JM13
10 Chantrey Road
London
SW9 9TE

6th May 2016

Ref: BIA/4507f

Dear Sirs,

13-15 John's Mews, London WC1N 2PA – Addendum Letter Report –
Revised Ground Movement & Damage Category Assessments

This letter report is supplementary to, and should be read in conjunction with, our Basement Impact Assessment (BIA) report, Ref: BIA/4507D, dated January 2016. The letter report covers revisions to various aspects of our BIA report which were requested by Louise Turley of FT Architects following a meeting with Gideon Whittington, Camden's planning officer. The three specific issues requested were:

i. *"Details of the groundwater levels most recently taken.*
ii. *"Confirmation that the piled scheme would reduce the ground movements from those previously predicted under the Trevor Scott scheme as we are now proceeding with a piled scheme.*
iii. *"Clarification that the reference to TS drawings on page 35 is a typo."*

Details of the revised method of construction for the temporary works have been provided on Barrett Mahony's revised drawing No.L14771-701-PL3 and in an email from Owen Carroll of Barrett Mahony dated 14th March 2016.

In order to satisfy Item (ii) above, the PDISP analyses have been completely re-done for the revised foundation geometries and revised method of construction proposed by Barrett Mahony (excluding the interim stage, when the loads from the basement and superstructure would be supported by only half the number of piles . As a result, the previous reference to TS Consulting's drawings in Item (iii) above has been superseded by the new Heave/ Settlement Assessment which is presented in Section 2 below, though the load takedown by TS Consulting was still used for the new analyses.

Chelmer Site Investigation Laboratories Ltd
Unit 15 East Hanningfield Industrial Estate, Old Church Road, East Hanningfield, Essex CM3 8AB
Essex: 01245 400930 | London: 0203 6409136 | info@siteinvestigations.co.uk | www.siteinvestigations.com

1. Groundwater

1.1 The latest groundwater level readings are presented in the attached table.

1.2 The additional borehole, BH5, was drilled on 18th August 2015, when a water strike was recorded at 4.5m below ground level (bgl), and the water level rose to 4.2m bgl. The seven subsequent groundwater level readings showed the groundwater level fluctuating between 3.80m and a highest reading of 2.92m bgl on 3rd March 2016, since when it has fallen slightly.

1.3 BH1B was the only original borehole to successfully penetrate through the Made Ground. Sporadic monitoring in that borehole from 30th July 2014 to 10th March 2015 recorded groundwater levels at 3.00-3.39m bgl. This is a smaller range than was recorded in BH5, but this standpipe was not accessible on 2nd and 9th September 2015 when the lowest levels were recorded in BH5. The depths to groundwater were always less in BH5 than in BH1BB, and when the slightly higher ground level in No.15 (BH5) than in No.13 (BH1B) is also taken into account this confirms the relative northwards flow of groundwater, broadly following the topography.

1.4 The maximum and minimum levels during this monitoring period are unlikely to have been recorded, and the previous comments in the paragraphs 10.1.2 of the Basement Impact Assessment (BIA) report remain valid. The recommended provisional design groundwater level at 1.0m bgl also remains unaltered based on these results.

2. Heave/Settlement Assessment

2.1 The revised method of construction for the temporary works was described in the email from Owen Carroll of Barrett Mahony dated 14th March 2016, as follows (with our Stage numbers added):

"With regard to the sequencing there are four conditions:

1. *During underpinning* (and installation of the piles & ground beams – Stage 1) *and bulk excavation* (Stage 2A)*, the superstructure is supported on cantilevering ground beams and temporary piles.*
2. *Once the lower lift of underpinning is complete and the basement excavated, half of the temporary ground beams and piles are broken out* (Stage 2B)*. At this stage, the superstructure is supported both on the underpins and by half of the ground beams and temporary piles.*
3. *The basement slab is cast and allowed to cure, with pockets left around the piles that have not yet been broken down. The slab is connected to the half of the piles that were broken down in (2) above. The other half of the piles are then broken down. At this stage, the superstructure is supported both on the underpins and by half of the piles.*
4. *The pockets are infilled and the superstructure is supported in the permanent case by the underpins and the piles* (Stage 3, short-term, & Stage 4, long-term)*."*

Basement Geometry and Stresses:

2.2 The attached Figure C10 illustrates the proposed basement as given on FT Architects' Drg No.200-32-18. The layout of the proposed underpins and piles is presented in Figure C11; this was compiled from information obtained from the following drawings by Barrett Mahony:

- Drg No. L14771/01-PL1 GA: Lower Ground Floor & Ground Floor Plans
- Drg No. L14771/701-PL3 Temporary Works: Method Statement and Temporary Propping Plans
- Drg No. L14771/702-PL1 Temporary Works: Ground Beam Plan and Stage 1 Section
- Drg No. L14771/703-PL2 Temporary Works: Stage 2 and Stage 3 Section.

2.3 Table 2A presents the net changes in vertical pressure resulting from a combination of the gross unloading from the excavation of the basement, the self-weight of the underpins and the maximum imposed loads from the superstructure, excluding live loads, as given by TS Consulting (see 'Load-01' sheet in our BIA report). An explanation of the stages analysed is given in paragraph 2.7 below.

Gross unloading:

- Depth of excavation = 4.2m (underpins)
  3.9m (basement slab)
- Estimated unit weight, γb = 19.0 kN/m^3.

Basement dimensions:

- 11.8m wide by 12.7m long, excluding strip footings.

