Chelmer
Consultancy Services
'Groundbreaking Services'

Ground Movement Assessment & Damage Category Assessment

Site	59 Solent Road London NW6 1TY
Client	Mahesh Varia
Date	March 2017
Our Ref	GMA/8461 Rev.A

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Foreword

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This assessment has involved consideration, using normal professional skill and care, of the findings of ground investigation data obtained from the Client and other sources. Ground investigations involve sampling a very small proportion of the ground of interest as a result of which it is inevitable that variations in ground conditions, including groundwater, will remain unrecorded around and between the exploratory hole locations; groundwater levels/pressures will also vary seasonally and with other man-induced influences; no liability can be accepted for any adverse consequences of such variations.

This report must be read in its entirety in order to obtain a full understanding of our recommendations and conclusions.

CONTENTS

1.0 INTRODUCTION

1.1 The excavation and construction of a single storey basement extension to extend beneath the full length of the existing building footprint and into the rear garden of 59 Solent Road, London NW6 1TY has been proposed. The site is located in the London Borough of Camden. This report is for planning and scheme development purposes and is not a design document.

1.2 This ground movement assessment (GMA) has been prepared by Chelmer Site Investigations Laboratories Limited (Chelmer) acting on behalf of Mahesh Varia.

1.3 A GMA, including damage category assessment (DCA), has been prepared using results from a site specific ground investigation undertaken by Chelmer (2016). This report presents the analyses undertaken and a damage category assessment.

1.4 The site is located at 59 Solent Road, London NW6 1TY (No. 59), approximate Ordnance Survey grid reference (OSNGR) 525100E, 185130N. The site location plan is displayed in Figure 1 below. The site currently comprises a terraced, three storey property. The property is adjoined by No. 61 Solent Road (No. 61) to the north and No. 57 Solent Road (No. 57) to the south. The British Geological Survey (BGS) GeoIndex indicates the site is on a slight north to south slope. The ground level at the northern end of Solent road is at approximately 60 mOD and drops to 51 mOD at the southern end; creating a slope of around 2°.

Figure 1. Site location Plan (Contains British Geological Survey materials © NERC 2016. Base mapping is provided by ESRI)

1.5 At this stage it is understood that a basement will be constructed beneath the full footprint of the existing property and beneath the planned extension alongside the existing kitchen at the rear of the property. A lightwell is proposed at the front of the property beanth the existing windows on the front wall. The basement perimeter walls will comprise reinforced concrete (RC) underpinning beneath the existing and proposed ground floor.

1.6 The assessment does not however, include a full basement impact assessment (BIA) and the study does not address the context of the proposed basement construction, any implications that the basement may have on groundwater and surface water regimes within the site and its environs and conversely how the wider site conditions may in turn impact the basement.

1.7 The following drawings and documents have been referred to in preparing this report. Drawings which were irrelevant to the basement have been ignored.

Hardman Structural Engineers

Drawing 2298-02 Oct 2016 (Existing Site Plan)

Drawing 2298-03 Oct 2016 (Existing Ground Floor Plan)

Drawing 2298-11 Oct 2016 (Proposed Basement Floor Plan)

Drawing 2298-12 Oct 2016 (Proposed Ground Floor Plan)

Drawing 2298-31 Oct 2016 (Proposed Section A-A)

Drawing 2298-32 Oct 2016 (Proposed Section B-B)

Drawing 2298-33 Oct 2016 (Proposed Section C-C)

Drawing 2298-101 Oct 2016 (Detail 01 Existing and Proposed)

Drawing 2298-102 Oct 2016 (Detail 02 Existing and Proposed)

2.0 GROUND MOVEMENT ASSESSMENT

2.1 *Basement Geometry and Stresses*

2.1.1 Analyses of vertical ground movements (heave or settlement) arising from changes in vertical stresses caused by excavation of the basement have been undertaken using proprietary software (Oasys PDISP™). The analysis is based on Boussinesq's theory of analysis for calculating stresses and strains in soils due to vertically applied loads; the predicted ground movements are derived by integration of vertical strains derived from Boussinesq's equations. These preliminary analyses have not modelled the horizontal forces on the retaining walls, and so have simplified the stress regime significantly. In addition, consistent with Boussinesq theory, the soils are assumed to comprise semi-infinite isotropically homogeneous elastic medium.

2.1.2 The layout of the basement used within the analysis is based on Drawing 2298-11 provided by Hardman Structural Engineers, and is presented in Figure 3 below. The proposed basement excavation and extension covers an area approximately 19.0 m long by 6.0 m wide with excavation generally extending to a depth of approximately 3.2 m below existing ground floor level (bgl) (as scaled from Drawings 2298-33). The basement is understood to be constructed by RC underpins and retaining walls as detailed in Section 1.5.

2.1.3 The excavation depths for the basement have been modelled using information provided by Hardman Structural Engineers to estimate the gross pressure reductions (unloading) across the development. Figure 3 below illustrates the layout of all load zones, positive and negative (unloading), used to model the proposed basement in PDISP. These include the excavation and loads on the underpins, retaining walls, excavation of central area from existing ground level and construction of the concrete slab.

2.1.4 The table in Appendix A presents the net changes in vertical pressure for each load zone for the four major stages in the sequence of stress changes which will result from excavation and construction of the basement (see 2.3.1 below for details).

