Client: Trademark Group Ltd

6 STREATLEY PLACE, LONDON, NW3 1HR

Ground movement analysis of proposed basement construction

Draft

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EXECUTIVE SUMMARY

On behalf of Trademark Group Limited, Soil Consultants Ltd (SCL) commissioned Geofem Limited to perform a ground movement analysis for the proposed works at 6 Streatley Place, London, NW3 1HR. Finite element analysis (FEA) was used to predict the ground movement beneath and around the proposed basement due to its construction.

The formation level of the proposed basement was predicted to heave up to about 3.5mm in the short-term on the NE side, decreasing to about 2.2mm on the SW side. The heave was predicted to reduce somewhat due to subsequent loading from the building and long-term consolidation. In the short-term case, the maximum deflection ratio Δ/L was predicted to be 1/47,000 and the maximum horizontal tensile strain 0.05%.

A deflection ratio Δ/L of 1/47,000 with a tensile strain of 0.05% would be expected to cause up to *very slight* damage on the Burland scale comprising less than 1mm wide cracks that are easily treated during normal redecoration and is acceptable on most projects.

Installation-induced ground movements were predicted by SCL using empirical relationships to be negligible.

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1 INTRODUCTION

On behalf of Trademark Group Limited, Soil Consultants Ltd (SCL) commissioned Geofem Limited to perform a ground movement analysis for the proposed works at 6 Streatley Place, London, NW3 1HR using finite element analysis (FEA). The aim of the FEA was to predict the ground movement beneath and around the proposed basement due to its construction. In particular, an assessment was made of the likely influence of these ground movements on adjacent properties.

SCL provided various information to prepare the FEAs. These included:

- Site Investigation Report and Basement Impact Assessment (ref. 10219/AP/JRCB/R3) dated January 2019 by Soil Consultants Ltd (SCL).
- Structural Feasibility Report (ref. 218075.100 rev. B) dated January 2019 by Ian Harban Consulting Engineers.
- Drawings showing proposed plans, elevations and sections by Martin Evans Architects dated 5th June 2018.

This report provides a description of the FEAs that Geofem performed for this project, together with a summary of the outputs and an assessment of the likely influence of predicted ground movements on adjacent properties.

2 GROUND MOVEMENT ANALYSIS

2.1 3D FEA of proposed basement

The 3D FEA was performed using the specialised geotechnical finite element analysis software Plaxis 3D 2017. The FEA is described in the following sections.

2.1.1 Ground model

The proposed basement, adjacent property foundations and surrounding ground were simulated in one finite element model. The vertical boundaries to the mesh were placed at about 50m horizontal distance from the basement such that boundary effects were not significant. Similarly, the fixed base of the mesh was placed at +70mOD, 35m below proposed basement formation level. The ground level was adopted as +108.4mOD at the site, generally sloping at 1:10 downwards to the east and upwards to the west, as shown in Figure 2.1.

The adjacent buildings are situated at different levels built into the natural ground slope. To create the geometries for these, an artificial perimeter wall was included about 10m from the study area to separate the assumed natural ground profile from the terraced site levels around the study area, as shown in

Figure 2.1. The natural ground level was assumed horizontal east and west of the artificial perimeter wall in order to simplify the geometry and to minimise the stress changes associated with forming the slope in the third stage of the analysis (see Section 2.1.5).

