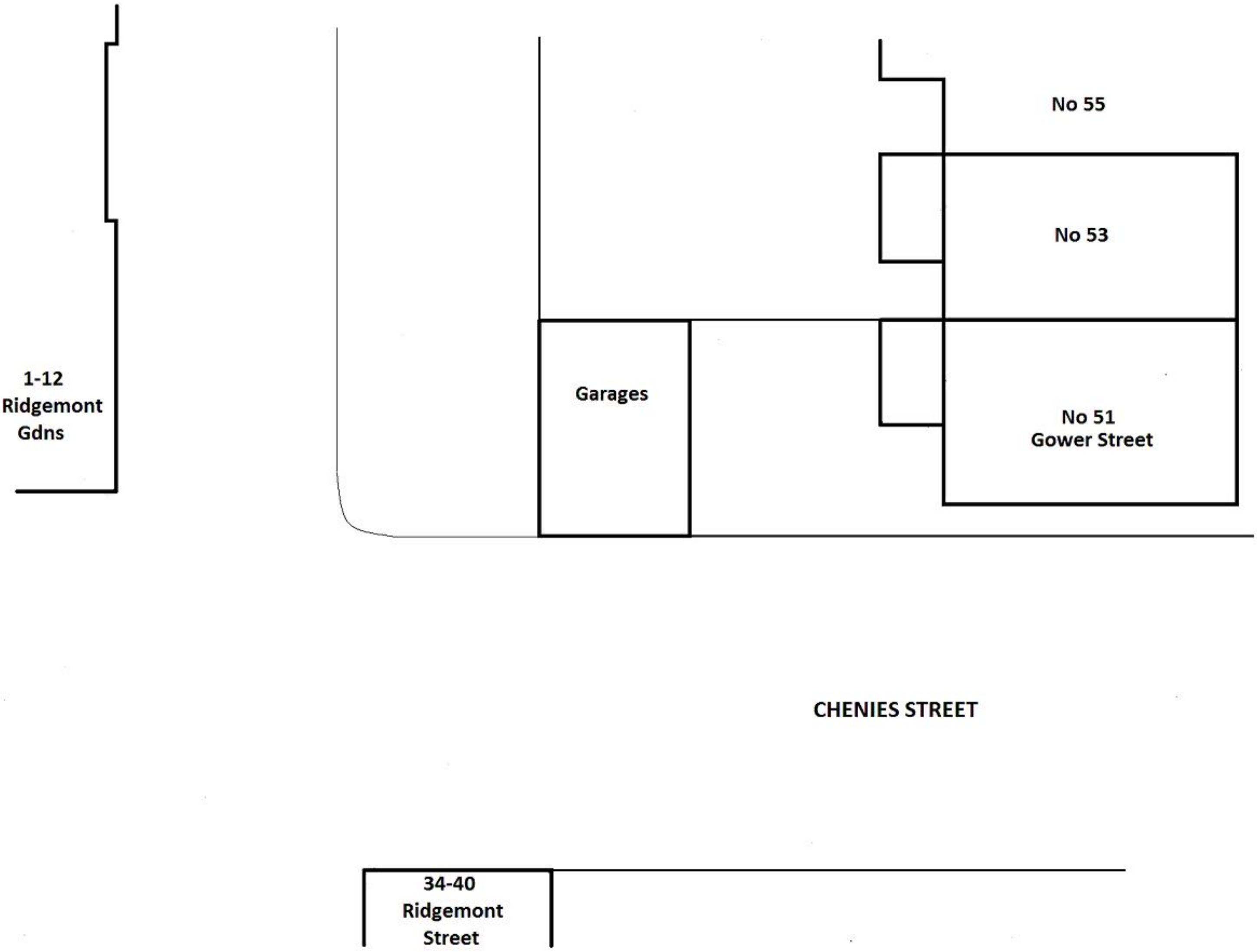
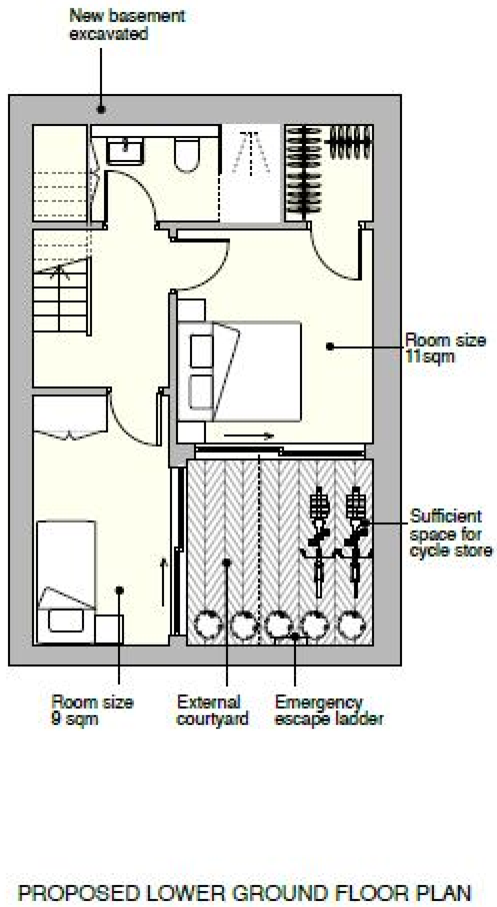
