Ground Movement Analysis

Site	55 Greencroft Gardens
	South Hampstead
	NW6 3LL
Client	Spencer Garcia
Date	September 2015
Our Ref	GMA/5352

Chelmer Site Investigation Laboratories Ltd
Unit 15 East Hanningfield Industrial Estate, Old Church Road, East Hanningfield, Essex CM3 8AB
Essex: 01245 400930 | London: 0203 67409136 | info@siteinvestigations.co.uk | www.siteinvestigations.com

Foreword

This report has been prepared in accordance with the scope and terms agreed with the Client, and the resources available, using all reasonable professional skill and care. The report is for the exclusive use of the Client and shall not be relied upon by any third party without explicit written agreement from Chelmer Site Investigations Laboratories Ltd.

This report is specific to the proposed site use or development, as appropriate, and as described in the report Chelmer Site Investigations Laboratories Ltd. accept no liability for any use of the report or its contents for any purpose other than the development or proposed site use described herein.

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 & TERMS OF REFERENCE

1.1 Chelmer Site Investigations Ltd (CSI) have been instructed to undertake a basement impact assessment for the construction of a single-storey basement beneath Flat 2, No.55 Greencroft Gardens, NW6 3LL. The assessment is in accordance with the requirements of the London Borough of Camden (LBC) Development Policy DP27 in relation to basement construction, and follows the requirements set out in LBC's guidance document CPG4 'Basements and Lightwells' (September 2013).

1.2 As an integral part of the BIA, a ground movement assessment (GMA) has been undertaken and this addendum report presents the findings of the GMA. In this regard the GMA should be considered and read in conjunction with the BIA reference CSI report BIA/5352 dated July 2015.

1.3 In preparation of the GMA, the following site-specific documents which describe the proposed new basement and have been presented in support of the planning application, have been considered.

SIMON GOLDSTEIN ARCHITECTURE (SGA):

Drg No. 15001/JA12_P_AL_001 rev.A	Existing Layouts
Drg No. 15001/JA12_E_S_001 rev.A	Existing Rear Elevation
Drg No. 15001/G200_P_AL_001 rev.A	Proposed Layouts (whole building)
Drg No. 15001/G200_P_AL_002 rev.B	Proposed Layouts (Flat 1 only)
Drg No. 15001/G200_E_S_001 rev.A	Proposed Rear Elevation
Drg No. 15001/G200_S_AA_001 rev.A	Proposed Section AA
Drg No. 15001/G200_S_BB_001 rev.A	Proposed Section BB

S.R.Brunswick (Structural Engineer):

Calculation sheets, including preliminary retaining wall analysis.
Load takedown annotated on plan.

2.0 GROUND MOVEMENT ANALYSIS

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 PDISP software. These preliminary analyses have not modelled the horizontal forces on the retaining walls, so have simplified the stress regime significantly.

2.1.2 The layout of the basement used within the analysis is based on SGA Drawings (specifically Drg No. 15001/G200_P_AL_001 rev.A and Drg No. 15001/G200_P_AL_002 rev.B) and is presented in Figure 1. The alignment of the underpinning system is presented in Figure 2 which is based on SGA scheme drawings as supplemented with information provided by the structural engineer, S R Brunswick. The maximum overall dimensions of the proposed basement are 5.6m wide by 17.2m long.

2.1.3 The net change in vertical stresses due to excavation and construction of the underpinning system will extend to a depth equal to twice the width of the affected area (below which the stress change is generally considered to be insignificant). The depths of excavation modelled are based on SGA scheme drawings (see above). This has been modelled within the PDISP software as a number of zones relating to the existing ground levels as well as the proposed basement geometry and the zonation developed and used within the analysis is shown in Figure 2.

2.1.4 The analysis carried out considers four different stages of construction, with the ground movements predicted for each stage, as follows:

- Stage 1 – Construction of underpins/retaining walls in short-term condition.
- Stage 2 – Bulk excavation of central area to formation level in short-term condition.
- Stage 3 – Construction of basement slab in short-term (undrained) condition.
- Stage 4 – Construction of basement slab in long-term (drained) condition.

2.1.5 The calculated net changes in vertical pressure for the four major stages in the stress history of the basement's construction are presented in Table 1 below for the zones used as shown in Figure 2.

Table 1 – Zones, Coordinates and Net Bearing Pressure for PDISP Analysis

ZONE	Centroid		Dimensions		Net change in vertical pressure (kPa)		
	Xc(m)	Yc(m)	X(m)	Y(m)	Stage 1	Stage 2	Stages 3 and 4
1	2.115	0.425	4.230	0.850	-24.35	-24.35	-21.85
2	7.260	0.675	6.060	1.350	133.52	133.52	136.02
3	15.065	0.600	9.550	1.200	152.04	152.04	154.54
4	19.065	2.460	1.550	2.520	45.12	45.12	47.62
5	13.100	6.700	5.620	1.200	118.79	118.79	121.29
6	2.315	6.700	4.630	1.200	-31.21	-31.21	-28.71
7	0.425	3.475	0.850	5.250	-24.35	-24.35	-21.85
8	4.605	3.725	0.750	4.750	-36.03	-36.03	-33.53
9	9.915	3.725	0.750	4.750	-9.37	-9.37	-6.87
10	2.540	3.475	3.380	5.250	0.00	-47.88	-40.38
11	7.260	3.725	4.560	4.750	0.00	-62.70	-55.20
12	13.100	3.650	5.620	4.900	0.00	-47.88	-40.38
13	17.100	4.910	2.380	2.380	0.00	-26.60	-19.10
14	17.100	2.460	2.380	2.520	0.00	-32.30	-24.80
15	19.065	4.910	1.550	2.380	50.82	50.82	53.32
16	17.875	6.700	3.930	1.200	140.07	140.07	142.57
17	7.460	6.700	5.660	1.200	103.97	103.97	106.47

2.2 Soil Parameters for Analysis

2.2.1 The ground profile used within the analysis of ground movements is based on the available site investigation information (see BIA report reference BIA/5352 dated July 2015 and CSI site investigation factual report reference FACT/5352 dated May 2015).

