Addendum to Basement Impact Assessment

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Client	Mr W McIntyre
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PREAMBLE

This addendum is supplementary to, and must be read in conjunction with, our Basement Impact Assessment (BIA) report (reference BIA5016 dated March 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 based on DVM architects' Drg No.1859-04. The alignment of the underpinning system is presented in Figure 2 based on Stroud Associates Drg No.SO699-01. The maximum overall dimensions of the proposed basement are 6.00m wide by 9.68m 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 were:

- Zones 3, 6 & 8 (existing cellar): 1.3m
- Zone 1 (basement slab within underpins): 2.15m
- All other zones (underpins): 2.3m

The depths of excavation for the underpins and Zone 1 allow for the presence of the significant crawl space beneath the suspended timber ground floor to the main part of No.57, which was estimated to be approximately 1m deep. No additional loading was included for any slab thickenings beneath the columns because the increase in net vertical pressure from such slab thickenings was anticipated to be minimal.

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 based on the load take down detail illustrated in Figure 4 based on Stroud Associates Drg No.SO699-SK001B.

Table 1: Coordinates and pressures 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	4.017	3.000	4.034	2.000	0.00	-43.00	-32.00
2	1.000	3.865	2.000	4.270	20.29	20.29	25.29
3	2.558	0.865	5.116	1.730	54.59	54.59	59.59
4	4.087	2.332	1.000	1.000	0.00	73.00	73.00
5	3.558	1.865	3.116	0.270	15.04	15.04	20.04
6	5.575	0.614	0.918	1.227	35.04	35.04	40.04
7	5.575	1.614	0.918	0.773	15.04	15.04	20.04

8	7.034	0.614	2.000	1.227	46.40	46.40	51.40
9	7.034	2.101	2.000	1.748	26.40	26.40	31.40
10	8.859	4.716	1.650	2.569	28.35	28.35	33.35
11	7.034	4.488	2.000	3.025	32.51	32.51	37.51
12	4.017	5.000	4.034	2.000	25.66	25.66	30.66
13	3.010	5.320	1.400	0.600	76.19	76.19	76.19
14	5.034	5.320	1.400	0.600	76.19	76.19	76.19

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 and 10.1 of BIA5016, 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	3.30 15.30	55 100	33 60

Where:

Undrained Young's Modulus, Eu = 55 + 3.75z MPa
Drained Young's Modulus, E' = 0.6 * Eu

In which:

z = depth below the founding level (3.30m bgl)

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, 3 and 4 are presented as contour plots on the appended Figures 5 to 7 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 only very small settlement movements less than 4mm are likely to develop beneath the external walls of the basement. The ranges of predicted short-term and long-term movements for each of the main walls are presented in Table 3 below.

Table 3: Summary of predicted settlement

Location	**Stage 2** (Figure 5)	**Stage 4** (Figure 7)
Front wall	0.4 – 1.5mm	1.2 – 3.5mm
57 flank wall	0.5 – 1.5mm	1.5 – 4.5mm
55/57 party wall	0.4 – 1.5mm	1.2 – 4mm
Rear wall	0.5 – 1.0mm	1.5 – 3.5mm
Courtyard walls (lightwell)	0.5 – 1.0mm	1.5 – 3.0mm
Basement slab (including column base)	0 – 1mm	1.0 – 3.0mm

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. The analyses indicated that the maximum predicted post-construction displacements beneath the slab are likely to be about 2mm settlement in a hogging type deformation.

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 The trial pits dug at No.57 showed that its front and rear walls were founded at depths of 0.885m/0.895m respectively. No.55 is at a slightly higher level than No.57 so its foundations were assumed to be at approximately 0.75m below the ground level around No.57. This indicates that the ground surface beneath No.57's crawl space is only just above the foundation level, so no adjustment to the damage category assessment to allow for the depth of the footing was considered appropriate in this case.

3.4 The PDISP analyses have predicted long-term settlements ranging from about 1.2mm to 4mm beneath the underpins to the party wall, 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. The internal layout in No.55 is not known, though it is possible that there is a transverse load-bearing wall, so separate damage category assessments have been made for the front wall of No.55 (and adjoining houses), where the least settlement was predicted, and for an internal transverse wall within No.55, where the maximum settlement was predicted. No separate analysis was warranted for the rear wall of No.55 where intermediate settlements were predicted by the PDISP analyses.

3.5 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.55 Front Wall:

3.6 The relevant geometries are as follows:

Depth of excavation = 2.3m

Width (L) = 2.3 x 4 = 9.2m, so the ground movements are only likely to extend to No.53.