Table 2A: Net changes in vertical pressure for PDISP Zones						
ZONE	**Net change in vertical pressure (kPa)**					
#	**Stage 1**	**Stage 1(20%)**	**Stage 2a**	**Stage 2a(20%)**	**Stages 3 and 4**	**Stages 3P and 4P**
1	10.19	42.71	-14.51	18.01	4.09	2.04
2	10.19	48.46	-14.51	23.76	4.09	2.04
3	10.19	43.98	-14.51	19.28	4.09	2.04
4a	353.96	287.85	428.06	361.95	0.00	49.15
5a	353.96	287.85	428.06	361.95	0.00	49.15
6a	353.96	287.85	428.06	361.95	0.00	49.15
7a	353.96	287.85	428.06	361.95	0.00	49.15
8a	353.96	287.85	428.06	361.95	0.00	49.15
9a	353.96	287.85	428.06	361.95	0.00	49.15
10a	236.05	193.52	310.15	267.62	0.00	49.15
11a	236.05	193.52	310.15	267.62	0.00	49.15
12a	236.05	193.52	310.15	267.62	0.00	49.15
13a	236.05	193.52	310.15	267.62	0.00	49.15
14a	236.05	193.52	310.15	267.62	0.00	49.15
15a	236.05	193.52	310.15	267.62	0.00	49.15
16a	236.05	193.52	310.15	267.62	0.00	49.15
17a	236.05	193.52	310.15	267.62	0.00	49.15
18a	236.05	193.52	310.15	267.62	0.00	49.15
19a	236.05	193.52	310.15	267.62	0.00	49.15
20a	236.05	193.52	310.15	267.62	0.00	49.15
21a	236.05	193.52	310.15	267.62	0.00	49.15

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Table 2A (cont'd): Net changes in vertical pressure for PDISP Zones						
ZONE	Net change in vertical pressure (kPa)					
#	Stage 1	Stage 1 (20%)	Stage 2a	Stage 2a (20%)	Stages 3 and 4	Stages 3P and 4P
22a	353.96	287.85	428.06	361.95	0.00	49.15
23a	353.96	287.85	428.06	361.95	0.00	49.15
24a	353.96	287.85	428.06	361.95	0.00	49.15
25a	353.96	287.85	428.06	361.95	0.00	49.15
26a	353.96	287.85	428.06	361.95	0.00	49.15
27a	353.96	287.85	428.06	361.95	0.00	49.15
28	0.00	0.00	-74.10	-74.10	4.09	2.04
29	0.00	0.00	-74.10	-74.10	4.09	2.04
30	10.19	22.36	-14.51	-2.34	4.09	2.04
31	10.19	22.36	-14.51	-2.34	4.09	2.04
32	10.19	25.68	-14.51	0.98	4.09	2.04
33	10.19	25.68	-14.51	0.98	4.09	2.04
4b,c,d	342.56	276.45	342.56	276.45	0.00	49.15
5b,c,d	342.56	276.45	342.56	276.45	0.00	49.15
6b,c,d	342.56	276.45	342.56	276.45	0.00	49.15
7b,c,d	342.56	276.45	342.56	276.45	0.00	49.15
8b,c,d	342.56	276.45	342.56	276.45	0.00	49.15
9b,c,d	342.56	276.45	342.56	276.45	0.00	49.15
10b,c,d	224.65	182.12	224.65	182.12	0.00	49.15
11b,c,d	224.65	182.12	224.65	182.12	0.00	49.15
12b,c,d	224.65	182.12	224.65	182.12	0.00	49.15
13b,c,d	224.65	182.12	224.65	182.12	0.00	49.15
14b,c,d	224.65	182.12	224.65	182.12	0.00	49.15
15b,c,d	224.65	182.12	224.65	182.12	0.00	49.15
16b,c,d	224.65	182.12	224.65	182.12	0.00	49.15
17b,c,d	224.65	182.12	224.65	182.12	0.00	49.15
18b,c,d	224.65	182.12	224.65	182.12	0.00	49.15
19b,c,d	224.65	182.12	224.65	182.12	0.00	49.15
20b,c,d	224.65	182.12	224.65	182.12	0.00	49.15
21b,c,d	224.65	182.12	224.65	182.12	0.00	49.15
22b,c,d	342.56	276.45	342.56	276.45	0.00	49.15
23b,c,d	342.56	276.45	342.56	276.45	0.00	49.15
24b,c,d	342.56	276.45	342.56	276.45	0.00	49.15
25b,c,d	342.56	276.45	342.56	276.45	0.00	49.15
26b,c,d	342.56	276.45	342.56	276.45	0.00	49.15
27b,c,d	342.56	276.45	342.56	276.45	0.00	49.15

Ground Conditions:

2.4 The ground profile was based on the site-specific ground investigation by Chelmer Site Investigations, as presented in Section 9 of the BIA, and the desk study information.

2.5 The geotechnical soil properties adopted for the PDISP analyses are summarized in Table 3A below, based on the log of borehole BH1B drilled by CSI and our previous experience of basement projects in the London Clay. BH5 encountered less Made Ground, extending to only 3.0m below ground level (bgl) though this Made Ground was loose to very loose and included voids (one SPT gave a zero blowcount). Soft, apparently re-worked, alluvial-type clays were recorded beneath the Made Ground, to a depth of 3.5m. While, at the location of BH5, all the very weak Made Ground and soft clay will be removed by the excavations (such that the basement will be founded on apparently in-situ River Terrace Deposits), it is possible that similar materials will extend to greater depth elsewhere.

Table 3A: Soil parameters for PDISP analyses

Strata	**Level** (m bgl)	**SPT blowcount** N	**Short term, undrained Young's Modulus,** Eu (MPa)	**Long term, drained Young's Modulus,** E' (MPa)
Made Ground	3.8-5.9	17	35	20
London Clay	5.9 27.5	20	40 120	25 70

Where:
Drained Young's Modulus = 2 x N
London Clay: Undrained shear strength, Cu assumed = 80kPa at 5.9m bgl
Eu = 500 * Cu Hence profile of Eu = 40 + 3.75z
Drained Young's Modulus was estimated based on E' = 0.6 Eu
where z = depth below the top of the London Clay stratum.

PDISP Assessment:

2.6 Three dimensional 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 and underpinning of the relevant walls. These analyses used the basement geometry, loads/stresses and ground conditions outlined above. The stages analysed by the initial PDISP analyses are identified within Barrett Mahony's construction sequence in paragraph 2.1 above.

2.7 The piles were sub-divided into four sections, with the upper section extending to the founding level of the basement slab (Zones 4a to 27a), and three equal, 2.0m long sections below the basement (Zones 4b, 4c, 4d to 27b, 27c, 27d). At bottom of each of these zones the piles were represented by circular footings with diameters slightly greater than the piles in order to allow for the load-shedding into the ground around each pile.