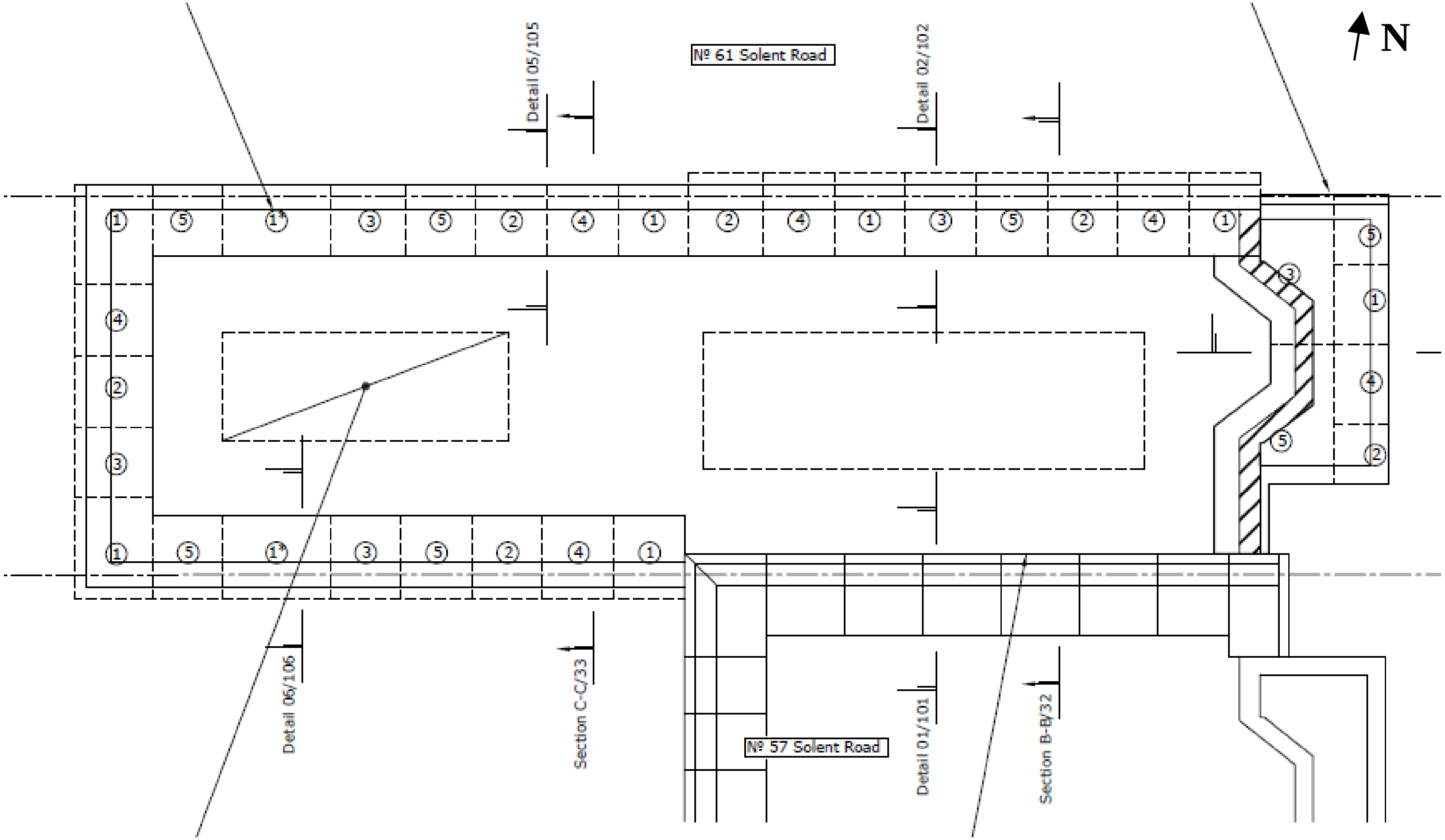

Figure 2. Layout of the proposed basement plan (Extract from Drawing 2298-11)

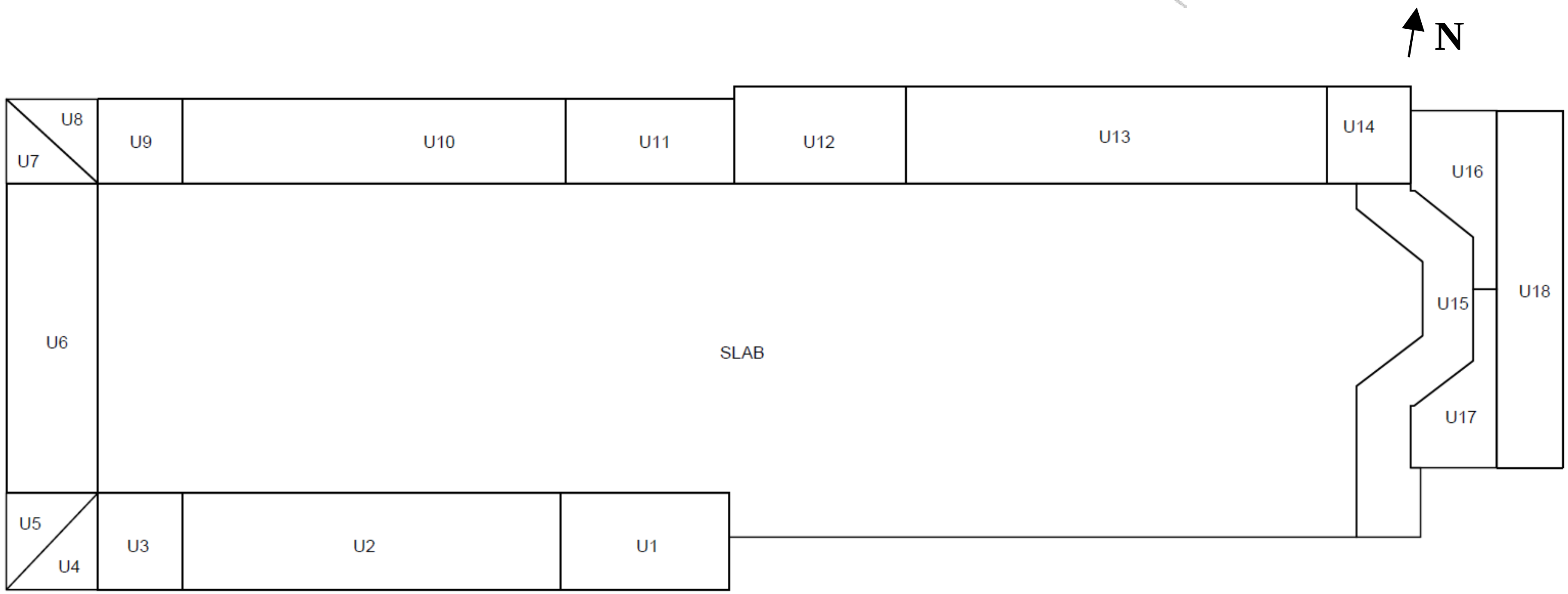

Figure 3. Detail of geometry introduced to PDISP
[U = Underpinning/retaining wall excavation and loads, Slab = Bulk excavation and slab loads]