2.2.2 The soil parameters used for the PDISP analyses are presented in Table 2 below. These parameters are based on the available site specific soils data and supplementary data based on our experience of similar ground conditions on other projects.

2.2.3 Excavation for the proposed basement to depths varying between 1.4m and 3.3mbgl (see BIA paragraphs 3.2 & 3.3), reducing to 0.77m where only minimal underpinning is required below the Flat 1/Flat 2 party wall. These excavations are likely to be within Made Ground over London Clay which is inferred to extend to depth below the site, with rigid boundary for elastic analysis assumed to be at a depth of -17.1mbgl.

Table 2 - Soil parameters for PDISP analyses			
Strata	***Depth*** *(m bgl)*	***Short-term, undrained Young's Modulus,*** *Eu (MPa)*	***Long-term, drained Young's Modulus,*** *E' (MPa)*
London Clay	2.50 17.1	45 100	27 60
*For London Clay Undrained Young's Modulus, Eu = 45 + 3.75z MPa from formation founding level (2.5mbgl in the original part of No.55) and Drained Young's Modulus, E' = 0.6 * Eu; in which z = depth below the founding level.* *A global Poissons ratio of 0.5 has been adopted for the London Clay over its modelled thickness.*			

2.3 Ground Movement Predictions – PDISP Analysis

2.3.1 The results of the three dimensional analysis as generated by the PDISP analysis are presented as contour plots and shown in Figure 3 to Figure 5 and present estimated ground displacements for Stage 2, Stage 3 and Stage 4 construction sequence respectively.

2.3.2 Excavation of the basement will cause immediate elastic heave in response to the stress reduction, followed by long-term plastic swelling as the underlying clays take up groundwater. The rate of plastic swelling in the clays will be determined largely by the availability of water and as a result, given the low permeability of the clays in the London Clay Formation, this can take decades to reach full equilibrium. The basement slab will need to be designed so as to enable it to accommodate the swelling displacements/pressures developed underneath it.

2.3.3 The PDISP analysis shows that the basement construction will generate relatively symmetrical ground movement giving rise to longitudinal 'tilting' with heave being experienced at the rear and settlement at the opposite internal front end of the basement. Movement of the party walls are similar for both sides of the proposed basement with settlement being slightly higher for the No.55/57party wall than the corresponding party wall for Flat 2/Flat 1 where underpins for Flat 1's basement are in place. In the short term predicted ground movements beneath the underpins and retaining walls are generally of the order of 2 to 4mm. In the longer term the ground movements are greater and range between 2 and 7mm.

2.3.4 The ranges of predicted short-term and long-term movements for each of the main walls as well as the central zone of the basement slab are presented in Table 3 below.

Table 3: Summary of predicted ground movements

Location	***Stage 2 (Figure 3)***	***Stage 4 (Figure 5)***
No 55 Flat 2 Internal front wall	2-4mm settlement	4-10mm settlement
No 55 Flat 2 Rear wall of original house	2mm settlement to 2mm heave	9mm settlement to 2mm heave
No 55 Flat 2 Rear wall of extension	1mm settlement to 3mm heave	5mm settlement to 4.5mm heave
No 55 Flat 2/Flat 1 Party wall	1-4mm settlement	1-10mm settlement
No. 55/57 Party wall	0-4mm settlement	0-9mm settlement
Basement slab (internal)	2mm settlement to 3mm heave	6mm settlement to 5mm heave
External terraces and en-suite, including basement slab:	1-3mm heave	1mm settlement to 5mm heave

2.3.5 All the short-term elastic displacements would have occurred as the excavations progress and before the new basement slab is cast, so only the post-construction incremental heave/settlements are relevant to the slab design. The analyses indicated that the maximum predicted post-construction displacements beneath the slab are likely to be about 3mm heave and 3mm settlement, giving 6mm in total.

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 calculations of predicted ground movements can never be rigorous. However, provided that the temporary support follows best practice as outlined in Section 10.4 of the BIA report, then extensive past experience has shown that the bulk movements of the ground alongside the basement caused by underpinning for a single storey basement (typical depth 3.5m) should not exceed 5mm in either horizontal or vertical directions.

3.2 In order to relate these typical ground movements to possible damage which adjoining 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 A basement has already been constructed beneath the adjoining Flat 1, whereas there is no basement beneath Flat 3 (it was possible to see below Flat 3 from the access point into Flat 1's crawl space, where a relatively modern block wall (sleeper wall?) was visible on the west side of the crawl space). As far as we are aware there is also no modern basement beneath the adjoining No.57, which is assumed to have a crawl space of similar depth and level as No.55's.

3.4 The PDISP analyses have predicted long-term settlements ranging from about nil to 10mm beneath the underpins to the party walls, although the model doesn't allow for the stiffness of the foundation so the range of settlements actually experienced is expected to be somewhat less. In addition, the ground beneath the Flat 1/Flat 2 party wall will already be stressed by Flat 1's underpinning so at least 50% of the predicted settlement will have occurred already.

3.5 The finished floor level (FFL) in the proposed basement is shown on SGA drawing 'Proposed Rear Elevation' (Drg No.15001/G200_P_S_001 rev.A) to be approximately 0.6m deeper than the existing basement to Flat 1. Drawings prepared by Simon Whitehead Architects show a uniform floor level throughout Flat 1 (except for the front lightwell, which is shown as deeper, though measured at 2.45m below top of external retaining wall). The adjoining walls in Flat 1 are located at the rear of the communal Lobby/Hallway, at the rear wall of the main 3-storey part of the house and at the rear wall of the extension/basement.

3.6 The internal layout in No.57 has been taken from the plan for the 1988 rear extension (Drg No. DL/88/1001A). This shows that the (load-bearing) walls adjoining No.55's Flat 2 were the 2-storey rear wall and the 3-storey internal transverse wall on the same line as the front wall to Flat 2. Both those walls extend only about 4.5m from the 55/57 party wall. The main rear wall to No.57 only adjoins No.55 at second floor level, though may still be affected by the proposed basement. Separate damage category assessments have been made for all these walls.