2.8 Various issues were encountered in undertaking these analyses owing to the complexity of the proposed scheme. The additional stages modelled and the issues encountered are summarised below:

- **Stage 1:** The first analysis showed the piled superstructure settling much more than the underpins, which was clearly unrealistic as the superstructure would force the underpins to settle by a similar amount. Only single stage underpinning to full depth has been modelled (see paragraph 4.5).
- **Stage 1 (20%):** An arbitrary 20% of the load from the superstructure was then transferred to the underpins. This reduced the settlement of the piles to 4-8mm. Piles are normally designed for up to 10mm of settlement, and some piling companies will design piles for a 5mm tolerance, so this 20% transfer was considered a reasonable estimate.
- **Stage 2a (20%):** The same 20% load transfer was applied to this stage, which modelled the bulk excavation of the ground prior to casting the basement slab.
- **Stage 2b:** An attempt was made to model the progressive construction of the base slab, with the slab sub-divided into six strips. As PDISP doesn't model soil-structure interaction a lengthy iterative process would be required to get close to a realistic result for this complex load transfer (or a finite element program should be used). We also consider that the construction sequence requires further refinement, as described in paragraph 4.4 below, which makes further analysis of the current proposal difficult to justify.
- **Stage 3 & 3P** (Short-term conditions)**:** At the completion of the basement slab, the building will be supported on a piled raft. The distribution of loads/bearing pressures will be complex, so analyses have been undertaken of the two extreme scenarios: with the basement slab acting only as a raft footing, and with the basement supported fully on the 24 piles (Stage 3P).
- **Stage 4 & 4P** (Long-term conditions)**:** The loads and bearing pressures for this stage were identical to those for Stage 3, except that long-term soil strength parameters were used.

2.9 The results of the analyses for Stages 1(20%), 2a(20%), 3P and 4P are presented as contour plots on the attached Figures C12 to C15.

2.10 The analyses indicated that the piles and underpins will initially undergo settlement, followed by modest heave movements when the main central basement is excavated. At Stage 2B (not presented, for the reasons given above) the analysis indicated that the movements would switch back to larger settlements, but once the basement structure has been completed only very small settlements were predicted. The ranges of predicted short-term and long-term movements for each of the main walls are presented in Table 4A below. These values are approximate, so should be used as a general guide to possible movements rather than definitive values, and will be influenced by the design of the piles, which should be specified to restrict settlements to not more than 5mm.

<table>
<tr><th colspan="5">Table 4A: Summary of predicted displacements</th></tr>
<tr><th>Location</th><th>Stage 1, 20%
(Figure C12)</th><th>Stage 2a, 20%
(Figure C13)</th><th>Stage 3P
(Figure C14)</th><th>Stage 4P
(Figure C15)</th></tr>
<tr><td>Front wall
(Wall D):</td><td>2 – 4mm
Settlement</td><td>0 – 4mm Heave</td><td rowspan="4">0 – 0.5mm
Settlement</td><td rowspan="4">0.5 – 1mm
Settlement</td></tr>
<tr><td>15/17 Party Wall
(Wall B):</td><td>2 – 4mm
Settlement</td><td>0 – 3mm Heave</td></tr>
<tr><td>Rear wall
(Wall C):</td><td>2 – 5mm
Settlement</td><td>0 – 4mm Heave</td></tr>
<tr><td>11/13 Party Wall
(Wall A):</td><td>2 – 4mm
Settlement</td><td>0 – 3mm Heave</td></tr>
<tr><td>Central wall
(Wall E):</td><td>3 – 6mm
Settlement</td><td>1 – 5mm Heave</td><td>0.5 – 1mm
Settlement</td><td>0.5 – 1.5mm
Settlement</td></tr>
<tr><td>Central basement area /
slab:</td><td>2 – 7mm
Settlement</td><td>3 – 9mm Heave</td><td>0.5 – 1.5mm
Settlement</td><td>0.5 – 2.5mm
Settlement</td></tr>
<tr><td>Piles:</td><td>4 – 8mm
Settlement</td><td>1 – 6mm Heave</td><td>0.5 – 1.5mm
Settlement</td><td>0.5 – 2.5mm
Settlement</td></tr>
</table>

2.11 When the analyses for Stages 3 & 4 were re-run excluding the piles, the predicted settlements decreased to less than 1.0mm for Stage 3 and less than 1.3mm for Stage 4, though these do not allow for the possible presence of very weak Made Ground or soft clays below the basement, or for the varying buoyant uplift forces that the groundwater fluctuations would cause, which were the original reason for recommending the use of piled foundations. In practice, the actual settlements are expected to be between those for the piled scenario and those that would occur if the basement were to be built without any piles (though the latter is NOT recommended).

2.12 The rate of plastic consolidation (and swelling) will be determined largely by the availability of water and as a result, given the low permeability of the London Clay, can take many years to reach full equilibrium. Normally all the short-term, elastic ground movement would have occurred before the concrete of the basement slab has cured, such that only the post-construction incremental heave/settlement is relevant to the slab, though in this case the progressive transfer of loads from the temporary works to the permanent structure would result in additional elastic movements after the slab has been cast. So the differential displacements experienced by the slab after it has cured will be larger than the difference between the displacements at Stages 3 & 4, and possibly in the order of up to 8mm.

2.13 The implications from these analyses are considered further in Section 4.

3. Damage Category Assessment

3.1 A damage category assessment has been requested only in relation to No.23 John Street, the rear garden to which adjoins the rear of No.13 John's Mews. (A damage category assessment for the adjoining properties was presented in paragraphs 10.4.8 to 10.4.10 of the BIA report.)

3.2 In order to relate the predicted vertical displacements from the PDISP analyses and the typical horizontal ground movements to possible damage which adjoining/ 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 For damage category assessments we are interested in the ground movements at the foundation levels of the neighbouring buildings, so it is the depth of excavation below foundation level that must be considered. Separate damage category assessments are required for each combination of foundation level, depth of excavation, geology, predicted heave/settlement (from the PDISP analyses), distance of adjacent structure from the proposed basement, and height to length ratio of the adjacent structure.

3.4 The ground level to the rear of No.13 is understood to be about 1m above the internal floor level, however No.23 John Street (23JS) has a lower ground floor set a full storey below ground level. Thus, 23JS is probably founded about 2m below the internal floor level in No.13, which means that the underpins for the proposed basement will be founded approximately 2.2m below 23JS's footings. The rear wall of 23JS's rear addition is approximately 8.5m from No.13's rear wall (scaled from the OS map) however 23JS is understood to have a rear vault. No reliable scaled drawing is available, but it is estimated that the vault extends to about 5m from No.13.