Height (H) = 6.7m to eaves level

Hence L/H = 1.37 = approx.1.5

Thus, for an anticipated 4mm maximum horizontal displacement (reduced pro-rata to the limited depth of excavation), the strain beneath No's 53 & 55 would, theoretically, be in the order of $\varepsilon_h = 4.3 \times 10^{-4}$ (0.043%).

The settlement predicted by the PDISP analysis at the front end of the party wall, allowing for the stiffness of the underpin base, is expected to be about 2mm; 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 6mm total predicted settlement of the ground at the level of No.55'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 = 1.0mm, which represents a deflection ratio, $\Delta/L = 1.1 \times 10^{-4}$ (0.011%).

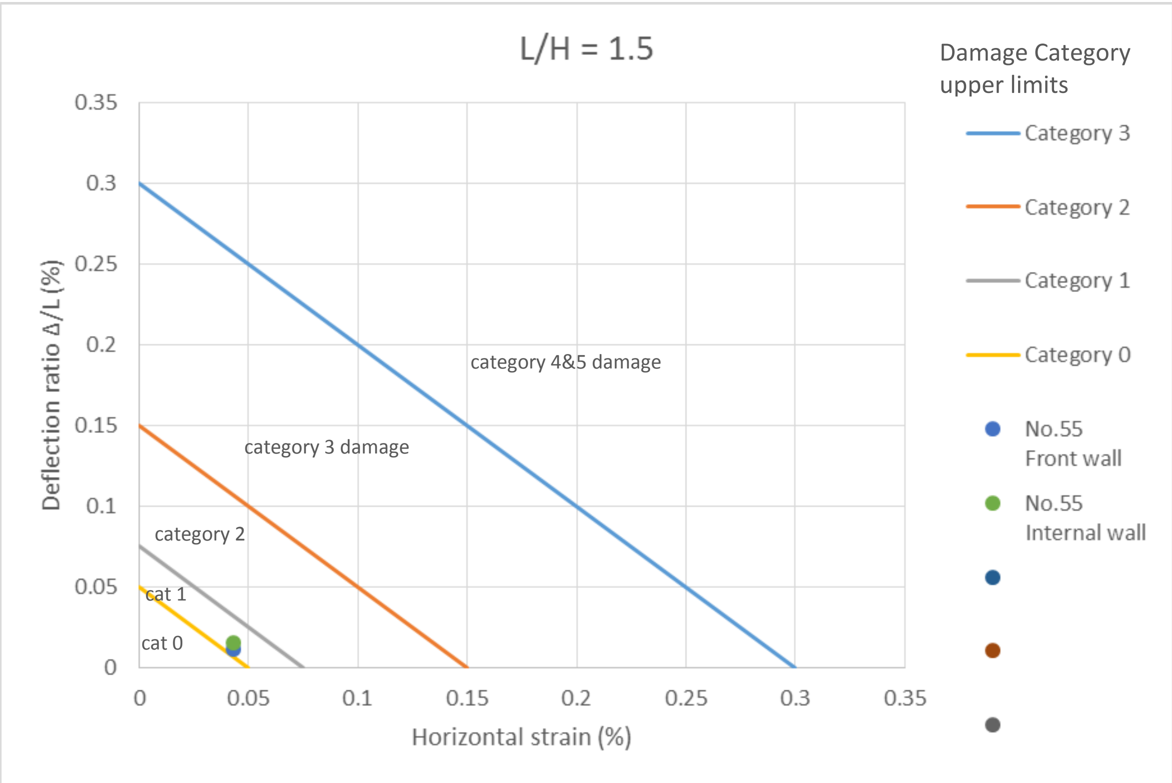

Figure 8: Damage category assessments for front wall and internal wall of No.55.

3.7 Using the graphs for L/H = 1.5, which is slightly conservative, these deformations represent a damage category of 'very slight' (Burland Category 1, ε_{lim} =0.05-0.075%), close to the boundary with Category 0 'negiligible', as given in CIRIA SP200, Table 3.1, and illustrated in Figure 8 above.

No.55 Internal Transverse Wall:

3.8 The relevant excavation and building geometries for an internal transverse wall remain as for the front wall analysed above. Thus, the maximum horizontal strain beneath No's 53 & 55 will also remain as above: $\varepsilon_h = 4.3 \times 10^{-4}$ (0.043%).

