1.0 Introduction

In connection with the proposal to redevelop No 4a Wadham Gardens, London NW3 3DP, by the construction of a new basement and first floor, Applied Geotechnical Engineering Ltd (AGE) has been instructed by Site Analytical Services Ltd (SAS), on behalf of their client, to provide information on the effect of basement construction on the neighbouring properties at Nos 4 and No 6 Wadham Gardens. The structural engineer for the project is Elliott Wood. A plan of the proposed basement of the property is given below in Figure 1.

It is understood that parts of the existing building are to be retained, but during construction the front wall, the internal structure and the roof will be removed. The basement is to be constructed beneath the full building footprint, and to extend a maximum of approximately 3m out beneath the front drive of the property. The construction of the basement will involve excavation from a current (external) ground level of approximately 50.2mOD to allow a proposed finished floor level of 46.7mOD (3.5mbgl).

An existing basement is in place beneath No 4 Wadham Gardens. This has a finished floor level of 46.7mOD (3.5mbgl) over the front part where it lies adjacent to No 4a; over the rear part there is a swimming pool, which will have a deeper floor level. No 4a constitutes an annexe to No 4, which lies to the right, both buildings being under the same ownership. The two structures are currently connected at ground floor level and, following the proposed works, will also be connected at basement and first floor levels.

Right and left are as viewed from the front of the property on Wadham Gardens.

The excavation will be undertaken within reinforced concrete underpins along the front, rear and left walls, the existing underpinning to the No 4 basement will form the right wall. The founding level of the proposed underpins is 45.65mOD (4.55m depth). The general basement excavation will be to 45.85mOD (4.35m depth).

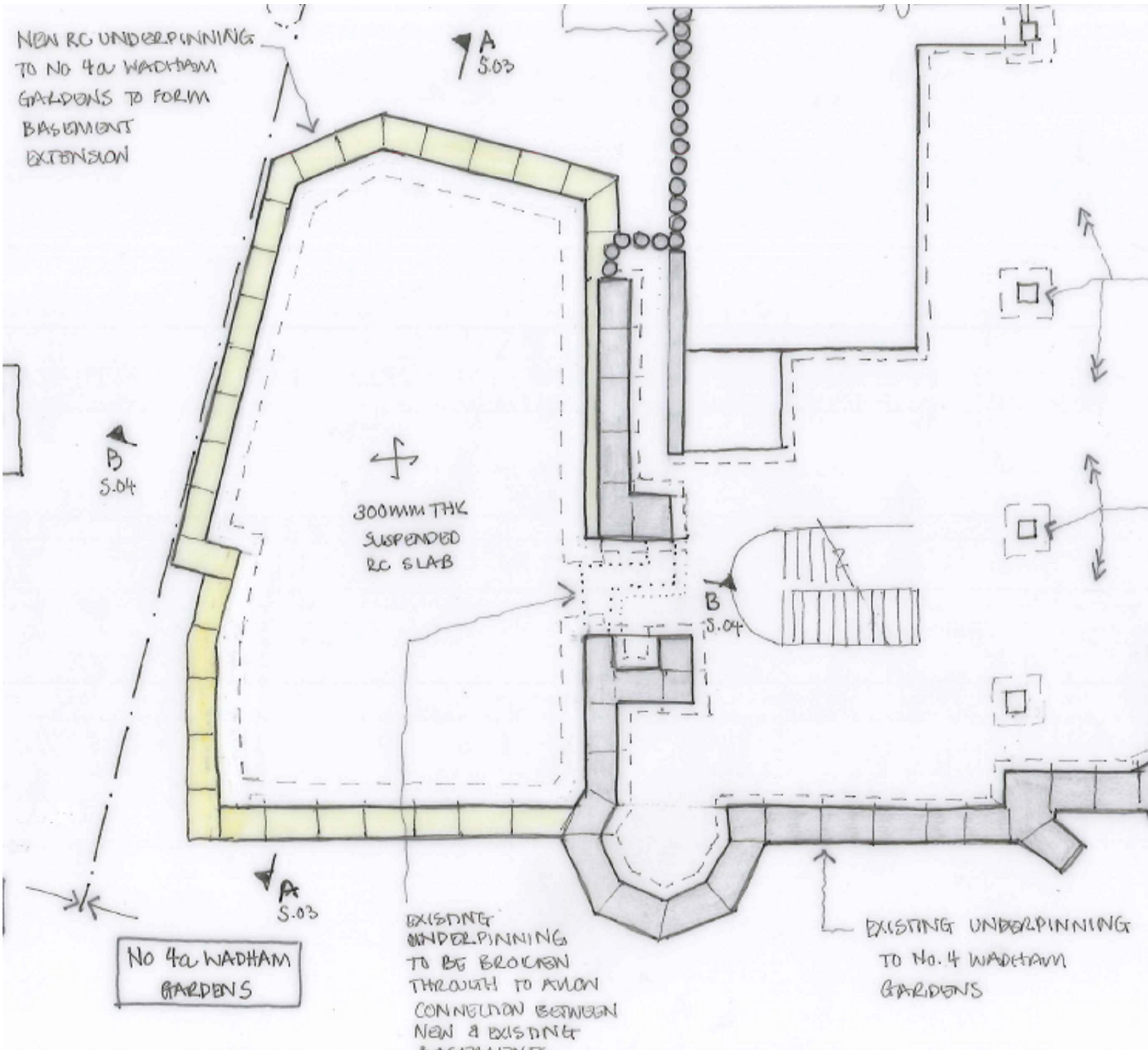
No 6 Wadham Gardens is a separate structure lying to the left of No 4/4a. The two buildings are separated by approximately 0.75m at their closest point. The separation distance varies along the boundary between the sites, as can be seen from the key plan in Figure 4. No 6 has a part-basement at the left end of the building, remote from No 4a. It will be assumed for the purposes of this report that the footings of No 6 close to No4a are set at a depth of 1.0m. The front and rear walls of No 6 are taken to be approximately 20m long; the right flank wall (adjacent to No 4a) is taken to be 12m long.

It is required that a predicted damage category assessment be made on Nos 4 and 6 Wadham Gardens.

2.0 Information Provided

The following relevant information has been used for these calculations:-
i) SAS Borehole and trial pit logs.
ii) Elliott Wood structural engineering report dated July 2014.
iii) Burwell Deakins Design and access statement dated 30 July 2014.
iv) Glanville Drawings GS8140612/100, 200-203, 300 dated June 2014.
v) Fluid Structures Dwg Nos 21526 (? number not clear)/P01-P04 revs A+B, EX01-06, DT01-05. Dated 2005-06, Basement to No 4 Wadham Gardens.
vi) Elliott Wood load markup sketch dated 10/12/14.
vii) Email correspondence SAS-AGE dated 9/12/14 to 12/1/15.

Rear



Front

Figure 1 –Proposed Basement Plan (extract of EW dwg 2140303/S01/P1)

3.0 Anticipated Ground Conditions

The external ground level across the 4/4a site varies between 50mOD and 50.6mOD, for the practical purposes of the current report the original ground surface at the site will be treated as horizontal at a level of 50.2mOD.

