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Grove Lodge, Admirals Walk, London

Ground Movement Assessment

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1. INTRODUCTION

It is proposed to develop Grove Lodge in Admiral's Walk in the London Borough of Camden. This will include the construction of a single basement level which will cover part of the existing building footprint and extend below the rear garden and into the front garden. The basement will be formed using a secant pile wall. The building will be used as habitable space.

Basement Impact Assessments (BIA) including retaining wall analyses have already been undertaken for the property (*HR Wallingford, Rep. No. MAM7409, August 2015; MBP, Rep. No. 5954, June 2015; R&E Geotechnical Consultants, Ref. No. RE1366, July 2015*). This report includes a brief summary of the geological, hydrogeological and hydrological conditions based on the findings of the above reports. Further analysis combines the heave, pile installation and excavation movements into a single prediction using readily available bespoke software as the analysis in this case does not warrant FE/FD coupled analysis. The latter in any case would not readily incorporate pile installation movements and would require modification to any output prediction. The analysis done provides a conservative estimate of the ground movement risk that the affected structures may be subject to. It is anticipated that with good construction practice the actual movement on the property and any nearby structures will be less than what is predicted by the adopted methodology.

The London Borough (LB) of Camden's *Planning Guidance: Basements and Lightwells,* was published in September 2013 and outlines requirements relating to basements within the borough. The assessment included in this report has had full regard to the requirements of Camden's Planning Policy Framework, including Policy CS14 of the Core Strategy and Policy DP27 of the Development Policies DPD, as well as Camden's Planning Guidance: Basements and Lightwells Document 2013. This report addresses criteria currently specified within the policy and guidance for LB Camden and should be reviewed as part of the Structural Methodology Statement for the proposed development. It includes details and assessments of:

- Site history, underlying geology and groundwater;
- The impact of the subterranean development on drainage, sewage, surface water and ground water, flows and levels; and

- Ground movement due to the proposed development (including demolition of the existing structures onsite) and corresponding damage to neighbouring properties and infrastructure.

2. SITE CONTEXT

2.1 Site location

The site is located in Admiral's Walk, London Borough of Camden, NW3 6RS. The Ordnance Survey Grid Reference for the approximate centre of the site is 526214E, 186092N. A site location plan is shown in Figure 1.

2.2 Site description

The site is broadly rectangular in plan with dimensions some 50m in length and some 35m in width (at the widest section). The dimensions of the proposed excavation are approximately 20m in length and some 15m in width, with the length orientated in the east-west direction parallel to Admiral's Walk. The site comprises a four level residential property including a lower level floor.

With reference to information provided by the Wallingford BIA, the typical elevation at the site is 127.5m AOD and the ground slopes towards the south and southwest. The proposed basement is to be constructed 3.8m lower at 123.7m AOD. Drawings showing the existing structure are included in Appendix A.

A boundary wall to the south of the site separates the property from Admiral's Walk. The property is adjacent to another residence on the northern side whilst to the east it is bounded by Admiral's House and to the west it is bounded by private gardens.

No underground structures or buried services that could affect the works or be affected by them have been identified in the proximity of the property.

A site plan depicting the information above is presented as Figure 2 (from available drawing No 5954/301-P1, dated June 2015). A typical section through the proposed basement is presented as Figure 3 (from available drawing No 5954/318-P1, dated June 2015).

2.3 Proposed development

The proposed development comprises the construction of a basement level which will extend beyond the existing house footprint mainly to the rear garden and to the south of the house.

The proposed basement will be constructed within a secant piled wall and a bottom up construction methodology will be adopted. Proposed basement formation level is at 123.7m AOD (which corresponds approximately to 4m below existing ground level).

Proposed development plans are included in Appendix B.

2.4 Site history

Based on information included in the previous BIA (Wallingford, ref: MAM7409-RT002, Revision 5, August 2015) the property was constructed in the early 18th century and is a Grade II listed building. It is adjacent to the Admiral's House and Netley Cottage, which were constructed at a later date in the 18th century. Alterations have been made to the house since its construction.

A review of the available literature and CGL's in-house resources indicated that mapping dated between c. 1870 and 1950 records the site to be a semi-detached house within a residential area. The *Hampstead Water Works Reservoir* is located approximately 100m north of the site which was covered with fill in the 1890's and an observatory was built within it in the 1950's. *Queen Mary's Maternity Home* was built in the 1930's some 150m northeast of the site.

The site is not recorded as having sustained any damage during the Second World War bombings[1]. The nearest buildings that sustained damage are located approximately 250m northwest of the site. Since Grove Lodge was developed long before the Second World War and there is no evidence of bombing damage within its boundaries or its proximity, it is considered that the risk of unexploded ordnance (UXO) on site is low.

2.5 London Underground Limited infrastructure

No London Underground Limited (LUL) tunnels run under or near the site.

2.6 Anticipated geology

2.6.1 Published geology

Both available Site Investigation Reports (Ground Engineering, February 2014 and Southern Testing, July 2014) confirm that the BGS maps[2] indicate that the geology at the site comprises sands of the Bagshot Formation underlain by the Claygate Member and London Clay.

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[1] Saunders, A (Ed.) (2005) *The London County Council Bomb Damage Maps 1939-1945.* London Topographical Society
[2] BGS Geological Map Extract, Sheet No 256: *North London, 2006.*

The Bagshot Formation comprises beds of sand with occasional seams of clay and silt and local beds of flint gravel. Its thickness in the area can reach up to 18m.

The Claygate Member constitutes the upper layers of London Clay Formation and comprises alternations of sands and clays with sands being predominant above and clays below. The thickness of the Claygate Member is commonly approximately 15m.

The London Clay Formation is an over-consolidated firm to very stiff, becoming hard with depth, fissured, blue to grey silty clay of low to very high plasticity. The upper and lower parts may contain silty or fine grained sand partings. The stratum may also contain laminated, structured, nodular claystone and rare sand partings. Crystals of gypsum (selenite) are often present within the formation. The stratum is generally horizontally bedded.

2.6.2 Unpublished geology

Following a research on the BGS website, it was found that no historical exploratory borehole records are available near the site (the nearest one was some 150m to the southwest). It is considered that any available borehole logs from the BGS archive would not contribute to the understanding of the underlying soils, since site specific ground investigation reports are already available.

2.7 Hydrology and hydrogeology

The Environment Agency (EA) [2] has produced an aquifer designation system consistent with the requirements of the Water Framework Directive. The designations have been set for superficial and bedrock geology and are based on the importance of aquifers for potable water supply, and their role in supporting surface water bodies and wetland ecosystems.

The map indicates that the site is underlain by a superficial Secondary A aquifer. These aquifers are defined as "*Permeable layers capable of supporting water supplies at a local rather than strategic scale, and in some cases forming an important source of base flow to rivers. These are generally aquifers formerly classified as minor aquifers*". The London Clay Formation has been classified as 'Unproductive Stratum', which consists of low permeability material that has negligible significance for water supply or river base flow. The site is not within a Groundwater Source Protection Zone. The site is not shown to be within an area at risk of flooding either from rivers, reservoirs or surface water.

[2] www.environment-agency.gov.uk

The nearest significant surface water feature to the site is the *Whitestone Pond* some 200m northeast of the site. Maps published by the London Borough of Camden indicate that there is a number of lost rivers within 100m from the site, flowing both towards the south and the north as the site is situated on a hill.

3. SCREENING PROCESS

3.1 Subterranean Flow and Surface Flow, Slope/Land Stability and Flooding

CGL has adopted a screening process based on current basement guidance for a number of boroughs in London. Relevant questions for the site in Camden and proposed development have been thoroughly addressed within the report produced by Wallingford (Rep. No MAM7409 – RT002, Revision 5, August 2015). As the relevant information is included within the aforementioned report, this process will not be repeated.

4. GROUND INVESTIGATION

4.1 Current site investigation

Two intrusive site investigations were undertaken, one by Ground Engineering in February 2014 and one by Southern Testing in July 2014.

4.1.1 Fieldwork

The investigation by Ground Engineering comprised two cable percussive boreholes (BH2, BH3), a single window sample borehole (WS1) and three foundation inspection pits (TP1, TP2 and TP3) at selected positions. BH2 and BH3 were drilled in the rear and main garden respectively to 15m bgl, WS1 was drilled adjacent to Admiral's Walk to 6m bgl and TP1 – TP3 were excavated adjacent to Terrace Lodge to 1.3m and 1.6m bgl.

Field testing comprised Standard Penetration Tests (SPT) and the installation of standpipes in WS1, BH2 and BH3 to depths of 6m, 10.5m and 10m bgl respectively, through Bagshot Sands and into the Claygate Member. Bulk, disturbed and undisturbed samples were retrieved from the window sample and the cable percussive boreholes, whereas bulk samples were retrieved from the trial pits.

The investigation by Southern Testing comprised a single cable percussive borehole (BH1), drilled in the front garden to a depth of 15m bgl.

Field testing comprised SPTs and Hand Penetrometer Tests whilst a standpipe for groundwater monitoring was installed after completion of the borehole between 1.1m and 7m bgl, within the Made Ground, through Bagshot Sand and into the underlying clay. Disturbed and undisturbed samples were retrieved for laboratory testing.

The two ground investigation reports referenced above have been included within Appendix C and they include borehole logs as well as in-situ and laboratory tests results.

4.1.2 Laboratory analysis

Laboratory tests on selected soil samples were performed by Ground Engineering and Southern Testing respectively. The tests included the following:

- Unconsolidated Undrained Triaxial compression;
- Atterberg Limits;

- Moisture content;
- Particle Size Distribution;
- Bulk and Dry Density;
- Chemical Tests.

Geotechnical test results are included in Appendix C.

5. GROUND AND GROUNDWATER CONDITIONS

5.1 Summary

The ground conditions encountered during the intrusive investigations are generally consistent with those expected from the desk study information reviewed in Section 2.6 of this report. The ground conditions on site are summarised in Table 1 below.

Table 1. Summary of ground conditions

Strata	Depth to top mbgl[a] [mAOD]	Thickness (m)
Brown gravelly silty SAND. Gravel is brick and concrete. [MADE GROUND]	0 [127.5 – 129.10]	1.20 – 2.30
Medium dense to dense brown silty gravelly SAND. Gravel is angular to rounded flint and quartzite. [BAGSHOT FORMATION]	1.20 – 2.30 [126.30 – 127.30]	4.0 – 5.7
Firm becoming stiff brown slightly sandy silty CLAY (Ground Engineering, BH2). Firm to stiff slightly sandy CLAY (2.7m thick) underlain by clayey fine SAND (Southern Testing, BH1) [CLAYGATE MEMBER/BAGSHOT FORMATION]	6.80 – 7.50 [121.20 – 121.70]	7.50 – 8.20 (Not proven)
Note: There is a discrepancy between the two site investigations regarding the layer of clay proven at 121.2m AOD by Southern Testing and 121.7m AOD by Ground Engineering, which indicates variability in the ground conditions. The Southern Testing GI has proven that the layer of clay is 2.7m thick and is underlain by clayey fine sand with clay laminations whereas the Ground Engineering GI indicates that the thickness of the clay is at least 8.2m (the base of the layer was not proven). Because of this difference a single ground model will not be representative of the whole site. For the purposes of this report however, it is more conservative to assume that the Bagshot Sand is underlain by firm clay.		

a) mbgl = metres below existing ground level; mAOD = metres above Ordnance Datum

Each stratum is discussed in the following sections together with the results of the geotechnical tests. Plots of SPT 'N' values and undrained shear strength, c_u (kPa) versus level (m bgl) are presented in Figure 4 and Figure 5, respectively.

5.2 Made Ground

Made Ground encountered comprised gravelly silty sand with thickness varying between 1.2m and 2.3m.

5.3 Bagshot Formation

Bagshot Formation was encountered directly below the Made Ground and comprised medium dense brown silty gravelly sand. The total thickness of the stratum varied between 4m and 5.7m. It should be noted that in borehole BH1 the formation was logged to 15m with a total thickness of 13m which also included a 2.7m thick clay stratum between 6.3m and 9.0m bgl (121.2m – 118.5m AOD). Therefore, it is expected in any case that the basement will be founded on clayey soils.

SPT 'N' values in Bagshot Sands commonly vary between 15 and 25. Values of 12 and 36 have been recorded as well but were individual cases.

Particle size distribution tests indicate that the fractions of sand and gravel are often poorly graded, however there is a significant fraction of silt.

The clayey stratum of the formation which was identified in borehole BH1 has been described as firm to stiff and hand penetration tests undertaken at 6.3m and 8m bgl indicate that the undrained shear strength is 60kPa and 75kPa respectively.

5.4 Claygate Member

The Claygate Member of London Clay was identified in boreholes BH2 and BH3 at 7.5m bgl (121.6mAOD) and 6.8m bgl (121.7mAOD) respectively. It consists of firm becoming stiff brown slightly sandy silty clay. SPT 'N' values range between 15 and 27, which is in general agreement with the log descriptions. Three undrained triaxial tests were undertaken and the derived undrained shear strength ranged between 27kPa and 92kPa. The low values are not consistent with the corresponding SPT 'N' values at similar depths.