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 4mm; 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 8mm total predicted settlement of the ground at the level of No.55'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 = 1.4mm, which represents a deflection ratio, $\Delta/L = 1.52 \times 10^{-4}$ (0.015%).

Using the graphs for L/H = 1.5, these deformations once again 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 8 above.

3.9 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.

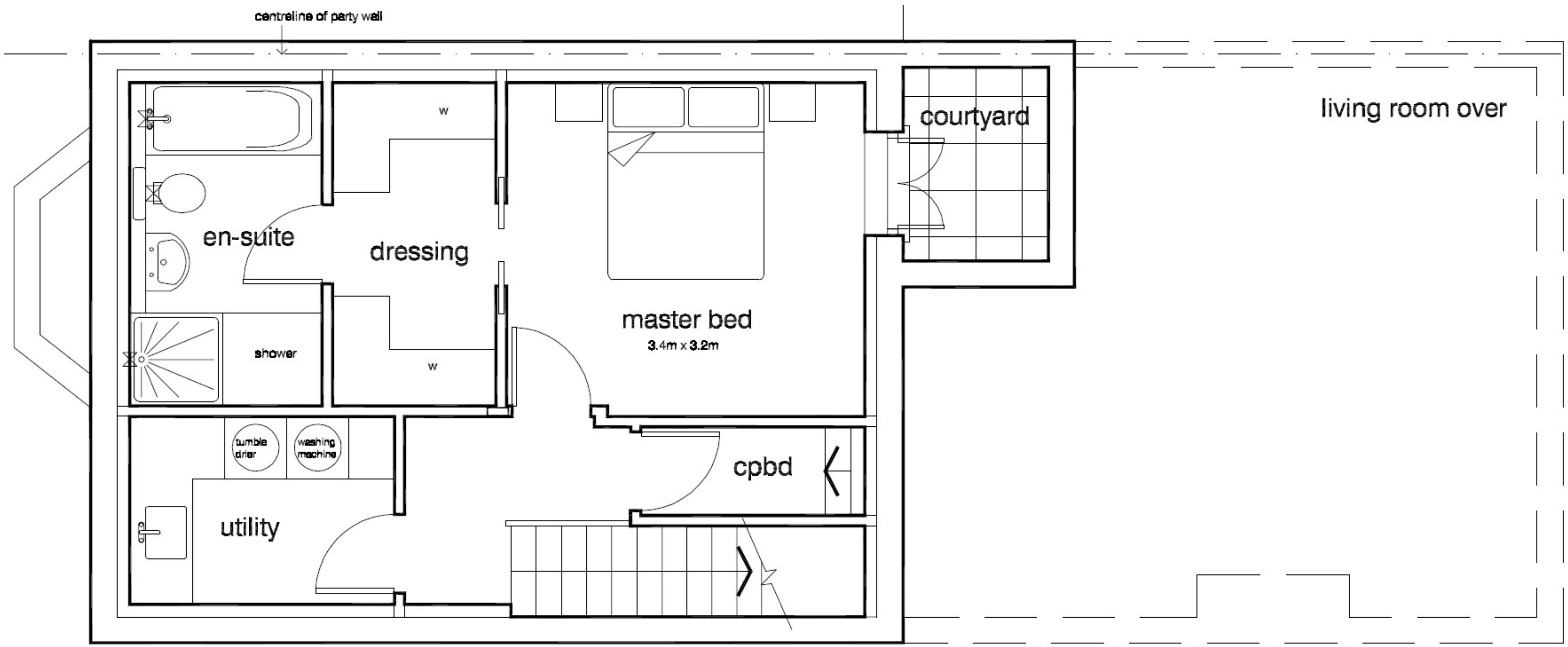

Figure 1. Layout of the proposed basement foundation plan