The published geological map (BGS 1:50 000 sheet 256: North London) indicates the site to lie on London Clay. On a developed site such as this Made Ground is also anticipated.

On the basis of the published mapping the base of the London Clay is anticipated to lie at approximately 50m depth (0mOD).

A ground investigation was undertaken at the site at the beginning of June 2014 (Item 'i' in Section 2 above). This comprised a rotary percussion borehole to 15m depth at the front of No 4a (BH1), and a continuous flight auger borehole, also to 15m depth, in the rear garden (BH2). Both boreholes were equipped with water-monitoring standpipes, with response zones from 1m to 5m depth.

A single trial pit was excavated adjacent to the rear wall of No 4a.

The boreholes confirmed clay/silt/sand Made Ground overlying stiff (becoming very stiff) London Clay. The Made Ground was present to 1m depth in BH1 and 1.7m depth in BH2, the difference in ground level between these boreholes (50mOD and 50.5mOD respectively) suggests the top of the London Clay can be taken to lie at approximately 49mOD.

Groundwater was not encountered during the boring. Subsequent readings made in the standpipes (3 July 2014) indicated BH1 to be dry while BH2 recorded water at 1.76mbgl. It is noted that the response zone of BH1 was probably sealed into the London Clay, while that of BH2 intersected the higher-permeability Made Ground. These readings are therefore taken to indicate a minor, and possibly impersistent, perched water table at the top of The London Clay.

On the basis of the above, the soil sequence at the site is taken to be:-

Ground Level 50.2mOD
Base of Made Ground 1.2mbgl (49mOD)
Base of London Clay approx 50mbgl (0mOD).

The Made Ground lies above excavation depth, and adjacent structures are likely to be founded onto the London Clay, therefore the Made Ground does not influence ground movements and will not be considered in detail.

In situ Standard Penetration Tests (SPT) were carried out in the London Clay in BH1, Vane tests were carried out on the arisings from BH2, and triaxial tests were carried out on samples from BH1. The results of these tests are plotted in Figure 2. The SPT results have been converted to undrained strength (Su) values using the method of Stroud (Ref 1) taking f1= 4.5. Vane tests typically overestimate the bulk strength of London Clay by a significant degree. The cell pressures used in the triaxial tests are considered to be too low to effect full saturation, particularly for the shallower samples, therefore these results are also considered to overestimate the bulk London Clay strength, therefore more weight has been accorded to the SPT results.

The (solid) trend-line given in Figure 2 has the equation:-

$Su = 75 + 12z$ (kPa)

Where z is the depth below the top of the London Clay.

This represents a significantly steeper rate of strength increase than is typically seen in this area of London, therefore a more conservative trend line has been adopted for the analysis, this is described by:-

$Su = 75 + 8z$ (kPa)

This strength profile is indicated by the dashed line in Figure 2 below.

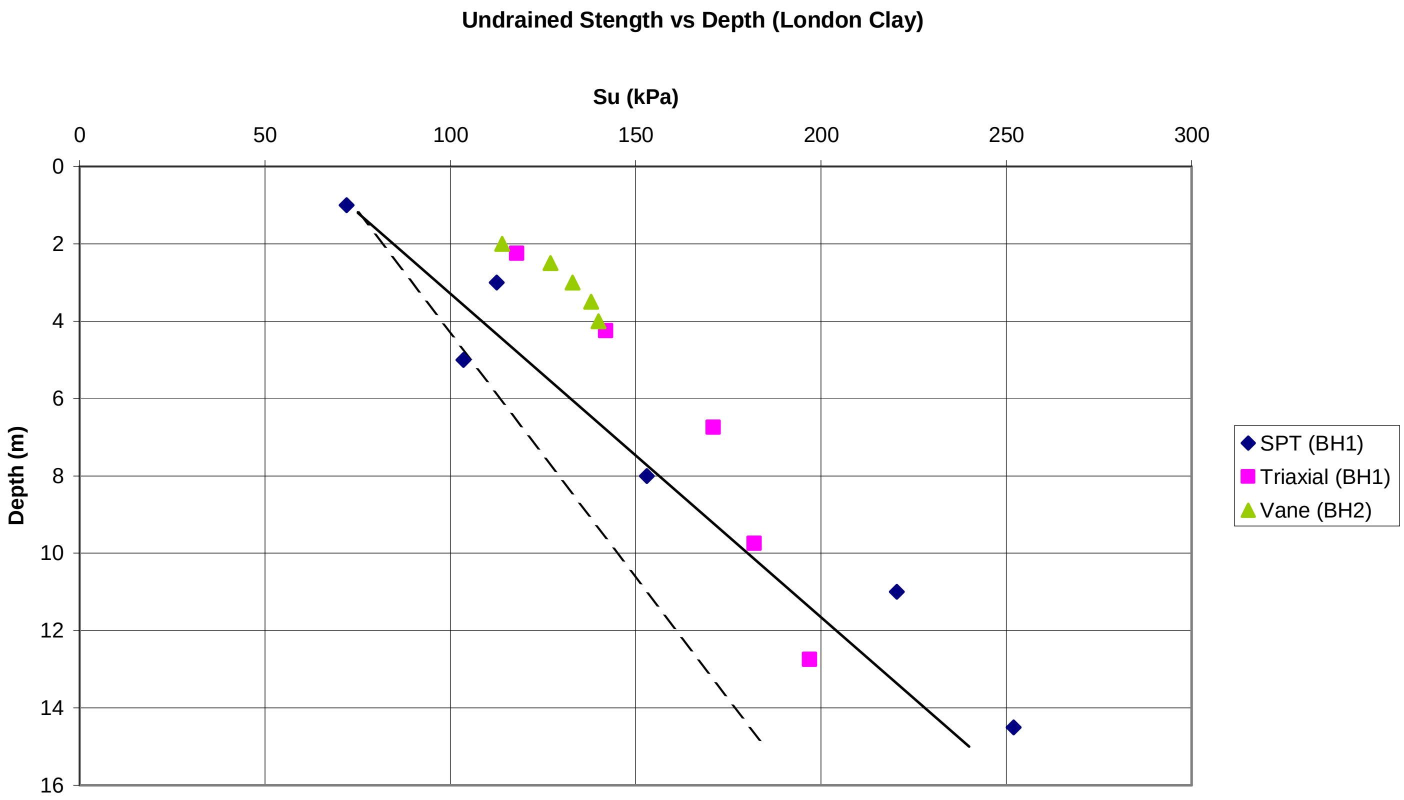


Figure 2
Undrained strength vs depth

4.0 Loads

The current building loads are taken to be imposed on cemented crushed brick footings (as revealed during the ground investigation) which are assumed to bear on the top of the London Clay at 49mOD. The loads are understood to equate to line loads of 45kN/m run of the perimeter walls of No 4a (Item 'vi' in Section 2 above).

It is understood that these superstructure loads will not alter significantly during or after the proposed works, except that they will be transferred to the new basement perimeter walls.

Excavation from existing ground level to the new basement formation level of 45.85mOD will yield a significant unload. In calculating this unload a bulk unit weight of $20kN/m^3$ has been adopted.