2.2 *Ground Conditions*

2.2.1 The ground profile was based on the Chelmer (2016) ground investigation, which comprised two C.F.A boreholes (BH1 and BH2) and two hand excavated trial pits (TP1 and TP2). The boreholes were undertaken on the front and rear gardens and advanced to 8.1 m bgl. The boreholes encountered 0.7 m of Made Ground overlying the London Clay Formation. The London Clay Formation was recorded as weathered, firm (becoming stiff at 2 m depth and very stiff from 6 m bgl) silty CLAY with occasional partings of silt and fine sand to the maximum drilling depth of 8.1 m bgl.

2.2.2 The geology is consistent with public domain geological information on the site from the British Geological Survey (BGS) Geology of Britain Viewer which indicates that the underlying geology at this site is the London Clay Formation with no overlying superficial deposits recorded.

2.2.3 No groundwater was noted during the intrusive site investigation within the boreholes to 8.1 m bgl during drilling. Groundwater monitoring visits were undertaken on 8th and 14th February 2017 and groundwater levels were recorded at depths of 0.65 m and 0.68 m bgl in BH1, and 1.83 m and 1.91 m bgl in BH2. No further groundwater monitoring visits are currently planned. Therefore, the basement should be designed to accommodate an uplift pressure of 32 kPa, which is likely to represent the worst credible scenario, in addition to the swelling displacements/pressures from the excavation and construction of the basement discussed below.

2.2.4 The short-term and long-term geotechnical properties used in the analysis are summarised in Table 1 below. These were based on both the ground investigation, and on data from previous Chelmer projects in similar ground conditions. All Made Ground will be excavated and therefore only the change in vertical pressure, due to its excavation, is required for the PDISP analyses. Geotechnical parameters for the Made Ground are not used in the analysis.

Table 1 - Soil parameters for PDISP analyses			
Strata	***Depth*** *(m bgl)*	***Short-term, undrained Young's Modulus, E_u*** *(MPa)*	***Long-term, drained Young's Modulus, E'*** *(MPa)*
London Clay Formation	3.2 6.5 10.0	42.0 62.5 77.5	25.2 37.5 46.5
Undrained Young's Modulus, $E_u = 500 * C_u$ Drained Young's Modulus, $E' = 0.6 * E_u$ Where no C_u data are available: Undrained Shear Strength, C_u assumed as $C_u = 80 + 7.5z$ kPa where z = depth below the highest founding level (m). A global Poissons ratio of 0.5 has been adopted for the London Clay Formation and Claygate Member strata, over their modelled thicknesses.			

2.3 *PDISP Analyses:*

2.3.1 Three dimensional analyses of vertical displacements have been undertaken using PDISP software and the basement geometry, loads/stresses and ground conditions outlined above in order to assess the potential magnitudes of ground movements (heave or settlement) which may result from the vertical stress changes caused by excavation of the basement. PDISP analyses have been carried out as follows:

- Stage 1 – Construction of underpins and retaining walls – Short-term (undrained) condition
- Stage 2 – Bulk excavation of central area to basement formation level – Short-term (undrained) conditions
- Stage 3 – Construction of the basement slab – Short-term (undrained) conditions
- Stage 4 – Construction of the basement slab – Long-term (drained) conditions

2.3.2 The results of the analyses for Stages 1, 2, 3 and 4 are presented as contour plots on Figures 4, 5, 6, and 7 respectively.