3.5 The Made Ground found beneath No's 13 & 15 was predominantly granular, as were the River Terrace Deposits in BH5. Figure 2.12 in CIRIA Report C580 shows that settlement in granular strata in response to excavation in front of a retaining wall typically extends no more than a distance equal to twice the depth of the excavation. Twice 2.2m is less than 5m so lateral relaxation of the ground alongside the basement is not expected to affect any part of No.23 John Street.

3.6 The worst case scenario identified by the PDISP analyses for 23JS occurred in Stage 1, when the 1mm settlement contour extended just over 1m from the proposed basement. The zero contour was not present within 4m from No.13, so it is estimated that no more than 0.5mm of settlement would occur beneath the nearest part of 23JS's rear vault. No visible damage is expected in No.23 John Street as a result of such movement, so this represents a damage category of 'negligible' (Burland Category 0, ε_{lim} = <0.05%), as given in CIRIA SP200 Table 3.1.

3.7 If the progressive transfer of load onto parts of the basement slab, as currently proposed is implemented, then greater displacements beneath No.23 Street would be expected, and a finite element analysis of the piled raft would be recommended to assess more reliably the displacements at and beyond the rear of the proposed basement.

4. Conclusions

4.1 These conclusions consider only the primary findings of this assessment; the whole letter report should be read to obtain a full understanding of the matters considered.

4.2 The additional groundwater monitoring has confirmed that the groundwater level is currently fluctuating between at least 2.9m and 3.8m bgl. Thus, the provisional design groundwater level at 1.0m bgl, as recommended in the BIA report, remains appropriate.

4.3 The PDISP analyses indicated that the piles and underpins will initially undergo settlement, followed by modest heave movements when the main central basement is excavated. If the construction method proposed by Barrett Mahony is followed, with removal of the temporary support to half the superstructure before the basement slab is structurally connected to all the piles (Stage 2B), then the movements would switch back to settlements, potentially larger than in Stage 1, the magnitude of which will depend primarily on the condition of the Made Ground (where present beneath some parts of the basement), the complex interaction between the partially complete basement, the variable condition of the underlying soils, and the ultimate capacity of the piles. Once the basement structure has been completed, however, only very small settlements of less than 2.5mm were predicted. These values are approximate, so should be used as a general guide to possible movements rather than definitive values, and will be influenced by the design of the piles, which should be specified to restrict settlements to not more than 5mm.

4.4 The revised scheme, with the superstructure supported on the temporary ground beams while the underpins are constructed, is an improvement on the previous scheme, but the complex transfer of loads onto the underpins and only 50% of the piles, while some underpins are founded on narrow footings in Made Ground of variable consistency, leaves the possibility of much larger movements being induced than have been indicated by these analyses. It is therefore recommended that the basement slab should be cast and allowed to cure, with full structural connections to all 24 piles, **before** the temporary support is removed from the superstructure.

4.5 The lack of a footing to the first lift of underpins increases the risk that they may settle away from the supported superstructure before the 2nd lift is cast. The presence of loose to very loose (SPT 'N' values from zero to 7) and possibly voided Made Ground (as recorded in BH5) increases further the risk that the two-stage underpinning currently proposed would experience unacceptably large settlements. The deeper Made Ground in BH1B gave medium dense SPT blowcounts, though may be more variable than has been proven to date, and may be underlain by soft alluvial clays, so single stage underpinning would be preferable; the condition of the formation would still need to be inspected very carefully, with local probing to check for the presence of any deeper soft alluvial clays, in order to confirm the adequacy of the short-term bearing capacity for the underpins.

Please contact us if any clarification is required of the matters discussed above.

Yours faithfully

K.R. Gabriel

Keith Gabriel
MSc DIC CGeol FGS
UK Registered Ground Engineering Adviser

Encs: - Groundwater monitoring results (on Landborne Gas Assessment sheet)
- Figures C10 to C15

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.
h) The assessments made in this report are based on the ground conditions as revealed by walkover survey and/or intrusive investigations, together with the results of any field or laboratory testing or chemical analysis undertaken and other relevant data, which may have been obtained including previous site investigations. In any event, ground contamination often exists as small discrete areas of contamination (hot spots) and there can be no certainty that any or all such areas have been located and/or sampled.
i) There may be special conditions appertaining to the site, which have not been taken into account in the report. The assessment may be subject to amendment in light of additional information becoming available.
j) Where any data supplied by the client or from other sources, including that from previous site investigations, have been used it has been assumed that the information is correct. No responsibility can be accepted by CSI for inaccuracies within the data supplied by other parties.
k) Whilst the report may express an opinion on possible ground conditions between or beyond trial pit or borehole locations, or on the possible presence of features based on either visual, verbal or published evidence this is for guidance only and no liability can be accepted for the accuracy thereof.
l) Comments on groundwater conditions are based on observations made at the time of the investigation unless otherwise stated. Groundwater conditions may vary due to seasonal or other effects.
m) This report is prepared and written in the context of the agreed scope of work and should not be used in a different context. Furthermore, new information, improved practices and changes in legislation may necessitate a reinterpretation of the report in whole or part after its original submission.
n) The copyright in the written materials shall remain the property of the CSI but with a royalty-free perpetual license to the client deemed to be granted on payment in full to CSI by the client of the outstanding amounts.
o) These terms apply in addition to the CSI Standard Terms of Engagement (or in addition to another written contract which may be in place instead thereof) unless specifically agreed in writing. (In the event of a conflict between these terms and the said Standard Terms of Engagement the said Standard Terms of Engagement shall prevail). In the absence of such a written contract the Standard Terms of Engagement will apply.
p) This report is issued on the condition that CSI will under no circumstances be liable for any loss arising directly or indirectly from subsequent information arising but not presented or discussed within the current Report.
q) In addition CSI will not be liable for any loss whatsoever arising directly or indirectly from any opinion within this report.