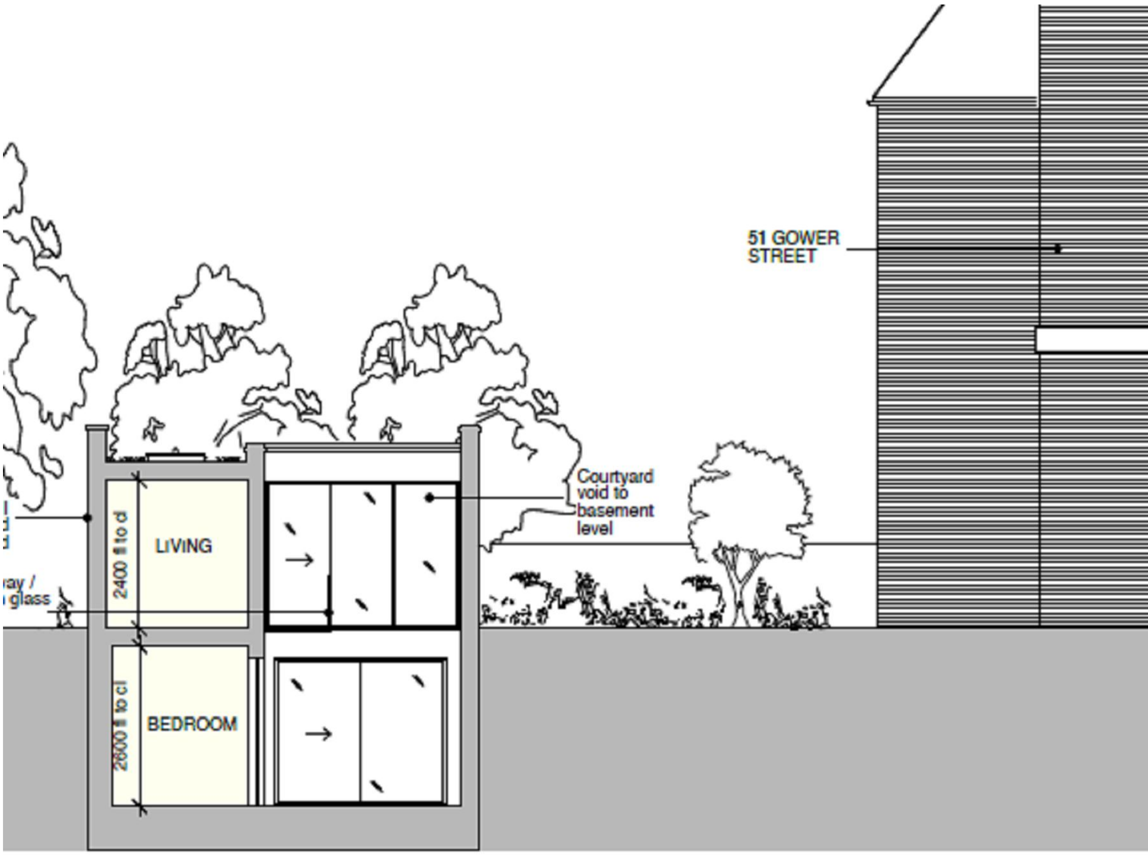