5.0 Estimated movement

5.1 Temporary support to the basement walls.

It is assumed within the following calculations that the basement perimeter retaining walls will be stiffly and safely propped at all stages of construction in line with BS5975:2008 and current good practice. Inadequate propping is likely to result in increased ground movements, and therefore increased damage to adjacent properties, as well as increased risk of injury to personnel.

It is generally recommended that consideration be given to the preloading of temporary basement wall props, and the monitoring of prop loads during critical stages of excavation.

5.2 Soil stiffness values

An equivalent-elastic analysis has been carried out using the program PDisp. The program takes no account of structural (building) stiffness.

The soil stiffness parameters are as given below.

The London Clay has been treated as a non-linear material. The small-strain stiffness is taken as 80% of the small-strain stiffness calculated from recent high quality data (Bond Street Station). These data yielded E_{uo} = 1940Su, therefore for the purposes of the current analysis take:-

$E_{uo} = 1550 \times Su$; (Poisson's ratio = 0.5)
$E'_o = 1240 \times Su$; (Poisson's ratio = 0.2)

Yielding :-

$E_{uo} = 116 + 12.4z$ (MPa)
$E'_o = 93 + 9.9z$ (MPa)

Where z = depth below top of London Clay in metres.

A non-linear degradation curve relating stiffness to strain, based on published data for the London Clay, has been used.

5.3 Causes of ground movement outside the excavation

The analysis considers three causes of ground movement outside the excavation, these are:-
i) Vertical ground movement due to vertical changes in load resulting from building works and excavation
ii) Vertical and horizontal movement due to installation of underpins
iii) Vertical and horizontal movement due to deflection of underpins, following removal of support from in front of underpins by excavation.

The first of these causes is investigated using equivalent-elastic analysis in the program PDISP. The second and third are based upon case-history data presented in Figures 2.8, 2.9 and 2.11 in CIRIA C580 (Ref 3). These data relate to installation in stiff clays. It is currently understood that the plots presented by CIRIA in the above figures include short-term movement arising from cause ‘i’ above. Therefore in this report short-term movements are calculated using the CIRIA data, and subsequent long-term movement is calculated using PDISP.

The CIRIA plots relate vertical and horizontal ground movement to the depth of the wall installed (for Cause ‘ii’ above), or to the depth of excavation within that wall (for Cause ‘iii’ above) as appropriate. Data relating to the secant bored pile wall case history in Ref 3 Figure 2.8 are considered to be unreliable and have been ignored. In addition, data relating to counterfort diaphragm walls have not been taken into account in this analysis. No data are presented by CIRIA for underpinned walls, and no other data are available from other sources for underpin walls. Underpin walls are therefore assumed to be similar in behaviour to plane diaphragm walls and bored pile walls.

The CIRIA data indicate that:-

a) Adjacent to the underpin, vertical ground settlement resulting from wall installation can be taken to equal 0.04% of wall depth, reducing linearly to zero at a distance of 2 x wall depth from the wall (Ref 3, Figures 2.8b and 2.9b).

b) Adjacent to the underpin, vertical ground settlement resulting from wall deflection can be taken to equal 0.04% of excavation depth, increasing to 0.08% of excavation depth at a distance of 0.6 x excavation depth from the wall, then reducing approximately linearly to zero at a distance of 3 x excavation depth from the wall. (Ref 3, Figure 2.11b).

c) Adjacent to the underpin, horizontal ground movement resulting from wall installation can be taken to equal 0.04% of wall depth, reducing linearly to zero at a distance of 1.5 x wall depth from the wall (Ref 3, Figures 2.8a and 2.9a).

d) Adjacent to the underpin, horizontal ground movement resulting from wall deflection can be taken to equal 0.15% of excavation depth, reducing linearly to zero at a distance of 4 x dig depth from the wall. (Ref 3, Figure 2.11a).

The above trends rely on good workmanship and stiffly-propped, stiff walls. Temporary support of excavations should be designed to BS5975 and BS8002.

Note that, in all the plots of vertical movement, settlement is taken as positive and heave as negative. The CIRIA data are understood to relate to movement at, or close to, ground level.

5.4 Predicted movement –6 Wadham Gardens, front and rear elevations

5.4.1 Vertical Movement

Profiles of short- and long-term vertical ground movement along the front and rear elevations of No 6 Wadham Gardens have been calculated and plotted in Figures 4 and 5 respectively. The plots present the short and long-term heave profiles calculated as described above.

These walls are both taken to extend 20m from the right flank wall of No 6, as shown on the plans in Figures 4 and 5. At the rear the right flank wall of No 6 lies 0.75m from the proposed basement of No 4a, while at the front the separation between the two properties widens to 2.2m due to the irregular geometry of No 4a. In both plots the proposed basement of No 4a lies at Y=0.

There is understood to be a basement beneath the left end of No 6, but the predicted vertical ground movement in this location is predicted to be negligible, so this basement is not taken into account in this analysis.

In calculating the short-term profiles using CIRIA C580, the excavation for the underpins to No4a is taken to descend from an existing ground level at 50.2mOD to a formation level of 45.65mOD; a depth of 4.55m. The basement excavation within the underpins is taken to descend from the same ground level to a formation level of 45.85mOD; a depth of 4.35m.

The settlement profiles are similar for the front and rear walls, with no practical difference between them save for the upturn at the right-hand end of the rear wall. This upturn occurs by virtue of the proximity of the rear wall to the proposed excavation at No 4a. The following comments relate to the rear wall.

The analysis indicates a maximum overall tilt of approximately 4.7mm over the 20m length of the wall. This equates to a whole-wall gradient of less than 1 in 4000. This is considerably less than the 1:400 gradient recognised as requiring remedial action.

Two modes of distortion are evident in the rear wall; hogging of approximately 2mm that affects virtually the entire wall length (18m), and sagging which affects the 9m or so length of wall lying closest to No 4a. (The front wall is affected only by the hogging mode of distortion).

In the hogging mode the maximum wall distortion (Delta – as defined by Burland, Ref 2) is 2.0mm within an 18m length of the wall. This equates to a deflection ratio of 2.0/18 000 = 0.011%. Taking the limiting tensile strain between the ‘very slight’ and ‘slight’ damage categories as being 0.075% (Ref 2) then the worst-case ratio of deflection ratio to limiting tensile strain = 0.011/0.075=0.15. By reference to Figure 3 (Ref 2 Figure 6) and taking the height of the No 6 rear wall as being approximately equal to half of its length, a horizontal strain/limiting tensile strain ratio of 0.80 is obtained, therefore a horizontal strain of 0.80 x 0.075% = 0.060% is acceptable for a ‘very slight’ category of damage.

In the sagging mode the maximum wall distortion (Delta – as defined by Burland, Ref 2) is 1.5mm within a 9m length of the wall. This equates to a deflection ratio of 1.5/9000 = 0.017%. Taking the limiting tensile strain between the ‘very slight’ and ‘slight’ damage categories as being 0.075% (Ref 2) then the worst-case ratio of deflection ratio to limiting tensile strain = 0.017/0.075=0.23. By reference to Figure 3 (Ref 2 Figure 6) and taking the height of the No 6 rear wall as being approximately equal to its width (over the sagging part of the wall), a horizontal strain/limiting tensile strain ratio of 0.85 is obtained, therefore a horizontal strain of 0.85 x 0.075% = 0.064% is acceptable for a 'very slight' category of damage.