Chelmer Consultancy Services 'Groundbreaking Services'

Landborne Gas Assessment

Site Ref: 4507F
Site Name: 13-15 John Mews, London, WC1N 2PA

Well	Date	Methane Peak	Methane Steady	Methane GSV	Carbon Dioxide Peak	Carbon Dioxide Steady	Carbon Dioxide GSV	Oxygen	Atmos.	Flow	Response Zone	Depth to Water	CO	H2S
		%v/v	%v/v	l/hr	%v/v	%v/v	l/hr	%v/v	mbar	l/hr	m bgl	m bgl	ppm	ppm
BH5	02.09.15	-	-		-	-		-	-	-	1.00-12.00	3.80	-	-
	09.09.15	0.0	0.0	0.0000	4.2	3.9	0.0210	14.2	1020	0.5		3.60	0	0
	14.09.15	-	-		-	-		-	-	-		3.48	-	-
	22.09.15	-	-		-	-		-	-	-		3.35	-	-
	20.01.16	0.1	0.1	0.0002	2.0	2.0	0.0040	19.4	1021	0.2		2.95	0	0
	03.03.16	-	-		-	-		-	-	-		2.92	-	-
	10.03.16	-	-		-	-		-	-	-		3.00	-	-
BH1B	30.07.14	0.1	<0.1	0.0002	0.1	0.1	0.0040	20.5	1015	0.5	1.00-8.00	3.39	0	0
	10.08.14	0.1	0.1	0.0002	1.1	0.8	0.0040	18.8	997	0.6		3.27	0	0
	14.09.15	-	-		-	-		-	-	-		3.18	-	-
	22.09.15	-	-		-	-		-	-	-		3.13	-	-
	20.01.15	0.1	0.1	0.0003	0.1	0.1	0.0003	21.2	1021	0.3		3.00	0	0
	03.03.16	-	-		-	-		-	-	-		3.09	-	-
	10.03.16	-	-		-	-		-	-	-		3.20	-	-