As mentioned in Table 1 above, BH1 indicates the presence of clayey fine sand at the same depths, interpreted as Bagshot Formation. For the ground model of this report it is assumed that the soil below 121.6m AOD is predominantly clayey.

5.5 Groundwater

Groundwater was encountered during the ground investigation in boreholes BH2 and WS1 at depths of 4.9m bgl (123.3m AOD) and 14.5m bgl (114m AOD) respectively.

In subsequent measurements, it was found that water in BH1 was at 3.8m bgl (123.7m AOD) whereas in BH2, BH3 and WS1 it ranged between 4.2m and 6.21m bgl (121.6m – 124m AOD). It is therefore likely that groundwater will be encountered at the base of the proposed excavation.

5.6 Geotechnical design parameters

Geotechnical design parameters for the proposed development are summarised in Table 2 below, these are based on the results of laboratory and in-situ testing. It should be noted that the parameters below are for heave/settlement calculations only.

Table 2. Geotechnical design parameters

Stratum	Design Level (Depth to top) (m bgl)* [m AOD]*	Bulk Unit Weight γ_b (kN/m^3)	Undrained Cohesion c_u (kPa) [c']	Friction Angle ϕ' (°)	Young's Modulus E_u (MPa) [E']
Made Ground	0 [127.5]	19[a]	- [-]	28[c]	[7][d]
Bagshot Formation	2.3 [125.2]	19[a]	- [-]	34[c]	[25][d]
Claygate Member/Bagshot Formation[f]	6.3 [121.2]	19[a]	65[b]	27[c]	33[d] [24][e]

*m bgl: metres below ground level
m AOD: metres above Ordnance Datum
Note: the elevation of 127.5m AOD has been selected as a reference level, however elevations across the site may vary and the elevations of the ground model strata will vary accordingly.

a. BS8002, assuming medium dense sand and firm clay.
b. Based on Cu=4.5N (assuming high plasticity clay), for N = 15
c. Site Investigation Report by Southern Testing (Report No J11827, June 2014).
d. Stiffness for Made Ground selected conservatively and based on available SPT values and soil description (predominantly sands/gravels). Stiffness for Bagshot Formation based on E'=1200(N+6) (Bowles, 5th edition). Stiffness for Claygate Member based on Eu = 500Cu (Lower bound).
e. Based on 0.75Eu - Burland, Standing J.R., and Jardine F.M. (eds) (2001), Building response to tunnelling, case studies from construction of the Jubilee Line Extension London, CIRIA Special Publication 200.
f. In order to adopt a conservative approach, it has been assumed that the soil underlying the Bagshot Sands is predominantly clayey.

6. SUBTERRANEAN (GROUNDWATER) FLOW

6.1 Impact on Groundwater Flows

The impact on the groundwater flow has already been assessed within the BIA (Rep. No MAM7409 – RT002, Revision 5, August 2015) and it was concluded that no significant effects on the groundwater regime are expected.

6.2 Impact on adjacent properties/infrastructure

As it is expected that no significant changes will occur in groundwater pressures around the site and given that granular deposits are predominant between ground surface and a minimum of 4m bgl, ground movements / settlement due to changing groundwater levels are not expected to be high.

6.3 Recommendations for Groundwater Control

The basement will be constructed using a secant piled wall with the male piles to be terminated within clayey soils (Bagshot Formation clay layer or Claygate Member) and the female piles to be stopped shorter in order to allow the flow of water beneath the basement.

Current design has taken the groundwater conditions at the site into account and relevant information can be found within the BIA by HR Wallingford (Rep. No MAM7409 – RT002, Revision 5, August 2015, Appendix D). Based on the available information, the risk of water ingression at the excavation level is low.

7. BASEMENT IMPACT ASSESSMENT – LAND STABILITY

7.1 Introduction

As noted in Section 1 of this report, horizontal deflections due to the excavation of the basement have already been undertaken by others (R&E Geotechnical Consultants). Although CGL has not undertaken a separate retaining wall analysis, the results provided by R&E Geotechnical Consultants have been reviewed and it is considered that the approach is appropriate and the resulting displacements are as would be expected for an excavation under the given conditions. This section provides calculations to determine ground movements that will result from stress release due to excavations and demolitions of existing structures and settlements that will be induced by the surcharge of the new structures. Vertical ground movements due to the installation of the secant pile wall will be estimated as well, in order to derive net ground movements which will enable the calculation of strains and the assessment of the potential damage to the buildings.

Based on current drawings it can be concluded that the installation of the secant pile wall will affect neighbouring and adjacent structures. The basement slab will only be affected by heave caused by the removal of soils during the excavation and demolition of certain structures before the excavation takes place. The net ground movements (after the basement construction is complete) should result in heave, as the loads of the proposed structures are expected to be lower than the load of the removed soil.

The following construction process and effects are likely to give rise to ground movements:

1. Installation of secant piled wall. This will generate lateral and vertical ground movements that are assumed to be proportional to the length of the piles. Based on the available drawings, the piles will not bear loads from other structures and therefore they are not expected to have an impact on any buried services that may lie underneath. Consideration will be given however to the effects the installation may have on adjacent and nearby structures.

2. Deflection of the secant piled wall. Deflections occur as the excavation proceeds and the piled wall is loaded with earth pressures, this can give rise to lateral and vertical ground movements. As mentioned above, a retaining wall analysis has already been undertaken. Based on the results of this analysis, lateral and vertical movements behind the wall will be predicted and combined with the ground movements from installation in order to predict the net effect of the construction of the wall.

3. Heave. Excavation of the basement and demolition of existing structures gives rise to undrained elastic heave, resulting in upwards movement of the foundation soils. Heave movements are not anticipated to be significant within the Bagshot Formation as it comprises sands and gravels which are not particularly affected by slow unloading. However, the relatively impermeable Claygate Member will be subject to heave due to stress relief. The amount of long-term heave depends on final construction loads and basement floor slab detailing. It is understood that a number of tension piles will be installed before the excavation, however the distances between them are high compared to their diameter and as they are not connected with a rigid pile cap, their contribution will be ignored in this analysis.

4. Underpinning. Underpinning for the support of the walls of the existing basement is not expected to generate significant ground movements as the process involves construction at intervals of relatively small width (~1m) and the reinforced concrete commonly used is of high stiffness. Therefore, underpinning will not be considered further in this assessment.

7.2 Critical sections for analysis

The critical constraints that will be considered for assessment are:

- Grove Lodge (Drawing Nos 5954/303 and 5954/310-318 – P1)
- Admiral's House (Drawing Nos 5954/307-309 – P1)
- Terrace Lodge (Drawing No 5954/306 – P1)

The plan locations of the critical sections are presented on Figure 2.

7.3 Ground movements arising from demolition and basement excavation

The clayey soils below basement formation level will be subject to stress relief during demolition and basement excavation. This is likely to give rise to a degree of elastic heave over the short term and potential heave or settlement over the long term as pore pressures recover in the Claygate Member and structural loads are reapplied. It is understood that a raft will form the foundation system. This will be designed to restrain the soils at formation level and thus negate any long-term heave effects that could occur over the design life of the structure.

The magnitude of ground movements has been calculated using OASYS Limited PDISP analysis software. PDISP assumes that the ground behaves as an elastic material under loading, with movements calculated based on the applied loads and the soil stiffness (E_u and E') for each stratum input.

For the short term analysis, the net stress change at proposed basement formation level due to various construction activities has been calculated and considers the three construction stages summarised below;

1. Stress decrease during demolition of garage and boundary garden walls;
2. Bulk excavation from ground level to proposed basement formation level.
3. Application of stresses induced by the basement slabs and the walls around the basement.

The load values used for each stage of construction and corresponding cumulative values applied in the PDISP analysis at proposed basement formation level (123.7m AOD) are summarised in Table 3.

For the demolition of the garden boundary walls and the garage, it has been assumed that the overall stress relief from demolition of structures will be 35kPa over the whole excavation surface.

The bulk unit weight of 19kN/m^3 for the excavated soils (Made Ground and Bagshot Formation) has been adopted in Table 3, whilst a depth of 4m bgl has been adopted for the excavation. The overall load which will be undertaken by the basement foundations after the construction works, is assumed to be 50kPa.

Table 3. Net load calculations for PDISP analysis

Stage 1[c]	**Stage 2[c]**	**Stage 3[c]**	**Stage 3[c]**
Demolition of structures (kN/m^2)[a]	**Total basement excavation from ground level (kN/m^2)[b]**	**Loads applied by basement slab (kN/m^2)**	**Loads applied by perimeter walls on secant piles (kN/m^2)[a]**
-35	-76	+7.2[d]	+50

a. Assumption.
b. Assumes 4m basement excavation depth.
c. Positive values (+) indicate stress increase and negative (-) values indicate stress reduction.
d. Based on 300mm thick slab with unit weight of 24kN/m^3.

The proposed development gives rise to a net unloading of the underlying strata during construction and over the long-term. At the excavation level (123.7m AOD), the maximum short term heave after the excavation is predicted to be 18mm whilst the maximum net long term heave is expected to be in the order of 3mm.

The amount of undrained unloading (short term) has been estimated and the effects on the structures are presented as displacement contour plots within Figure 6. The ground movements within the contour plots are taken from two displacement grids applied at the level of the foundations of the existing buildings (approximately 125m AOD) and at the basement formation level, to illustrate the effects of the construction stage on the basement and the surrounding buildings. Displacement lines have been added to the PDISP models to illustrate the ground movement profiles at the locations of the surrounding structures and to undertake an impact/damage assessment for each of the affected structures.

Imposed loads and drained conditions have also been considered in a similar manner for the estimation of ground movements after the construction. The results are presented in Figure 7 which shows the predicted settlements at the level of the existing buildings foundations as well as the level of the excavation.

Maximum undrained heave values, which occur during the excavation, have been estimated at 18mm for the centre of the excavation. Maximum ground movements in the long term after the structural loads are imposed have been estimated at 3mm of heave, whilst total movements will be up to 21mm of heave, again for the centre of the excavation. Only the long term heave is likely to have any effect on the basement slab. The stiffness of the slab will further reduce the effects.

The vertical heave movements for the rest of the structures will be reduced due to their distance from the centre of the excavation. The secant piles at the perimeter of the excavation are expected to limit the heave effects outside the excavation boundaries however their contribution has been ignored.

The results of the vertical ground movement analysis are summarised in Table 4 below for both short and long term. The PDISP output can be provided separately upon request.

Table 4. Summary of maximum heave movements within excavation and at constraint locations

Stage	Centre of excavation (123.7m AOD) (mm)	Grove Lodge[a] (127.5m AOD) (mm)	Terrace Lodge [a] (127.5m AOD) (mm)	Admiral's House [a] (125.0m AOD) (mm)
Short term movement	-17.9	-11.0	-5.5	-5.6
Long term movement [b]	-2.8	-1.8	-1.0	-1.0

a. Based on results of displacement line at level and plan location of constraint
b. Positive values (+) indicate settlement and negative (-) values indicate heave.

The results of the above assessment and corresponding ground movement profiles have been brought forward into Section 8 where the cumulative impact due to demolition, excavation, pile wall installation and deflection on neighbouring properties has been assessed.

7.4 Ground movement due to retaining wall deflection

As mentioned above the retaining wall analysis has been undertaken by others (R&E Geotechnical Consultants) and is included within Appendix D. Analyses have been undertaken on three sections as shown in the accompanying sketch for both Serviceability Limit State (SLS) and Ultimate Limit State (ULS). This section presents the results of retaining wall analysis to provide predictions of ground movement behind basement walls in the location of the critical constraints. The proposed construction methodology and sequence which was adopted is summarised below:

1. Install secant piled wall (ground level at 127.5m AOD).
2. Install supporting prop at top of the pile.
3. Excavate to 123.7m AOD.
4. Install strut at 124.0m AOD.
5. Remove temporary prop at the top.

7.4.1 Analysis results

Analysis of the retaining wall has been undertaken by R&E Geotechnical Consultants using WALLAP embedded retaining wall analysis software. Serviceability limit state (SLS) criteria have been used to determine wall deflections. Calculation sheets are provided within Appendix D, whilst the total predicted horizontal deflections at the back of the wall are calculated as the sum of the movements induced by the excavation and the movements induced by the installation of the piles (taken as 0.08% of the pile length as a conservative assessment for a clayey sand, based on interpretation of CIRIA C580, Table 2.2 page 50). The results have been summarised within Table 5. The corresponding ground settlements at the critical constraints are also provided.

It has been conservatively assumed that the water table will be at the excavation level (123.7m AOD). For long term conditions it has been assumed that there will be a difference of 1.4m between the water table in the active side and the water table in the passive side, after the excavation.

The estimation of the horizontal and vertical ground movements at a distance behind the wall has been undertaken by CGL. The distance to negligible horizontal movements behind the wall has been taken as twice the excavation depth. Linear interpolation was then undertaken to determine the displacements between the two ends.

Vertical ground movement has been calculated by taking 0.25% of the excavation depth as suggested by CIRIA Report C580 for excavations in sand. The reduction of the settlements at various distances behind the wall are reduced based on the factors provided within the relevant plot in page 56 of C580.