Figure 1 – Site Context



Proposed Basement Plan
(Extract of FT Architects drawing 231_14_01)

Figure 2-1



Proposed Basement Section
(Extract of FT Architects drawing 231_14_03)
Figure 2-2

3.0 Anticipated Ground Conditions

The external ground level in the area of the proposed basement is understood to be approximately 27.0mOD.

The published geological map (BGS 1:50 000 sheet 256: North London) indicates the site to lie on the Lynch Hill Gravel (sand and gravel, occasionally clayey) overlying London Clay (silty clay). On a developed site such as this Made Ground is also anticipated.

On the basis of the published mapping, the base of the London Clay is anticipated to lie at approximately 0mOD (approx 27m depth).

A ground investigation was undertaken at the site in March 2015 by Ground and Water Limited (Item 'i' in Section 2 above, p35/120 to p78/120). This comprised a borehole (BH1) sunk to 10m depth in the entrance apron to the existing garage, using a windowless sampler. Additionally, a trial pit (TP1) was excavated externally, adjacent to the existing left side wall near the front of the garage.

Based on the available information the commencement level of the borehole is taken to be approximately 26.9mOD.

The borehole confirmed 150mm of concrete over Made Ground to 4.4m depth (22.5mOD). The Made Ground was found to be predominantly granular in nature to 2.2m depth, becoming predominantly cohesive below that depth. The thickness of the Made Ground may be highly variable. The base of the Made Ground identified in the borehole lies approximately 500mm below the base of the proposed excavation, as currently understood. It is generally not recommended that buildings be founded on Made Ground due to the inherent variability of that deposit, and as a minimum it is recommended that a thorough investigation of the formation be undertaken, before foundations are cast onto Made Ground, in order that poor ground can be removed.

Beneath the Made Ground the borehole revealed sandy clay to 5.5m, then sand and gravel to 7.6m depth (19.3mOD). Both of these deposits are thought to represent the Lynch Hill gravel. London Clay was then encountered to the base of the borehole.

The trial pit revealed the existing left side garage wall to be founded at 1.3m depth onto the granular Made Ground.

For the purposes of this report only, the foundations of all the neighbouring buildings are taken to be founded on Lynch Hill Gravel (clay or granular facies) at depths of between 2m and 3m. Small variations from this assumption have no significant effect on the outcome of the analysis.

Groundwater was not noted during the excavation of the trial pit. Water was struck at 6.4m depth in BH1 during boring, this is within the sand+gravel facies of the Lynch Hill gravel, below the proposed excavation level. Recent observations in a standpipe installed to 3.8m depth within a response zone from 1m to 3.8m, gave water level readings of 3.5m to 3.6m bgl (approx 23.4mOD) which is within the excavation depth, in the clayey Made Ground.

For the purposes of the analysis, the soil sequence in the area of the proposed basement is taken to be:-

Ground Level 27.0mOD
Base of Made Ground variable, taken to lie between 22.5 and 25.0mOD
Base of Lynch Hill Gravel 19.3mOD
Base of London Clay 0mOD.

It is considered that the neighbouring structures are unlikely to be founded onto significant thicknesses of the Made Ground, but rather onto the Lynch Hill Gravel. The stiffness of the Made Ground is therefore not considered to be a primary influence on ground movements beneath these structures.

Standard Penetration Tests (SPT) were undertaken in BH1 and the results are plotted against depth in Figure 3. No suitable SPT data are available locally, for comparison, from the BGS archive.