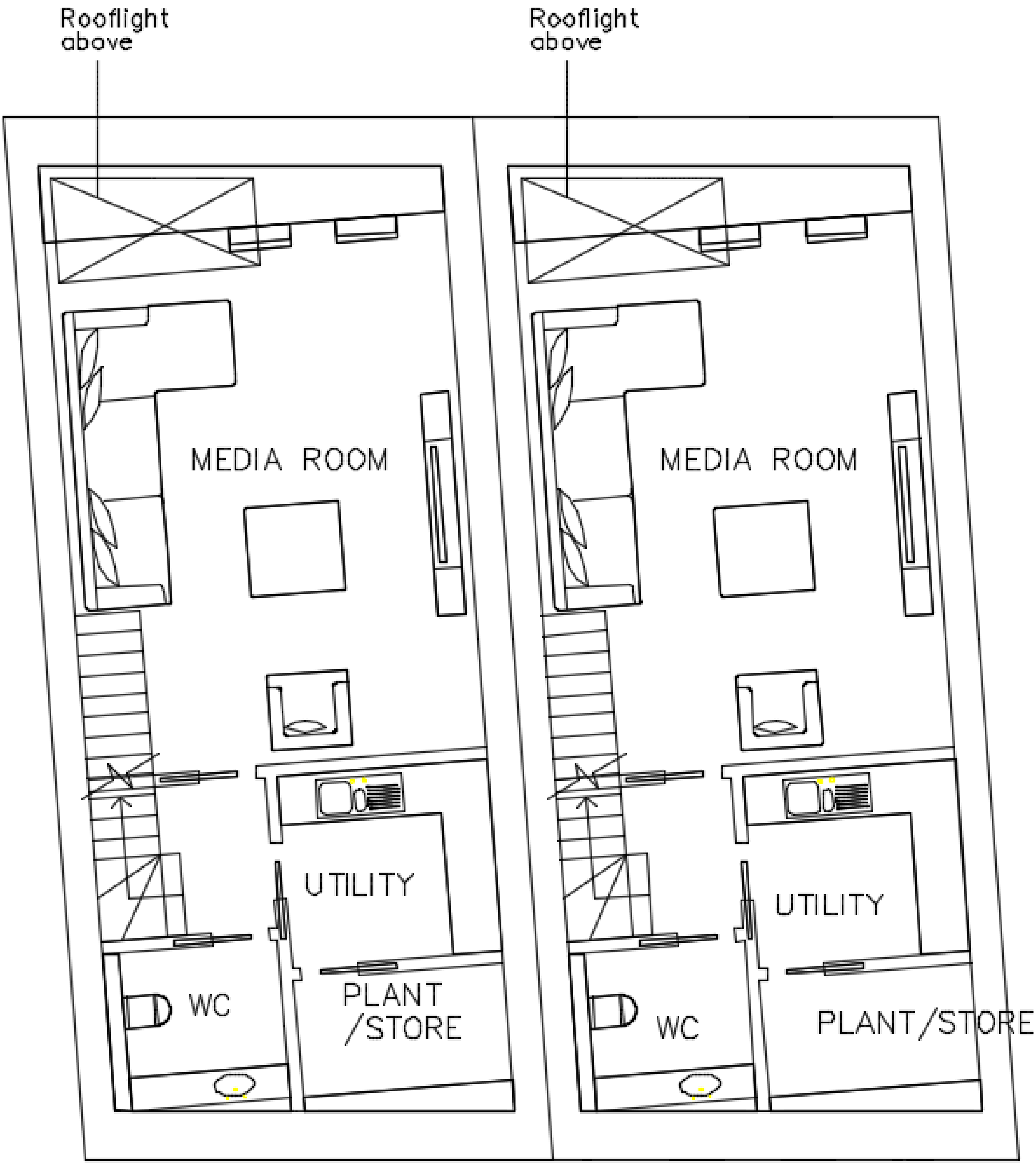

Figure C10. Layout of the proposed basement

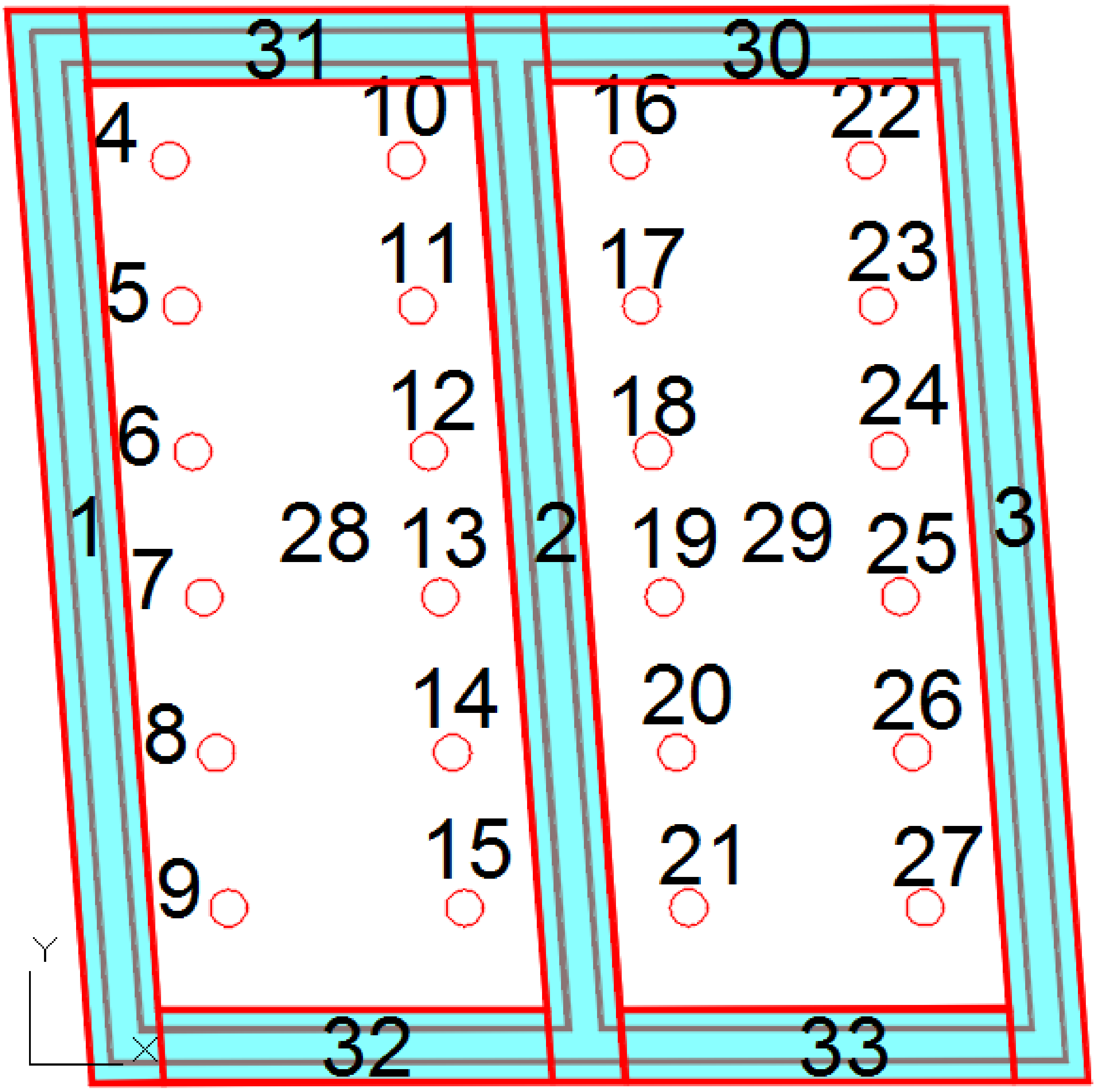

Figure C11. Detail of geometry introduced to PDISP

Chelmer
Consultancy Services
'Groundbreaking Services'