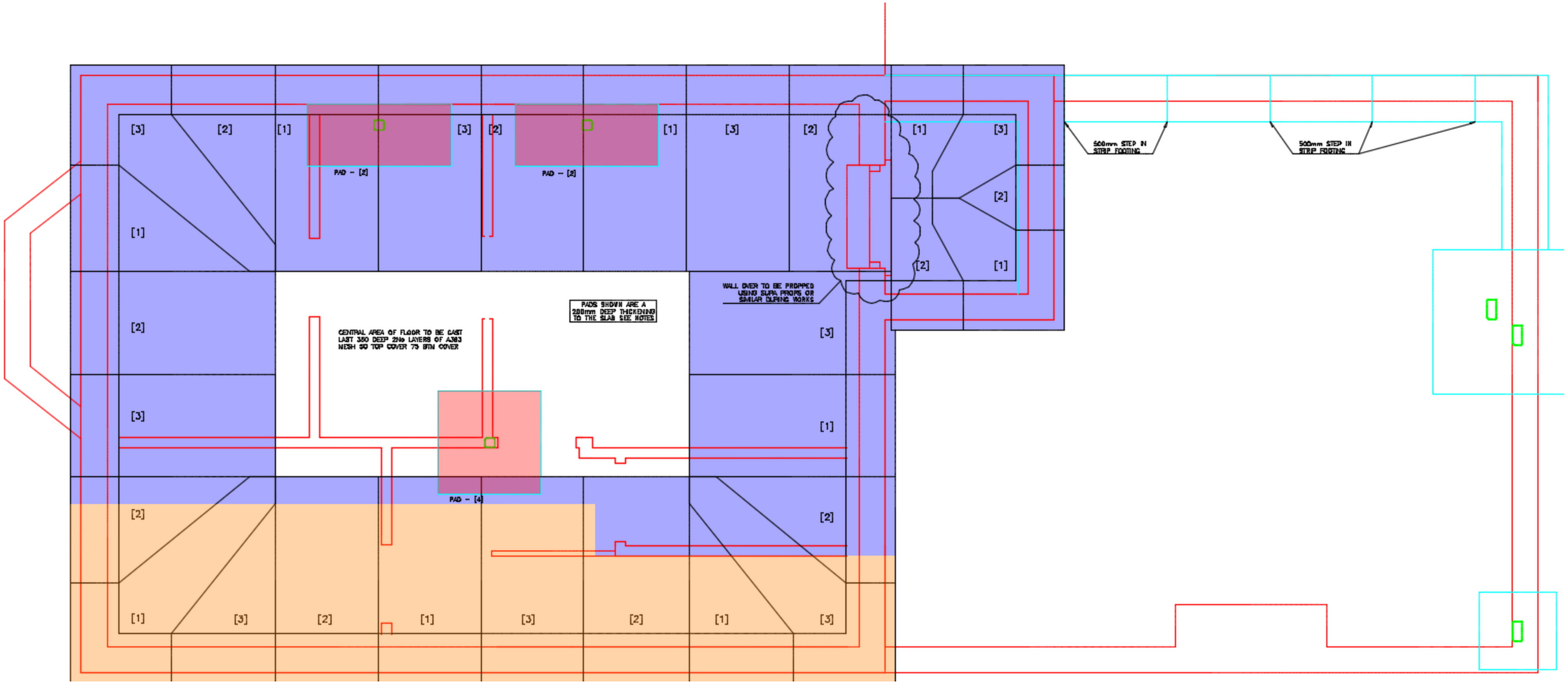

Figure 2. Layout of the underpinning system

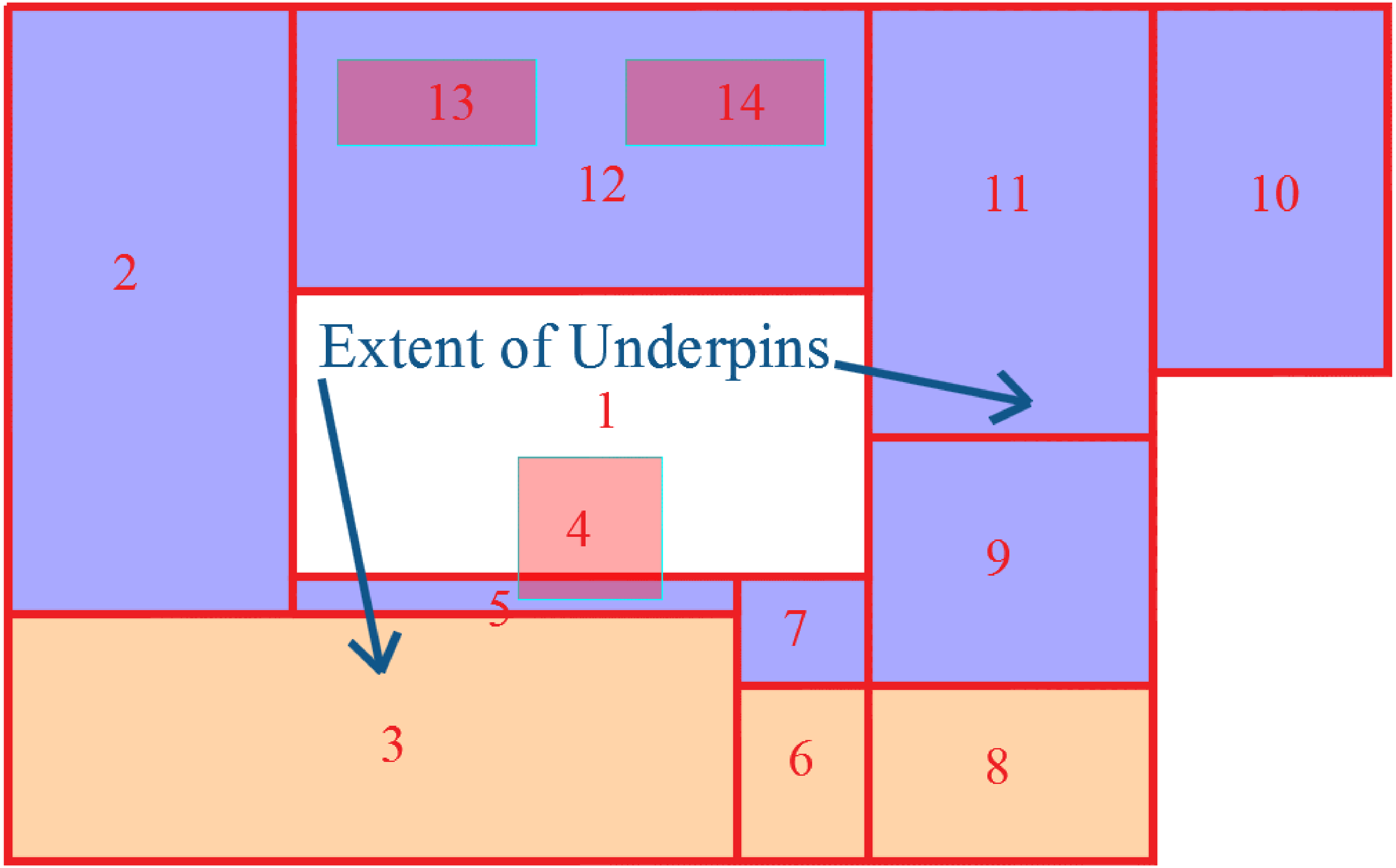

Figure 3. Detail of geometry introduced to PDISP

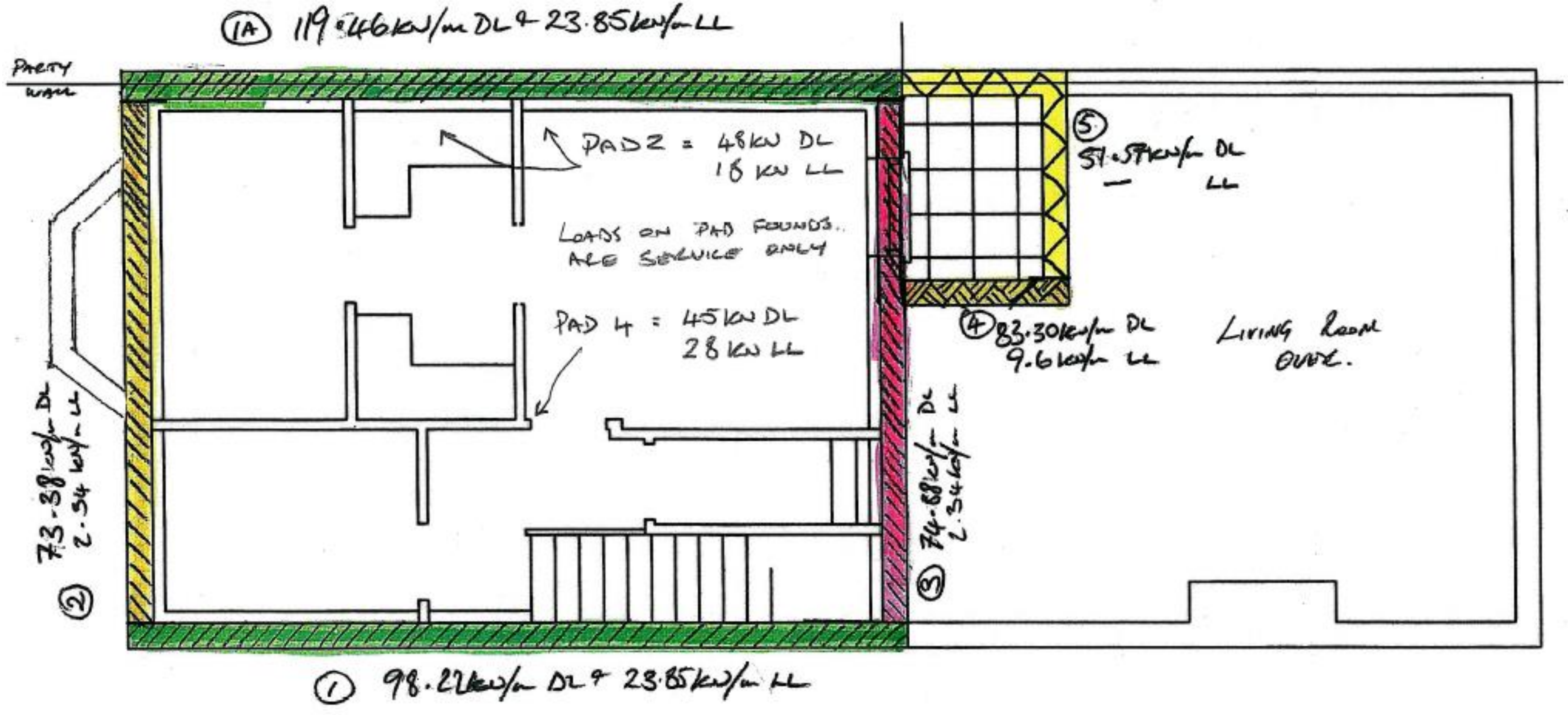

Figure 4. Load Take Down Detail

Chelmer Consultancy Services 'Groundbreaking Services'