Excavation to 4m depth is proposed, and there are no neighbouring buildings immediately adjacent to the site (the closest is approximately 8m distant) therefore the movement of the neighbouring buildings will be dictated by deep-seated ground distortions and as a result, any residual Made Ground beneath their footings will not significantly influence these movements.

A single SPT determination was made in the upper clayey part of the Lynch Hill Gravel, this yielded an ‘N’ value of 13. The bulk strength of the clay can be estimated from this result using the work of Stroud (Ref 1). A correlation coefficient, f1, of 4.8 has been adopted based on a measured plasticity index of 26%, yielding an undrained strength of approximately 60kPa for this material. This is considered reasonable for this deposit in this context and has been adopted. The clayey materials in the Lynch Hill Gravels are often of limited lateral extent, and gravelly facies usually predominate.

Two SPTs in the granular part of the Lynch Hill gravel both yielded ‘N’ values in excess of 35. Lower values are often seen in Terrace Gravels of this nature, therefore a value of N=25 has been adopted as representative of the Lynch Hill Gravel. This is considered conservative.

Two SPT results are available from the London Clay. Again the work of Stroud (Ref 1) has been adopted to estimate the bulk undrained strength from these results. A correlation coefficient, f1, of 4.5 has been adopted based on a measured plasticity value of 50%, yielding bulk undrained strengths near the top of the London Clay of 63kPa and 117kPa. This is in broad accordance with previous experience and published results from other sites in a similar geological context, and on this basis the undrained strength (Su) profile for the London Clay is taken as:-

$Su = 80 + 7z_1$ (kPa)

Where z_1 is the depth in metres below the top of the London Clay (taken to be 19.3mOD)