3.7 For No.55 Flat 3, where the length of both the internal partition wall and the 55/57 party wall are the same, only the party wall has been considered because their geometries and predicted settlements were almost identical.

3.8 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.

<u>No.55 Flat 1 internal transverse wall at rear of communal Lobby/Hallway:</u>

3.9 The relevant geometries are:

Depth of excavation = 0.6m (assumes underpin construction thickness is identical to that proposed for Flat 2).

Width (L) = 0.6 x 4 = 2.4m, so the ground movements will extend less than the full width of Flat 1.

Height (H) = 12.15 + 0.45 = 12.6m (basement FFL to eaves level, plus assumed thickness of Flat 1's underpin bases)

Hence L/H = 0.19, which is well below 0.5, the lowest ratio for which graphs are available.

3.10 Thus, for an anticipated 1mm maximum horizontal displacement (reduced pro-rata to the limited depth of excavation), the strain beneath Flat 1 would be in the order of $\varepsilon_h = 4.2 \times 10^{-4}$ (0.042%).

3.11 The maximum settlement predicted by the PDISP analysis beneath this point on the Flats 1/2 party wall, allowing for the stiffness of the underpin base was 9mm; with allowance for the settlement already developed by Flat 1's underpinning the settlement resulting from Flat 2's underpinning is expected to be in the order of 4.5mm (50% of 9mm, see paragraph 3.4); this must be added to the typical settlement caused by relaxation of the ground alongside the basement in response to excavation of the underpins, giving a maximum predicted settlement of the ground of approximately 5.5mm at the formation level of the underpinning to Flat 1's transverse internal wall. The settlement profile is expected to be convex, as also illustrated by the PDISP contours, with a worst case (low stiffness) deflection, Δ = 17% of the predicted combined settlement profile. Hence, Δ = 0.94mm, which represents a deflection ratio, $\Delta/L = 3.92 \times 10^{-4}$ (0.039%).

3.12 Using the graphs for L/H = 0.5, these deformations represent a damage category of 'very slight' (Burland Category 1, ε_{lim} =0.05-0.075%) as given in CIRIA SP200, Table 3.1, and illustrated in Figure 9 below.

<u>No.55 Flat 1 rear wall of main house:</u>

3.13 The same geometries as given in paragraph 3.9 above apply here, so the same horizontal strain will apply. The maximum settlement predicted by PDISP analyses was 8mm so, on the same basis as above, the deflection ratio, $\Delta/L = 3.54 \times 10^{-4}$ (0.035%).

3.14 Using the same graphs for L/H = 0.5, these deformations once again represent a damage category of 'very slight' (Burland Category 1) as illustrated in Figure 9 below.

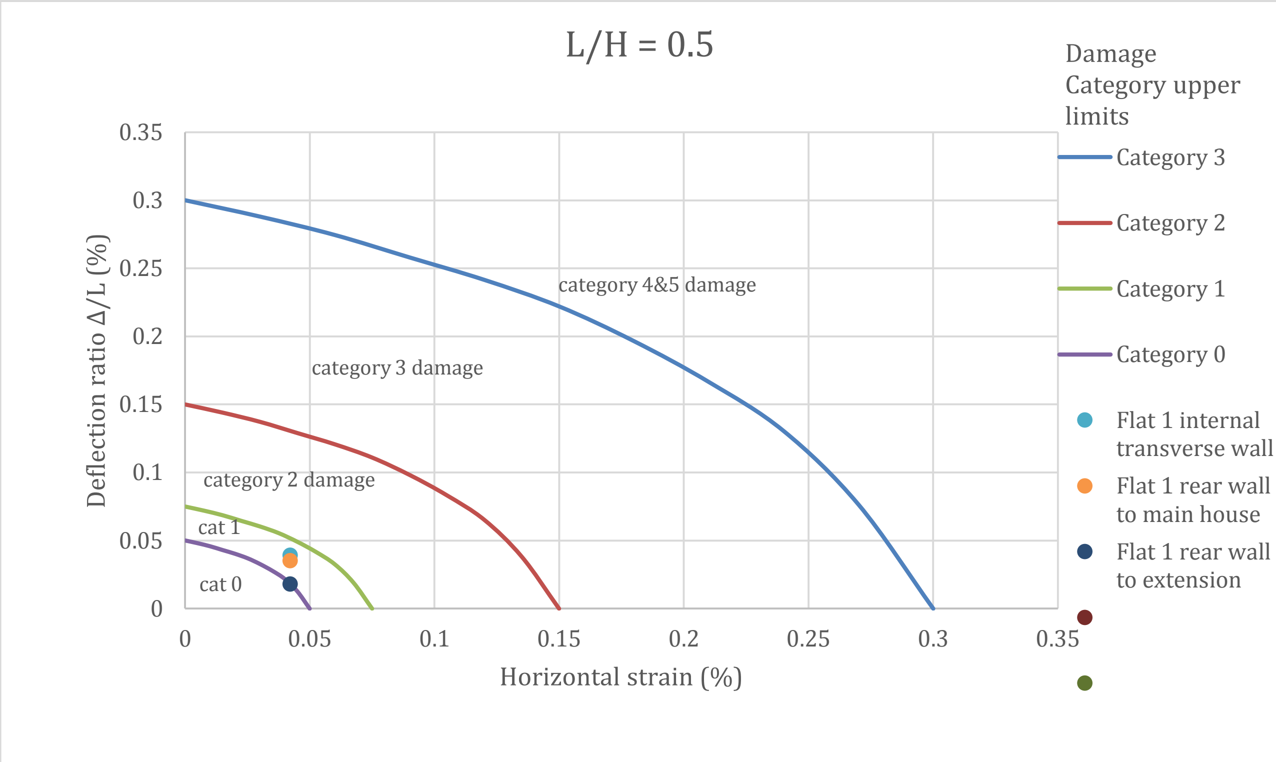

Figure 9: Damage category assessments for No.55's Flat 1 – internal transverse wall, rear wall of the main house and rear wall of Flat 1's extension/basement.

No.55 Flat 1 rear wall of extension:

3.15 The relevant geometries are:

Depth of excavation = 0.6m (assumes underpin construction thickness is identical to that proposed for Flat 2).

