Damage Category Assessment

Site	1 Ardwick Road London NW2 2BX
Client	Green Structural Engineering
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PREAMBLE

This addendum is supplementary to, and must be read in conjunction with, our Basement Impact Assessment (BIA) report (reference BIA5217 dated May 2015).

1. PDISP HEAVE ANALYSES

1.1 Basement Geometry and Stresses:

1.1.1 Analyses of vertical ground movements (heave or settlement) have been undertaken using PDISP software in order to assess the potential magnitudes of movements which may result from the changes of vertical stresses caused by excavation of the basement. These preliminary analyses have not modelled the horizontal forces on the retaining walls, so have simplified the stress regime significantly.

1.1.2 Figure 1 illustrates the layout of the proposed basement using an extract from the Lower Ground Floor Plan by Green Structural Engineer (GSE Drg No.12693-GA/01 P1, which is based on Metropolitan Development Consultancy's Drg No. 7852/21C). The alignment of the underpinning system and the load takedown details are presented in Figure 2 based on Green Structural Engineering's Drg No.12693 – SK06. The maximum overall dimensions of the proposed basement are 12.47m wide by 27.34m long.

1.1.3 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 GSE's Drg No.12693 – SK06 presented in Figure No. 2. GSE's sections, Drgs No's 12693 – S/01-S/03, show that the underpins and the basement slab will all be founded at the same level. Thus, variations in the depths of excavation generally reflect the varying existing ground levels.

1.1.4 Table 1 below presents the co-ordinates of the zones used to input the main elements of the basement's geometry into PDISP based on the illustration in Figure 3, together with the net changes in vertical pressure for the four major stages in the stress history of the basement's construction, as detailed in paragraph 1.3.1 below. The ability to consider non-rectangular zones has recently been introduced to PDisp and has been used in these analyses for the zones which include or adjoin the two bay windows. Definition of the non-rectangular zones is by the precise coordinates of all corners from which the programme has calculated the geometrical properties of each zone automatically. As centroids and dimensions are no longer relevant for those zones the relevant fields in Table 1 have been left blank.

Table 1: Coordinates and net bearing pressure for PDISP							
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	9.366	0.900	4.488	1.800	-48.01	-48.01	-43.01
2	8.020	2.828	1.795	2.055	-56.46	-56.46	-51.46
3	6.075	4.605	5.175	1.500	-14.61	-14.61	-9.61
4	1.744	4.824	3.487	1.937	-45.33	-45.33	-40.33
5	1.726	6.830	3.522	2.076	-42.55	-42.55	-37.55
6	1.709	8.868	3.563	2.128	-42.00	-42.00	-37.00
7					-30.49	-30.49	-25.49
8					-32.28	-32.28	-27.28
9					-30.67	-30.67	-25.67
10	1.701	13.951	3.757	0.999	19.06	19.06	24.06
11					24.72	24.72	29.72
12					24.95	24.95	29.95

Table 1: Coordinates and net bearing pressure for PDISP (Cont'd)							
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
13					24.99	24.99	29.99
14					69.32	69.32	74.32
15	3.256	18.775	1.054	0.896	1.96	1.96	6.96
16	2.581	19.648	2.405	0.850	8.22	8.22	13.22
17	1.803	20.982	0.850	1.818	8.22	8.22	13.22
18	2.581	22.316	2.405	0.850	8.22	8.22	13.22
19	3.358	24.616	0.850	3.750	23.35	23.35	28.35
20	7.692	26.916	9.517	0.850	23.35	23.35	28.35
21	12.025	24.618	0.850	3.746	23.35	23.35	28.35
22	11.710	20.534	1.480	4.414	0.19	0.19	5.19
23	11.885	15.919	1.830	4.816	70.41	70.41	75.41
24	11.885	8.263	1.830	10.496	48.75	48.75	53.75
25	10.713	2.408	1.795	1.215	-56.46	-56.46	-51.46
26	9.366	2.408	0.898	1.215	-70.11	-70.11	-58.11
27	9.944	3.410	2.053	0.790	-70.11	-70.11	-58.11
28	9.816	4.580	2.308	1.550	-45.79	-45.79	-33.79
29	7.229	5.574	7.483	0.437	0.00	-45.79	-33.79
30	7.229	7.862	7.483	4.140	0.00	-45.79	-33.79
31					0.00	-45.79	-33.79
32					0.00	-45.79	-33.79
33					0.00	-45.79	-33.79
34	7.275	13.981	7.391	0.939	0.00	-24.13	-12.13