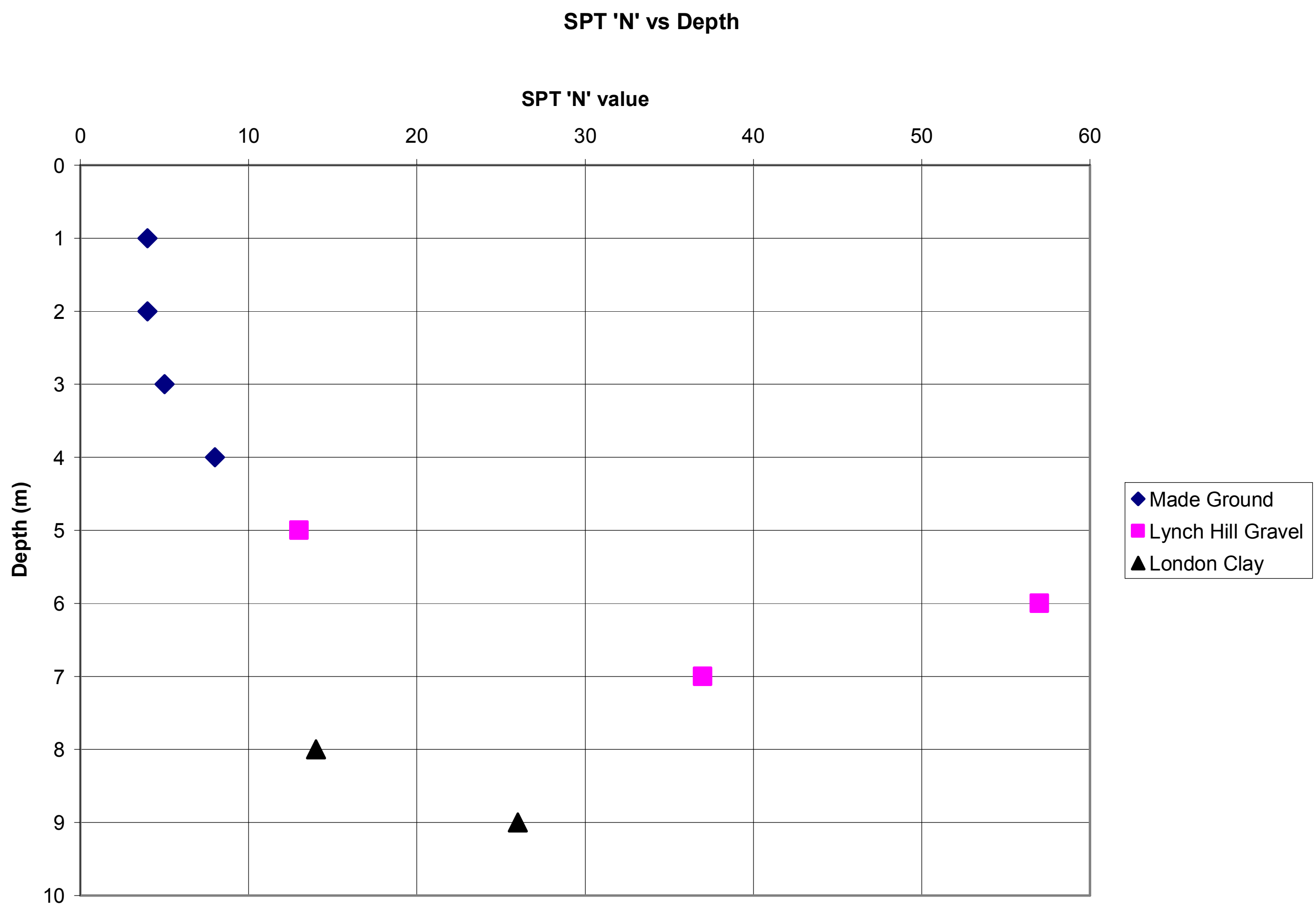


Figure 3 – SPT profile

4.0 Expertise

This report has been prepared by:-

M G Brice CGeol, UK Registered Ground Engineering Adviser.
Eur Ing N A Smith CEng, FICE, UK Registered Ground Engineering Adviser.

Both have more than 25years experience in Geotechnical engineering related to the urban environment.

5.0 Basement impact assessment (Ground Stability) – Screening Phase

The following is based on guidance contained in Camden Planning Guidance (CPG) 4 (Ref 5).

Slope stability

Q1 – Does the existing site include slopes, natural or man made, greater than 7º?	No
Q2 – Will the proposed reprofiling of landscaping at the site change slopes at the property boundary to more than 7º (approximately 1 in 8)?	No
Q3 – Does the development neighbour land, including railway cuttings and the like, with a slope greater than 7º?	No
Q4 – Is the site within a wider hillside setting in which the general slope is greater than 7º?	No (see note)
Q5 – Is the London Clay the shallowest stratum at the site?	No (see note)
Q6 – Will any trees be felled as part of the proposed development and/or are any works proposed within any tree protection zones where trees are to be retained? (Note that consent is required from LB Camden to undertake work to any tree/s protected by a Tree Protection Order or to tree/s in a Conservation Area if the tree is over certain dimensions)	Unknown (see note)
Q7 – Is there a history of seasonal shrink/swell subsidence in the local area, and/or evidence of such effects at the site?	No (see note)
Q8 – Is the site within 100m of a watercourse or a potential spring line?	No (see note)
Q9 – Is the site within an area of previously worked ground?	No (see note)
Q10 – Is the site within an aquifer? If so, will the proposed basement extend beneath the water table such that dewatering may be required during construction?	Yes, unknown (see note)
Q11 – Is the site within 50m of the Hampstead Heath ponds?	No
Q12 – Is the site within 5m of a highway or pedestrian right of way?	Yes (see note)
Q13 – Will the proposed basement significantly increase the differential depth of foundations relative to neighbouring properties?	Yes (see note)
Q14 – Is the site over (or within the exclusion zone of) any tunnels, eg railway tunnels?	No (see note)