5.4.2 Lateral movement.

From Section 5.3 above, taking wall depth to be 4.55m the maximum lateral movement due to underpin wall installation is calculated to be 1.8mm, reducing to zero at approximately 7m distance from the basement. This yields a horizontal ground strain of 1.8/7000 = 0.026% within that 7m distance.

Also from Section 5.3 above, taking general excavation depth to be 4.35m, the ground movement due to the subsequent deflection of the underpin wall, following excavation of the basement, is calculated as 6.5mm, reducing to zero at a distance of 17.5m (yielding an average strain of 6.5/17 500 = 0.037%). This lateral ground strain encompasses much of the rear wall of No 6 (but a smaller proportion of the front wall).

The total lateral ground strain beneath that part of the rear wall of No 6 which lies within approximately 7m of the basement excavation at No 4a is therefore assessed as 0.026% + 0.037% = 0.063%. This is less than the upper limit of 0.064% for 'very slight' damage derived above for this (sagging) length of wall.

In contrast, the hogging mode of distortion affects that part of the front and rear walls that lie more than 7m from the proposed basement excavation. Here the horizontal ground strain is calculated to be 0.037%. This is less than the upper limit of 0.060% for 'very slight' damage derived above for this (hogging) length of wall.

The analysis is conservative insofar as it takes no account of the stiffness of the walls, which would tend to limit the effect of ground strains in both the vertical and lateral directions.

The predicted level of damage to these walls can therefore be taken as 'very slight'.

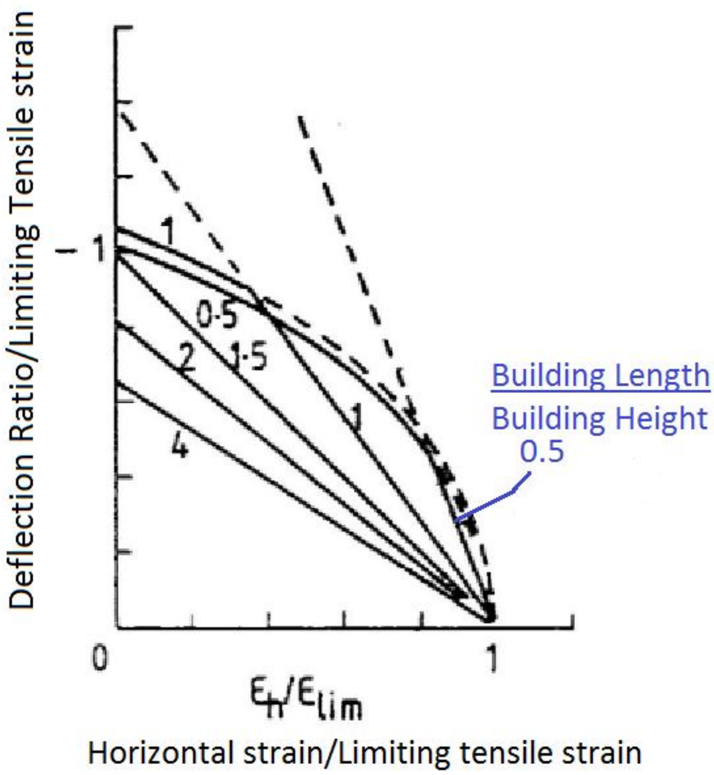


Figure 3 (from Ref 2)

5.5 Predicted movement – No 6 Wadham Gardens, right flank wall.

Profiles of short- and long-term vertical ground movement along the right flank wall of No 6 Wadham Gardens have been calculated and plotted in Figure 6. This wall extends from Y = 1.4m at the front, to Y = 13.4m at the rear. Over the rearmost 8m or so of this wall the excavation for the basement to No 4a lies approximately 0.75m away, but this dimension increases to approximately 2.2m at the front, due to the irregular shape, in plan, of No 4a.

The analysis indicates a maximum overall tilt of approximately 0.9mm over the 12m length of the wall. This equates to a whole-wall gradient of less than 1 in 12000. This is considerably less than the 1:400 gradient recognised as requiring remedial action.

The maximum predicted wall distortion (Delta – as defined by Burland, Ref 2) is 0.8mm within the 12m length of the wall. This is negligible, and taking into account that there is also likely to be negligible horizontal ground strain along the length of this wall, as a result of the proposed works at No 4a, the predicted damage category can be taken as ‘very slight’ or less, by inspection.

5.6 Predicted movement – 4 Wadham Gardens, front wall.

5.6.1 Vertical Movement

Profiles of short- and long-term vertical ground movement along the front elevation of No 4 Wadham Gardens have been calculated and plotted in Figure 7.

The existing basement to No 4 is understood to be approximately the same depth as the proposed basement to No 4a. The front part of the basement at No 4 was excavated within underpin walls. Due to this proximity to the existing basement of similar depth, vertical ground movements are likely to arise only due to the unloading of the ground beneath No 4a, as calculated by Pdisp (Cause 'i' in Section 5.3 above). Movements calculated by reference to CIRIA C580 do not apply, as the underpins forming the boundary between the existing basement to No 4 and the proposed basement to No 4a, are already in place and there will be no imbalance of earth pressure acting on this boundary wall following excavation of the 4a basement.

The front wall of No 4 is taken to extend approximately 11m from the boundary wall with No 4a at X=8.7m.

The analysis indicates a maximum overall tilt of 1.5mm along the 11m length of the wall. This equates to a whole-wall gradient of less than 1 in 7000. This is considerably less than the 1:400 gradient recognised as requiring remedial action.

The maximum wall distortion (Delta – as defined by Burland, Ref 2) is 0.6mm within the 11m wall length. This equates to a deflection ratio of 0.6/11000 = 0.005%. Taking the limiting tensile strain between the 'very slight' and 'slight' damage categories as being 0.075% (Ref 2) then the worst-case ratio of deflection ratio to limiting tensile strain = 0.005/0.075=0.07. By reference to Figure 3 (Ref 2 Figure 6) and taking the length of the No 4 front wall as approximately equal to its height, a horizontal strain/limiting tensile strain ratio of 0.95 is obtained, therefore a horizontal strain of 0.95 x 0.075% = 0.07% is acceptable for a 'very slight' category of damage. This analysis does not take account of the stiffness of the walls of No 4, the result is therefore conservative in this respect.

5.6.2 Lateral movement.

As discussed above, due to the presence of the existing basement to No 4, immediately adjacent to, and of similar depth as, the proposed basement to No 4a, the ground movements due to wall installation and basement excavation predicted by CIRIA C580 do not apply. Any horizontal movements undergone by No 4 will therefore be the result of the relaxation of that structure, as the earth pressures it is currently subjected to, from the site of No 4a, are removed.

The prediction of such movements would require structural analysis which is beyond the scope of this report, but by inspection would be expected to be considerably less than those resulting from the usual CIRIA C580 predictions. CIRIA C580 usually predicts a maximum horizontal ground strain of approximately 0.064%, which is less than the 0.07% acceptable limit calculated above, therefore the predicted level of damage to the front wall of No 4 can be taken as 'very slight' or less.