Table 5: Results of WALLAP analysis

Section	Maximum wall deflection (mm)[a]	Total Horizontal deflection at location/level of constraint (mm)[a]	Vertical settlement below location of constraints (mm)[b]
Grove Lodge	3.0	7.4	14.0
Terrace Lodge	7.0	7.1	8.2
Admiral's House	3.0	3.7	4.0

a. Positive values indicate lateral wall deflection towards the basement excavation. Values derived from adding movements both from installation and excavation

b. Positive values (+) indicate settlement and negative (-) values indicate heave behind the piled wall. Values derived from adding settlements both from installation and excavation.

The vertical and horizontal ground movements predicted by the above calculations are conservative as they do not take into account increased stiffness at the corners of the wall, they have been assumed to dissipate linearly with increasing distance from the wall and have been assumed to take place in green field conditions.

With regard to indicative wall displacements that may be expected during excavation, it should be noted that WALLAP uses a Winkler spring analysis to determine the wall displacements. In a Winkler medium, springs are used to represent a continuum and there is no transfer of shear stresses between the springs. In general, the application of this concept can lead to an overestimation of structural deformations; hence the resulting wall displacements and corresponding impact on the nearby structures and infrastructure may be over-predicted by the WALLAP program.

7.5 Ground movement due to retaining wall installation

With reference to CIRIA C580[3], vertical and horizontal surface movements due to installation of a secant piled wall are generally in the region of 0.05% and 0.08% of the wall depth, respectively. Based on an installation depth of 8m, the results are a horizontal movement of approximately 6.4mm at the top of the wall and a vertical movement of approximately 4mm behind the wall. It has been assumed that these displacements dissipate linearly with distance.

The combined cumulative impact of ground movements associated with the pile wall installation, pile deflection and heave due to excavation (short and long term) on neighbouring properties and infrastructure is discussed in greater detail in Section 8.

[3] CIRIA C580 (2003) *Embedded Retaining Walls – guidance for economic design*

8. BUILDING DAMAGE ASSESSMENT

The calculated ground movements have been used to assess potential 'damage categories' that may apply to neighbouring structures due to the proposed basement construction method and assumed construction sequence. The methodology proposed by Burland and Wroth[4] and later supplemented by the work of Boscardin and Cording[5] has been used, as described in *CIRIA Special Publication 200*[6] and *CIRIA C580*.

General damage categories are summarised in Table 6 below:

Table 6. Classification of damage visible to walls (reproduction of Table 2.5, CIRIA C580)

Category	Description
0 (Negligible)	Negligible – hairline cracks
1 (Very slight)	Fine cracks that can easily be treated during normal decoration (crack width <1mm)
2 (Slight)	Cracks easily filled, redecoration probably required. Some repointing may be required externally (crack width <5mm).
3 (Moderate)	The cracks require some opening up and can be patched by a mason. Recurrent cracks can be masked by suitable linings. Repointing of external brickwork and possibly a small amount of brickwork to be replaced (crack width 5 to 15mm or a number of cracks > 3mm).
4 (Severe)	Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows (crack width 15mm to 25mm but also depends on number of cracks).
5 (Very Severe)	This requires a major repair involving partial or complete re-building (crack width usually >25mm but depends on number of cracks).

The above assessment criteria are primarily relevant for assessing masonry structures founded on strip footings. Therefore, this methodology will be adopted within the damage assessment for the affected structures in Admiral's Walk.

[4] Burland, J.B., and Wroth, C.P. (1974). *Settlement of buildings and associated damage*, State of the art review. Conf on Settlement of Structures, Cambridge, Pentech Press, London, pp611-654

[5] Boscardin, M.D., and Cording, E.G., (1989). *Building response to excavation induced settlement*. J Geotech Eng, ASCE, 115 (1); pp 1-21.

[6] Burland, Standing J.R., and Jardine F.M. (eds) (2001), *Building response to tunnelling, case studies from construction of the Jubilee Line Extension London*, CIRIA Special Publication 200.

8.1 Impact Assessment – Grove Lodge

The results of the predicted ground movement below Grove Lodge due to the proposed basement development have been compiled to determine the overall lateral and vertical deflection of the structure.

Figure 8 shows the combined lateral movement of the piled wall due to pile installation and deflection. The maximum lateral deflection of the piled wall at the level of Grove Lodge foundations (125m AOD) is predicted to be 3mm. Adding the horizontal movement due to installation, results in a total horizontal movement of 7.4mm which corresponds to a horizontal strain of 0.062% (calculated over a total building length of 12m).

Combined cumulative vertical movement profiles below the structure, including short and long term heave due to excavation, retaining wall installation and wall deflection due to excavation are presented within Figure 9. The profile indicates that the maximum total vertical movement below Grove Lodge is 4mm of heave at a distance of 1m behind the secant pile wall. The maximum differential movement along the building is 2.5mm, resulting in a distortion of 1/4,800 (i.e. 2.5mm/12,000mm) which is within published limits[7, 8] for preventing excess cracking and damage to load bearing walls and partitions. The corresponding maximum deflection is 5mm corresponding to a deflection ratio of 0.042%, calculated over 12m of length (which is assumed to be the length of the building where maximum ground movements occur).

Table 7 incorporates a summary of the maximum lateral and vertical deflection (mm) of Grove Lodge and prediction of the corresponding horizontal strain and vertical deflection ratio.

Table 7. Summary of ground movements and corresponding damage category

Constraint	Net Horizontal movements (mm)[c]	Maximum deflection (mm)	Horizontal Strain ε_h[b] (%)	Deflection ratio Δ/L[a] (%)	Damage category
Grove Lodge	7.4	4.5	0.062	0.0375	1 – Very Slight

a. *See Figure 2.18 (a) CIRIA C580 (2003) Embedded retaining walls guidance for economic design. (L = length of adjacent structure in metres, perpendicular to basement; Δ = relative deflection)*
b. *See Box 2.5 (v) CIRIA C580 (2003) Embedded retaining walls guidance for economic design. (δ_h = horizontal movement in metres).*
c. Net horizontal movement along neighbouring structure.

Based on the above and assuming a good standard of workmanship the estimated maximum damage category imposed on Grove Lodge is 'Category 1' corresponding to 'very

[7] Skempton, A. W. & Mac Donald, D. H. (1956). The Allowable settlement of buildings. Proceedings of the Institution of Civil Engineers, Part 3, No. 5, pp 727-784.
[8] Polshin, D. E. & Tokar, R. A. (1957). Maximum allowable non-uniform settlement of structures. Proc. 4th Int. Conf. SM&FE, Wiesbaden, No. 1, pp. 285.

slight' damage, or fine cracks of up to 1mm in width. Although the deflection ratio and the horizontal strain correspond to a point at the boundary between Category 1 and Category 2, any inflicted damage is not expected to exceed Category 1. This can be justified by the fact that the above approach is conservative and the actual deflections and strains are expected to be less than those predicted.

8.2 Impact Assessment – Terrace Lodge

The results of the predicted ground movement below Terrace Lodge due to the proposed basement development have been compiled to determine the overall lateral and vertical deflection of the structure.

Figure 10 shows the combined lateral movement of the piled wall due to pile installation and deflection. The maximum lateral deflection of the piled wall at the level of Terrace Lodge foundations (125m AOD) is predicted to be 7mm. Adding the horizontal movement due to installation (4.8mm), results in a total horizontal movement of 11.8mm which at a distance of 2m behind the wall, where the building is situated, reduces to 7mm. This corresponds to a horizontal strain of 0.064% (calculated over a total building length of 11m).

Combined cumulative vertical movement profiles below the structure, including short and long term heave due to excavation, retaining wall installation and wall deflection due to excavation are presented within Figure 11. The profile indicates that the maximum total vertical movement below Terrace Lodge is 3mm of heave at the point which is closest to the secant pile wall. The maximum differential movement along the building is 3mm, resulting in a distortion of 1/3,333 (i.e. 3mm/10,000mm) which is within published limits[9, 10] for preventing excess cracking and damage to load bearing walls and partitions. The corresponding maximum deflection is 1.5mm corresponding to a deflection ratio of 0.015%, calculated over 10m of length (which is assumed to be the length of the building where maximum ground movements occur).

Table 8 incorporates a summary of the maximum lateral and vertical deflection (mm) of Terrace Lodge and prediction of the corresponding horizontal strain and vertical deflection ratio.

[9] Skempton, A. W. & Mac Donald, D. H. (1956). The Allowable settlement of buildings. Proceedings of the Institution of Civil Engineers, Part 3, No. 5, pp 727-784.

[10] Polshin, D. E. & Tokar, R. A. (1957). Maximum allowable non-uniform settlement of structures. Proc. 4th Int. Conf. SM&FE, Wiesbaden, No. 1, pp. 285.

Table 8. Summary of ground movements and corresponding damage category

Constraint	Net Horizontal movements (mm)[c]	Maximum deflection (mm)	Horizontal Strain ε_h[b] (%)	Deflection ratio Δ/L[a] (%)	Damage category
Terrace Lodge	7.1	1.5	0.071	0.015	1 – Very Slight

a. *See Figure 2.18 (a) CIRIA C580 (2003) Embedded retaining walls guidance for economic design. (L = length of adjacent structure in metres, perpendicular to basement; Δ = relative deflection)*
b. *See Box 2.5 (v) CIRIA C580 (2003) Embedded retaining walls guidance for economic design. (δ_h = horizontal movement in metres).*
c. Net horizontal movement along neighbouring structure.

Based on the above and assuming a good standard of workmanship the estimated maximum damage category imposed on Terrace Lodge is 'Category 1' corresponding to 'very slight' damage, or fine cracks of up to 1mm in width. Although the deflection ratio and the horizontal strain correspond to a point at the boundary between Category 1 and Category 2, any inflicted damage is not expected to exceed Category 1. This can be justified by the fact that the above approach is conservative and the actual deflections and strains are expected to be less than those predicted.

8.3 Admiral's House

The results of the predicted ground movement below Admiral's House due to the proposed basement development have been compiled to determine the overall lateral and vertical deflection of the structure.

Figure 12 shows the combined lateral movement of the piled wall due to pile installation and deflection. The maximum lateral deflection of the piled wall at the level of Admiral's House foundations (125m AOD) is predicted to be 3mm. Adding the horizontal movement due to installation (6.4mm), results in a total horizontal movement of 9.4mm which at a distance of 4m behind the wall, where the building is situated, reduces to 3.7mm. This corresponds to a horizontal strain of 0.031% (calculated over a total building length of 12m).

Combined cumulative vertical movement profiles below the structure, including short and long term heave due to excavation, retaining wall installation and wall deflection due to excavation are presented within Figure 13. The profile indicates that the maximum total vertical movement below Grove Lodge is 1.5mm of heave at the point closest to the excavation. The maximum differential movement along the building is 1.5mm, resulting in a distortion of 1/8,000 (i.e. 1.5mm/12,000mm) which is within published limits[11, 12] for

[11] Skempton, A. W. & Mac Donald, D. H. (1956). The Allowable settlement of buildings. Proceedings of the Institution of Civil Engineers, Part 3, No. 5, pp 727-784.

preventing excess cracking and damage to load bearing walls and partitions. The corresponding maximum deflection is 0.5mm corresponding to a deflection ratio of 0.004%, calculated over 12m of length (which is assumed to be the length of the building where maximum ground movements occur).

Table 9 incorporates a summary of the maximum lateral and vertical deflection (mm) of Grove Lodge and prediction of the corresponding horizontal strain and vertical deflection ratio.

Table 9. Summary of ground movements and corresponding damage category

Constraint	Net Horizontal movements (mm)[c]	Maximum deflection (mm)	Horizontal Strain ε_h[b] (%)	Deflection ratio Δ/L[a] (%)	Damage category
Admiral's House	3.7	0.5	0.031	0.004	0 – Negligible

a. *See Figure 2.18 (a) CIRIA C580 (2003) Embedded retaining walls guidance for economic design. (L = length of adjacent structure in metres, perpendicular to basement; Δ = relative deflection)*
b. *See Box 2.5 (v) CIRIA C580 (2003) Embedded retaining walls guidance for economic design. (δ_h = horizontal movement in metres).*
c. Net horizontal movement along neighbouring structure.

Based on the above and assuming a good standard of workmanship the estimated maximum damage category imposed on Admiral's House is 'Category 0' corresponding to 'negligible' damage.

8.4 Summary

The building damage assessment for the affected structures is summarised in Table 10 below. The Damage Assessment Plot is presented within Figure 14 for all structures.

Table 10. Damage Assessment for Affected Structures in Admiral's Walk

Constraint	Damage category
Grove Lodge	1 – Very Slight
Terrace Lodge	1 – Very Slight
Admiral's House	0 – Negligible

[12] Polshin, D. E. & Tokar, R. A. (1957). Maximum allowable non-uniform settlement of structures. Proc. 4th Int. Conf. SM&FE, Wiesbaden, No. 1, pp. 285.

9. SURFACE FLOW AND FLOODING

A review of the available information and development proposals indicates that the proposed basement is not likely to affect the existing groundwater flow patterns significantly. The site is not located within an area susceptible to floods.

10. CONSTRUCTION MONITORING

The results of the ground movement analysis suggest that with good construction control, damage to the structures surrounding the basement, generated by the assumed construction methods and sequence is likely to be (within Category 1) 'very slight damage'. The locations of buried services are not known, however based on the predicted ground movements, strain levels on such services are expected to be sustainable.