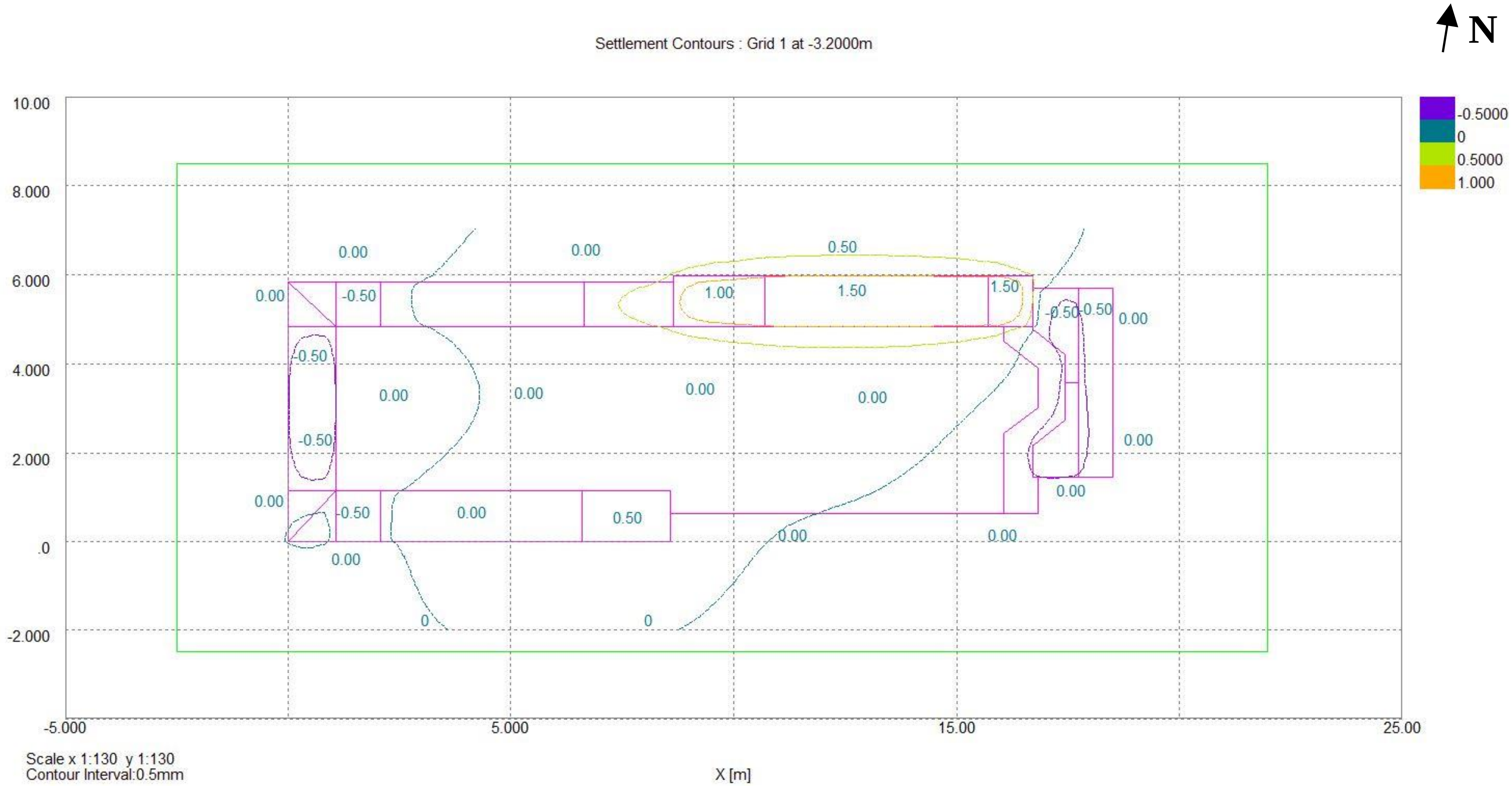

Figure 4. Stage 1 – Construction of underpins and retaining walls – Short-term (undrained) condition (0.5mm settlement contours)

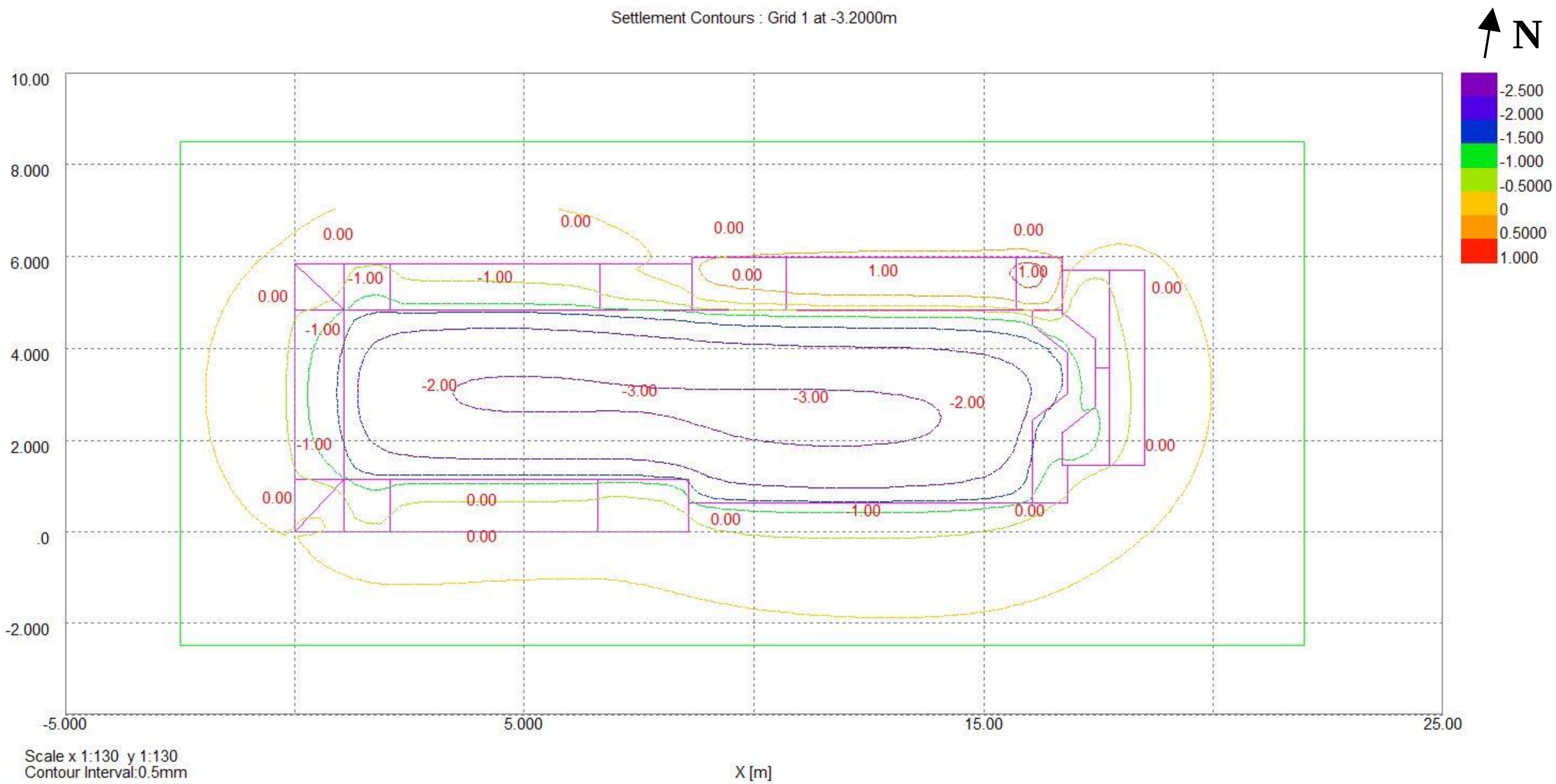

Figure 5. Stage 2 – Bulk excavation of central area to basement formation – Short-term (undrained) condition (0.5 mm settlement contours)