35					0.00	-24.13	-12.13
36					0.00	-24.13	-12.13
37	7.377	20.534	7.187	4.414	0.00	-24.13	-12.13
38	3.006	20.982	1.555	1.818	-24.13	-24.13	-12.13
39	7.692	24.616	7.817	3.750	0.00	-19.00	-7.00

1.2 Ground Conditions:

1.2.1 The ground profile was based on the site-specific ground investigation by Chelmer Site Investigations, as presented in Sections 9 & 10.1 of BIA5217, and the desk study information.

1.2.2 The short-term and long-term geotechnical properties of the soil strata used for the PDISP analyses are presented in Table 2 below, based on this investigation and data from other projects.

Table 2: Soil parameters for PDISP analyses

Strata	**Level** (m bgl)	**Short-term, undrained Young's Modulus,** Eu (MPa)	**Long-term, drained Young's Modulus,** E' (MPa)
London Clay	3.15 28.10	50 144	30 86

Where:

Undrained Young's Modulus, Eu = 50 + 3.75z MPa

Drained Young's Modulus, E' = 0.6 * Eu

In which:

z = depth below the founding level

1.3 PDISP Analyses:

1.3.1 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/retaining walls – Short-term condition
- Stage 2 – Bulk excavation of central area to formation level – Short-term condition
- Stage 3 – Construction of basement slab – Short-term (undrained) condition
- Stage 4 – As Stage 3, except – Long-term (drained) condition.

1.3.2 The results of the analyses for the Stages 2 and 4 are presented as contour plots on the appended Figures 4 and 5, respectively.

2. HEAVE/SETTLEMENT ASSESSMENT

2.1 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, 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.

2.2 The PDISP analyses indicated that only small movements in the order of 3.5mm heave to 1mm settlement are likely to develop beneath the external walls of the basement in short-term, increasing to 6mm heave to 2mm settlement in the long-term. Predicted displacements beneath the slab were up to 6mm total and differential heave in the long term. 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 4)**	**Stage 4 (Figure 5)**
Front wall & Lightwell	3.5mm Heave – 0.5mm Settlement	6mm Heave – 1mm Settlement
1/3 party wall	1mm Heave – 1mm Settlement	2mm Heave – 3.5mm Settlement
Ardwick Road flank wall	0.5 – 3.5mm Heave	4.5mm Heave – 1.5mm Settlement
Courtyard Garden Retaining Wall	3mm Heave – 0.5mm Settlement	6mm Heave – 1.5mm Settlement
Basement slab	0.5-4.0mm Heave	0 – 6mm Heave
Extension (single storey)	0 – 0.5mm Heave	0.5 – 1.0mm Settlement

2.3 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. In Stage 3, following construction of the basement slab slight settlement reduced the predicted slab displacements to 3mm heave – 0.5mm settlement, however the slab will be integral with the perimeter walls so the increase in settlement of the 1/3 party wall to 2.5mm must also be considered. Thus, the analyses indicated that the maximum predicted post-construction displacements beneath the slab are likely to be about 3mm heave with a differential displacement of about 5mm across the slab (centre to zone 23).

2.4 These analyses do not allow for the stiffness of the basement slab, which in practice will tend to 'smooth out' the predicted movements, especially where these occur over short distances. This influence may have little or no impact on the extremes of the predicted displacements, depending on the stiffness of the structural member concerned.

3. 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 No.1 Ardwick Road adjoins No.3. Trial pits were only dug alongside the external walls to No.1, with TP1 located at the front left corner of the house where the footing was approximately 1.5m deep. No.3 has an integral garage alongside the party wall, so it seems reasonable to assume that the party wall is likely to be founded at the same level as seen in TP1.

3.4 Beneath the front part of No.1 there was a void/crawl space below the ground floor; to the rear, where there was a lower ground floor, the ground level stepped down. As a result the proposed depths of excavation within No.1 alongside the party wall will vary from 2.41m at the front to 1.27m alongside a short section of the party wall (short because No.3 does not extend as far south as No.1). Moreover, the maximum excavation depth below the footings will be approximately 2.2m (= 3.69 - 1.5m), and the garage floor (and party wall footing) may be even deeper, though that has been ignored in the absence of specific information. Thus, the typical bulk movements of the ground alongside the basement in response to the proposed underpinning were therefore adjusted to allow for the depth of the footing and reduced depth of excavation.