The rear wall of No 4 is underlain by a basement void. The depth of this basement is greater than that at the front due to the presence of a swimming pool basin in the rear basement to No 4. By inspection therefore, the degree of damage predicted at the rear wall of No 4 will also be 'very slight' or less.

5.7 Predicted movement – No 4 Wadham Gardens, party wall with No 4a.

Profiles of short- and long-term vertical ground movement along the No 4/4a party wall have been estimated and plotted in Figure 8. This wall is taken to extend from Y = 0m at the front, to Y= 13m at the rear, as shown in the plan accompanying Figure 8. This wall sits atop the existing underpin basement wall of No 4. Therefore the discussion above in Section 5.6.1 applies, and the predicted ground movement will be that arising from the unloading of the soil beneath No 4a, but not from the installation and deflection of the underpins as predicted by CIRIA C580.

The analysis indicates a negligible overall tilt along the 13m length of the wall.

The maximum wall distortion (Delta – as defined by Burland, Ref 2) is 1.8mm within the 13m wall length. This equates to a deflection ratio of 1.8/13000 = 0.014%. Taking the limiting tensile strain between the 'very slight' and 'slight' damage categories as being 0.075% (Ref 2) then the worst-case ratio of deflection ratio to limiting tensile strain = 0.014/0.075=0.18. By reference to Figure 3 (Ref 2 Figure 6) and taking the length of the No 4/4a party wall as approximately equal to 1.5 x its height, a horizontal strain/limiting tensile strain ratio of 0.85 is obtained, therefore a horizontal strain of 0.85 x 0.075% = 0.064% is acceptable for a 'very slight' category of damage. This analysis does not take account of the stiffness of the party wall; the result is therefore conservative in this respect.

The nature of the proposed works is such that significant horizontal strains are not likely to arise in the plane of this wall, therefore the damage classification can be taken as 'very slight' or less.

5.8 Predicted damage summary

On the basis of the above, the level of damage to Nos 4 and 6 Wadham Gardens is predicted to be 'very slight' or less, as defined in Ref 2. This conclusion assumes a high standard of workmanship and adequate propping of the basement excavation.

6.0 Groundwater

It is proposed to excavate to a maximum depth of approximately 4.6m through up to 1.7m of Made Ground into a thick deposit of London Clay. No groundwater was encountered during the ground investigation works. A standpipe installed in BH2 at the rear of the property indicated groundwater level coincident with the top of the London Clay when monitored 1 month after installation. A similar standpipe at the front of the property was dry. This is taken to indicate the likely presence of a minor and possibly impersistent perched water table in the Made Ground on top of the London Clay.

It appears that there is no potential for significant groundwater flow within the proposed basement depth, and that therefore the development will not affect the local groundwater regime.

7.0 Conclusions and Recommendations

From the above, it is concluded that, given good workmanship, the basement to 4a Wadham Gardens can be constructed without imposing more than very slight damage on the adjoining properties. The development is not likely to disrupt any existing local groundwater flows.

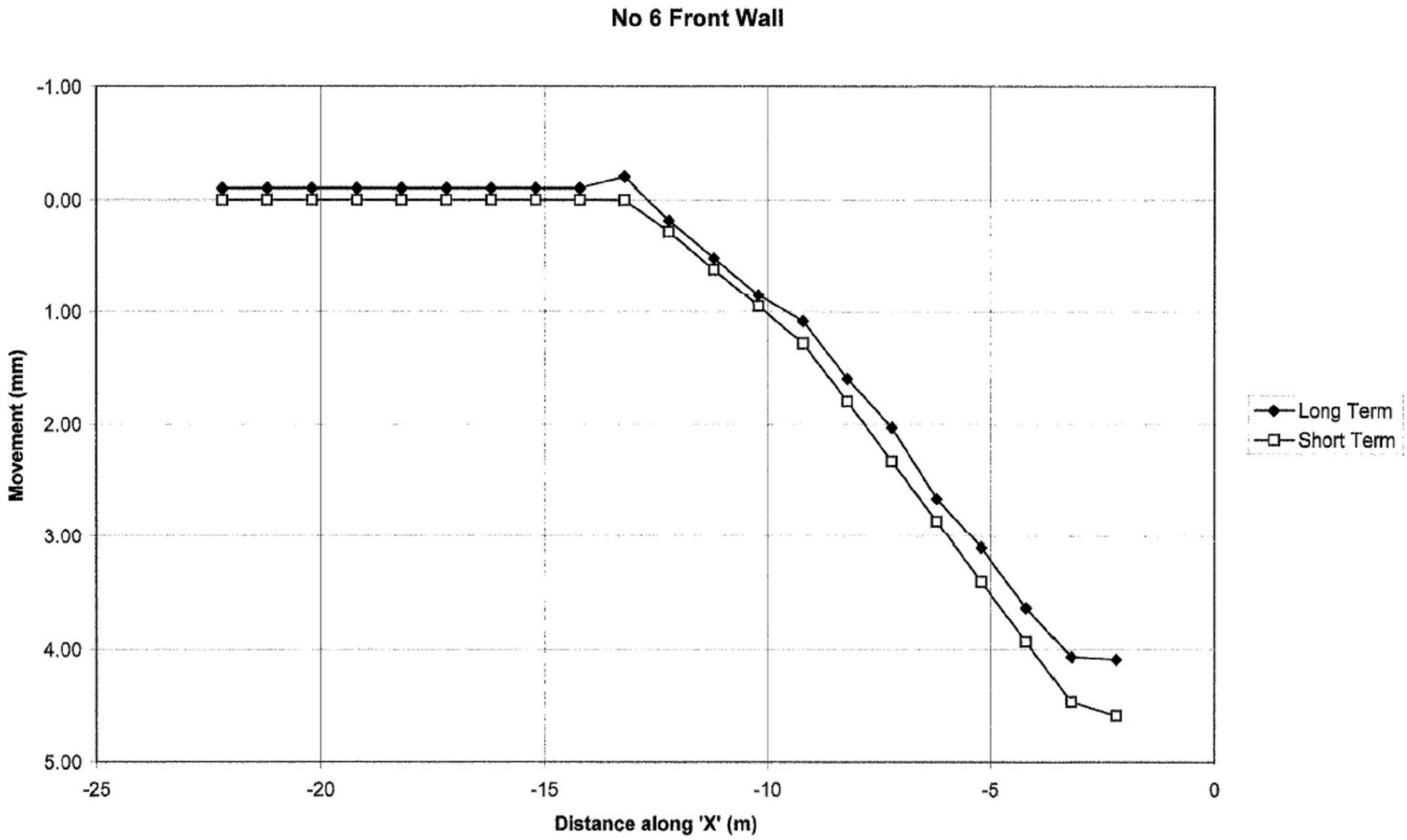
References:

1 Stroud M A (1989) 'The standard penetration test – its application and interpretation'. In 'Penetration testing in the UK', Thomas Telford pub.