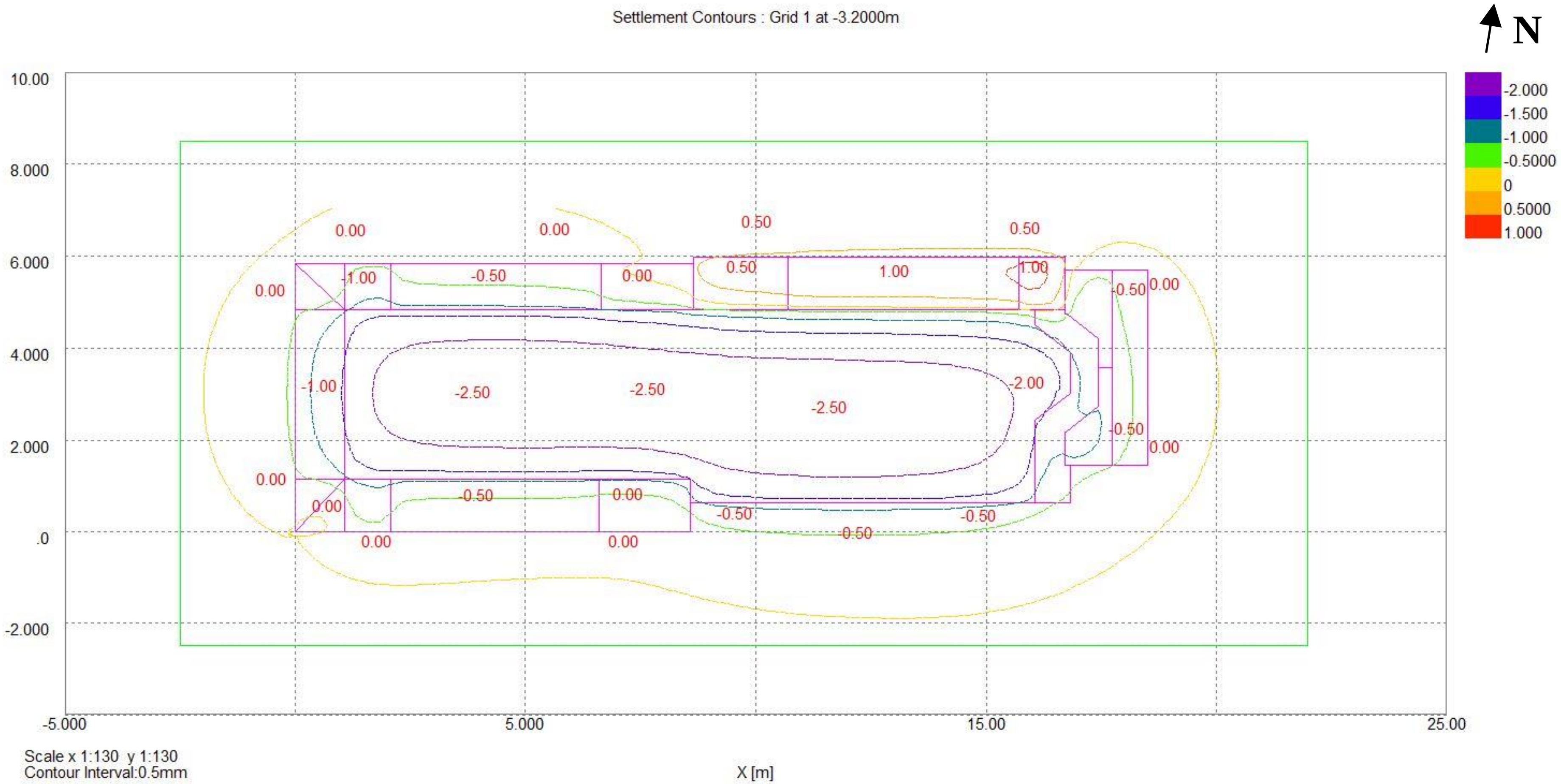

Figure 6. Stage 3 – Construction of the basement slab – Short-term (undrained) conditions (0.5mm settlement contours)