3.5 The PDISP analyses have predicted long-term settlements beneath the underpins to the party wall ranging from nil at the front end of the party wall to about 3.5mm alongside the rear part of No.3, although the model doesn't allow for the stiffness of the foundation so the range of settlements actually experienced may be somewhat less. Separate damage category assessments have been made for the front and rear walls of No.3 (and adjoining houses), where, respectively, the minimum and maximum settlements was predicted by the PDISP analyses.

3.6 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.

No.3 Front Wall:

3.7 The relevant geometries are as follows:

Depth of excavation = 2.2m
Width (L) = 2.2 x 4 = 8.8m.
Height (H) = 10.9m (= 9.4m to No.3's parapet, plus 1.5m footing)
Hence L/H = 0.81 = approx.1.0 (which is conservative)

Thus, for an anticipated 4mm maximum horizontal displacement (reduced pro-rata to the limited depth of excavation), the strain beneath No.3 would, theoretically, be in the order of $\varepsilon_h = 4.55 \times 10^{-4}$ (0.046%).

Settlements of the ground alongside the basement predicted by the PDISP analysis should be added to the settlement caused by relaxation of the ground alongside the basement in response to excavation of the underpins. In this case that settlement was zero, so the predicted settlement of the ground at the assumed level of No.3's footings was 4mm. The settlement profile is expected to be convex with a worst case (low stiffness) deflection, Δ = 17% of the predicted combined settlement profile. Hence, Δ = 0.7mm, which represents a deflection ratio, $\Delta/L = 7.95 \times 10^{-5}$ (0.008%).