Width (L) = 0.6 x 4 = 2.4m, so the ground movements will extend less than the full width of Flat 1.

Height (H) = 5.7 + 0.45 = 6.15m (basement FFL to top of parapet wall, plus assumed thickness of Flat 1's underpin bases)

Hence L/H = 0.39, so graph for L/H = 0.5 is still conservative.

3.16 The excavation geometry remains as above, so the horizontal strain beneath Flat 1 would once again be in the order of $\varepsilon_h = 4.2 \times 10^{-4}$ (0.042%).

3.17 The maximum settlement predicted by the PDISP analysis beneath this point on the Flats 1/2 party wall was varying rapidly (see Figure G5); with allowance for the stiffness of the underpin base a settlement of about 3mm seems likely. As previously:

- 50% reduction for settlement which will already have occurred in response to Flat 1's underpinning, hence: 1.5mm settlement in response to Flat 2's proposed underpinning.

- Add typical settlement caused by relaxation of the ground alongside the basement in response to excavation of the underpins (reduced pro-rata to the limited depth of excavation): hence, approximate maximum predicted settlement of the ground at the formation level of the foundation to Flat 1's rear wall = 2.5mm.
- Convex settlement profile, with a worst case (low stiffness) deflection, gives Δ = 17% of total predicted settlement. Hence, Δ = 0.43mm, which represents a deflection ratio, $\Delta/L = 1.79 \times 10^{-4}$ (0.018%).

3.18 Using once again the graphs for L/H = 0.5, these deformations represent a damage category on the boundary between 'negligible' and 'very slight' (Burland Categories 0 and 1, ε_{lim} <0.05% to 0.05-0.075%) as illustrated in Figure 9 above.

<u>No.57's two-storey rear wall:</u>

3.19 The level of the foundation to the 55/57 party wall, and No.57's adjoining load-bearing walls, can reasonably be assumed to be 1.75m below the internal ground floor FFL, which would be 0.05m below the ground level under the access hatch into the crawl space. Thus, the relevant geometries are:

Depth of excavation below footing = 3.45 – 1.75 = 1.7m.

Width (L) = 4.5m (= 4.65 – 0.15, ie: rear wall of 2-storey projection less half the width of the underpin stem).

Width, zone of affected soils = 1.7 x 4 = 6.8m, so ground movements will extend beyond the 2-storey rear wall.

Height (H) = 6.8 + 1.75 = 8.55m (ground floor FFL to top of parapet, plus assumed depth of rear wall's footings)

Hence L/H = 0.53.

3.20 Thus, for an anticipated 3mm maximum horizontal displacement (reduced pro-rata to the limited depth of excavation), the strain beneath No.57 would be in the order of $\varepsilon_h = 4.41 \times 10^{-4}$ (0.044%).

3.21 The maximum settlement predicted by the PDISP analysis beneath this point on the 55/57 party wall, allowing for the stiffness of the underpin base, was 5mm. As previously:

- Add typical settlement caused by relaxation of the ground alongside the basement in response to excavation of the underpins (reduced pro-rata to the limited depth of excavation): hence, approximate maximum predicted settlement of the ground at the level of the 2-storey rear wall's foundation = 8mm.
- Convex settlement profile, with a worst case (low stiffness) deflection, gives Δ = 17% of total predicted settlement. Hence, Δ = 1.36mm, which represents a deflection ratio, $\Delta/L = 3.02 \times 10^{-4}$ (0.030%).

3.22 Using once again the graphs for L/H = 0.5, these deformations represent a damage category of 'very slight' (Burland Category 1) as illustrated in Figure 10 below.