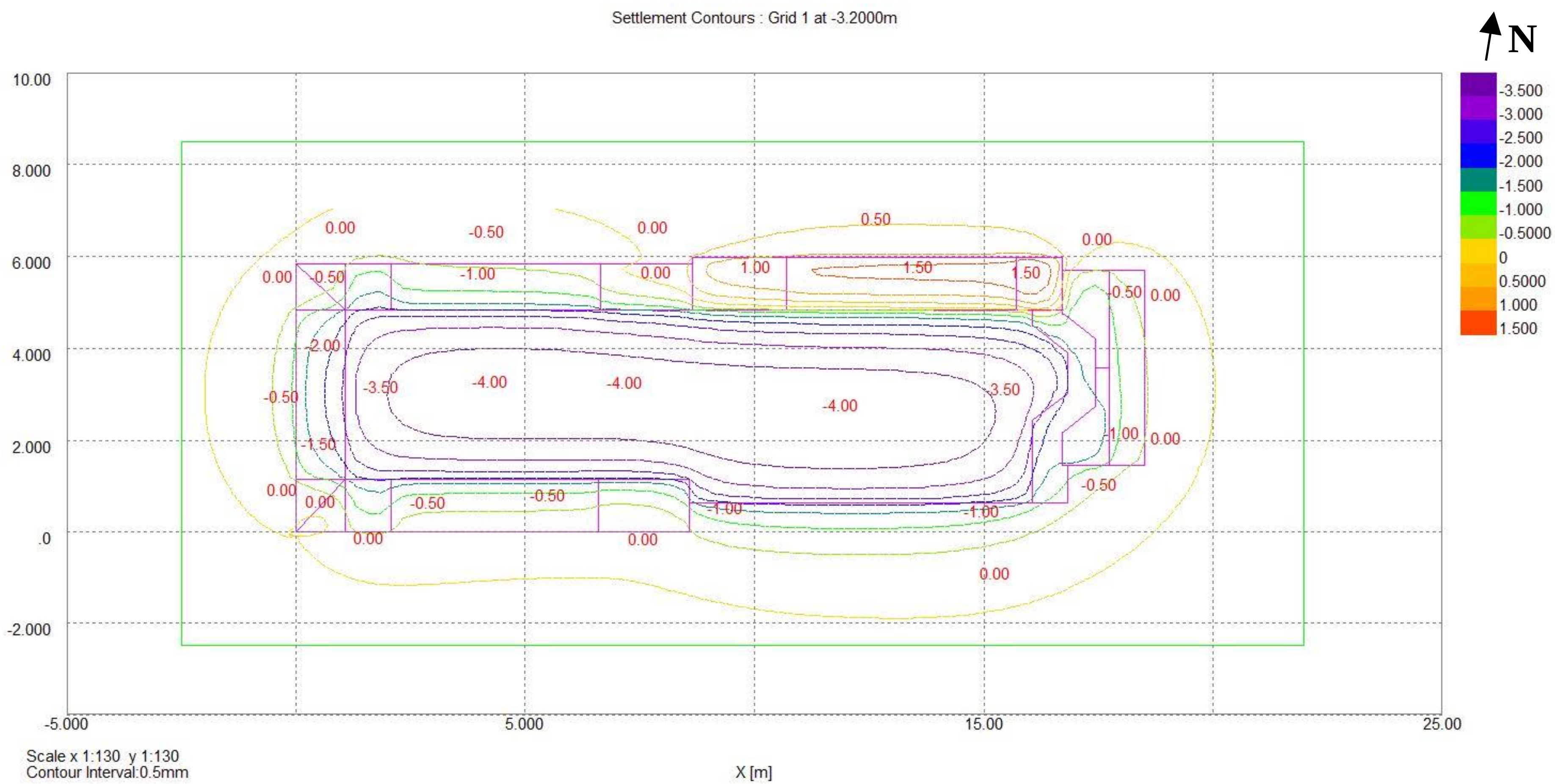

Figure 7. Stage 4 – Construction of the basement slab – Long-term (drained) conditions (0.5 mm settlement contours)

2.4 *Heave/Settlement Analysis*

2.4.1 Excavation of the basement and construction of the underpins will cause immediate elastic heave/settlements in response to the stress changes, followed by long term plastic swelling/settlement as the underlying clays take up groundwater or consolidation occurs. The rate of plastic swelling/consolidation will be determined largely by the availability of water and as a result, given the low permeability of the London Clay Formation, can take many years to reach full equilibrium. The basement slab will need to be designed to enable it to accommodate the swelling displacements/pressures developed underneath it.

2.4.2 The ranges of predicted short-term and long-term movements for each of the main sections of the proposed basement are presented in Table 2 below. These analyses indicate that the perimeter walls are predicted to undergo movements ranging from 1.5 mm settlement to 2.0 mm heave. The basement slab is predicted to undergo displacements, between 0.0 and 4.0 mm heave. All values are approximate owing to the simplification of the stress regime and include only displacements caused by stress changes in the ground beneath the basement.

Table 2: Summary of Predicted Ground Movements from PDISP

Location / Building Element	*Stage 1 (short term)*	*Stage 2 (short term)*	*Stage 3 (short term)*	*Stage 4 (long term)*
Underpins and retaining wall along north boundary	0.5 mm Heave to 1.5 mm Settlement	1.0 mm Heave to 1.0 mm Settlement	1.0 mm Heave to 1.0 mm Settlement	1.0 mm Heave to 1.5 mm Settlement
Underpins and retaining wall along south boundary	0.5 mm Heave to 0.5 mm Settlement	0.0 – 1.0 mm Heave	0.0 – 0.5 mm Heave	0.0 – 1.0 mm Heave
Underpins and retaining wall along east boundary.	0.0 – 0.5 mm Settlement	0.0 mm	0.0 – 0.5 mm Heave	0.50 – 1.0 mm Heave
Underpins and retaining wall along west boundary	0.5 mm Heave to 0.5 mm Settlement	0.0 – 1.0 mm Heave	0.0 – 0.5 mm Heave	0.0 – 2.0 mm Heave
Basement slab	0.0 mm	1.0 – 3.0 mm Heave	1.0 – 2.5 mm Heave	0.0 – 4.0 mm Heave

2.4.3 All the short-term elastic displacements would have occurred before the basement slab is cast, so only the post-construction incremental heave/settlements (the difference from Stages 3, short-term, to 4, long-term) are relevant to the slab design.

3.0 DAMAGE CATEGORY ASSESSMENT

3.1 When underpinning it is inevitable that the ground will be un-supported or only partially supported for a short period during excavation of each pin, even when support is installed sequentially as the excavation progresses. This means that the behaviour of the ground will depend on the quality of workmanship and suitability of the methods used, so rigorous calculations of predicted ground movements are not practical. However, provided that the temporary support follows best practice, then extensive past experience has shown that the bulk movements of the ground alongside underpins for a single storey basement should not exceed 5 mm horizontally. This figure should be adjusted pro-rata for shallower or deeper basements.

3.2 In order to relate these predicted ground movements to possible damage which adjacent properties might suffer, it is necessary to consider the strains and the angular distortion (as a deflection ratio) which they might generate using the method proposed by Burland (2001, in CIRIA Special Publication 200, which developed earlier work by himself and others).

3.3 The London Borough of Camden's planning website displays a planning application (2010/5437/P) for a basement at No. 57. Drawing 2298-02 by Hardman Structural Engineers indicates this basement extends 8 m from the front wall towards the rear with a light well extending approximately 1.7 m from the front wall into the front garden. No evidence has been found that any other properties in close proximity to No. 59 along the row of terraced houses have a basement beneath them.

3.4 The uniform founding level beneath the proposed basement means that the potentially critical locations will be determined by the displacements predicted by the PDISP analyses and the geometries of the adjoining buildings. For these damage category assessments we are interested in the ground movements at the foundation level of the neighbouring buildings, so it is the depth of the proposed excavation below foundation level of the neighbouring properties that must be considered.

3.5 The geometries and distances relevant to the damage category assessments are presented in Figure 8 below. The worst case scenario is considered to be the front wall of No. 61 due to the higher settlements predicted by PDISP in this location and the presence of a basement already beneath the majority of No. 57.