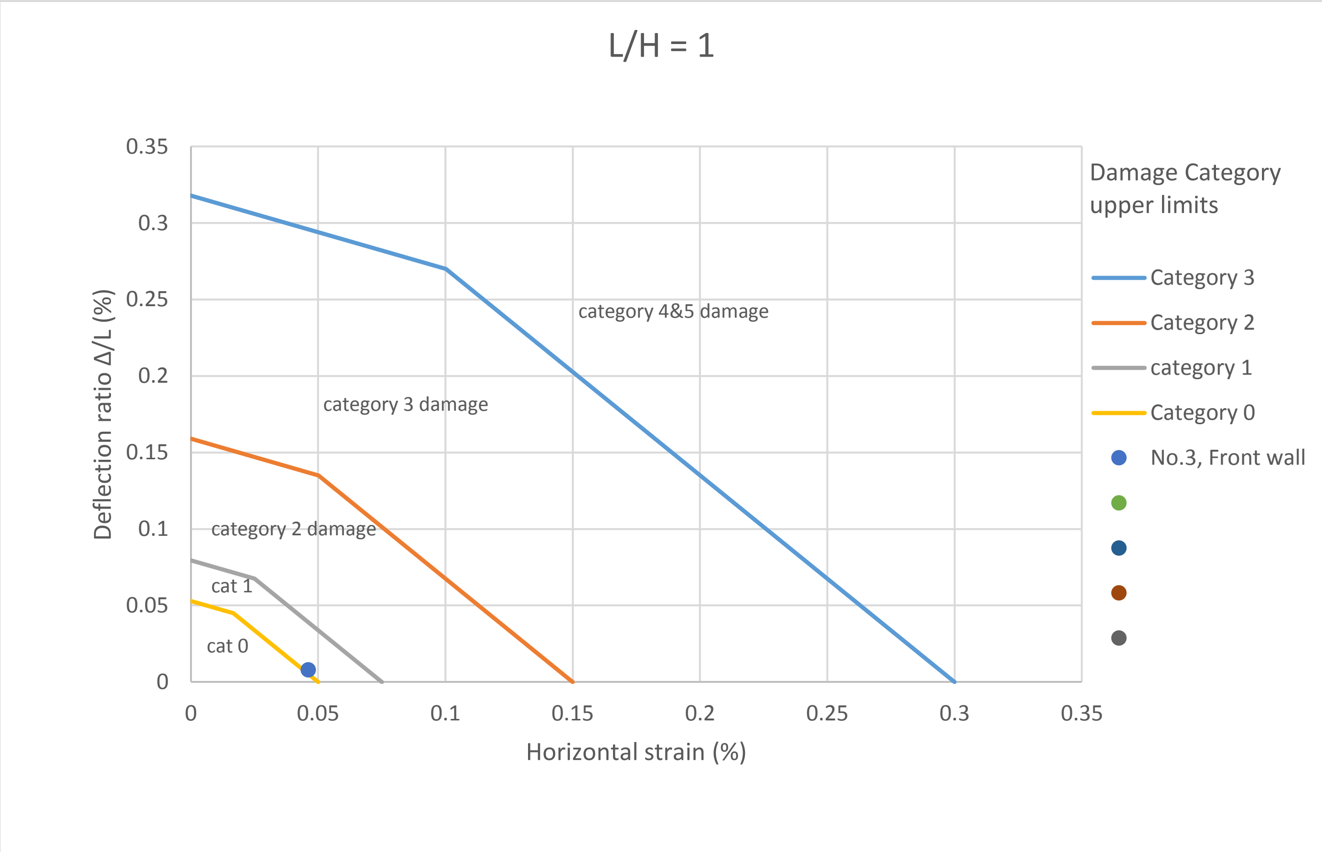

Figure 6: Damage category assessment for front wall of No.3.

3.8 Using the graphs for L/H = 1.0, which is slightly conservative, these deformations represent a damage category of 'very slight' (Burland Category 1, ε_{lim} =0.05-0.075%) almost on the boundary with Category 0 'negligible', as given in CIRIA SP200, Table 3.1, and illustrated in Figure 6 above.

<u>No.3 Rear Wall:</u>

3.9 The relevant geometries are as follows:

Depth of excavation = 1.27m
Width (L) = 1.27 x 4 = 5.1m.
Height (H) = 10.9m (= 9.4m to No.3's parapet, plus 1.5m footing)
Hence L/H = 0.47 = approx.0.5

Thus, for an anticipated 2mm maximum horizontal displacement (reduced pro-rata to the limited depth of excavation), the strain beneath No.3 would, theoretically, be in the order of $\varepsilon_h = 3.92 \times 10^{-4}$ (0.039%).

The settlement predicted by the PDISP analysis beneath the middle of the party wall, allowing for the stiffness of the underpin base, might be in the order of 3.5mm; 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 approximately 5mm total predicted settlement of the ground at the level of No.3's footings. The settlement profile is expected to be convex with a worst case (low stiffness) deflection, Δ = 17% of the predicted combined settlement profile. Hence, Δ = 0.85mm, which represents a deflection ratio, $\Delta/L = 1.67 \times 10^{-4}$ (0.017%).

Using the graphs for L/H = 0.5, these deformations represent a damage category of 'negligible' (Burland Category 0, ε_{lim} = <0.05%), as given in CIRIA SP200, Table 3.1, and illustrated in Figure 7 below.

3.10 Use of best practice construction methods, as outlined in the BIA report, will be essential to ensure that the ground movements are kept in line with the above predictions.

