Ref: 11/17802
August 2016

Basement Impact Assessment (Hydrogeological and Structural)

At

Ornan Court, 2 Ornan Road, London, NW3 4PT

For

Ornan Court Limited

VOLUME 4 of 8

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3.1 Ground Movement Assessment Report (Part 2)

Figure 1 –Proposed Basement Plan

3.0 Anticipated Ground Conditions

The general ground surface within the site is understood to be effectively horizontal, but there are retaining walls on both the Ornan Road and Haverstock Hill sides of the property. The road pavements lie approximately 1m below site level. For the purposes of the current report the site will be treated as level for practical purposes.

The published geological map (BGS 1:50 000 sheet 256: North London) indicates the site to lie on London Clay, with a propensity for some Head to be present. The outcrop of the Claygate Member (silt and fine-grained sand) underlying Hampstead Heath lies some 100-200m to the west of the site. 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 80m depth.

The level of the site, relative to Ordnance Datum, is understood to be approximately 78mOD. However, levels to Ordnance Datum were not available during the preparation of this report, therefore, for the purposes of this report, Site Datum (0.0mSD) will be taken at existing ground level inside Ornan Court.

Two phases of ground investigation have been undertaken at the site (Item 'i' in Section 2 above). This comprised five boreholes, each to 5m depth, drilled in the garden areas to the front, right and rear of the property. Boreholes BH1 and 2 were excavated in 2011 using continuous flight auger methods, while boreholes BH3-5, bored in 2014, used window-sampling methods. Water monitoring standpipes were installed at approximately 4.9m depth in all boreholes.

The boreholes confirmed approximately 1.0m of sandy clay or clayey sand Made Ground, overlying stiff or very stiff London Clay.

Trial Pits were also undertaken and the results indicate the existing footings of the external walls of the structure to lie at between 1.0m and 1.2m depth, and to consist of spread footings of approximately 950mm width (MRA Drawing 06.462/20A). For the purposes of this report, the footings of the internal load-bearing walls are assumed to be similar. The footings are therefore taken to bear on stiff London Clay.

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

Base of Made Ground = -1.0mSD
Base of London Clay approx -80mSD.

The Made Ground lies above excavation depth, it does not influence ground movements and will not be considered in detail.

Standard Penetration Tests (SPT) were carried out in the London Clay in boreholes 3-5, and hand-vane tests were carried out on the arisings from boreholes 1+2. The results of the SPT 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. These give an undrained strength profile of :-

$Su = 60 + 7z$ (kPa). Where z is depth below top of London Clay in metres.

Vane tests tend to overestimate the bulk strength of London Clay due to scale effects. The tests carried out on boreholes 1+2 yielded Su values of approximately 80kPa at the top of the London Clay (-1.0mSD) rising to >140kPa at –1.5mSD. These values have not been adopted and reliance has been placed, instead, on the SPT values. Therefore, for the purposes of the current report only, the above Su profile has been adopted.

No groundwater was encountered during the ground investigations. Readings taken in the water monitoring standpipes in Boreholes 1-5 (in November 2011) show water levels of between 1.73mbgl (BH5) and 2.87mbgl (BH4).