Q4 – The Camden geological and hydro-geological study, Fig 16 indicates slope gradients in the immediate area of the site do not exceed 7º.
Q5 – Boring indicates Made Ground over Lynch Hill Gravel over London Clay.
Q6 – Refer to arboriculturalist's report.
Q7 – A walkover survey by CSE found no evidence of shrink-swell subsidence. None would be expected in this geological context.
Q8 – 'Lost Rivers of London' (Barton, 1962) indicates a (culverted) tributary of the River Fleet lies approximately 400m North of the site. The edge of the Lynch Hill Gravel, where it overlies the London Clay, represents the closest potential spring line. Published mapping indicates this boundary to lie some 600m N of the site at its closest approach (BGS 1:50 000 series Sheet 256 – North London).
Q9 – Made Ground was revealed to 4.4m depth in borings, but this is likely to be associated with development of the area, not worked ground. Review of historical mapping by CSE indicates the area has been in residential use for 150 years.
Q10 – The Lynch Hill Gravel represents a minor aquifer. The proposed excavation will not penetrate any significant depth into the Lynch Hill Gravel. Water level readings are not conclusive: observations during boring indicated the water table to lie within the Lynch Hill Gravel at a depth of 6.4m, but standpipe observations indicate a water table within the Made Ground at a depth of 3.5m. This is discussed in Section 6 below, and clarification is needed regarding any requirement for dewatering. Carried forward to Scoping.
Q12 – The front of the excavation lies immediately adjacent to the footpath and road pavement of Chenies Street. Predicted ground movements are discussed in Section 7 below. Highway loading will be incorporated into retaining wall design by CSE.
Q13 – The proposal involves the deepening of foundations from approximately 1.3m (current) to approximately 4m (proposed). These foundations lie more than 6m distant from the nearest building foundations therefore there is no direct risk of undermining. The effect of the excavation on the neighbouring buildings is analysed in Section 7 below.
Q14 – There are no road, rail or underground train tunnels beneath the site. The northern line runs beneath Tottenham Court Road some 150-200m west of the site. The existence of other tunnels is not known.

6.0 Basement impact assessment (Ground Stability) – Scoping Phase

A ground investigation has been carried out as described above and no further investigation of soil conditions is required for the Land Stability aspects of the BIA.

Groundwater conditions are not currently fully understood. During borehole excavation a water strike was made at 6.4m depth in the granular Lynch Hill Gravel. This is well below proposed excavation level (understood to be 4m depth).

However, recent observations in a standpipe installed at 3.8m depth within a response zone from 1m to 3.8m depth indicate a water level at 3.5-3.6m bgl (23.4mOD), some 400mm above proposed excavation level, as it is currently understood.

Transient perched water might be expected to accumulate at the boundary between the granular and clayey Made Ground (2.2m bgl) after rainfall events and such water may make its way into the standpipe, perhaps giving rise to the high water level observed. This would not be a particular concern and such minor water ingress that might occur during construction could be controlled by sump pumping. We consider that this is most likely to be the case.

However, if these standpipe observations accurately reflect the groundwater level in the underlying strata then they suggest the main water table within the Lynch Hill Gravel may have risen to within excavation depth. This would be of considerable concern, as it would be highly problematic to excavate below the water table without effective advance groundwater control measures being in place.

It is recommended that this uncertainty be resolved at an early stage of the construction works, in order that effective groundwater control measures can be put in place before excavation below the equilibrium water table, if required.

7.0 Basement impact assessment (Ground Stability) – Impact Assessment

7.1 Loads

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

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

7.2 Temporary support to the basement walls.

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

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

7.3 Soil stiffness values

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

The soil stiffness parameters are as given below.

The stiffness of the Made Ground is difficult to predict, and may be highly variable, but in the current situation it is considered acceptable to derive a stiffness value as a proportion of the clayey Lynch Hill Gravel stiffness, on the basis of the SPT results in these two deposits. The measured 'N' value in the Made Ground was approximately 50% of that in the clayey Lynch Hill Gravel (see below) therefore the stiffness of the Made Ground has been taken as 12000kPa and 9600kPa in the short and long-terms respectively.

The short-term stiffness of the clayey Lynch Hill Gravel is taken as 400Su, and the long-term stiffness as 320Su (where Su is the bulk undrained shear strength) therefore:-

Eu = 24 000kPa
E' = 19 200kPa.

In the current context, in which the predominant change in vertical stress is one of unloading, the granular Lynch Hill Gravel is treated as an overconsolidated deposit.