Figure 8. Approximate widths of affected walls of adjacent structures (Not To Scale)

3.6 The damage category assessments undertaken will consider the following:

- ground movements arising from the vertical stress changes, as assessed by the PDISP analyses;
- ground movements alongside the proposed underpins and retaining walls caused by relaxation of the ground in response to the excavations.

Some ground movement is inevitable when basements are constructed. Ground movements associated with the construction of retaining walls in clay soils have been shown to extend to a distance up to 4 times the depth of the excavation, as detailed in Table 2.4 of CIRIA C580 (Gaba et al,, 2003).

3.7 For worst case 'low support stiffness' walls (which is appropriate to the underpinning construction method) the estimated vertical ground movements resulting from the excavation in front of the proposed basement wall would be as defined in Table 2.4 of CIRIA C580. This predicts a settlement 0.35% of the maximum excavation depth. Therefore, for a 3.2 m excavation (the approximate excavation depth for each assessed case) the total settlements immediately alongside the proposed basement walls due to the excavation of the soil would be 11.2 mm.

Front wall of No. 61:

3.8 The relevant geometries are as follows:

Relative depth of foundations = 0.5 m (as identified by the site investigation (Chelmer, 2016))
Depth of excavation = 3.2 – 0.5 = 2.7 m
Width of zone of affected ground = 2.7 x 4 = 10.8 m

Width of No. 61's front wall = 5.8 m (as scaled from Drawing 2298-02) therefore the majority of No. 63's front wall will also be affected;
Affected width (L) = 10.8 m
Height of buildings affected (H) = 7.0 m (assumed, appears slightly higher than No. 59) + 0.5 m (footing depth) = 7.5 m
Hence L/H = 1.4

3.9 The predicted 5 mm maximum horizontal displacement (see Section 3.1) decreases pro-rata to 3.9 mm when the depth of excavation is taken into account. Thus, the horizontal strain beneath the front wall would, theoretically, be in the order of $\varepsilon_h = 3.61 \times 10^{-4}$ (0.036%).

3.10 The maximum settlement produced by the PDISP analysis beneath the location where the proposed development meets the front wall of the adjoining No. 61 was in Stage 4 where 1.7 mm settlement was predicted. This must be added to the settlement profile presented in Figure 2.11(b) of CIRIA Report C580 for a worst case (low stiffness ground support) scenario, which is appropriate to the underpinning construction method.

3.11 The total predicted settlement (due to excavation) of 11.2 mm (see Section 3.7) is reduced to 9.5 mm when the assumed depth to the adjacent buildings footings are taken into account. The total combined settlement of 11.2 mm, 9.5 mm predicted by the CIRIA methods plus the 1.7 mm predicted by PDISP, is detailed as the point immediately alongside the proposed basement (0 m) in Figure 9 below. Figure 9 presents the settlement curve from the basement wall to the maximum distance of affected ground, 10.8 m (see Section 3.8).

3.12 The deflection along the front walls of the No. 61 and 63 is calculated as the difference between the tangent of the relevant width of the affected wall (10.8 m) and the total predicted ground surface movements curve (from Figure 2.11(b) of CIRIA C580). For the low stiffness ground support case, settlement is convex and gives a maximum vertical deflection, $\Delta = 2.8$ mm as displayed in Figure 9 below, which represents a deflection ratio $\Delta/L = 2.59 \times 10^{-4}$ (0.026%).

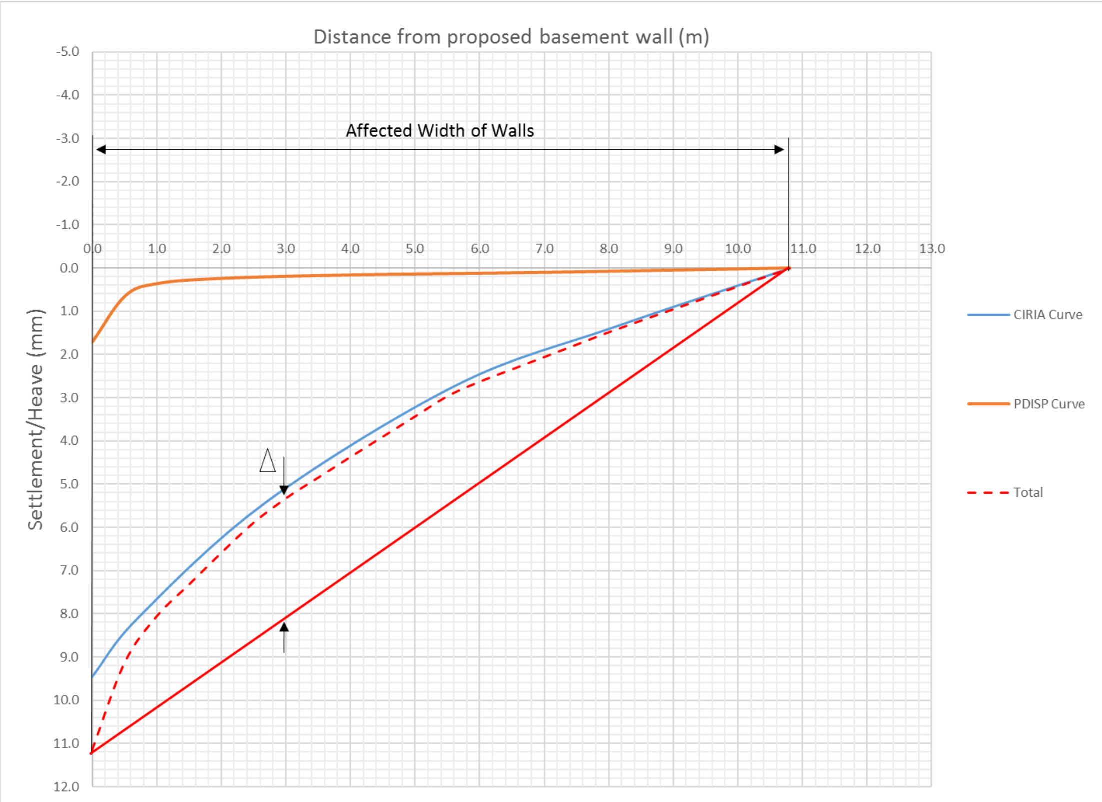

Figure 9. Combined displacements for the front walls of No. 61 and 63 due to excavation of proposed basement

3.13 Using the damage category ratings and graphs given in CIRIA SP200, for L/H = 1.5 (conservative for the 1.4 defined in Section 3.8), these deformations represent a damage category of 'very slight' (Burland Category 1), as illustrated in Figure 10 below.