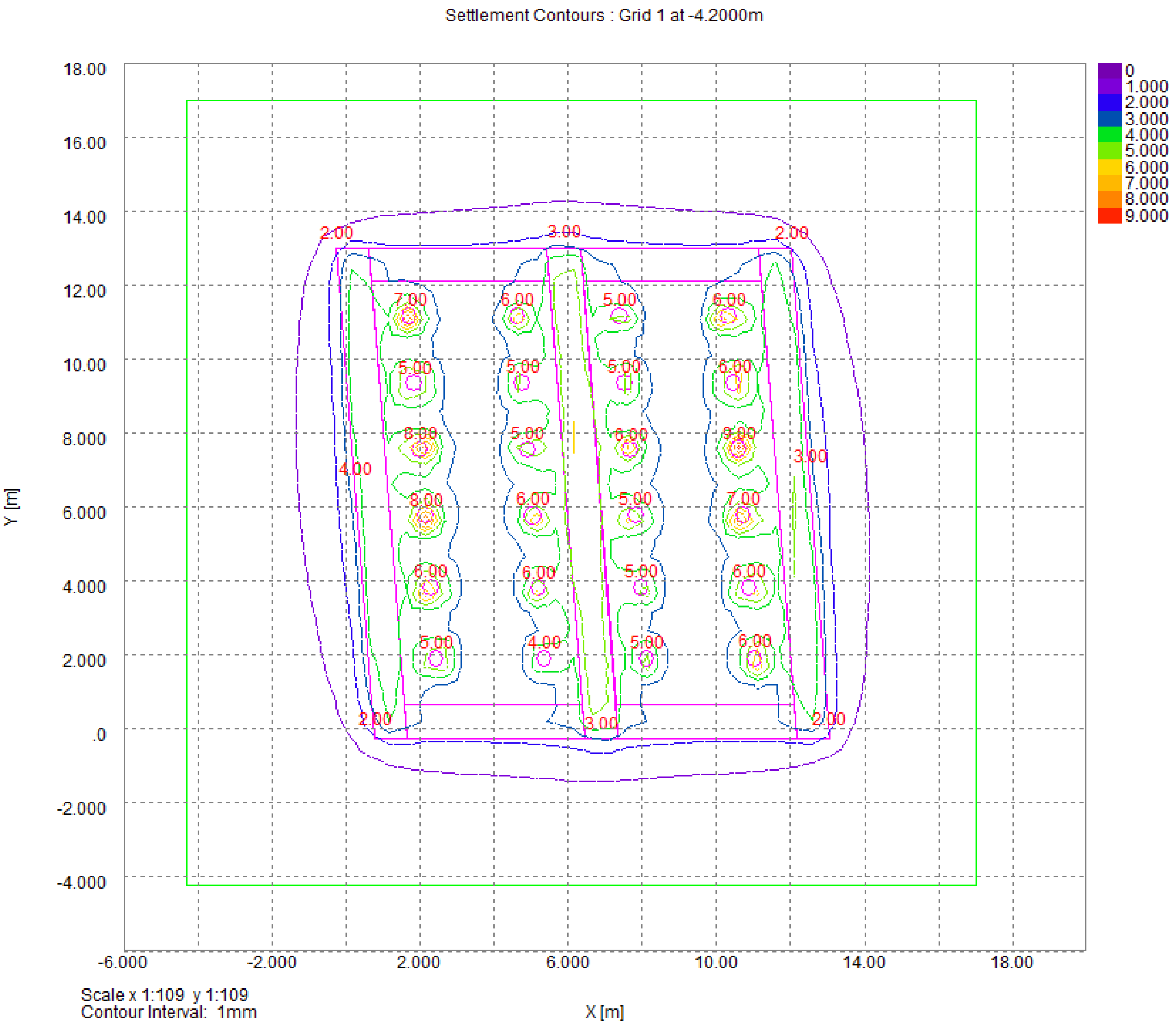

Figure C12. Short term (Stage 1) heave assessment contour

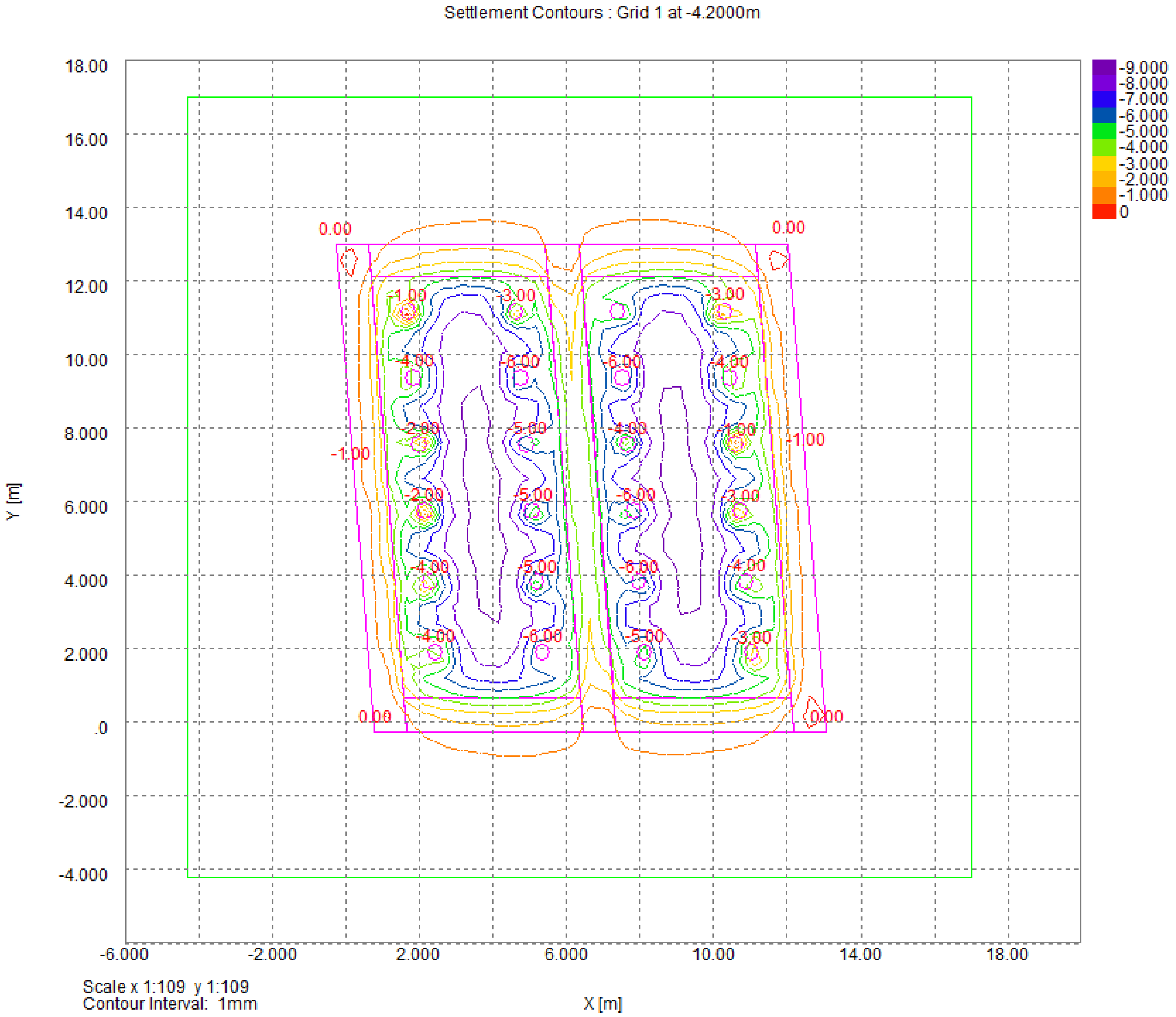

Figure C13. Short term (Stage 2) heave assessment contour