From Ref 1, the Young's modulus of the Lynch Hill Gravel can be taken to be equal to 2.0 x SPT'N', therefore the stiffness derived from Section 3 above is described by:-

E' = 50 (MPa).

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

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

Yielding :-

$E_{uo} = 124 + 10.85z_l$ (MPa).

and:-

$E'_o = 99 + 8.7z_l$ (MPa).

Where z_l is the depth in metres below the top of the London Clay.

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

7.4 Causes of ground movement outside the excavation

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

The first of these causes is investigated using equivalent-elastic analysis in the program PDISP. The second and third are based upon case-history data presented in Figures 2.8, 2.9 and 2.11 in CIRIA C580 (Ref 3). These data relate to installation in stiff clays, though reference to the individual case histories indicates there were substantial thicknesses of Made Ground and Terrace Gravel present at many of the sites, therefore the results are taken to be applicable to this site. It is currently understood that the plots presented by CIRIA in the above figures include short-term movement arising from cause 'i' above. Therefore in this report short-term movements are calculated using the CIRIA data, and subsequent long-term movement is calculated using PDISP.

The CIRIA plots relate vertical and horizontal ground movement to the depth of the wall installed (for Cause 'ii' above), or to the depth of excavation within that wall (for Cause 'iii' above) as appropriate. Data relating to the secant bored pile wall case history in Ref 3 Figure 2.8 are considered to be unreliable and have been ignored. In addition, data relating to counterfort diaphragm walls have not been taken into account in this analysis. No data are presented by CIRIA for underpinned walls, and no other data are available from other sources for underpin walls. Underpin walls are therefore, of necessity, assumed to be similar in behaviour to plane diaphragm walls and bored pile walls.

The CIRIA data indicate that:-

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

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

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

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

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

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

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

7.5 Predicted vertical movement

The predicted short-term vertical ground movement (settlement) due to the installation of the underpin walls, and the general excavation within these walls, is shown in Figure 4 below. It will be seen that predicted settlements beneath the neighbouring buildings are less than 1mm. The rear projection of No 51 itself undergoes slightly more than 1mm of settlement.

In the long-term the predicted additional movement is consistently less than 0.1mm at the locations of all the neighbouring buildings.

Very limited degrees of tilt are therefore predicted at all these buildings; certainly well below the 1:400 gradient recognised as requiring remedial action. In addition, the predicted degree of distortion of the walls is, by inspection, negligible for all neighbouring properties (including No 51 itself). Therefore a horizontal strain of 0.075% will be acceptable for a 'very slight' category of damage (Ref 2).

7.6 Predicted Lateral movement.

On the basis of the CIRIA data the horizontal ground movement immediately adjacent to the excavation would be expected to be of the order of 7-8mm, reducing to approximately 4mm at a distance of 6m from the excavation and to zero at a distance of approximately 16m.

From Section 7.4 above, the greatest average horizontal ground strain adjacent to the proposed excavation at the garage behind No 51 is predicted to be 0.064%, reducing to 0.0375% some 6m or so from the excavation. This is significantly less than the 0.075% limit for very slight damage calculated above.

7.7 Burland Damage Category Assessment - Conclusions

The predicted level of damage to the neighbouring buildings can therefore be taken as 'very slight' or less (Category 1 or 0).

The analysis indicates that short-term settlement of the adjacent footpath and road pavements on Chenies St is expected to be of the order of 3mm. It is not expected that this degree of movement would cause damage to the flexible pavement structure.

From the above, it is concluded that, given good workmanship, the proposed basement to the rear of No51 Gower Street can be constructed without imposing more than 'very slight' damage on the neighbouring properties and no significant damage to the adjacent pavement structure. This conclusion is made on the basis that any groundwater control works will be adequately designed and operated.