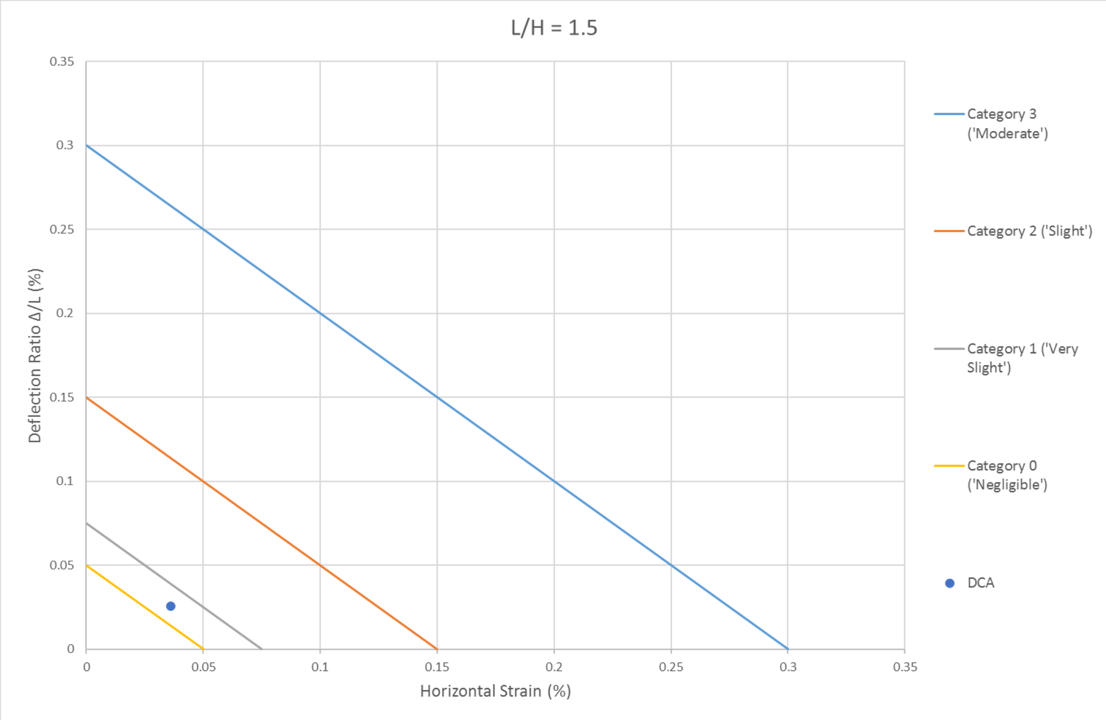

Figure 10: DCA for the front walls of No. 61 and 63

3.14 Use of best practice construction methods will be essential to ensure that the ground movements are kept in line with the above predictions.

3.15 No long term groundwater monitoring data is available and therefore there is a risk that levels higher than the proposed basement foundation level may be experienced. Care should be taken to ensure that any seepages from the exposed clay are collected and removed efficiently, and that water is not allowed to pond on the exposed clay in the excavation at the toe of the underpins.

4.0 SUMMARY

4.1 This summary considers only the primary findings of this assessment; the whole report should be read to obtain a full understanding of the matters considered.

4.2 Contour plots of displacement in response to the changes in vertical pressure caused by the excavation and construction of the proposed basement are presented in Figures 4 – 7.

4.3 A damage category assessment was undertaken for the worst case scenario in the adjoining properties, based on the maximum displacements predicted by the PDISP analyses, combined with the ground movements alongside the basement in response to the lateral stress release, as predicted by CIRIA C580, Figure 2.11.

4.4 In the assessed case, the front walls of the adjoining No.'s 61 and 63 Solent Road fell within Burland Category 1 'very slight' (as given in CIRIA SP200, Table 3.1). The damage category result has been plotted graphically in Figure 10.

4.5 No further damage category assessments have been carried out as other structures in the vicinity are further away and/or in areas with less predicted ground movements and therefore considered lower risk. Therefore, all other walls are considered to be classified as Category 1 'very slight' or Category 0 'negligible'.

4.7 Use of best practice construction methods will be essential to ensure that the ground movements are kept in line with the above predictions. Pre-construction condition surveys of neighbouring properties are also recommended and a system of monitoring adjoining and adjacent structures should be established before the works start.

References

Burland J.B., et al (2001). Building response to tunnelling. Case studies from the Jubilee line Extension, London. CIRIA Special Publication 200.

Gaba A.R., et al (2003). Embedded retaining walls – guidance for economic design. CIRIA Report C580.

Chelmer Site Investigation Laboratories Limited (2016). Factual Report, 59 Solent Road, London NW6 1TY. Report FACT/7543.

End of report

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APPENDIX A

Table 1: Coordinates and net bearing pressure for PDISP			
ZONE	Net change in vertical pressure (kPa)		
#	Stage 1	Stage 2	Stages 3 and 4
U1	15.47	15.47	15.47
U2	8.20	8.20	8.20
U3	-21.28	-21.28	-21.28
U4	18.24	18.24	18.24
U5	5.44	5.44	5.44
U6	-27.68	-27.68	-27.68
U7	5.44	5.44	5.44
U8	11.40	11.40	11.40
U9	-24.70	-24.70	-24.70
U10	4.47	4.47	4.47
U11	20.35	20.35	20.35
U12	49.24	49.24	49.24
U13	54.57	54.57	54.57
U14	76.42	76.42	76.42
U15	-9.94	-9.94	-9.94
U16	-41.52	-41.52	-41.52
U17	-40.26	-40.26	-40.26
U18	-10.51	-10.51	-10.51
SLAB	0.00	-60.80	-55.35

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