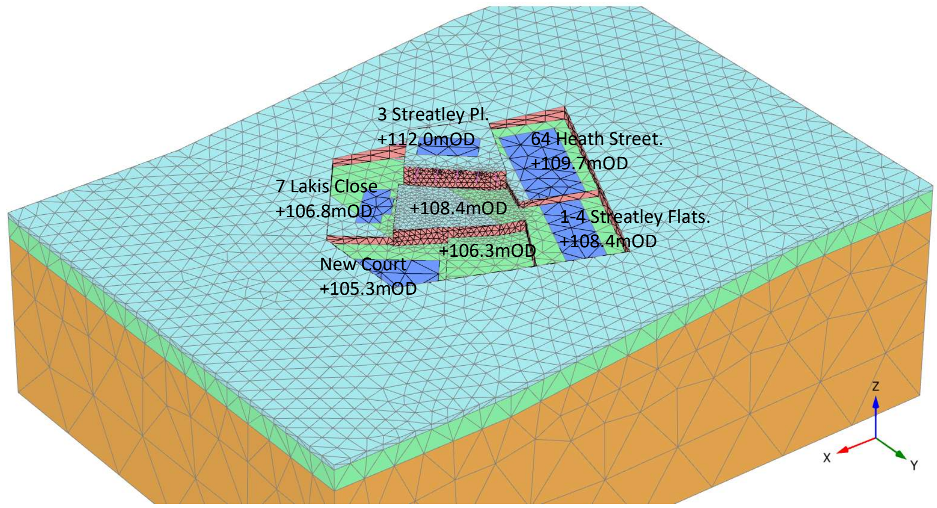

Figure 2.1: FEA model geometry (with existing sub-structures)

The adopted site levels for the adjacent buildings are shown in Figure 2.1. These were separated by retaining walls as shown in red and described in Section 2.1.2.

During the ground investigation by SCL, the borehole within the site encountered 0.6m of Made Ground overlying about 3.9m of Bagshot Formation at +108.4mOD level, overlying Claygate Member at +104.5mOD extending to at least the full depth of the borehole at +91.55mOD. In the FEA model, a thicker 1.0m surface layer of weak Made Ground (shown as pale blue in Figure 2.1) was adopted for conservatism, overlying a 4.0m thickness of Bagshot Formation (green in Figure 2.1), overlying Claygate Member (brown in Figure 2.1). According to the SCL report, published BGS information indicates that London Clay occurs below the Claygate Member extending to at least 30m depth. Since London Clay at depth would be expected to have similar properties to the Claygate Member encountered at this site, the Claygate Member was adopted to the base of the model at +70mOD level. This sequence was adopted throughout the model, assumed parallel to the adopted ground surface. This resulted in Bagshot Formation outcropping at some site levels, which was considered reasonable because adjacent buildings would have been constructed on competent formation rather than on weak Made Ground.

For the Made Ground layer, a basic linear elastic perfectly-plastic soil model was used with Mohr-Coulomb failure criterion, with parameters (see Table 2.1) based on the findings of the ground investigation. The properties of this surface layer were not considered to have a significant influence on the critical outputs from the model.

The Bagshot Formation and Claygate Member had a more significant influence on the prediction of ground movements associated with excavation of the basement and loading changes. The prediction was improved by including the stress- and strain-dependency of stiffness in their definition in the constitutive models. Although the advanced parameter testing required to obtain such parameters was not undertaken for this site, conservative parameters were estimated based on the information available including soil characterisation test results. This is considered preferable to estimating the parameters for less accurate basic soil models that can produce highly erroneous ground movement predictions, particularly in unloading situations such as a basement excavation. The small-strain stiffness version of the Hardening soil model (HSsmall) was adopted for these soils with the parameters shown in Table 2.1 (glossary of terms and derivation of G_0 in Appendix 1).

Hardening small strain model		
	Bagshot Formation	Claygate Member
E_{50}^{ref}	20MPa	30MPa
E_{ur}^{ref}	60MPa	90MPa
E_{oed}^{ref}	20MPa	30MPa
m	0.8	1.0
ν_{ur}	0.2	0.2
p^{ref}	100kPa	100kPa
φ'	27°	23°
c'	3kPa	5kPa
ψ	0°	0°
K_0^{nc}	0.55	0.609
R_f	0.9	0.9
p_p	0.5MPa	2MPa
G_0^{ref}	35MPa	53MPa
$\gamma_{0.7}$	$1x10^{-4}$	$8x10^{-5}$
γ	19kN/m^3	20kN/m^3
K_0	0.7	1.0