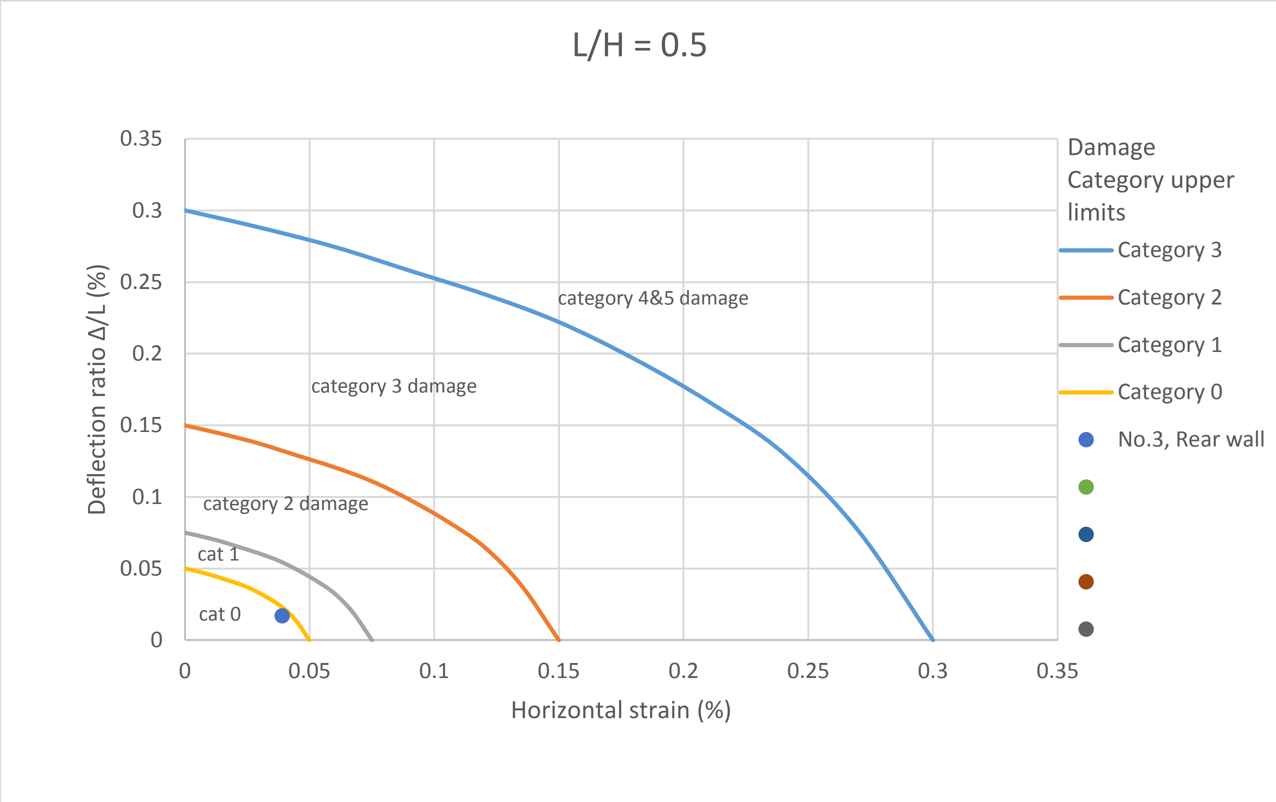

Figure 7: Damage category assessment for rear wall of No.3.