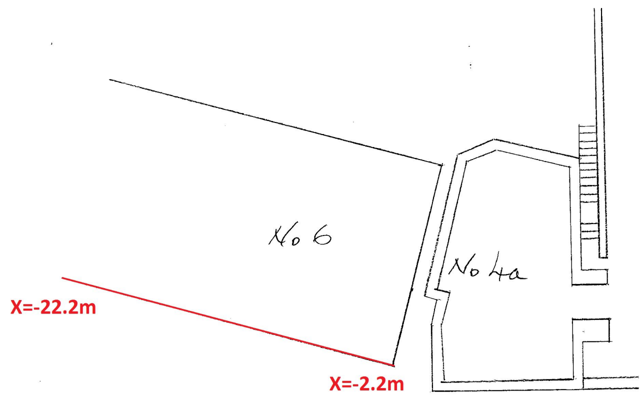
2 Burland JB (1997). 'Assessment of risk of damage to buildings due to tunnelling and excavation'. In 'Earthquake Geotechnical engineering' Ishihara (Ed). Balkema pub.

3 Gaba A R, Simpson B, Powrie W, Beadman D R (2003) Embedded retaining walls - guidance for economic design, CIRIA Report C580, London. ISBN: 978-0-86017-580-3.

(Figures 4-8 follow below)

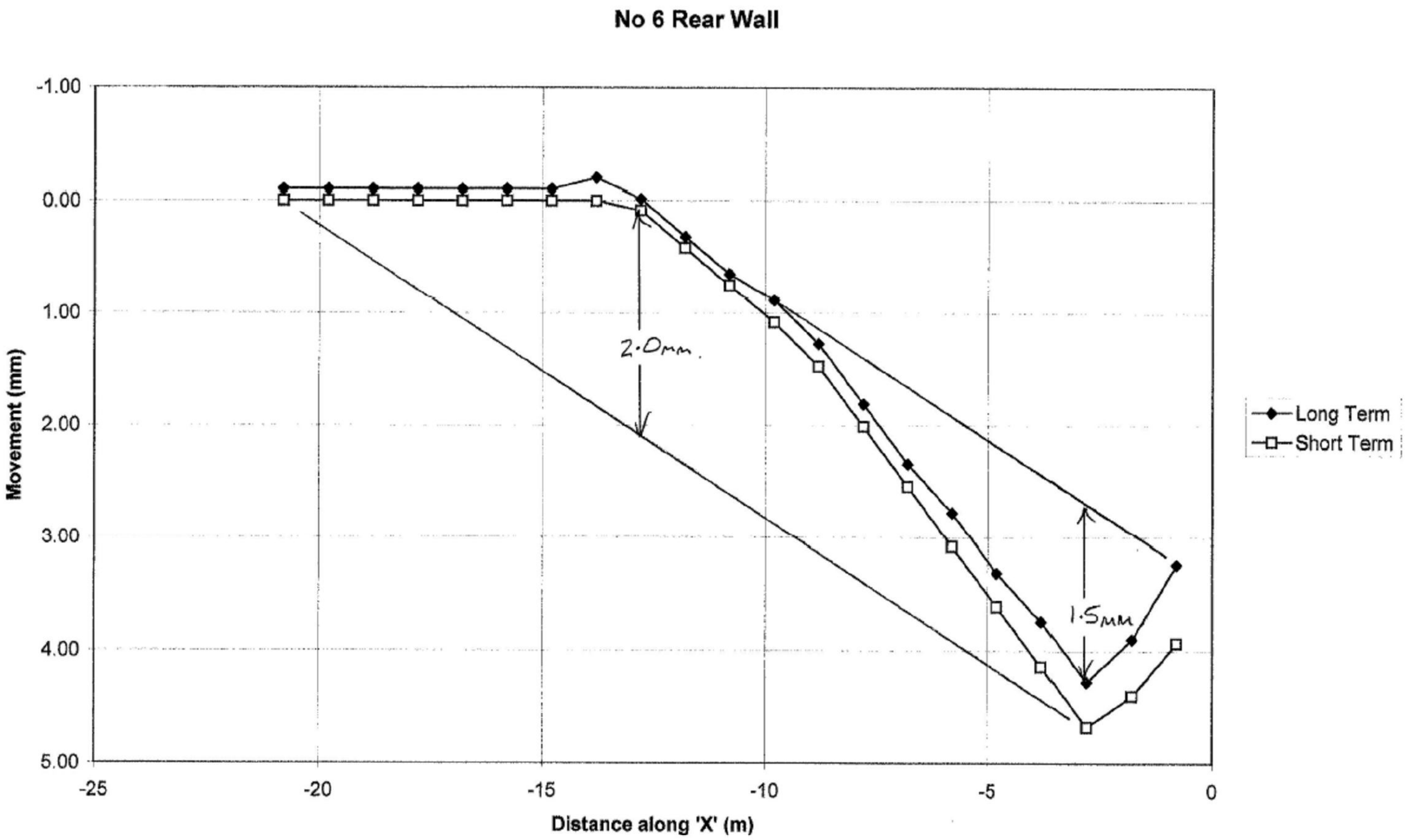


No 6 Front Wall
-1.00
0.00
1.00
2.00
3.00
4.00
5.00
Movement (mm)
-25
-20
-15
-10
-5
0
Distance along 'X' (m)
Long Term
Short Term



No 6
No 4a
X=-22.2m
X=-2.2m

Figure 4


No 6 Rear Wall
Movement (mm)
-1.00
0.00
1.00
2.00
3.00
4.00
5.00
-25
-20
-15
-10
-5
0
Distance along 'X' (m)
2.0mm.
1.5mm
Long Term
Short Term