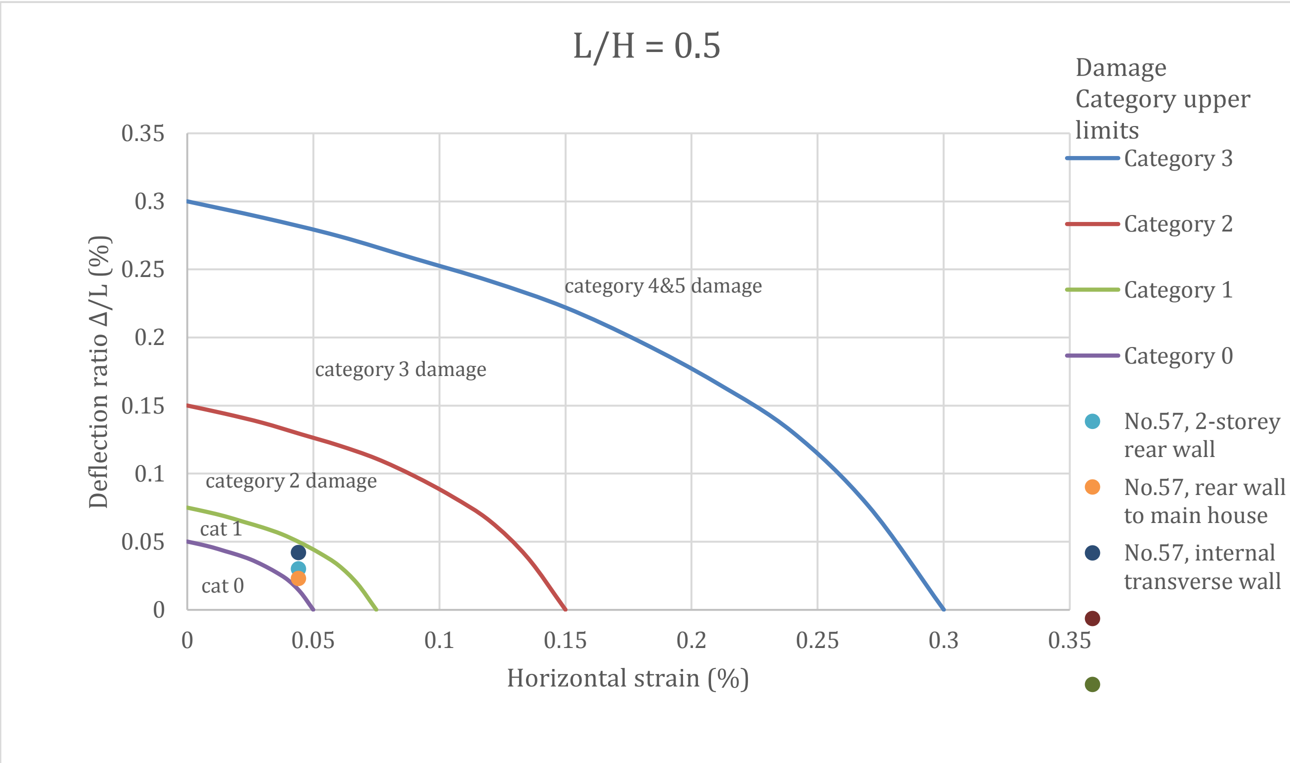

Figure 10: Damage category assessments for No.57: 2-storey rear wall, 3-storey rear wall to the main house, and internal transverse wall.
Note: 55/57 party wall forward of Flat 2 gave same result as No.57's internal transverse wall.

No.57's three-storey rear wall to the main house:

3.23 The foundation and excavation depths remain as above, so:

Depth of excavation below footing = 3.45 – 1.75 = 1.7m.

Width (L) = 1.7 x 4 = 6.8m, so ground movements will extend less than the full width of No.57.

Height (H) = 12.15 + 1.75 = 13.9m (ground floor FFL to eaves level, plus assumed depth of party wall footings)

Hence L/H = 0.49.

3.24 Thus, maximum horizontal strain beneath No.57 would remain in the order of ε_h = 4.41 x 10^{-4} (0.044%).

3.25 The maximum settlement predicted by the PDISP analysis beneath this point on the 55/57 party wall, allowing for the stiffness of the underpin base, was 6mm. As previously:

- Add typical settlement caused by relaxation of the ground alongside the basement in response to excavation of the underpins (reduced pro-rata to the limited depth of excavation): hence, approximate maximum predicted settlement of the ground at the level of the 2-storey rear wall's foundation = 9mm.

- Convex settlement profile, with a worst case (low stiffness) deflection, gives Δ = 17% of total predicted settlement. Hence, Δ = 1.53mm, which represents a deflection ratio, $\Delta/L = 2.25 \times 10^{-4}$ (0.023%).

3.26 Using once again the graphs for L/H = 0.5, these deformations represent a damage category of 'very slight' (Burland Category 1) as illustrated in Figure 10 above.

No.57's internal transverse wall (between front & rear reception rooms):

3.27 This wall is in line with the front wall to Flat 2. The foundation and excavation depths remain as above, but the width of this wall is the same as the width of the 2-storey rear wall, so:

Depth of excavation below footing = 3.45 – 1.75 = 1.7m.

Width (L) = 4.5m (= 4.65 – 0.15, internal transverse wall less half the width of the underpin stem).

Width, zone of affected soils = 1.7 x 4 = 6.8m, so ground movements will extend beyond this transverse wall.

Height (H) = 12.15 + 1.75 = 13.9m (ground floor FFL to eaves level, plus assumed depth of party wall footings).

Hence L/H = 0.32.

3.28 Thus, maximum horizontal strain beneath No.57 would remain in the order of $\varepsilon_h = 4.41 \times 10^{-4}$ (0.044%).

3.29 The maximum settlement predicted by the PDISP analysis beneath this point on the 55/57 party wall, allowing for the stiffness of the underpin base, was 8mm. As previously:

- Add typical settlement caused by relaxation of the ground alongside the basement in response to excavation of the underpins (reduced pro-rata to the limited depth of excavation): hence, approximate maximum predicted settlement of the ground at the level of the transverse wall's foundation = 11mm.
- Convex settlement profile, with a worst case (low stiffness) deflection, gives Δ = 17% of total predicted settlement. Hence, Δ = 1.87mm, which represents a deflection ratio, $\Delta/L = 4.16 \times 10^{-4}$ (0.042%).

3.30 Using once again the graphs for L/H = 0.5, these deformations represent a damage category of 'very slight' (Burland Category 1) as illustrated in Figure 10 above.

55/57 party wall forward from Flat 2 (alongside Flat 3):

3.31 The foundation and excavation depths remain as above, so:

Depth of excavation below footing = 3.45 – 1.75 = 1.7m.

Width (L) = 6.3m (to front wall).

Width, zone of affected soils = 1.7 x 4 = 6.8m, so ground movements will extend slightly beyond the front wall.

Height (H) = 12.15 + 1.75 = 13.9m (as above)

Hence L/H = 0.45.

3.32 Thus, maximum horizontal strain beneath No.57 would remain in the order of ε_h = 4.41×10^{-4} (0.044%).

3.33 The maximum settlement predicted by the PDISP analysis beneath this point on the 55/57 party wall, allowing for the stiffness of the underpin base, was once again 8mm. So the deflection ratio will also be the same as for the internal transverse wall, $\Delta/L = 4.16 \times 10^{-4}$ (0.042%), and the damage category will once again be 'very slight' (Burland Category 1).

SUMMARY

3.34 Damage category assessments have been undertaken for seven walls in neighbouring properties, all of which adjoin the proposed basement. The predicted damage in every case fell with Burland Category 1 'very slight' (ε_{lim} =0.05-0.075%, as given in CIRIA SP200, Table 3.1). One result fell on the boundary between 'negligible' and 'very slight' (Burland Categories 0 and 1 respectively).

3.35 The results have been plotted graphically in Figures 9 & 10.

3.36 No allowance has been made at the corners of the proposed basement for the beneficial restriction on displacements that will be provided, where relevant, by the adjacent ground which remains in-situ.