A formal monitoring strategy has been proposed for the observation and control of ground movements during construction.

In summary, the system comprises Total Station Monitoring of selected targets and Precise Levelling Stations on the highways adjacent to the site.

An appropriate monitoring strategy and trigger limits should be agreed with stakeholders prior to any works. It is likely that a baseline survey will be required by the owners of the affected assets prior to demolition of the existing garden boundary walls on site.

It is also recommended that vibration monitoring be considered during the demolition and piling works and that a current condition survey is undertaken of the neighbouring properties.

Monitoring data should be checked against predefined trigger limits and can also be further analysed to assess and manage the damage category of the adjacent buildings as construction progresses. The data could also potentially be used to undertake back analysis calculations and value engineer certain elements of the construction.

11. CONCLUSIONS

- The proposed development in Admiral's Walk comprises the construction of a single basement beneath the rear garden and partly beneath Grove Lodge. No party walls are to be underpinned.

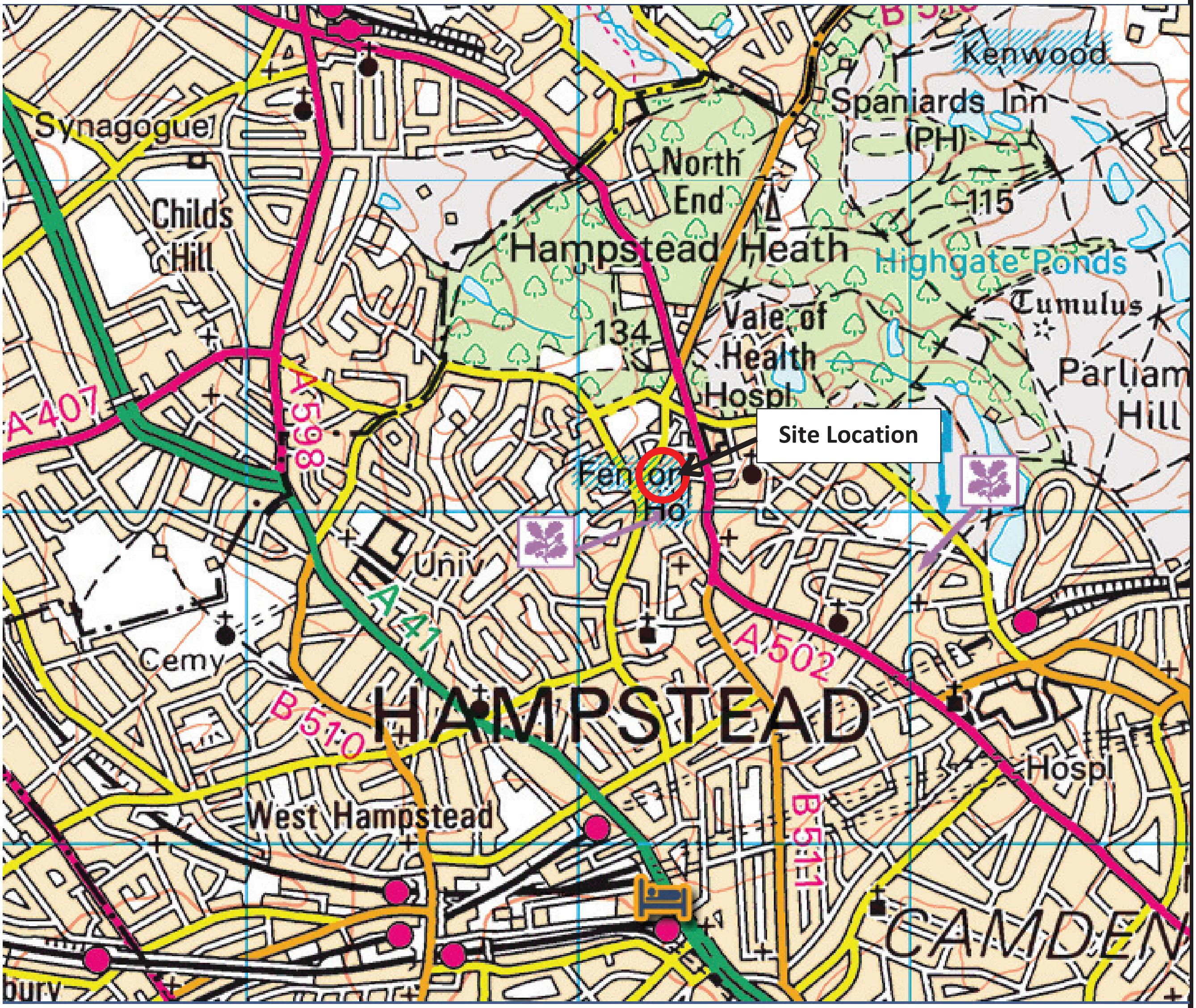
- Based on available information from previous work (Rep. No MAM7409 – RT002, Revision 5, August 2015), the proposed basement development is expected to have a negligible effect on groundwater flow, surface water and flooding at this site.

- The construction of the basement will generate ground movements due to a variety of causes including heave due to demolition and excavation, ground settlement due to pile wall installation, and deflection during basement excavation. It is understood that underpinning is part of the construction sequence, however any ground movements that might be generated during underpinning, are expected to be negligible.

- An assessment of the results of the detailed ground movement analysis and displacement profiles indicates that these movements will give rise to a damage category within 'Category 1' (very slight damage) for Grove Lodge and Terrace Lodge. Less damage is expected to be inflicted on Admiral's House (Category 0). This damage category is within allowable limits as specified by London Borough of Camden's *Camden Planning Guidance: Basements and Lightwells,* September 2013.

- An appropriate monitoring regime has been adopted to manage risk and potential damage to the neighbouring properties and any existing buried services.

FIGURES



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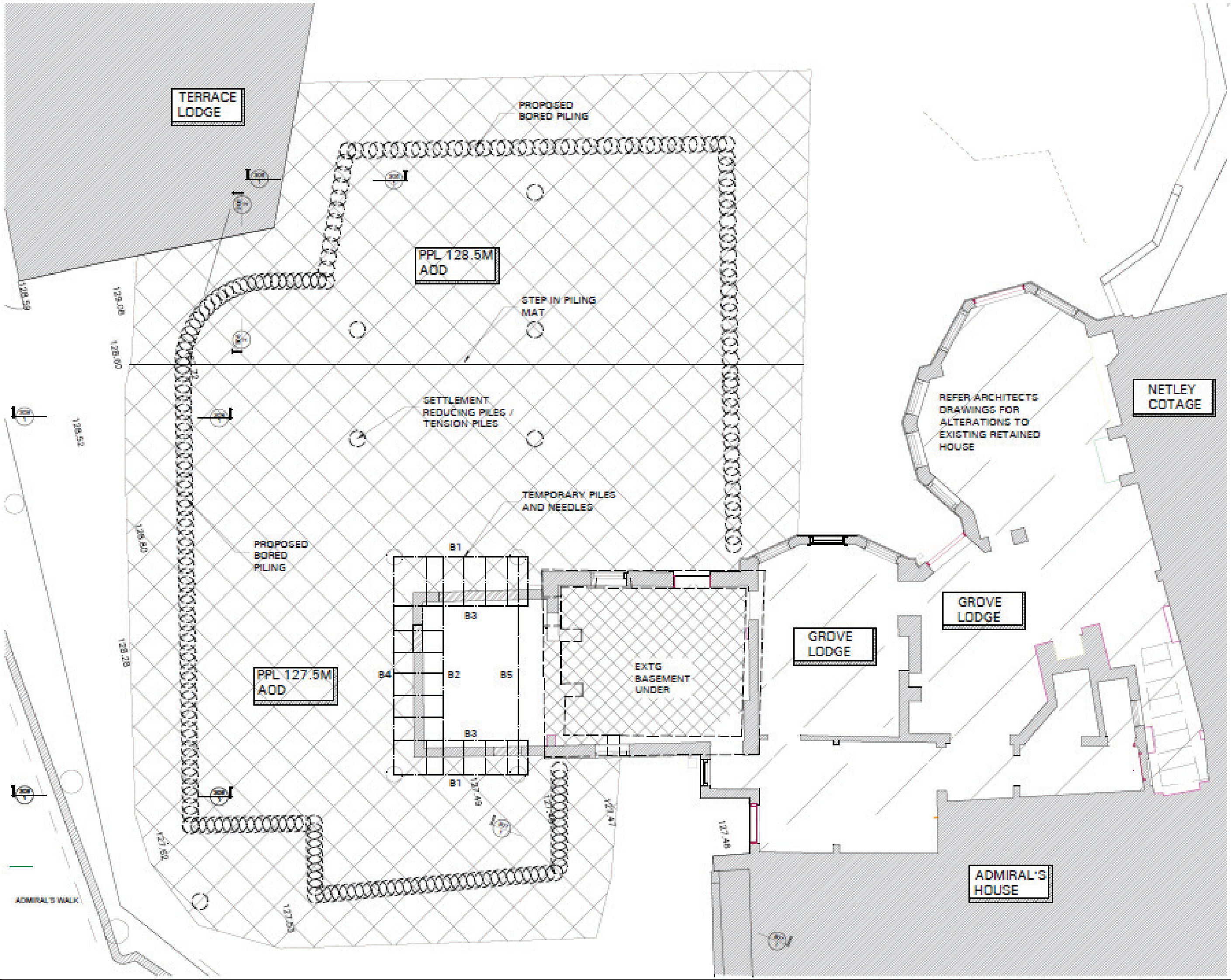
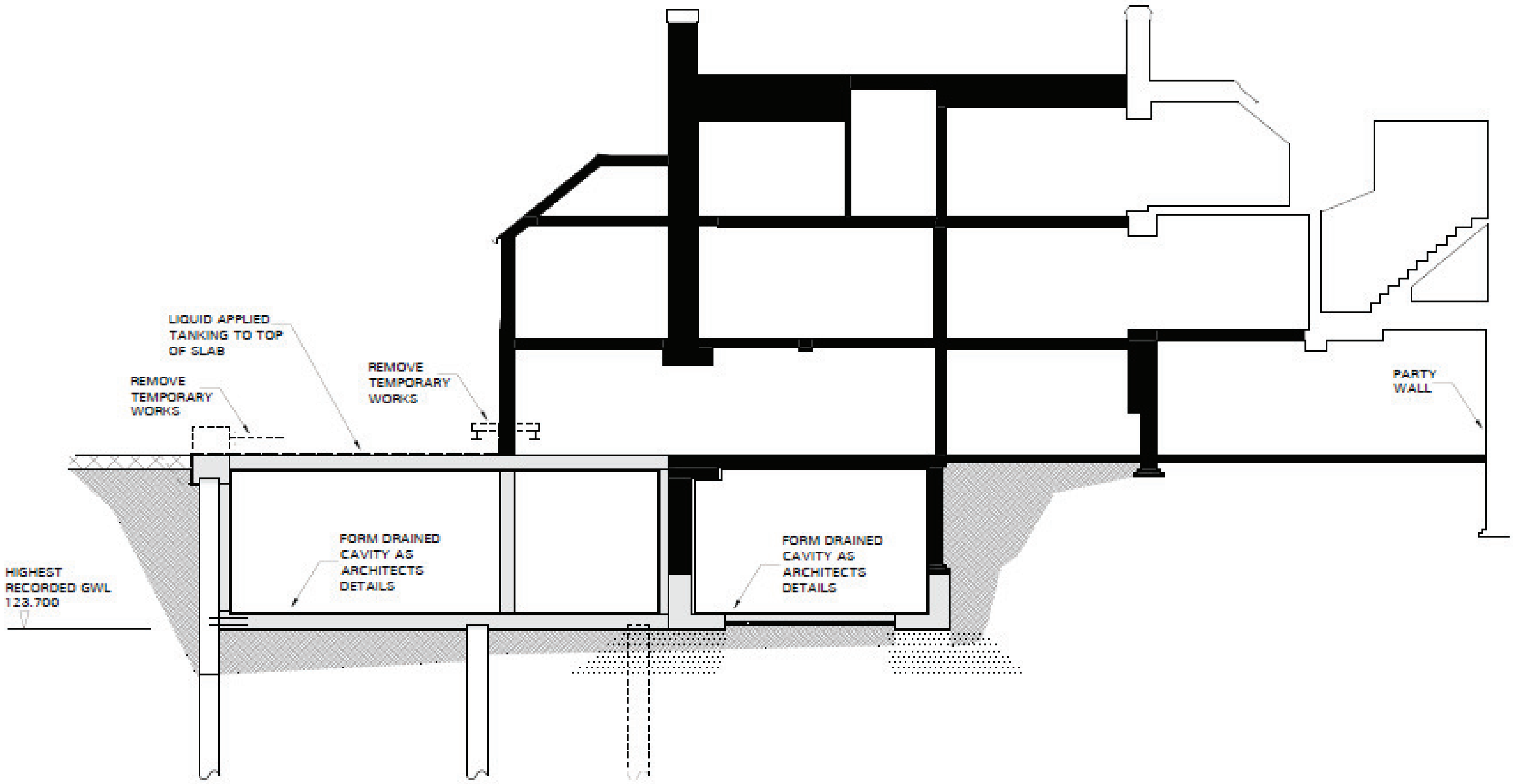
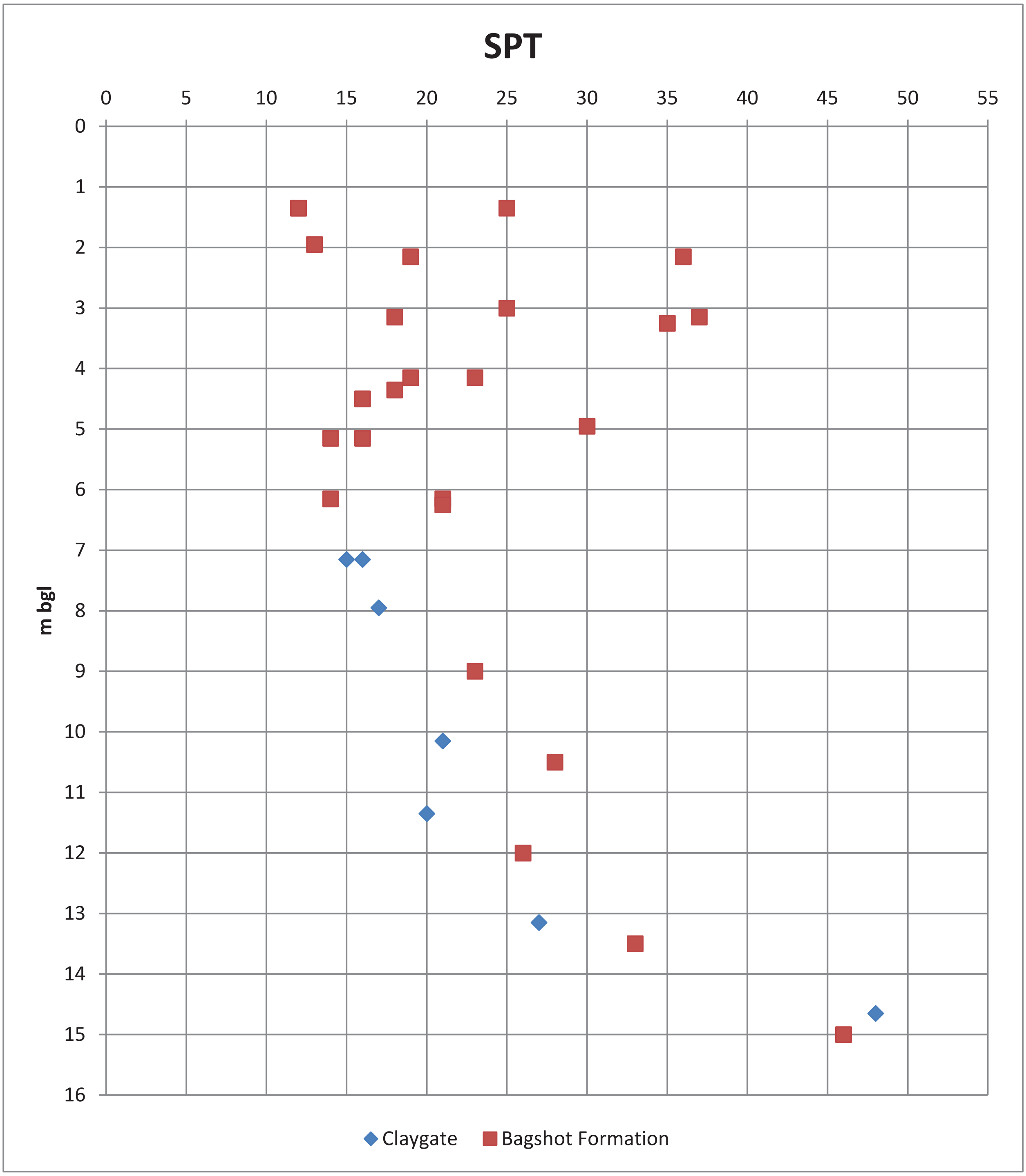
Client	Project	Job No
Mr Caspar Berendsen	**Admiral's Walk, London**	**CG/09013**
CGL	Title **Site location plan**	**Figure 1**