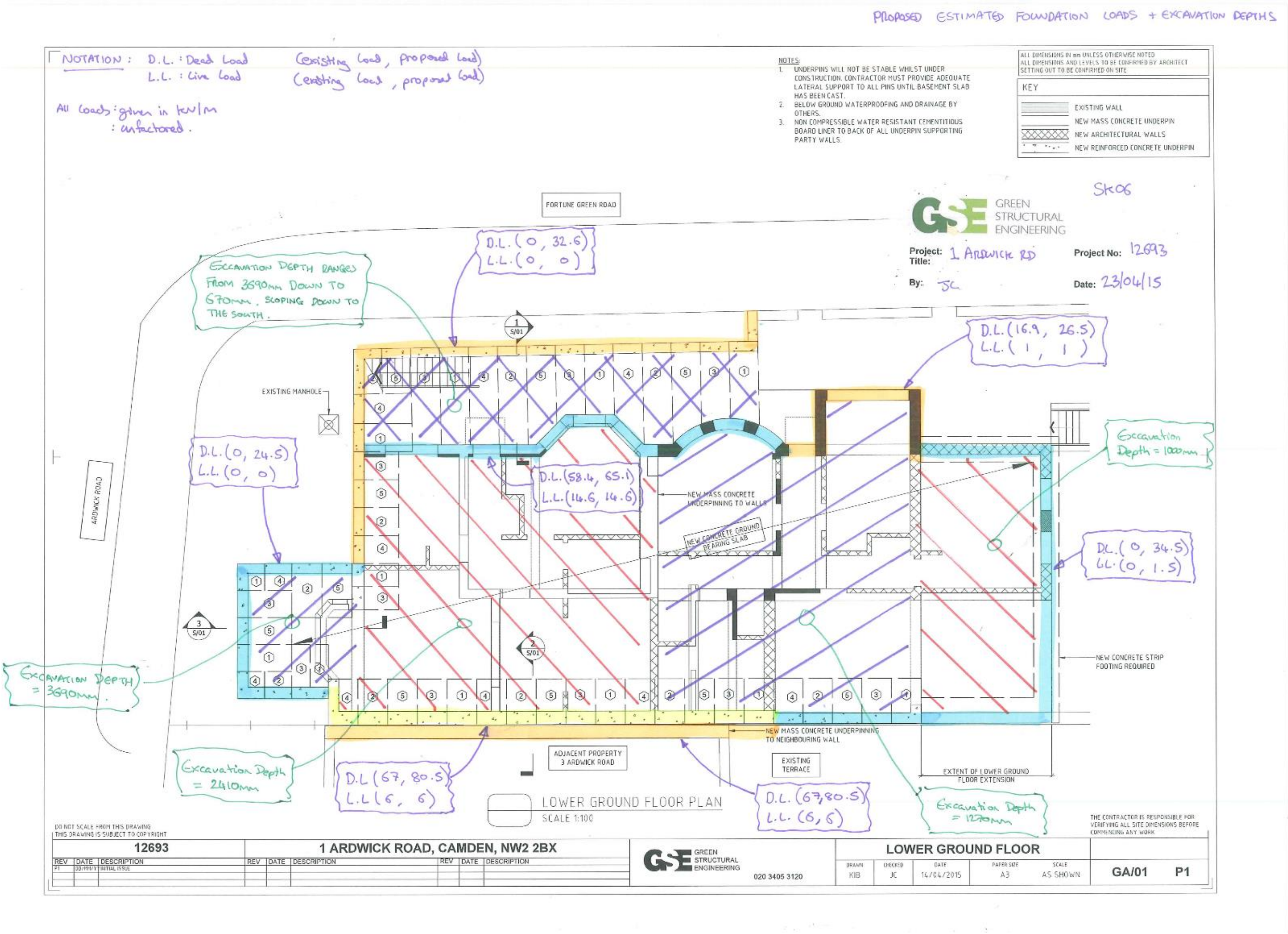

Figure 2. Layout of the underpinning system and Load Take down Detail and Excavation Depth Detail