Mohr-Coulomb linear elastic	
	Made Ground
E	20MPa
ν	0.2
φ'	25°
c'	0.1kPa
ψ	0°
γ	18kN/m^3
K_0	0.58

Linear elastic model		
	rc	brickwork
E	30GPa	5GPa
ν	0.2	0.2
γ	0kN/m^3	0kN/m^3

Table 2.1: FEA material parameters

Undrained (short-term) or drained (long-term) conditions were assumed for the Bagshot and Claygate layers as described in the construction stages in Section 2.1.5. A further advantage of using the advanced HSsmall model for these soils was the more accurate prediction of undrained behaviour when using effective stress parameters. Drained conditions were assumed for the Made Ground.

Groundwater seepage was encountered during the SCL ground investigation at 7.35m depth. Accordingly, in the FEA model, groundwater level was adopted at 7.0m below and parallel with the ground level. Hydrostatic conditions were assumed for the groundwater and zero pore pressure above groundwater level.

2.1.2 Existing building geometry

The site contains a few lightweight existing buildings which were not considered in the FEA model because their bearing pressures would be low and it is conservative in terms of ground movement to assume zero pre-loading from existing buildings on the site.

The adjacent buildings were simulated as surface foundations composed of plate elements covering all or a part of the building footprint closest to the study site. The flat 2D plate elements shown as blue in Figures 2.1 and 2.2 had the bending stiffness equivalent to 0.3m thick reinforced concrete with a Young's modulus of 30GPa.

The retaining walls supporting terraces in the ground were also composed of plate elements, as shown in red in Figure 2.1. These elements had the bending stiffness equivalent to 0.3m thick brickwork with an assumed Young's modulus of 5GPa. These walls were extended an appropriate depth below ground level to maintain stability since they act more as embedded cantilever walls than gravity walls.

The highest retaining wall to 3 Streatley Place needed additional support in the form of some bracing. In the absence of further details on existing support, spring supports of arbitrary stiffness 50kN/mm were installed at 2 locations near the top of the walls as shown by the pink lines in Figure 2.3.