End of report

FIGURES

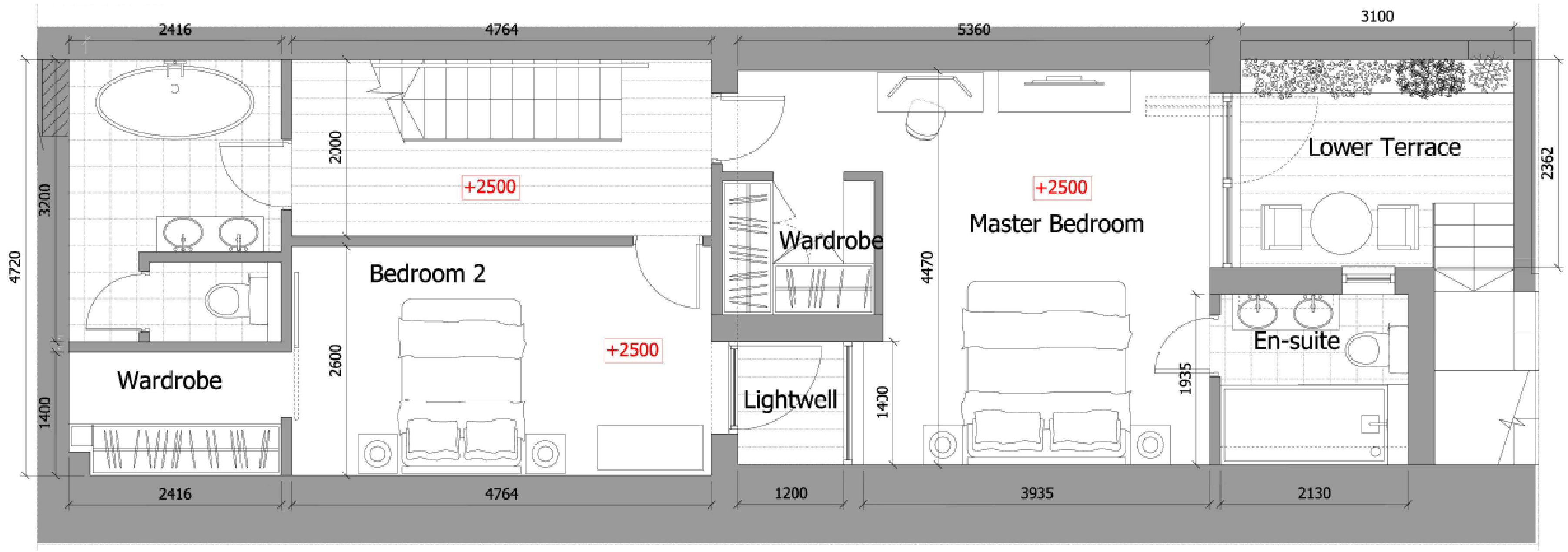

Figure 1. Layout of the proposed basement foundation plan

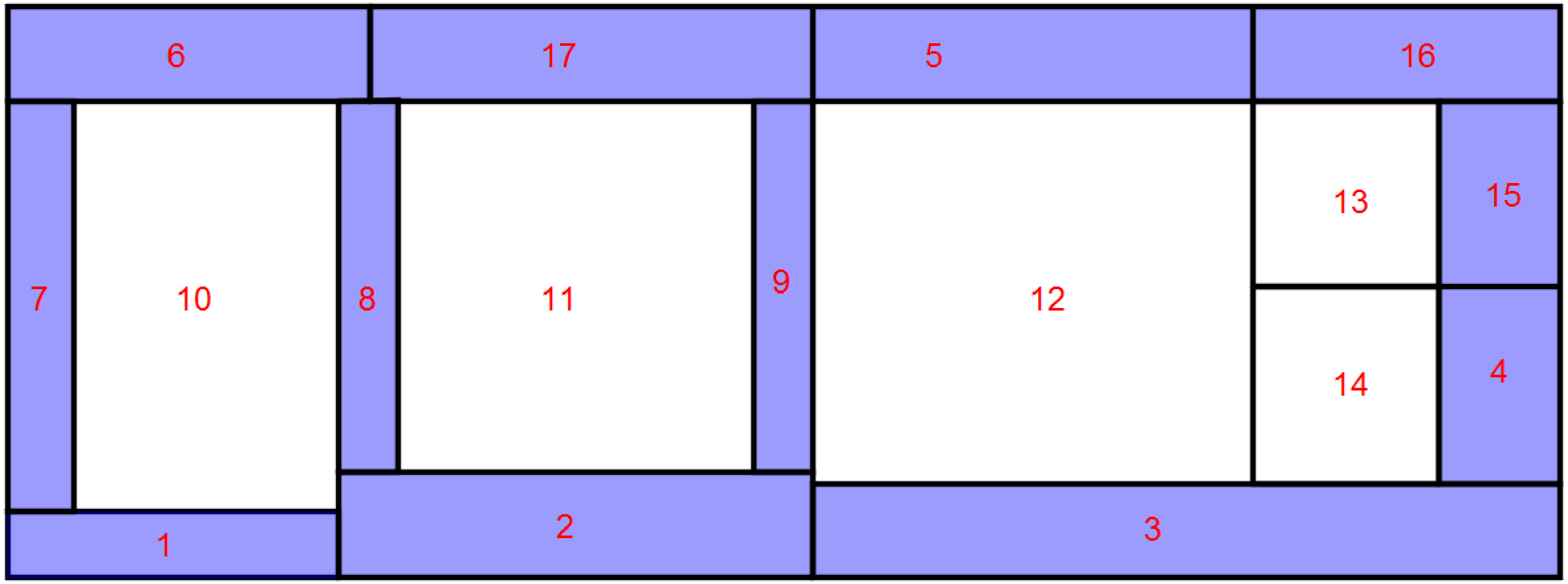

Figure 2. Detail of geometry introduced to PDISP

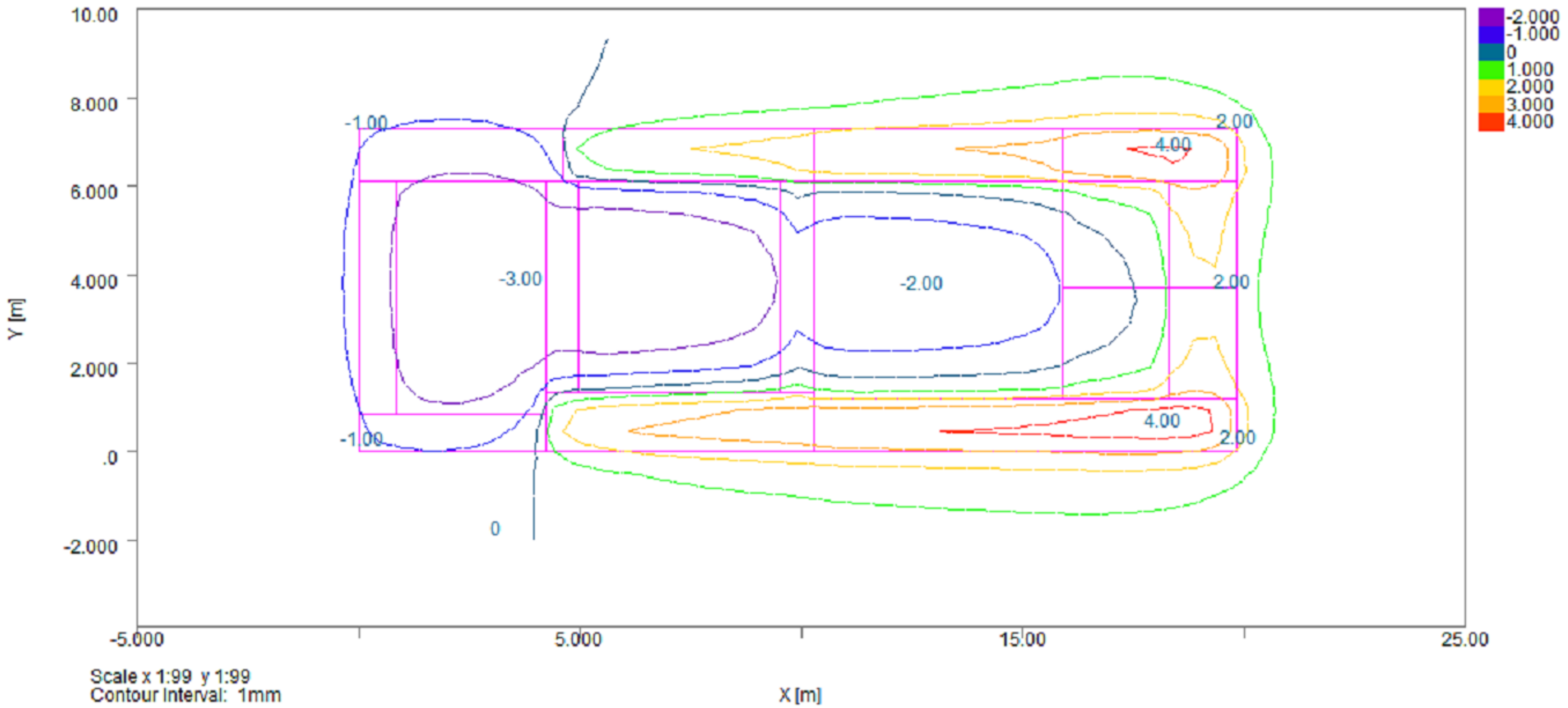

Figure 3. Short term (Stage 2) heave assessment contour

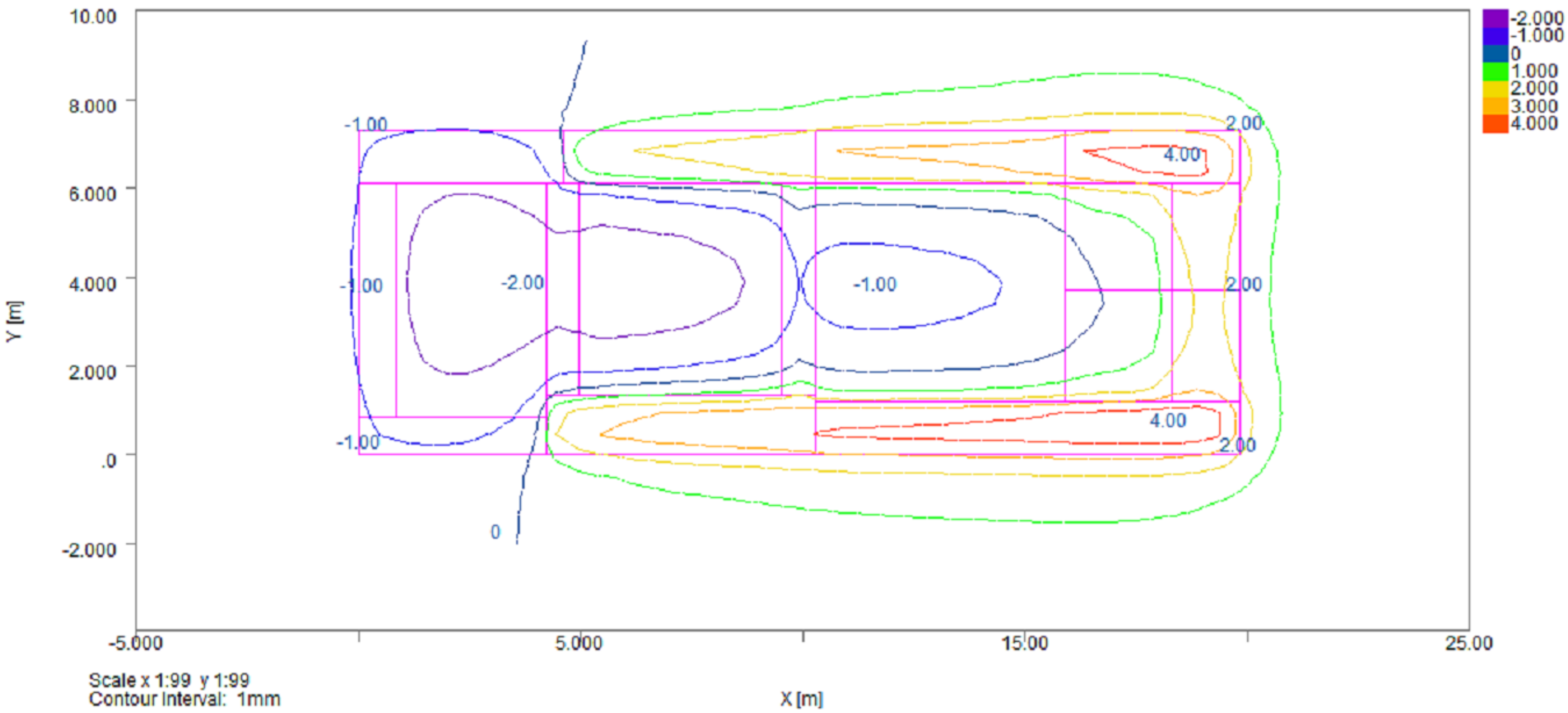

Figure 4. Short term (Stage 3) heave assessment contour

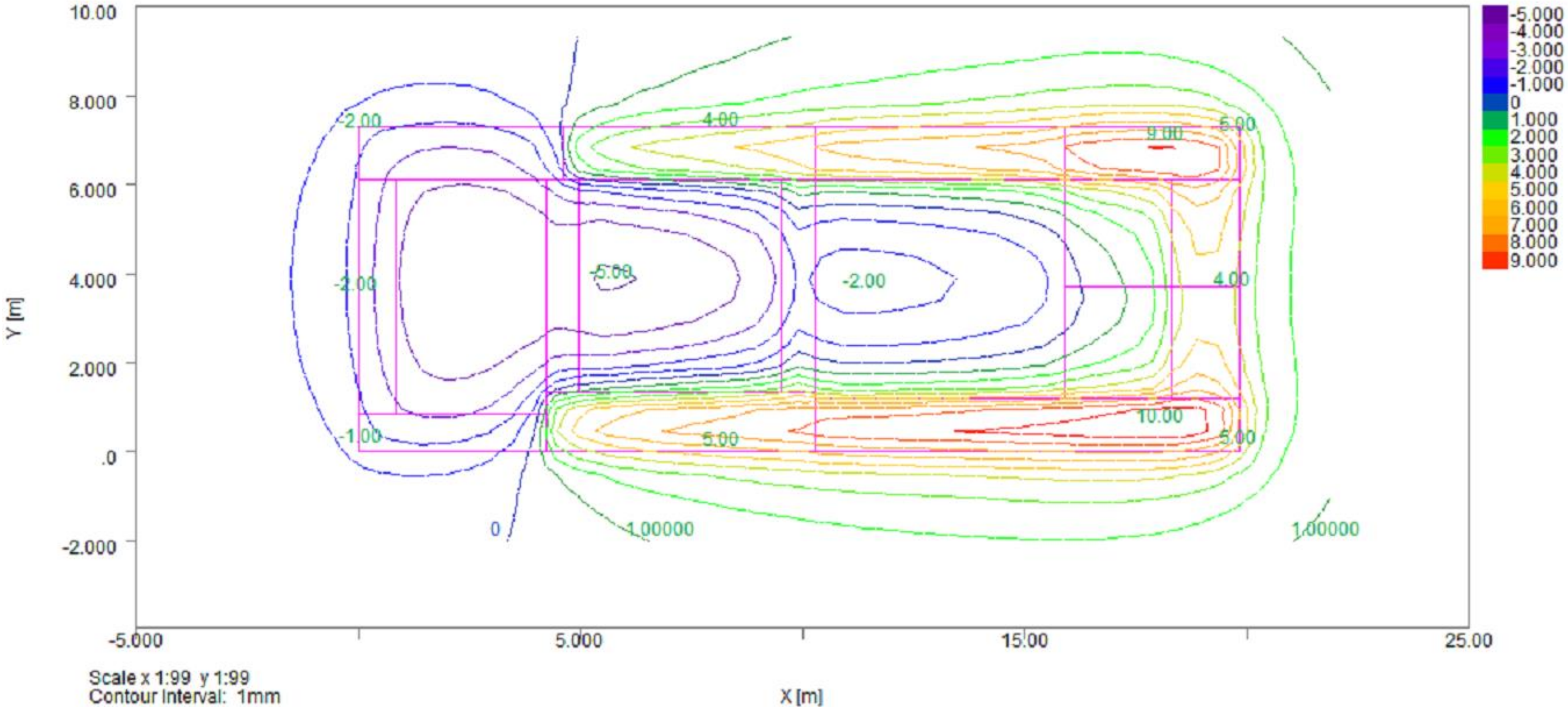

Figure Figure 5. Long term (Stage 4) heave assessment contour

TERMS & CONDITIONS

a) This report has been prepared for the purpose of providing advice to the client pursuant to its appointment of Chelmer Site Investigation Laboratories Limited (CSI) to act as a consultant.
b) Save for the client no duty is undertaken or warranty or representation made to any party in respect of the opinions, advice, recommendations or conclusions herein set out.