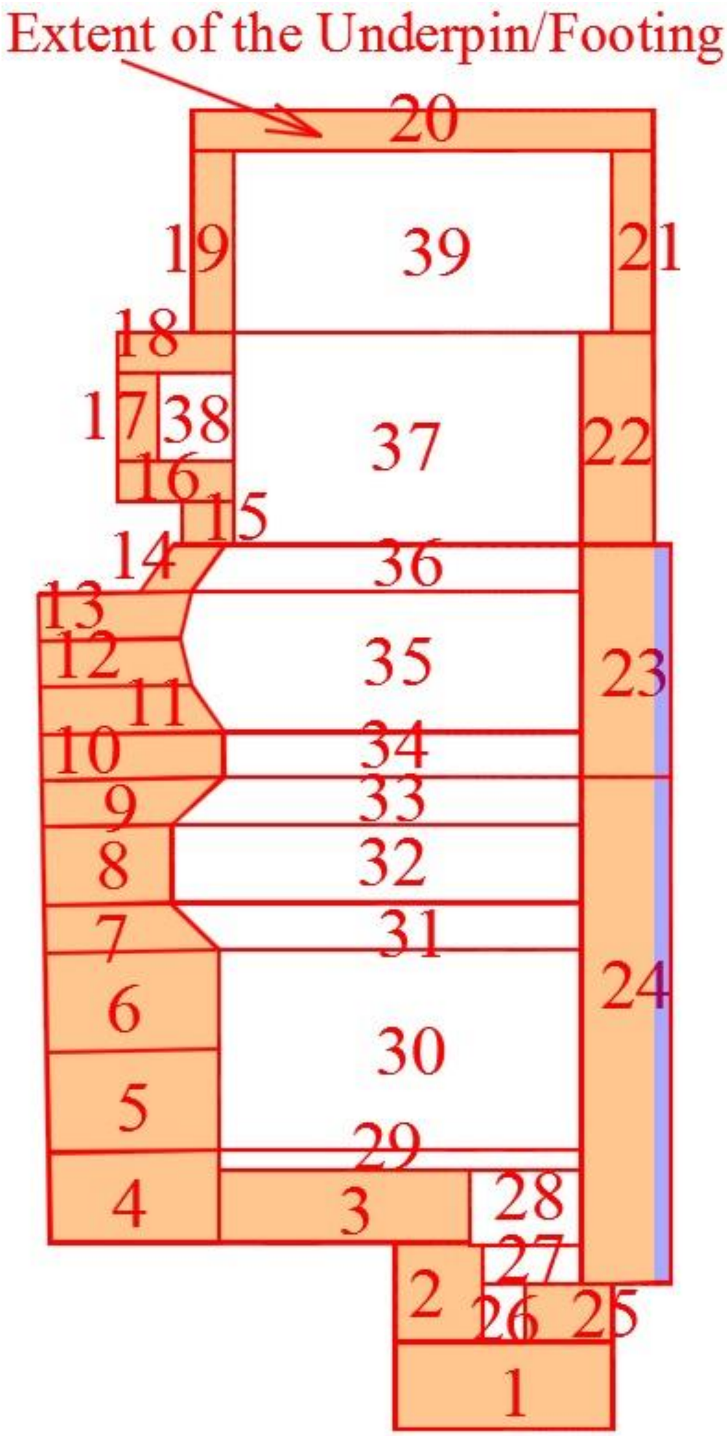

Figure 3. Detail of geometry introduced to PDISP

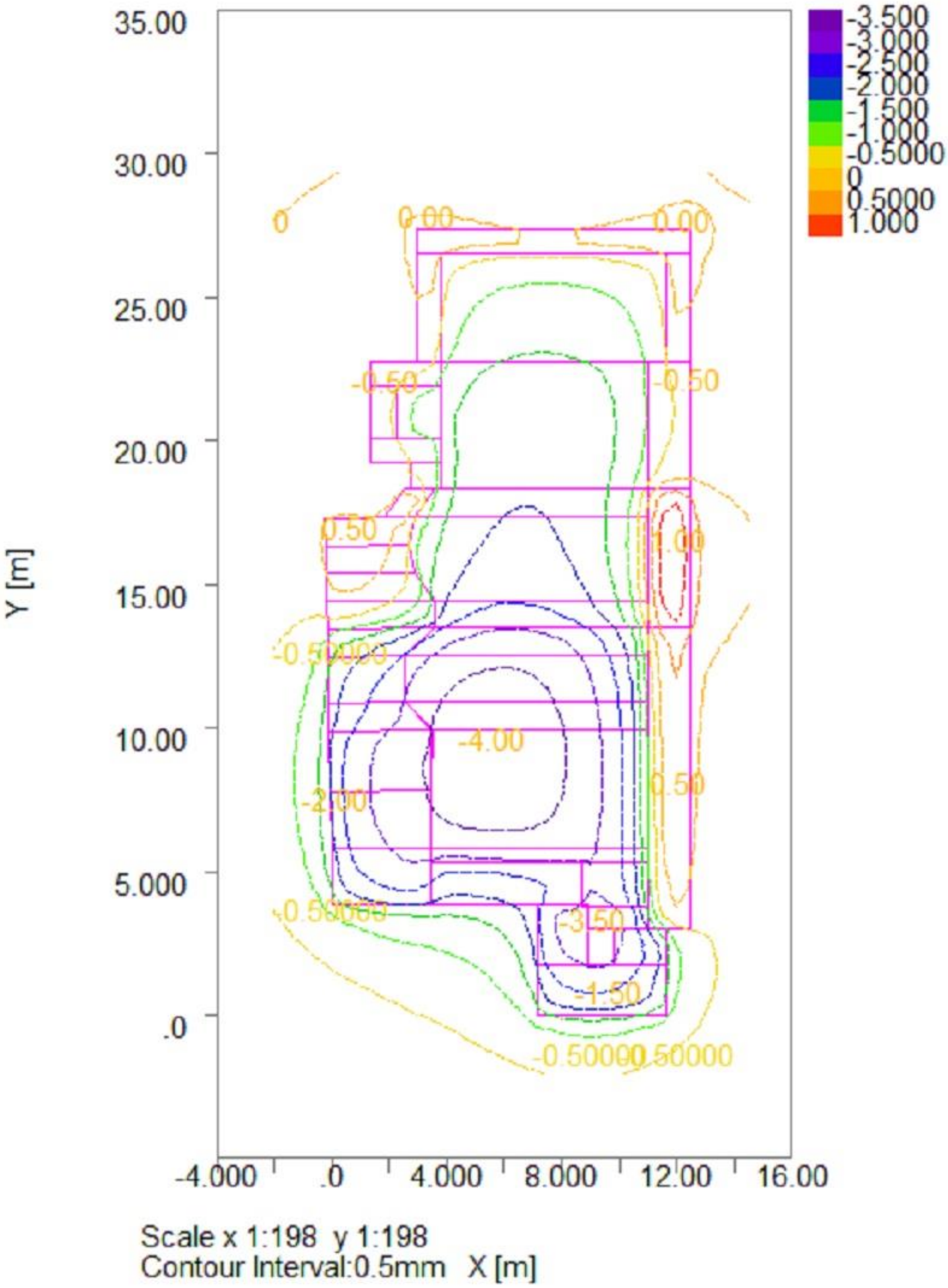

Figure 4. Short term (Stage 2) heave assessment contour