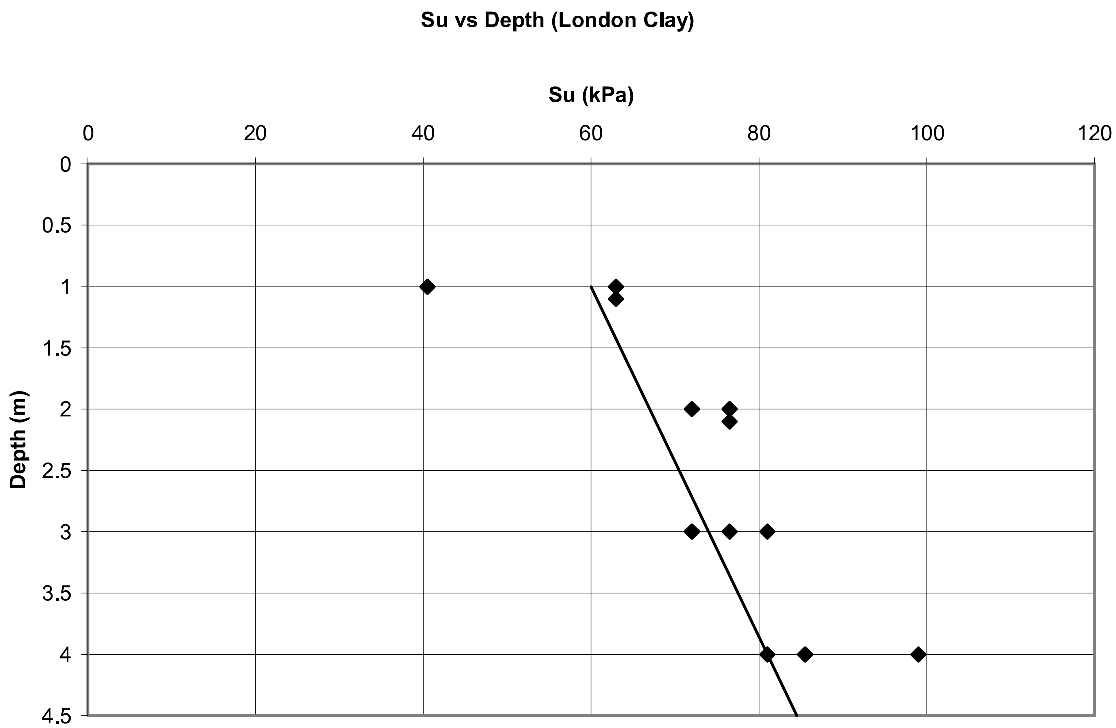

Figure 2 – Undrained strength vs depth (London Clay)

4.0 Loads

4.1 General

A plan of the site is shown in Figure 1. Ornan Road runs along the bottom of the figure, Haverstock Hill runs up the right side.

Based on the findings of the trial pits the existing footings are taken to bear at a level of -1.0 to -1.2mSD. It is understood that the underpinning will bear at a level of –3.5mSD, and the general excavation level will be –3.2mSD.

The load changes imposed by the works on the Made Ground are modest, and the soil so affected will be excavated as part of the underpinning works. The influence of the Made Ground on the ground movements is therefore not significant and its behaviour is not considered in detail.

The building loads have been provided by MRA (Item 'iii' in Section 2 above), and are summarised below.

4.2 Existing wall loads (at founding depth)

Ornan Ct-Rosslyn Ct party wall – 160kN/m run
Rear Wall – 152kN/m run
Right flank wall – 160kN/m run
Front Wall – 179kN/m run
Rear single storey extension rear wall – 30kN/m
Internal walls – 80kN/m to 190kN/m run.

4.3 Underpinning.

Wall loads in 4.2 above, transferred from -1.0mSD to –3.5mSD, and enhanced where appropriate by underpin self-weight taken as 16kN/m run.

4.4 Excavation

Excavation has been modelled as 3.2m reduction in level, at 20kPa per metre depth.

4.5 Proposed long-term loads (at new founding depth)

As in section 4.3.

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 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 following soil stiffness parameters have been adopted for the purpose of this analysis:-

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 (from Section 3.0 above) :-

$E_{uo} = 93 + 10.85z$ (MPa)
$E'_o = 74.4 + 8.68z$ (MPa)

Where z = depth below top of London Clay (at –1.0mSD) 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 resulting from 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, these are assumed to be similar in behaviour to plane diaphragm walls and bored pile walls. The CIRIA data indicate that:-

a) Adjacent to the pile wall or 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 pile wall or 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 pile wall or 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 pile wall or 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.

Note that, in all the plots of vertical movement, settlement is taken as positive and heave as negative.

5.4 Predicted movement – Rosslyn Court, 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 Rosslyn Court 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. For the purposes of this report the front and rear elevations of Rosslyn Court are taken to be 23m long (similar to Ornan Court), and to extend from X=-23 at the left flank wall, to X=0 at the party wall with Ornan Court (see indicative axis directions on Figure 1).

The settlement profiles are similar for the front and rear walls, with no practical difference between them. The following comments relate to the front wall.

The analysis indicates a maximum overall tilt of (2.7mm-0.0mm=) 2.7mm over the 23m length of the wall in the short term. This equates to a whole-wall gradient of less than 1 in 8000. 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 2.6mm within a 10m length of the wall. This equates to a deflection ratio of 2.6/10000 = 0.026%. 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 = 0.026/0.075=0.35. By reference to Figure 3 (Ref 2 Figure 6) and taking the height of the Rosslyn Court front wall as being slightly greater than the 10m length of wall under consideration, a horizontal strain/limiting tensile strain ratio of 0.8 is obtained, therefore a horizontal strain of 0.8 x 0.075% = 0.06% is acceptable for a 'very slight' category of damage.