c) All work carried out in preparing this report has used, and is based upon, our professional knowledge and understanding of the current relevant English and European Community standards, approved codes of practice, technology and legislation.
d) Changes in the above may cause the opinion, advice, recommendations or conclusions set out in this report to become inappropriate or incorrect. However, in giving its opinions, advice, recommendations and conclusions, CSI has considered pending changes to environmental legislation and regulations of which it is currently aware. Following delivery of this report, we will have no obligation to advise the client of any such changes, or of their repercussions.
e) CSI acknowledges that it is being retained, in part, because of its knowledge and experience with respect to environmental matters. CSI will consider and analyse all information provided to it in the context of our knowledge and experience and all other relevant information known to us. To the extent that the information provided to us is not inconsistent or incompatible therewith, CSI shall be entitled to rely upon and assume, without independent verification, the accuracy and completeness of such information.
f) The content of this report represents the professional opinion of experienced environmental consultants. CSI does not provide specialist legal advice and the advice of lawyers may be required.
g) In the Summary and Recommendations sections of this report, CSI has set out our key findings and provided a summary and overview of our advice, opinions and recommendations. However, other parts of this report will often indicate the limitations of the information obtained by CSI and therefore any advice, opinions or recommendations set out in the Executive Summary, Summary and Recommendations sections ought not to be relied upon unless they are considered in the context of the whole report.
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