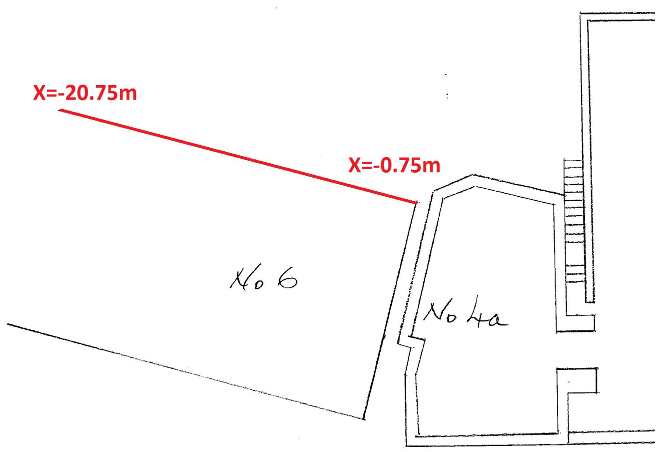
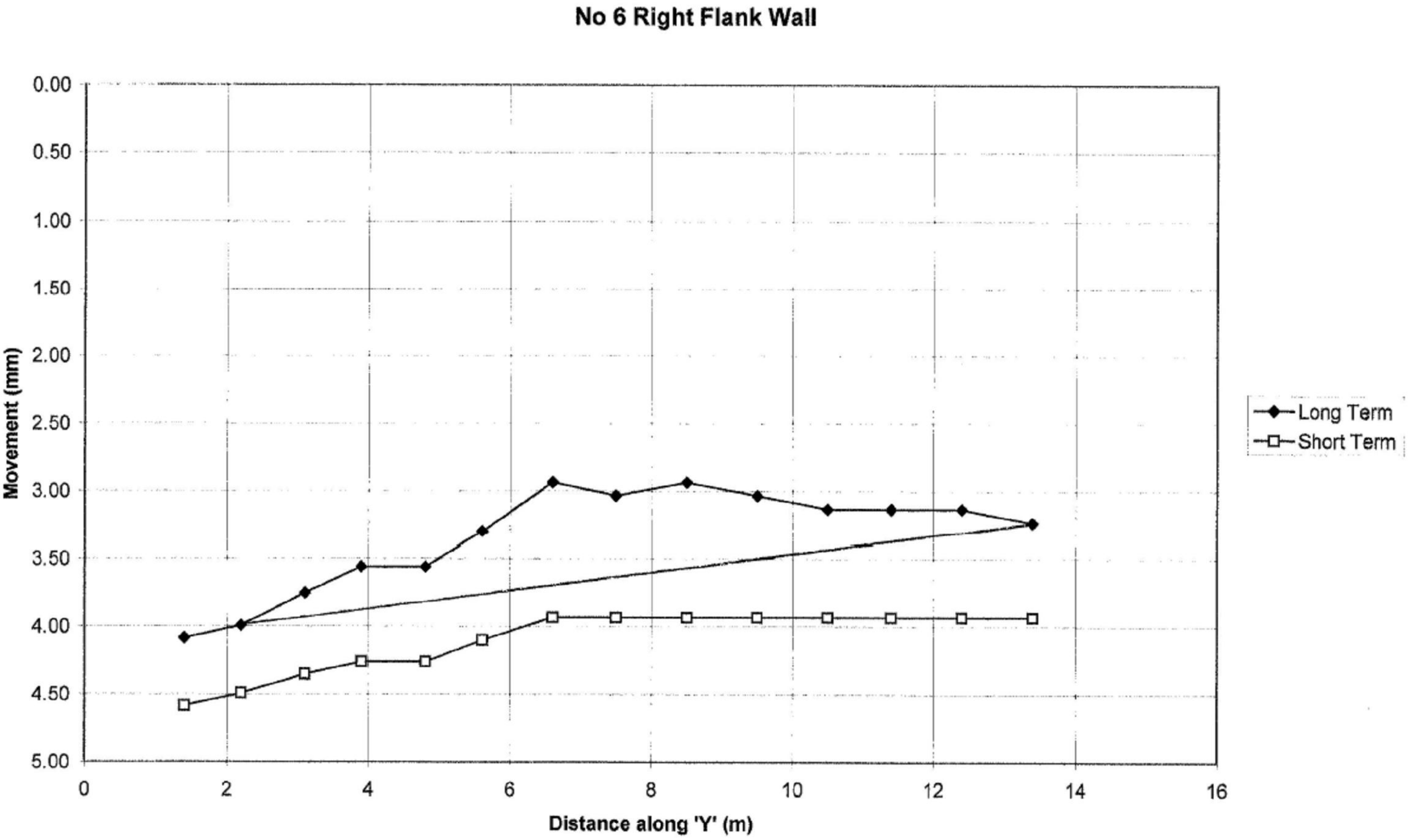
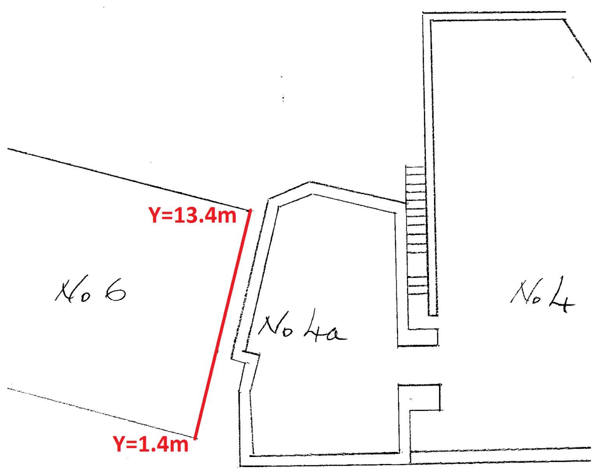

X=-20.75m
X=-0.75m
No 6
No 4a

Figure 5

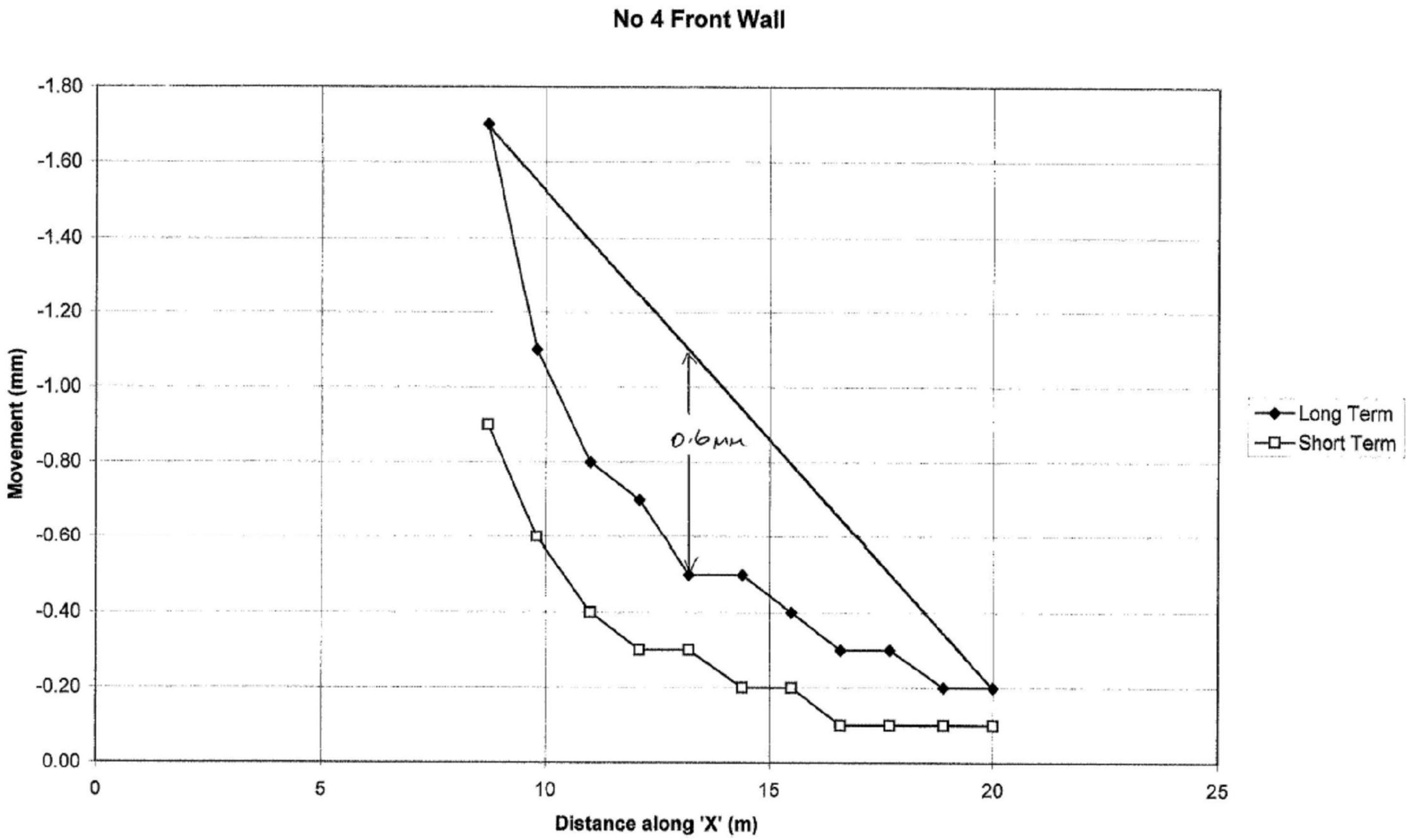


No 6 Right Flank Wall
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
4.50
5.00
Movement (mm)
0
2
4
6
8
10
12
14
16
Distance along 'Y' (m)
Long Term
Short Term