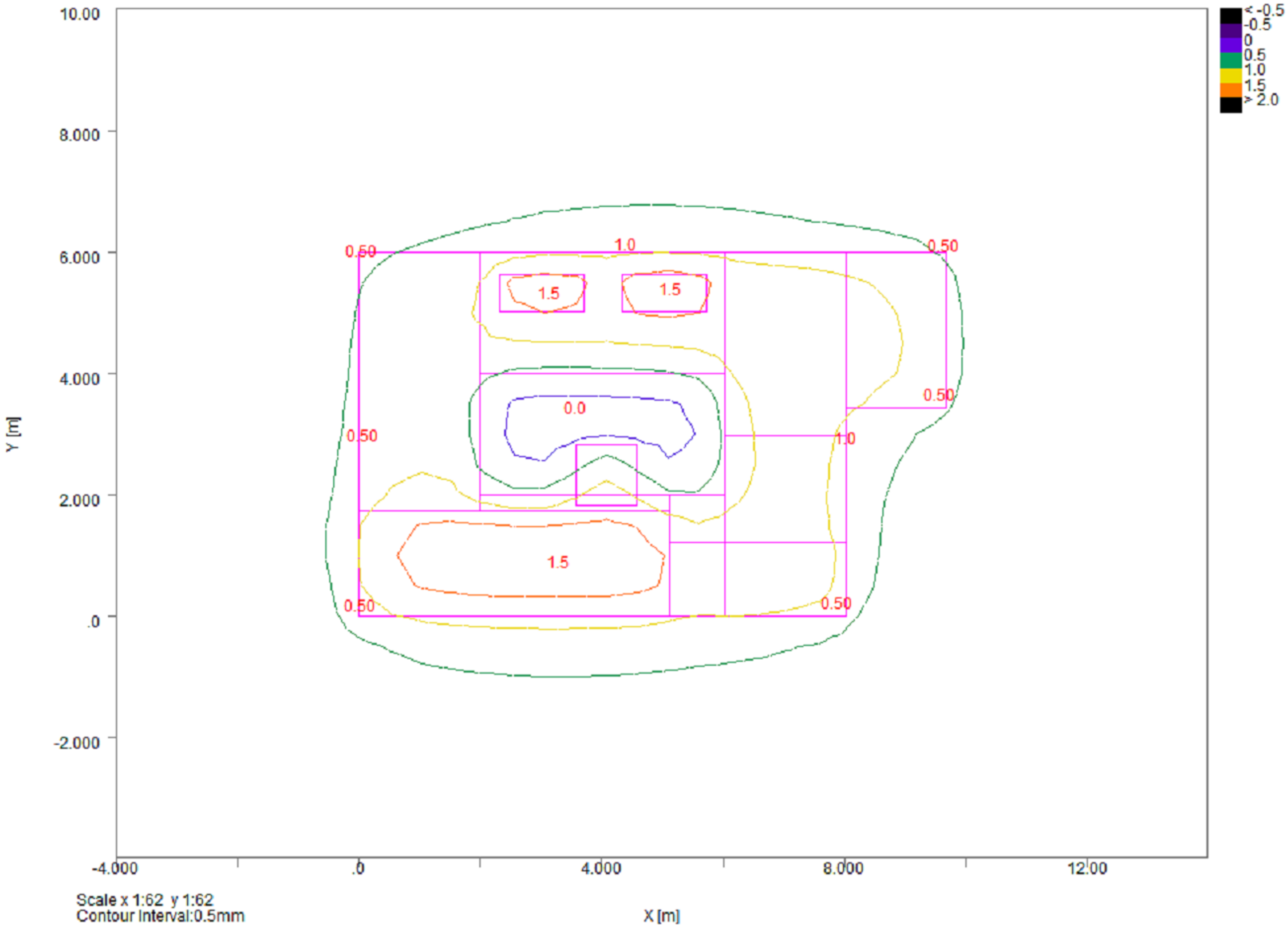

Figure 5. Short term (Stage 2) heave assessment contour