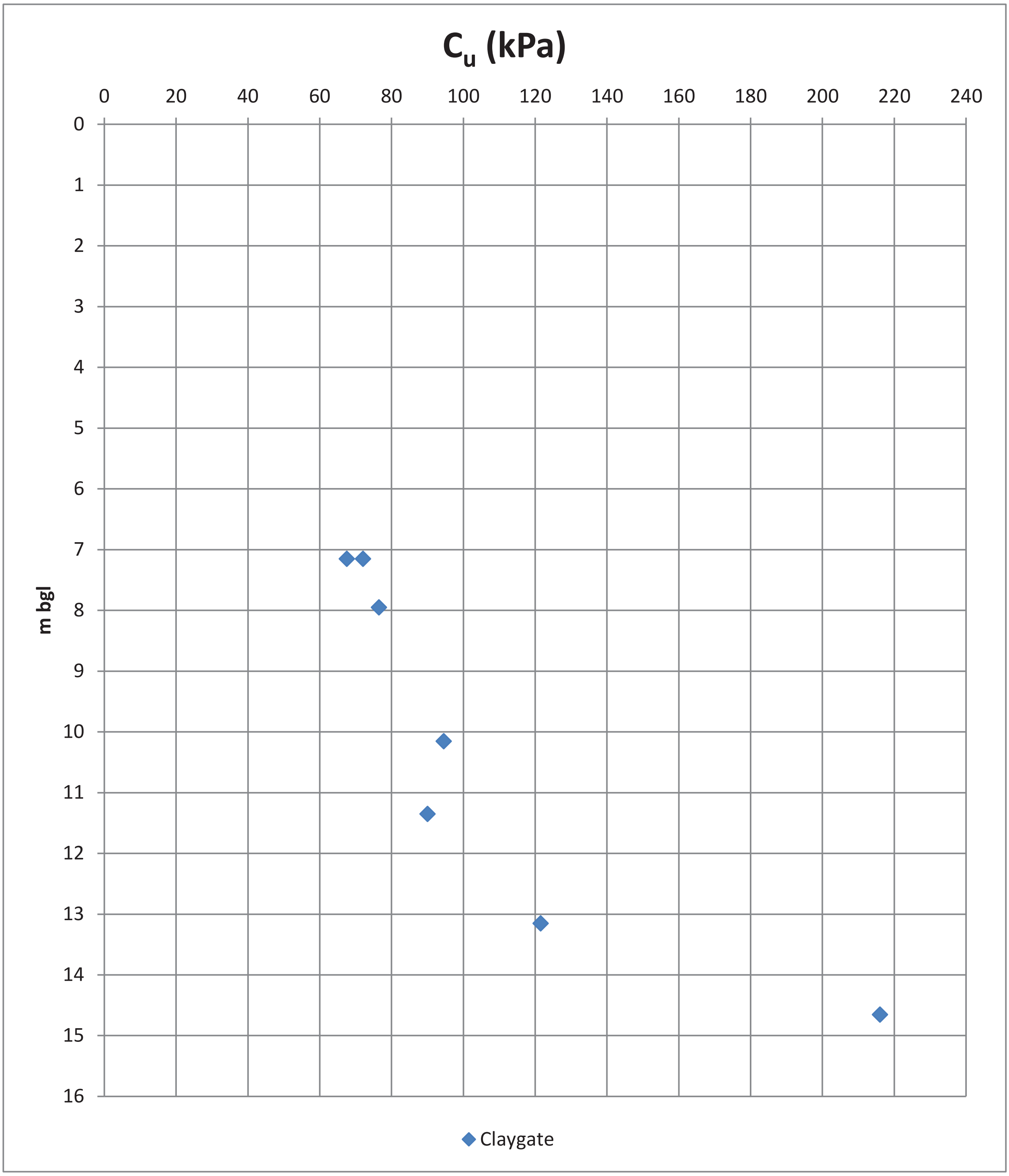
Client	Project	Job No
Mr Caspar Berendsen	Admiral's Walk, London	CG/09013
CGL	Title: Site Layout Plan and Nearby Properties	Figure 2



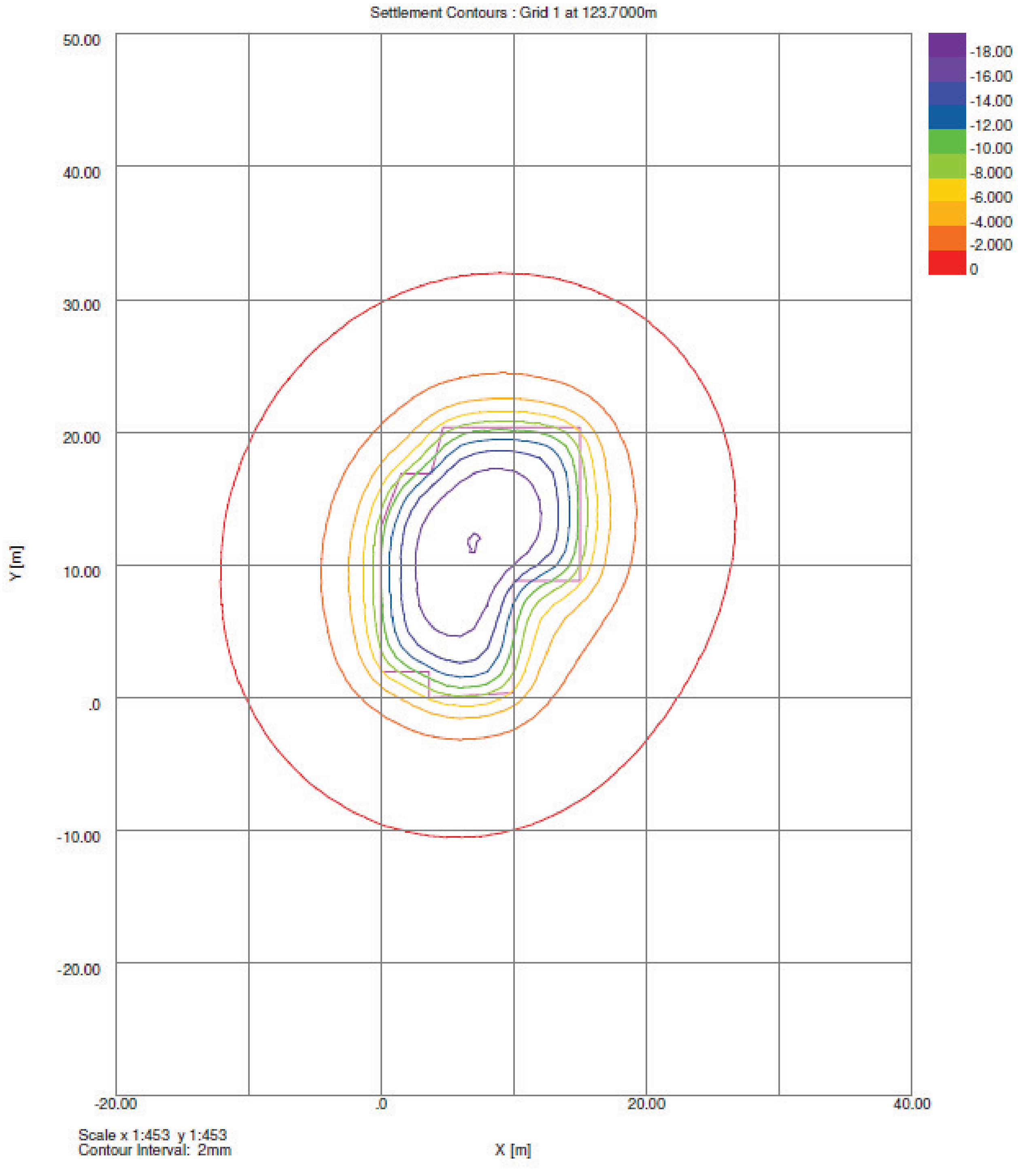
Client	Project	Job No
Mr Caspar Berendsen	Admiral's Walk, London	CG/09013
CGL	Title: Typical Cross Section	Figure 3