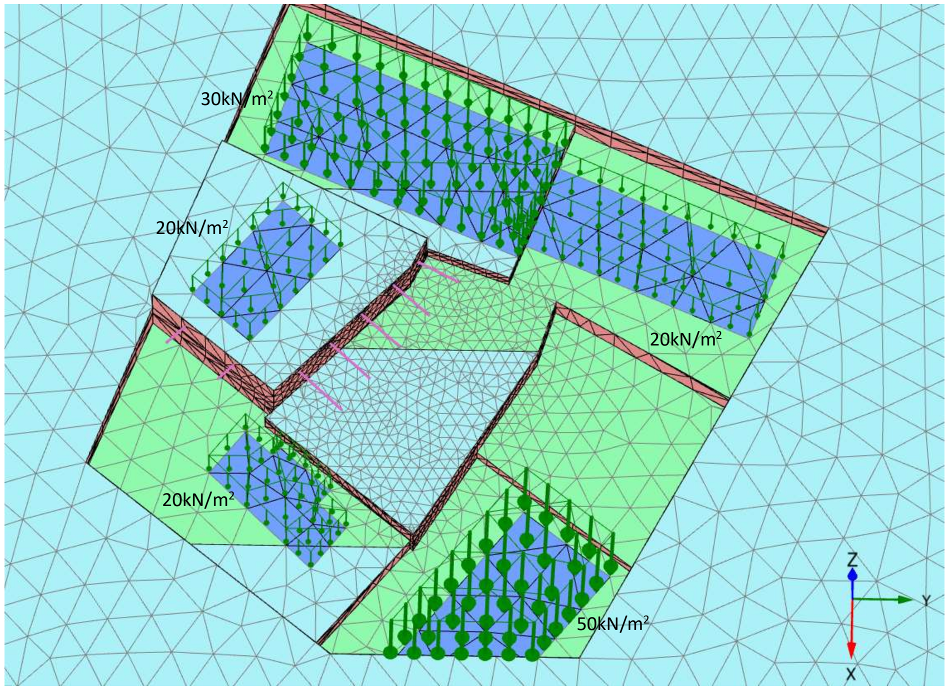

Figure 2.2: FEA model of existing geometry showing building loads

2.1.3 Proposed basement geometry

Existing retaining walls around and above the proposed basement were stiffened by substituting the plate materials for the properties of assumed 0.5m thick reinforced concrete, as shown coloured grey in Figure 2.3. Other new basement perimeter walls and underpinning were modelled in the same way. The ground floor and lower ground floors slabs were modelled with plate elements with full moment connection to the basement walls as 0.3m (blue in Figure 2.3) and 0.4m (mauve in Figure 2.4) thick respectively reinforced concrete.

The proposed piled foundations were modelled with the "embedded beam" feature in Plaxis which uses a linear elastic beam element to model the bending and axial stiffness of the pile while the soil within the pile's radius has its stiffness enhanced to that of the pile material in order to model pile/soil interaction effects. An assumed arrangement of 36no. vertical piles (6 rows of 6) and 5 pairs of raking piles, as shown in dark green in Figure 2.5, were modelled as 0.3m diameter reinforced concrete, extending from the ground floor level initially to a toe level of +98.0mOD. The pile lengths between ground and lower ground floor level were deactivated in an intermediate analysis stage as described in Section 2.1.5. Note that the pile arrangement adopted in the FEA model was broadly based on the construction sequence sketches in the Structural Feasibility Report in order to estimate ground movement but should not be regarded as a design proposal. The interface friction was set to 70% of internal soil friction.