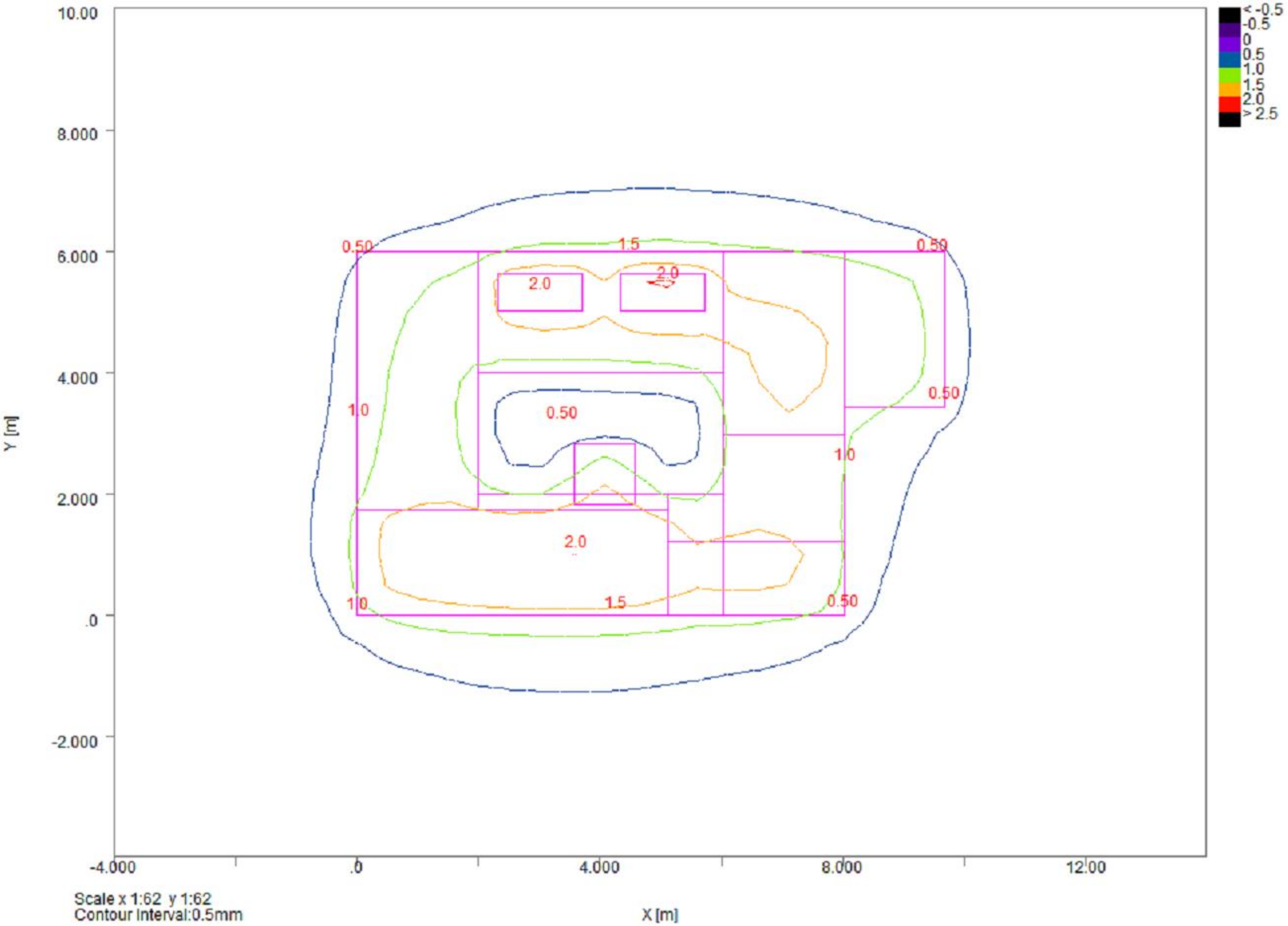

Figure 6. Short term (Stage 3) heave assessment contour