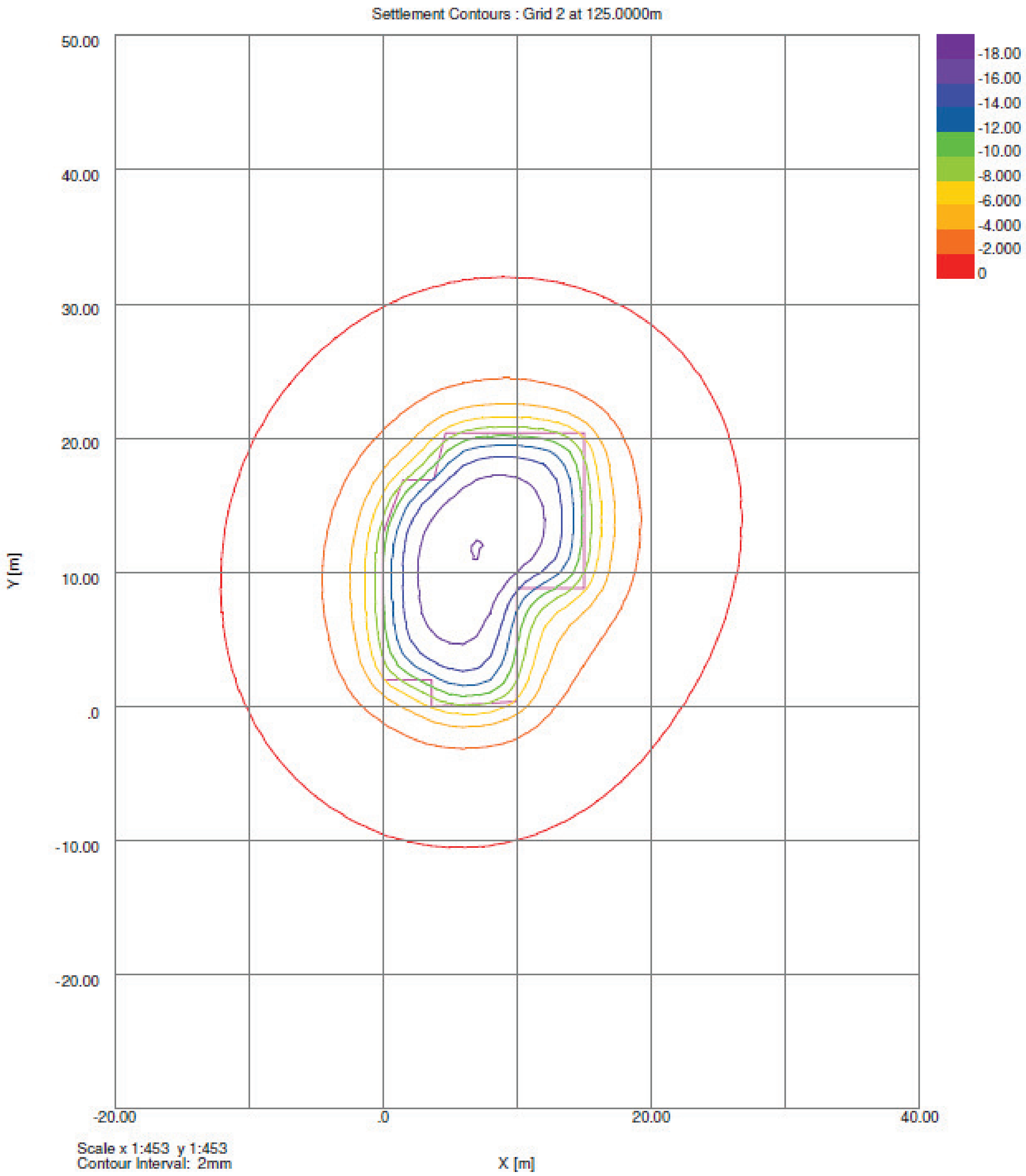
Client	Project	Job No
Mr Caspar Berendsen	**Admiral's Walk, London**	**CG/09013**
CGL	Title **SPT Results**	**Figure 4**



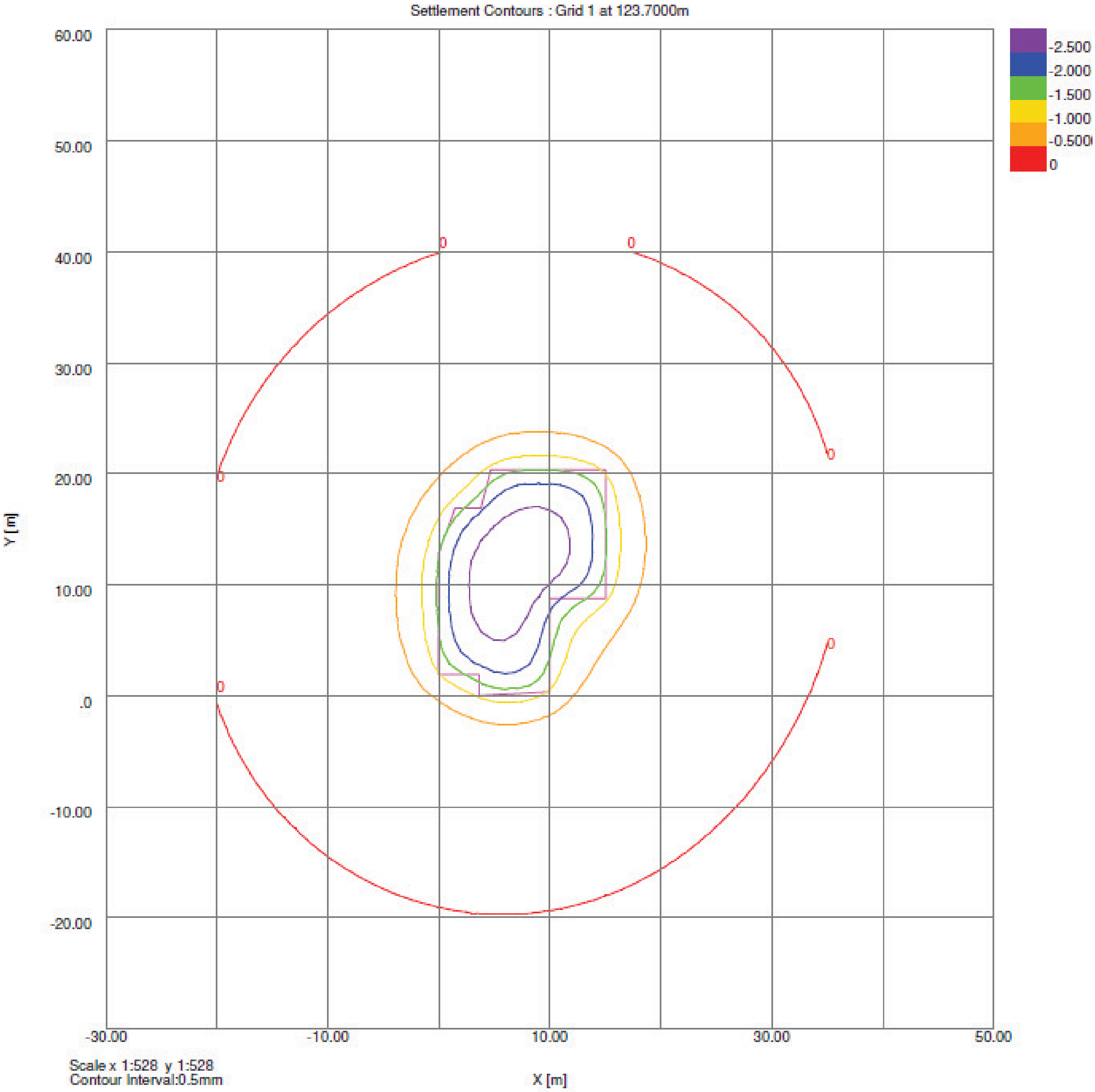
Client	Project	Job No
Mr Caspar Berendsen	**Admiral's Walk, London**	**CG/09013**
CGL	Title **C_u – Depth Plot**	**Figure 5**



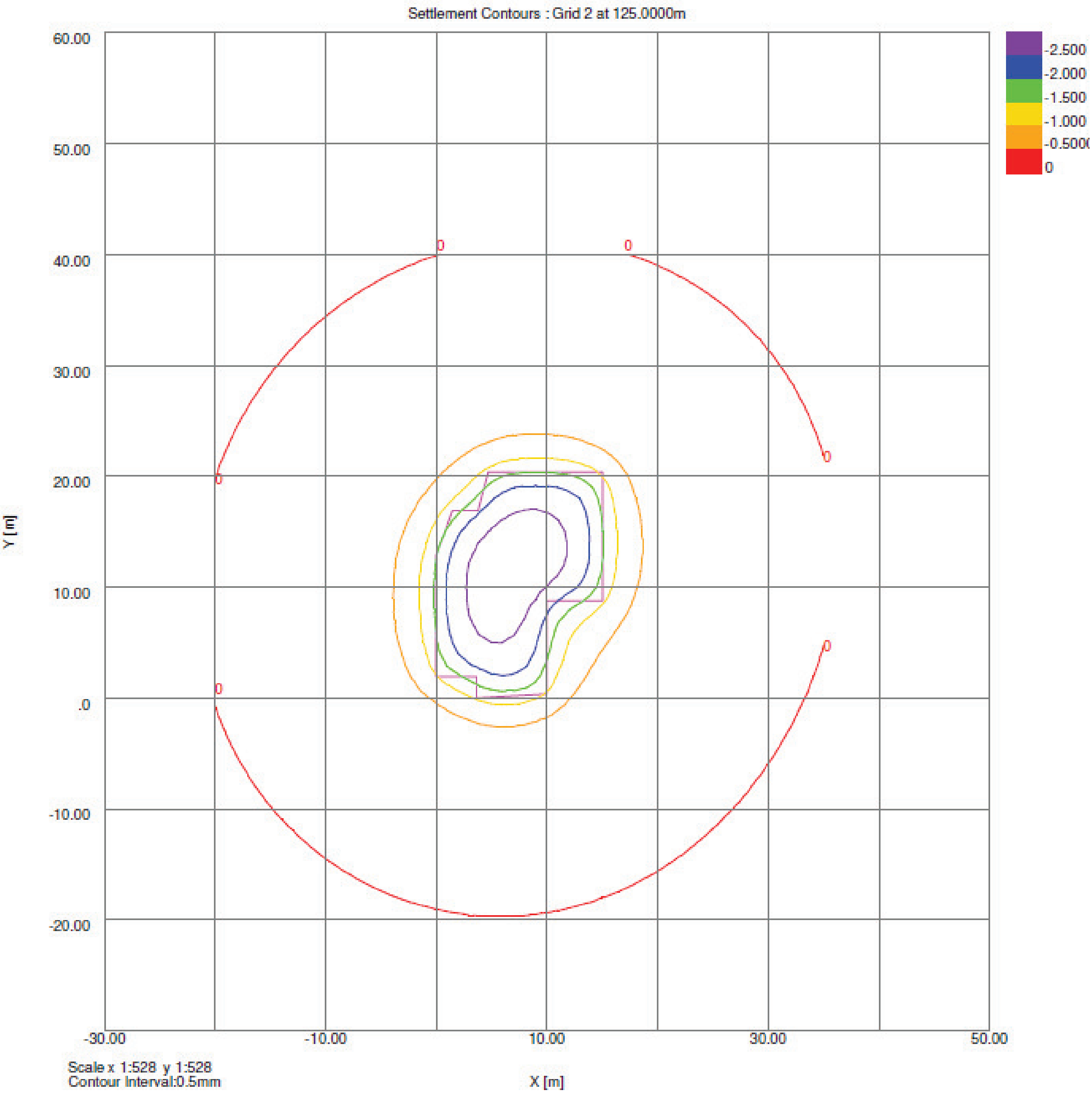
Client	Project	Job No
Mr Caspar Berendsen	**Admiral's Walk, London**	**CG/09013**
CGL	Title **Undrained Heave Contours (Excavation Level 123.7mAOD)**	**Figure 6a**



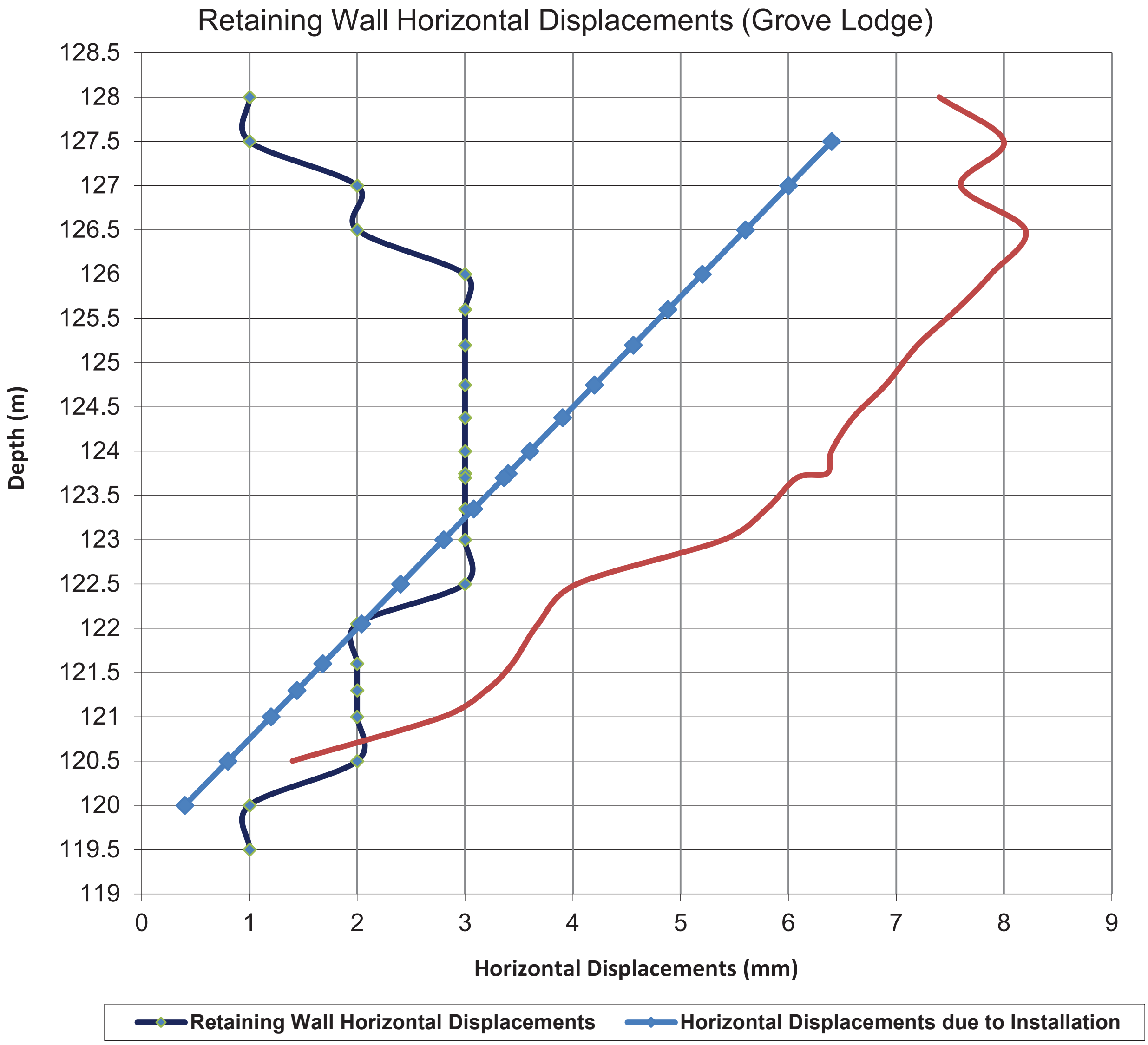
Client	Project	Job No
Mr Caspar Berendsen	**Admiral's Walk, London**	**CG/09013**
CGL	Title **Undrained Heave Contours (Foundation Level – 125mAOD)**	**Figure 6b**