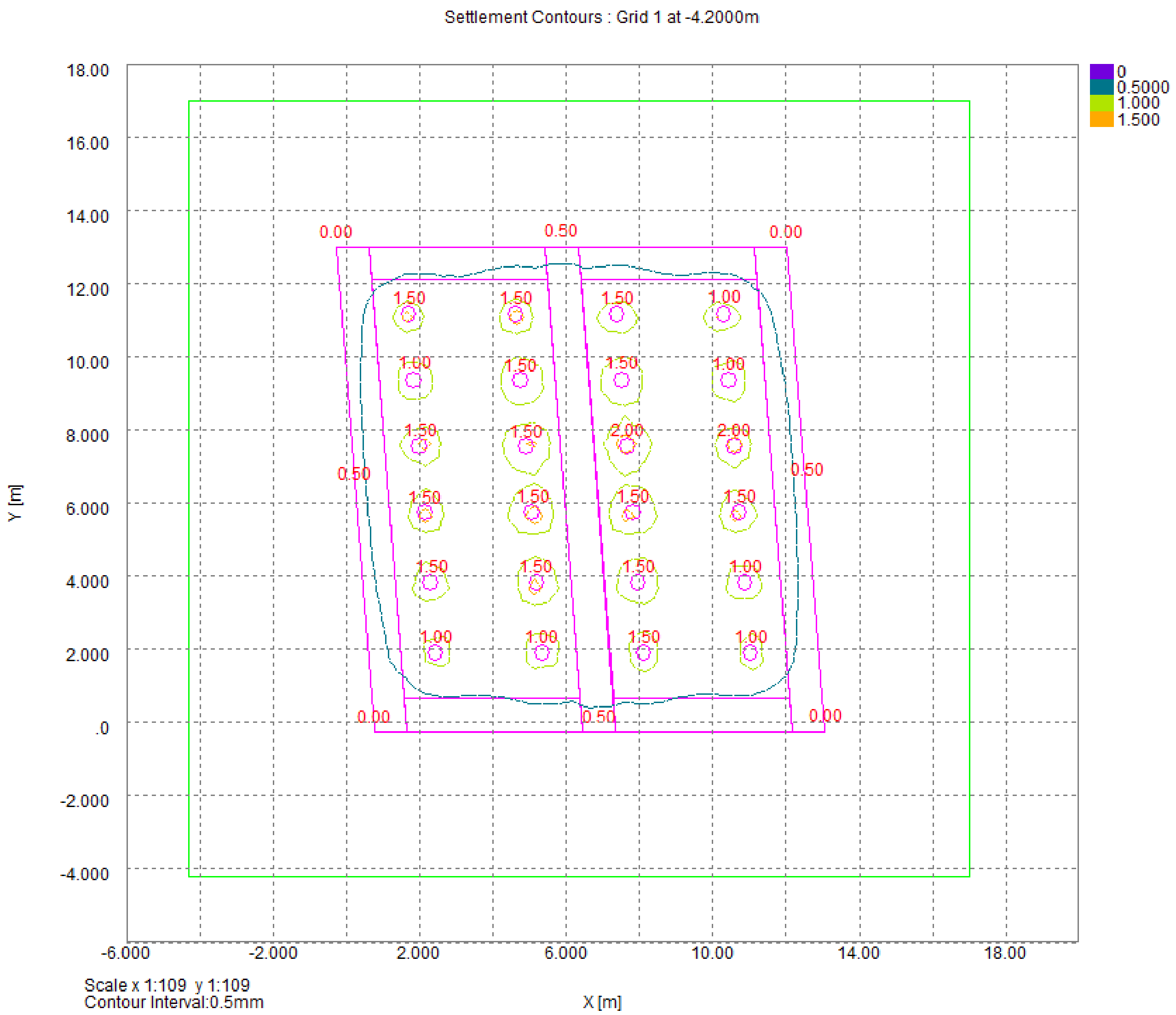

Figure C14. Short term (Stage 3) heave assessment contour

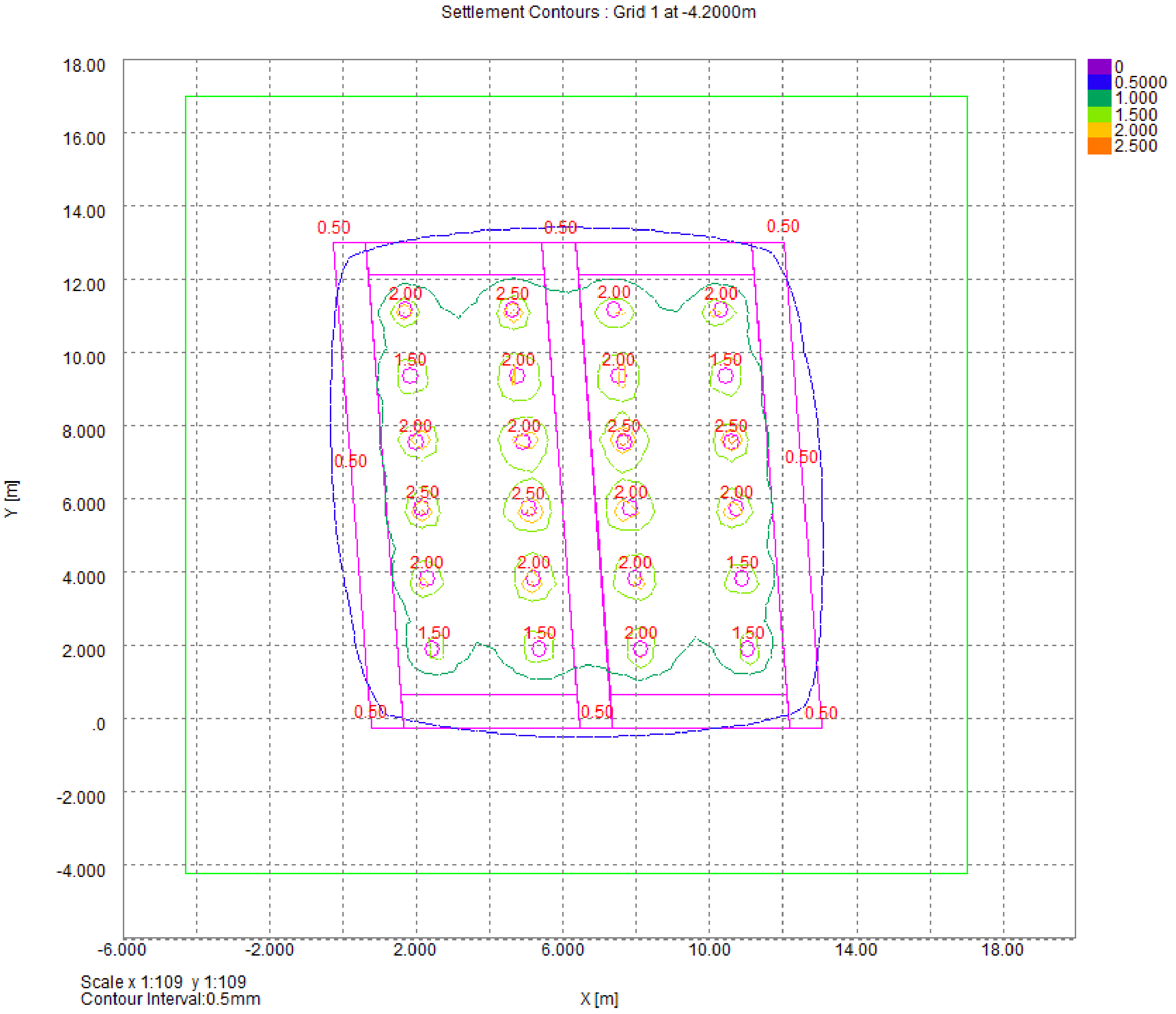

Figure C15. Long term (Stage 4) heave assessment contour