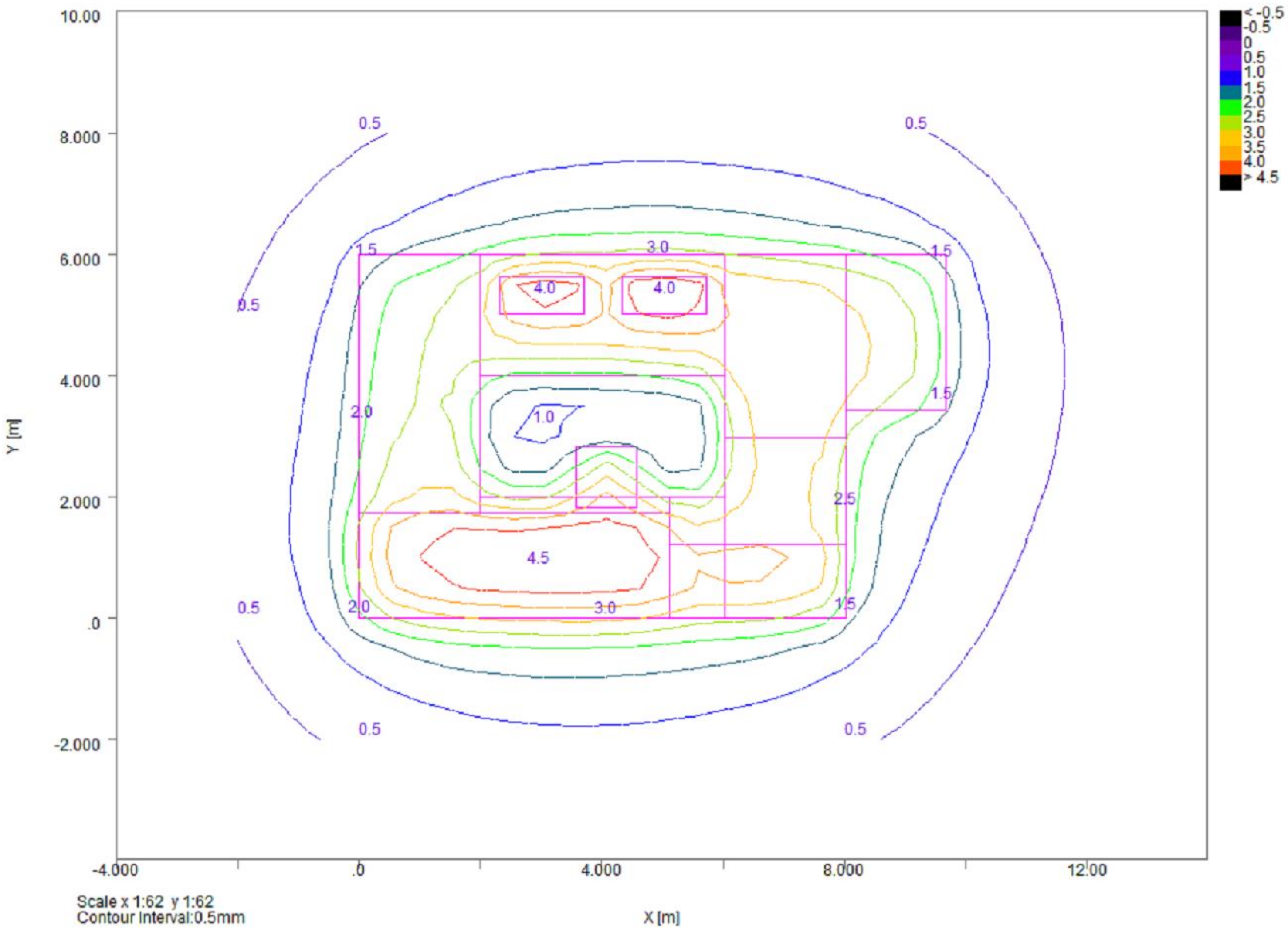

Figure 7. Long term (Stage 4) heave assessment contour