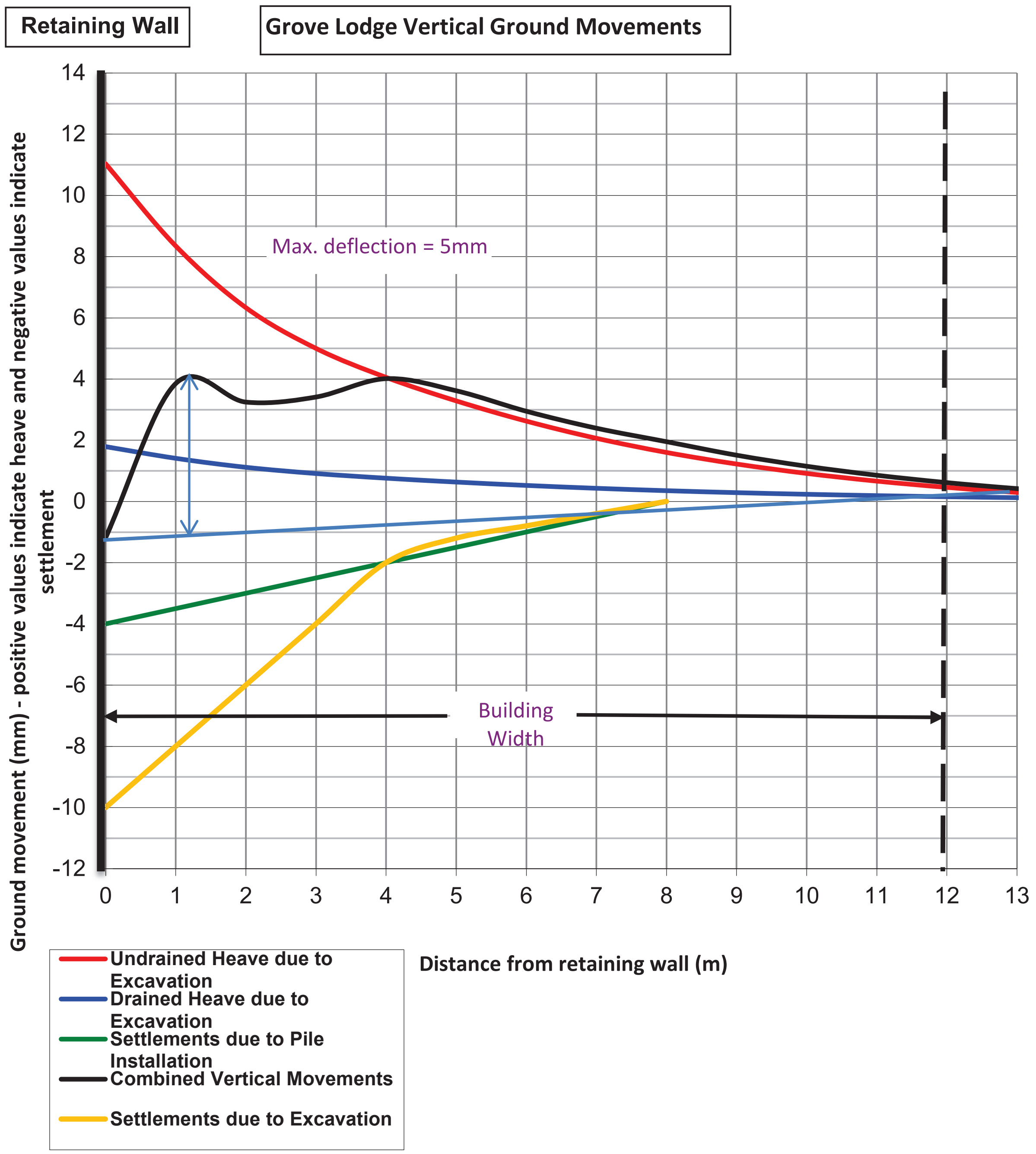
Client	Project	Job No
Mr Caspar Berendsen	**Admiral's Walk, London**	**CG/09013**
CGL	Title **Drained Heave Contours (Excavation Level 123.7mAOD)**	**Figure 7a**



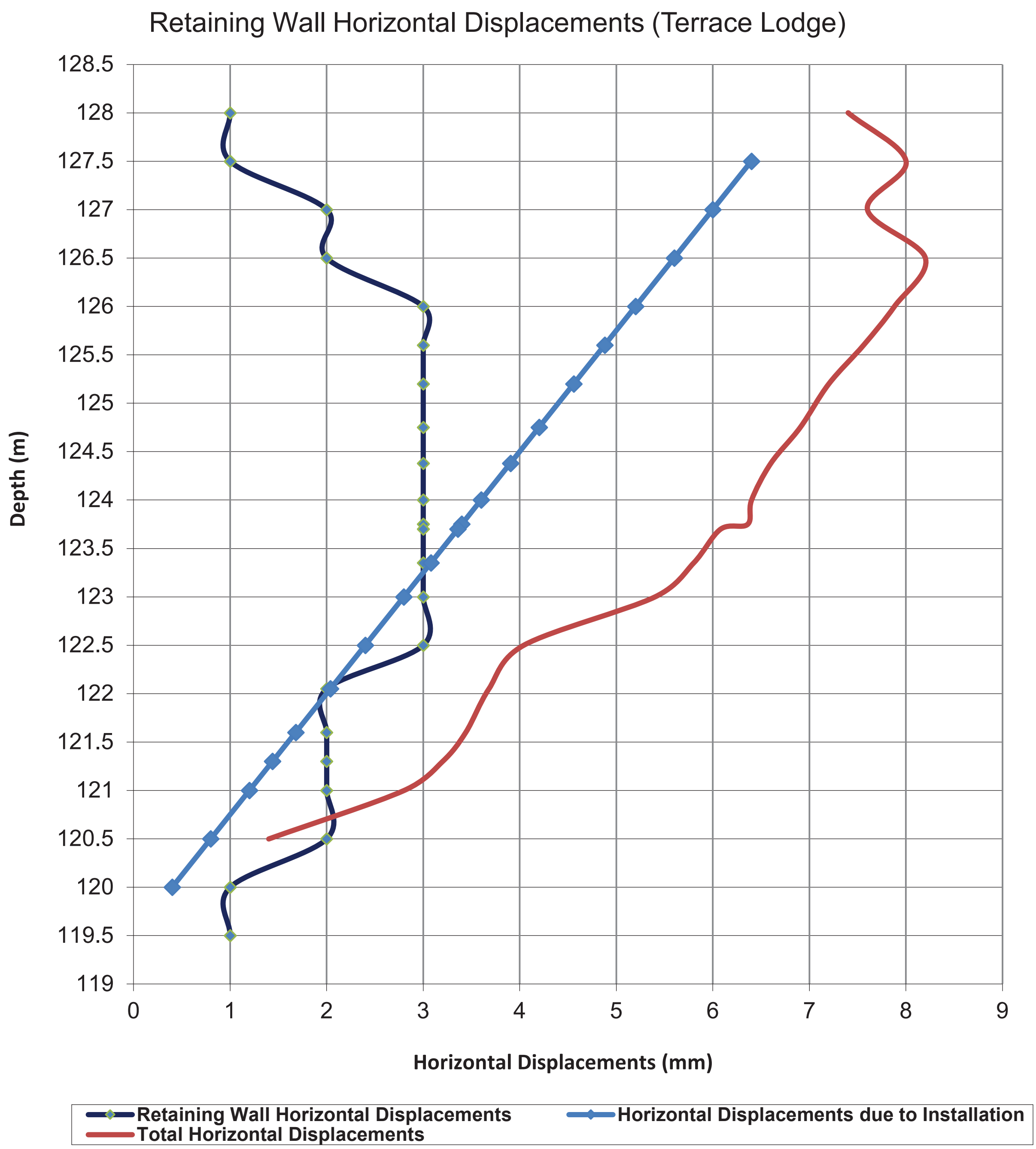
Client	Project	Job No
Mr Caspar Berendsen	**Admiral's Walk, London**	**CG/09013**
CGL	Title **Drained Heave Contours (Foundation Level – 125mAOD)**	**Figure 7b**



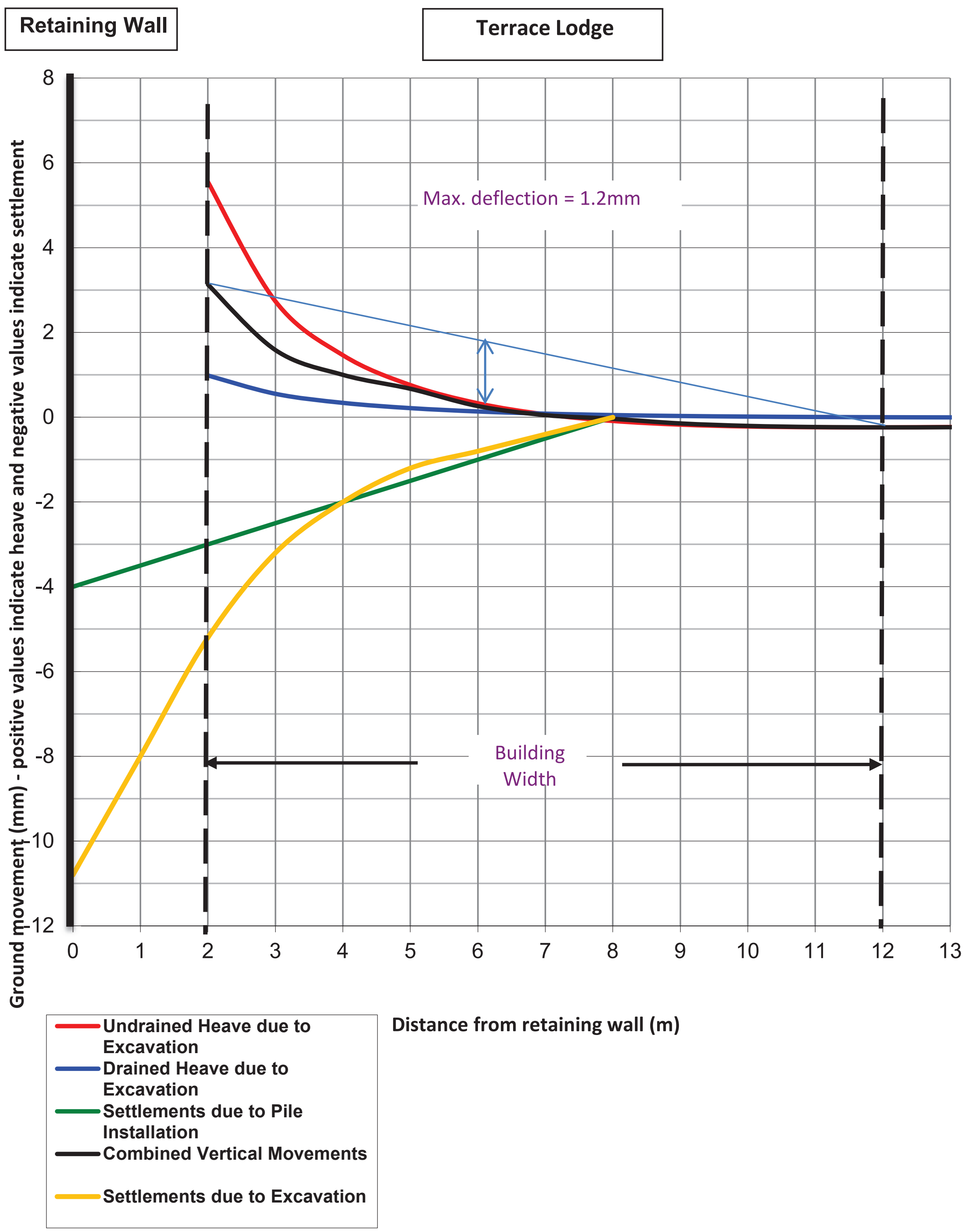
Client	Project	Job No
Mr Caspar Berendsen	**Admiral's Walk, London**	**CG/09013**
CGL	Title **Horizontal Ground Movements**	**Figure 8**