5.4.2 Lateral movement.

From Section 5.3 above, taking wall depth to be 3.5m and excavation depth to be 3.2m, the maximum lateral movement due to underpin wall installation is calculated to be 1.4mm, reducing to zero at 5.25m distance (yielding a strain of 1.4/5250 = 0.026%). On the same basis, the ground movement due to the subsequent deflection of the underpin wall, following excavation of the basement, is calculated as 4.8mm, reducing to zero at a distance of 12.8m (yielding a strain of 4.8/12800 = 0.038%).

The total lateral ground strain beneath the front and rear walls of Rosslyn Court is therefore assessed as 0.064%. This is greater than the upper limit of 0.060% for 'very slight' damage derived above, suggesting damage at the lower end of the 'slight' range (which in this case would extend from a lateral ground strain of 0.06% to 0.135%). However, the above analysis is conservative as the stiffness of the walls is not taken into account, and the predicted mode of distortion is sagging, which is significantly less damaging than the hogging mode that Burland considered in his original analysis (Ref 2). As a result, it is considered most likely that a 'very slight' level of damage will arise at Rosslyn Court.

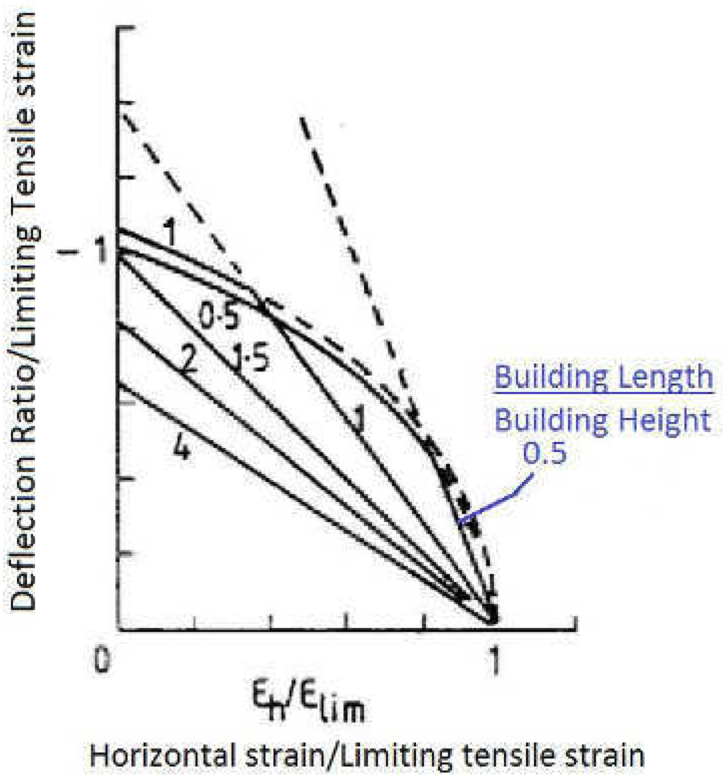

Figure 3 (from Ref 2)

5.5 Predicted movement – Ornan Court/Rosslyn Court party wall.

Profiles of short- and long-term vertical ground movement along the party wall have been estimated and plotted in Figure 6. This wall extends from Y = 7.5m at the front, to Y= 18.5m at the rear. The proposed excavation at Ornan Court extends 2m or so beyond the ends of the party wall (Y = 5.7m to Y = 20.8m) in order to provide front and rear lightwells to the basement.

Because the party wall defines the limit of the excavation in Ornan Court, its movement is not defined by the CIRIA C580 data, which apply outside the excavation. Instead the short-term settlement of the party wall above ground will be controlled by movements occurring during the underpin construction process. However, such movements depend on the condition of the existing wall, the precise underpinning technique and the quality of workmanship and so cannot reliably be predicted. Experience shows that, in most cases, such movements are minimal and may go unnoticed. However, in adverse circumstances, some millimetres of movement could be realised from this cause.

For the purposes of this report the short-term wall settlement due to underpinning has arbitrarily been assumed to equal that predicted by CIRIA C580 for the ground immediately adjacent to the wall, in this case this is 2.7mm, the actual settlement due to underpinning could be more or less, but is likely to be reasonably constant along the wall length, assuming good workmanship.

In the long-term the settlement along the line of the wall is also predicted to be reasonably consistent at approximately 1.5mm.