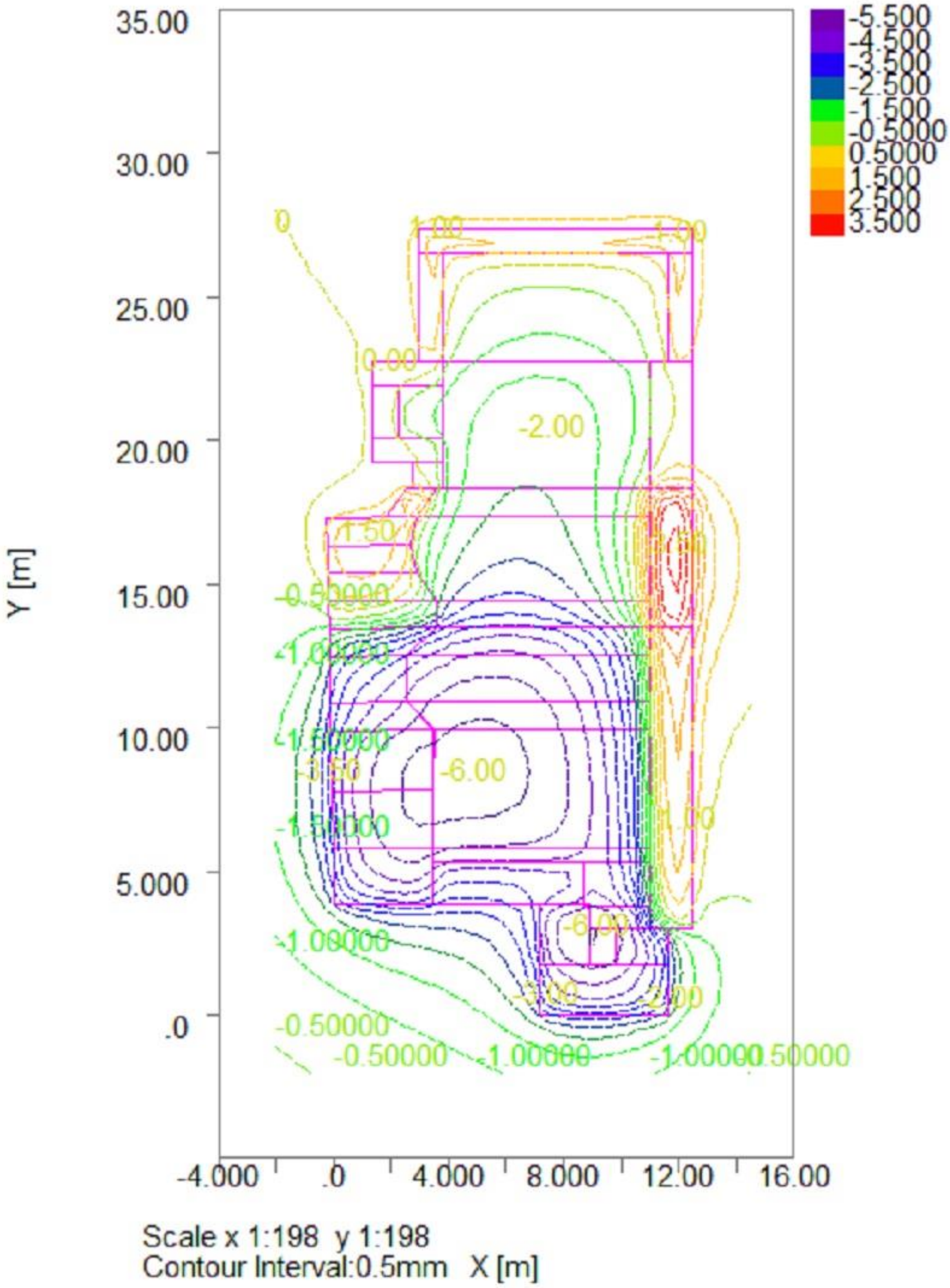

Figure 5. Long term (Stage 4) heave assessment contour

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