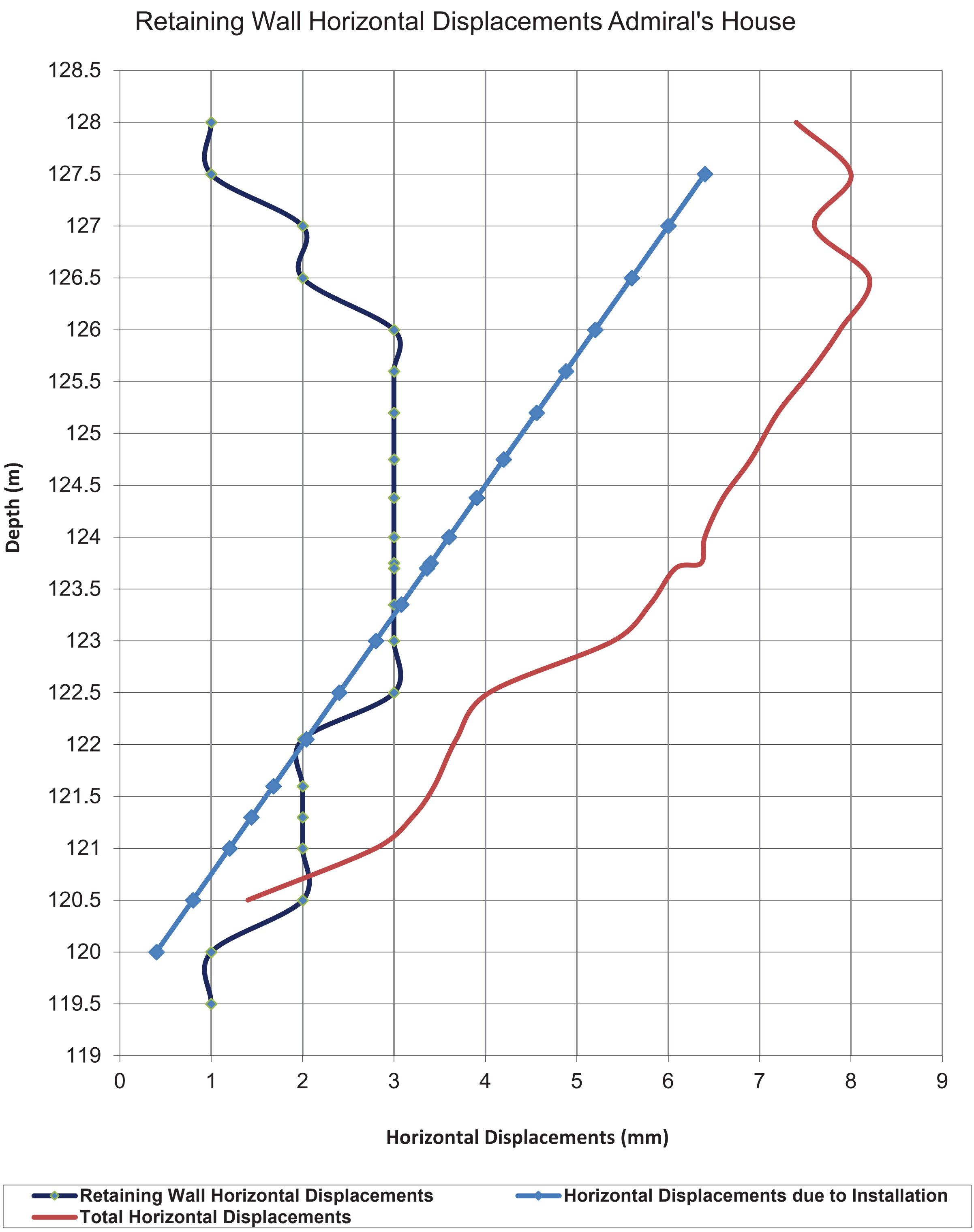
Client	Project	Job No
Mr Caspar Berendsen	**Admiral's Walk, London**	**CG/09013**
CGL	Title: **Horizontal Ground Movements**	**Figure 9**



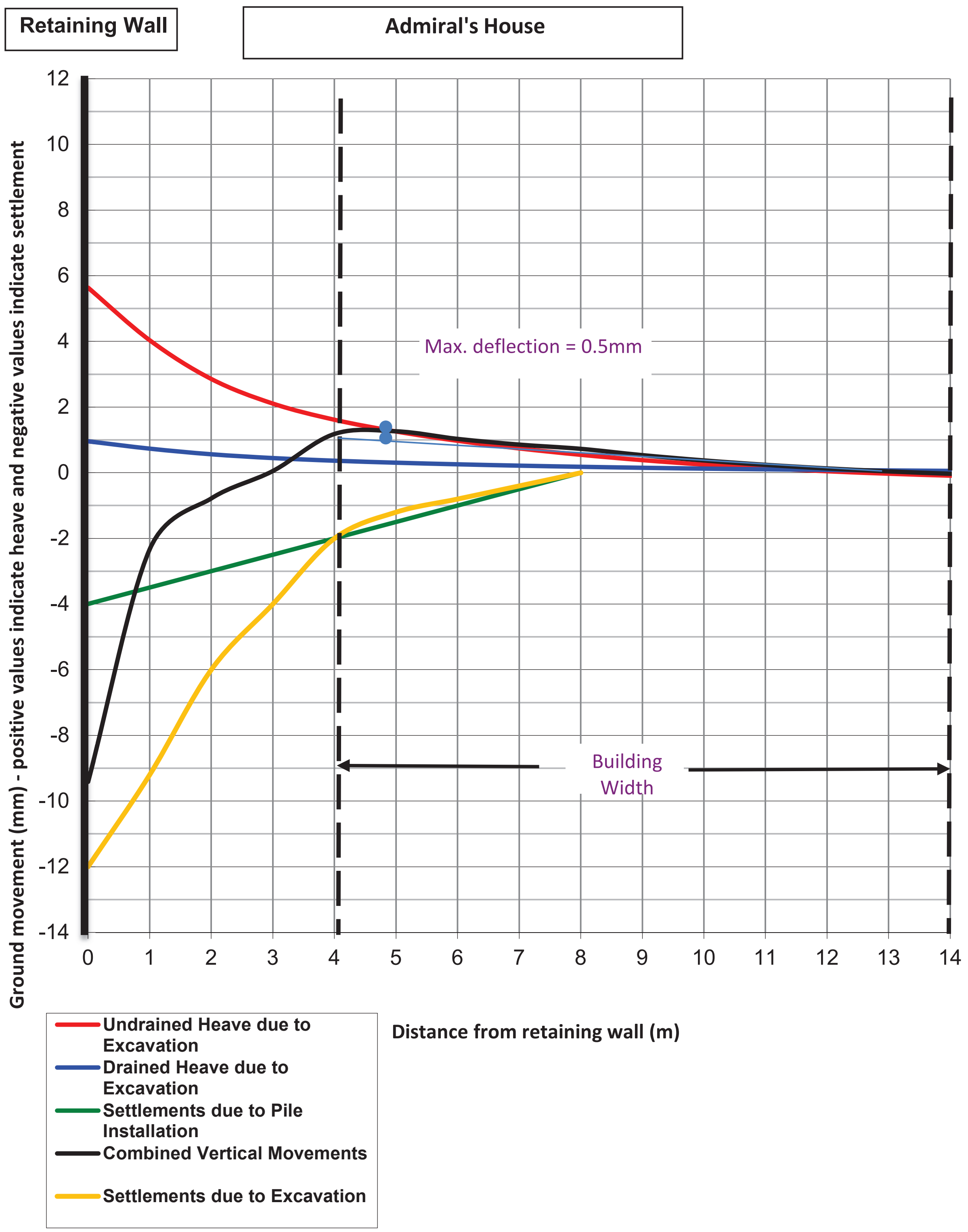
Client	Project	Job No
Mr Caspar Berendsen	**Admiral's Walk, London**	**CG/09013**
CGL	Title **Horizontal Ground Movements**	**Figure 10**



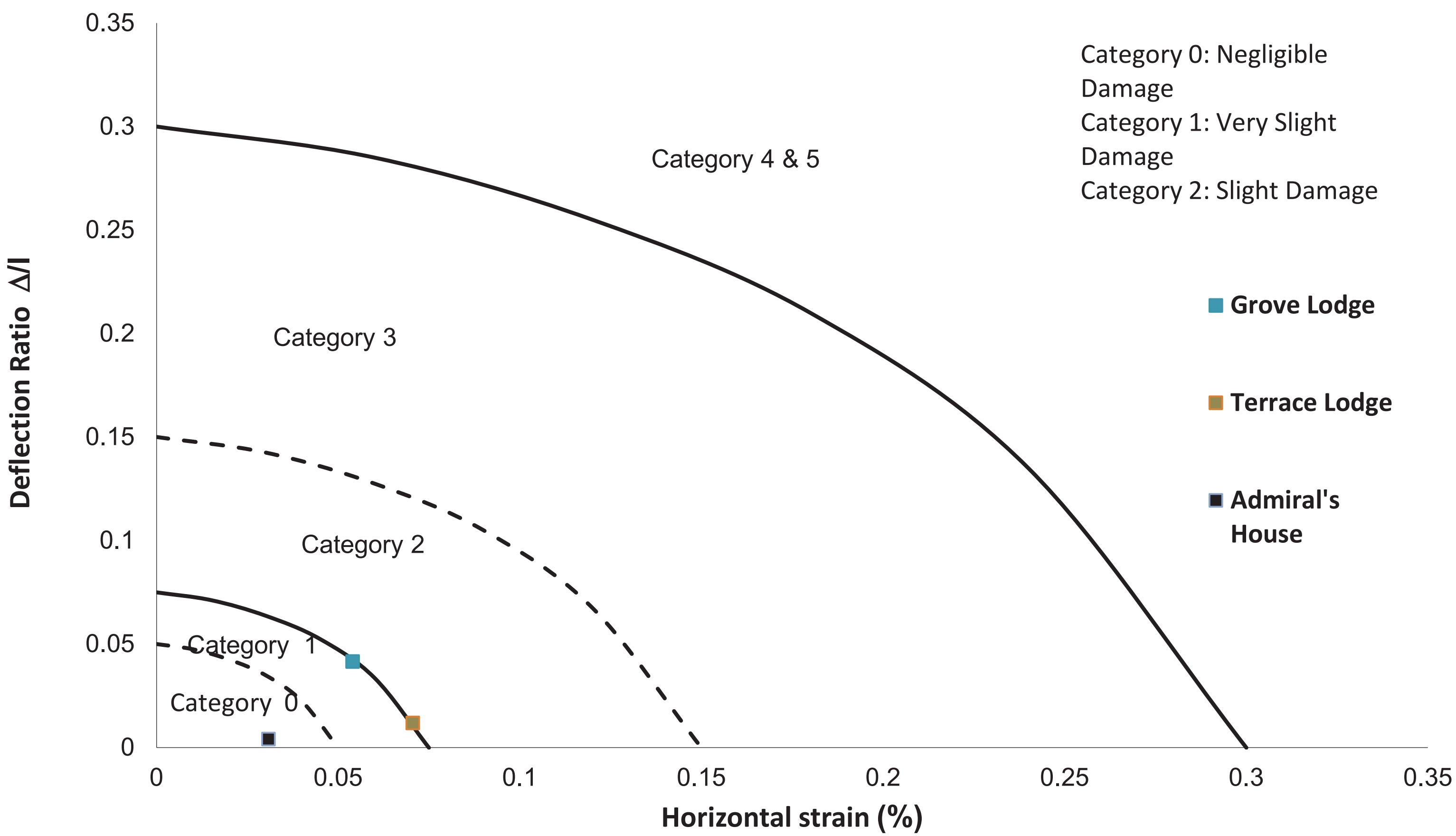
Client	Project	Job No
Mr Caspar Berendsen	**Admiral's Walk, London**	**CG/09013**
CGL	Title **Horizontal Ground Movements**	**Figure 11**



Client	Project	Job No
Mr Caspar Berendsen	**Admiral's Walk, London**	**CG/09013**
CGL	Title **Horizontal Ground Movements**	**Figure 12**



Client	Project	Job No
Mr Caspar Berendsen	**Admiral's Walk, London**	**CG/09013**
CGL	Title: **Horizontal Ground Movements**	**Figure 13**



Client	Project	Job No
Mr Caspar Berendsen	Admiral’s Walk, London	CG/09013
CGL	Title: Building Damage Assessment	Figure 14