Provided the wall settlement due to underpin construction is not too severe, then both the tilt and the distortion of the party wall are predicted, by inspection, to yield damage within the 'very slight' category, as defined by Burland. Should significant settlement of the main party wall occur due to underpinning works then any damage is likely to be located close to the junction with Rosslyn Court, and as a precaution this area should be closely monitored.

5.6 Predicted movement – 239 Haverstock Hill, front and rear elevations (as viewed from Haverstock Hill)

5.6.1 Vertical Movement

Profiles of short- and long-term vertical ground movement along the front and rear elevations of No 239 Haverstock Hill have been calculated and plotted in Figures 7 and 8 respectively. The plots present the short and long-term heave profiles calculated as described above.

These walls both extend rearwards from the back of the Ornan Court excavation which lies at approximately Y= 23m (see indicative axis direction on Figure 1). The front and rear walls of No 239 are understood to extend from approximately Y = 29m. The length of each of these walls has been taken as 20m.

The settlement profiles are reasonably similar for the front and rear walls, but slightly greater tilt and distortion is predicted for the front wall. The following comments therefore relate to the front wall.

The analysis indicates a maximum overall tilt of (1.5-0.0=) 1.5mm along the 20m length of the wall in the short-term. This equates to a whole-wall gradient of less than 1 in 13000. 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 1.4mm within the 20m wall length. This equates to a deflection ratio of 1.4/20 000 = 0.007%. 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 = 0.007/0.075=0.09. By reference to Figure 3 (Ref 2 Figure 6) and taking the height of the No 239 front and rear walls as approximately half their length, a horizontal strain/limiting tensile strain ratio of 0.9 is obtained, therefore a horizontal strain of 0.9 x 0.075% = 0.068% is acceptable for a 'very slight' category of damage.

5.6.2 Lateral movement.

From Section 5.3 above, taking wall depth to be 3.5m and excavation depth to be 3.2m, the maximum lateral movement due to underpin wall installation is calculated to be 1.4mm, reducing to zero at 5.25m distance, this lateral ground movement therefore peters out in the intervening 6m of ground between Ornan Court and No 239 Haverstock Hill. On the same basis, the ground movement due to the subsequent deflection of the underpin wall, following excavation of the basement, is calculated as 4.8mm, reducing to zero at a distance of 12.8m (yielding a strain of 4.8/12800 = 0.038%).

The lateral ground strain beneath the closest few metres of the front and rear walls of No 239 is therefore assessed as 0.038%. This is less than the upper limit of 0.064% for 'very slight' damage derived above, and is therefore acceptable. Furthermore, the above analysis is conservative, as the stiffness of the walls is not taken into account and the depth of the foundations of No 239 has not been modelled in the above CIRIA analysis.

5.7 Predicted movement – 239 Haverstock Hill, left flank wall (as viewed from Haverstock Hill)

5.7.1 Vertical Movement

Profiles of short- and long-term vertical ground movement along the left flank wall of No 239 Haverstock Hill have been calculated and plotted in Figure 9. The plot presents the short and long-term heave profiles calculated as described above.

This wall runs parallel to the back of the Ornan Court excavation and is believed to extend from approximately X = 12.4m to approximately 23.4m.

The settlement profiles take account of the varying distance from the Ornan Court excavation to the 239 flank wall (due to the irregular shape of the excavation) but make no allowance for the smoothing effect of the intervening soil on the movement profile.

The analysis indicates a maximum overall tilt of 0.5mm along the 11m length of the wall in the long-term. This is insignificant.

The maximum wall distortion (Delta – as defined by Burland, Ref 2) is 0.6mm within a 7m length of the wall in the long term. This equates to a deflection ratio of 0.6/7000 = 0.009%. 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 = 0.009/0.075=0.11. By reference to Figure 3 (Ref 2 Figure 6) and taking the height of the No 239 left flank wall to be roughly equal to its length, a horizontal strain/limiting tensile strain ratio of 0.9 is obtained, therefore a horizontal strain of 0.9 x 0.075% = 0.068% is acceptable for a 'very slight' category of damage.

5.7.2 Lateral movement.

The nature of the works is such that there is unlikely to be any longitudinal strain along the plane of the left flank wall of No 239, therefore the level of damage predicted for the left flank wall of No 239 is predicted to be very slight or less.

5.8 Predicted damage summary

On the basis of the above, the level of damage to Rosslyn Court and to No 239 Haverstock Hill 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.