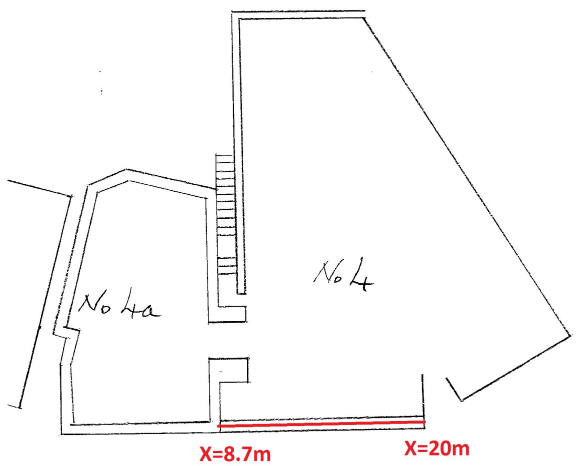


Y=13.4m
Y=1.4m
No 6
No 4a
No 4

Figure 6

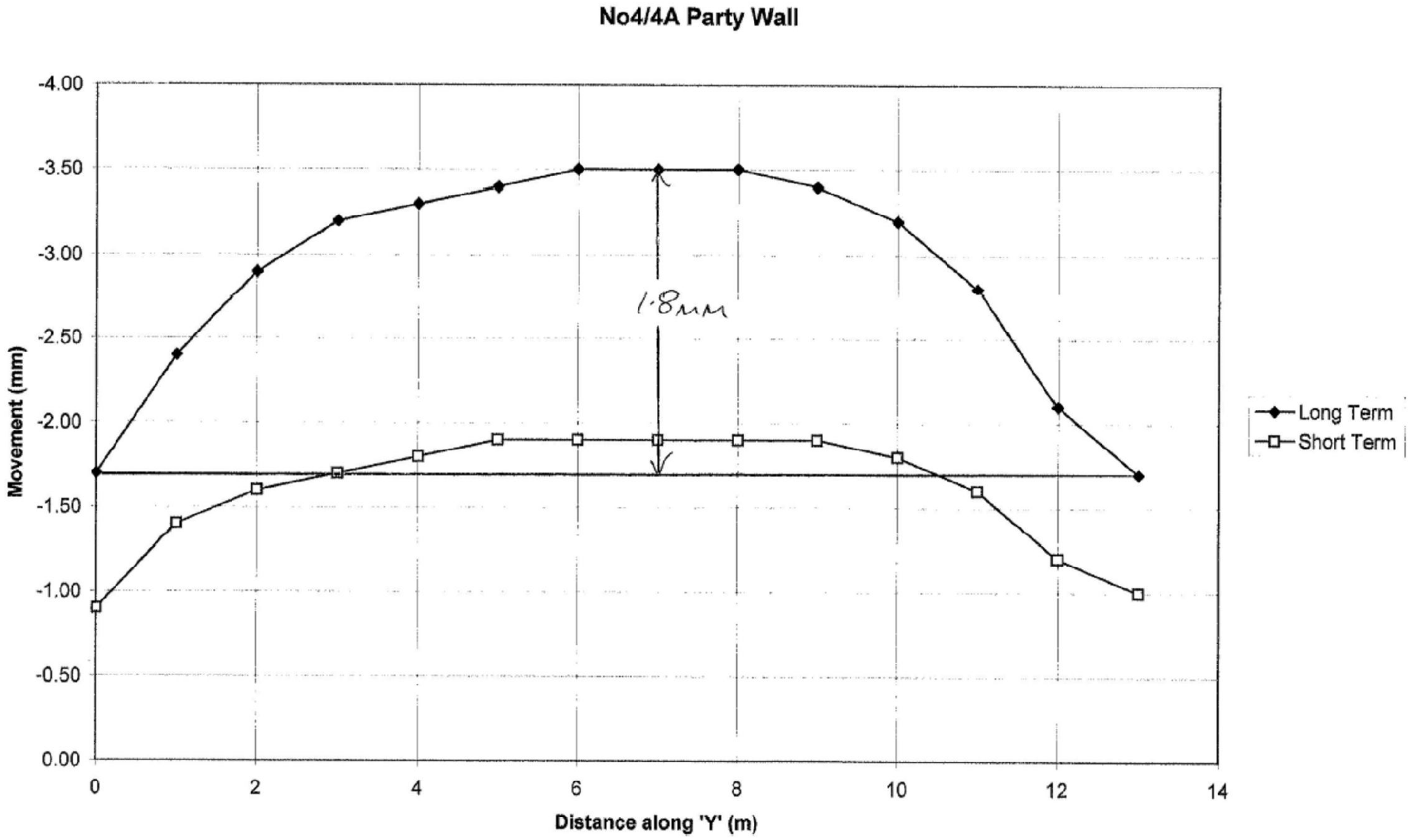


No 4 Front Wall
Movement (mm)
-1.80
-1.60
-1.40
-1.20
-1.00
-0.80
-0.60
-0.40
-0.20
0.00
0
5
10
15
20
25
Distance along 'X' (m)
Long Term
Short Term
0.6mm

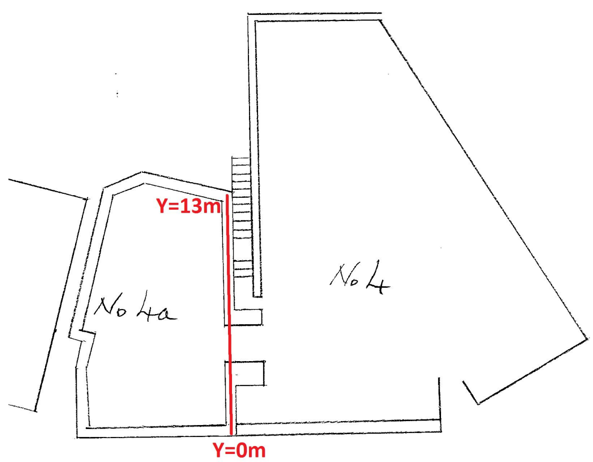


No 4a
No 4
X=8.7m
X=20m

Figure 7



No4/4A Party Wall
Movement (mm)
-4.00
-3.50
-3.00
-2.50
-2.00
-1.50
-1.00
-0.50
0.00
0
2
4
6
8
10
12
14
Distance along 'Y' (m)
1·8mm
Long Term
Short Term



Y=13m
Y=0m
No 4a
No 4

Figure 8