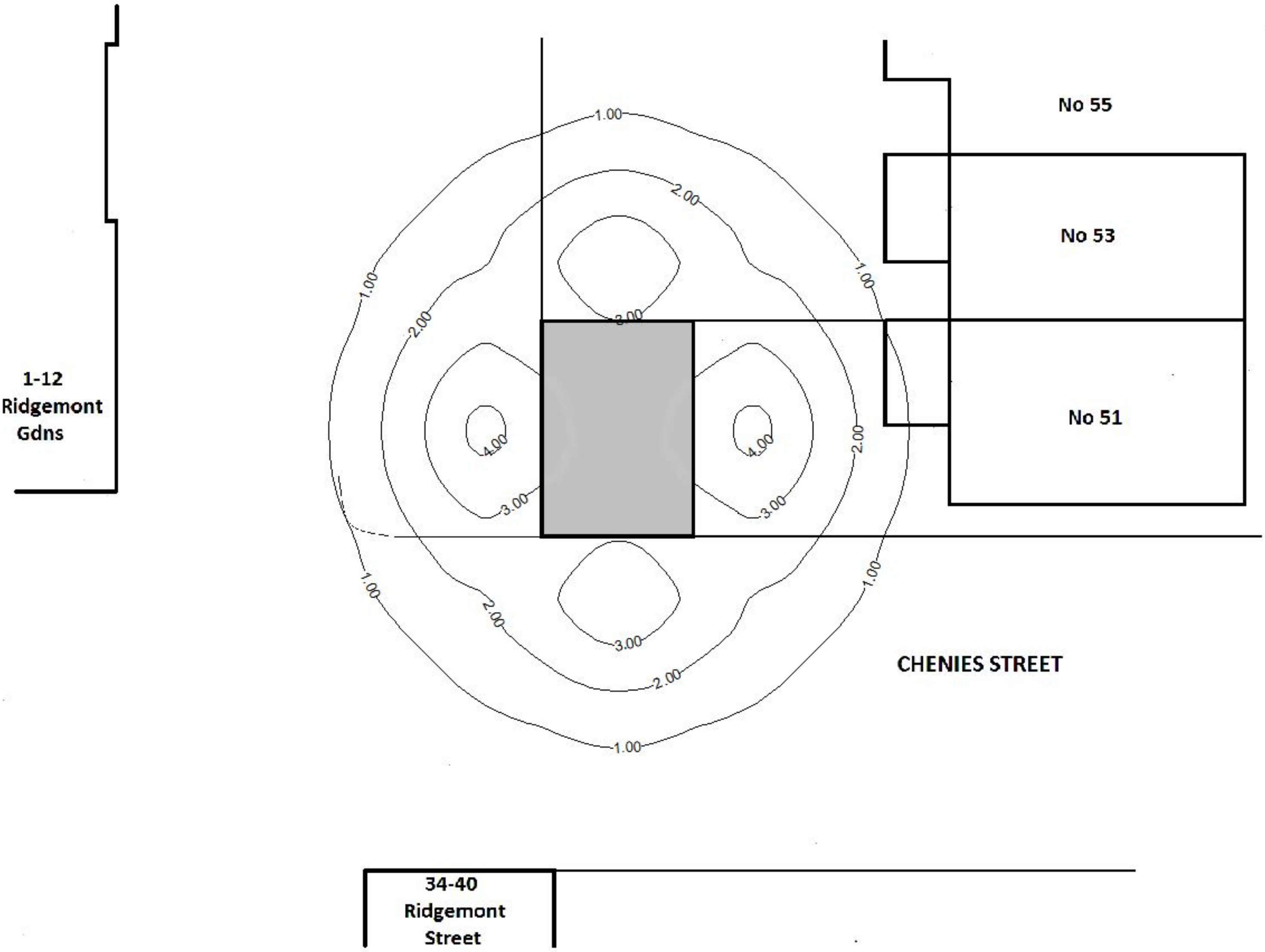


Figure 4
(Short-term ground settlement contours)

References:

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

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

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

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

5 London Borough of Camden (2015). 'Camden Planning Guidance – Basements and lightwells. CPG4'. Dated July 2015.

6 Arup (2010). 'London Borough of Camden. Camden geological, hydrogeological and hydrological study. Guidance for subterranean development.'