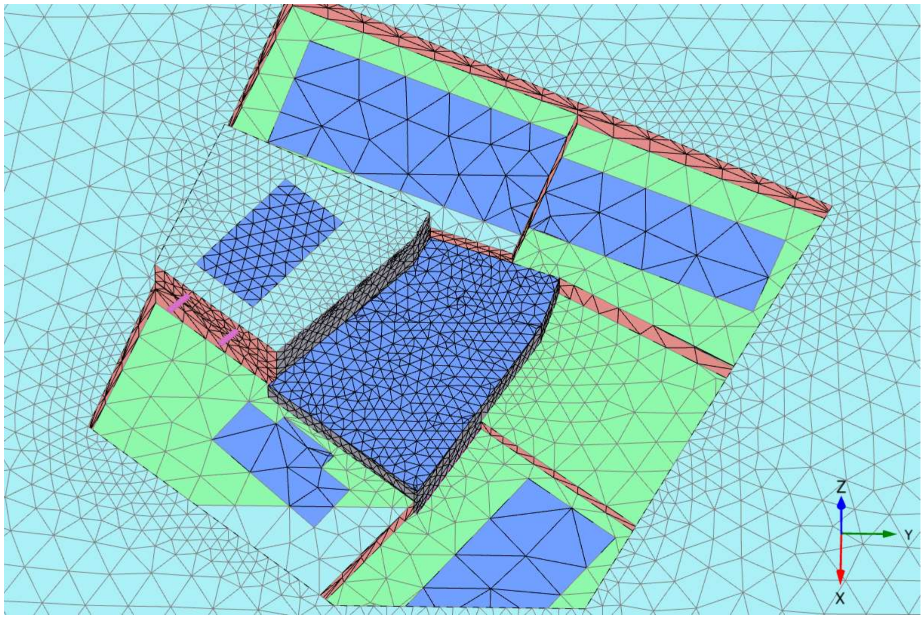

Figure 2.3: FEA model geometry showing proposed basement structure

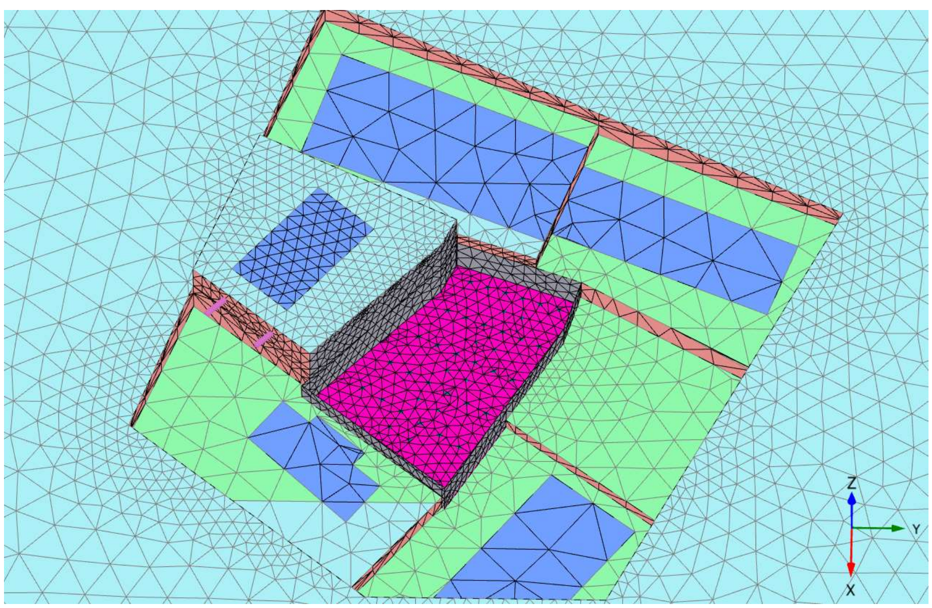

Figure 2.4: FEA model geometry showing proposed basement structure (ground floor slab hidden)

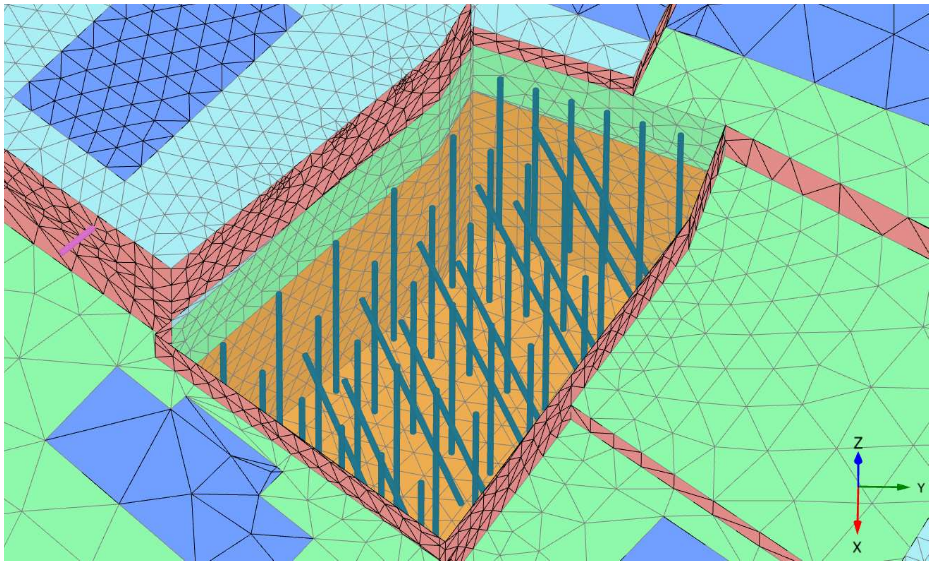

Figure 2.5: FEA model geometry showing assumed foundation piling arrangement (soil around piles hidden)

2.1.4 Loading

Uniformly distributed area loads were applied to the adjacent building footprints as shown in Figure 2.2. On completion of construction, a uniformly distributed load of 40kN/m^2 was applied to the lower ground floor slab to represent the building load. The structural materials had zero self-weight.

2.1.5 Construction sequence

The construction sequence shown in Table 2.2 was adopted in the analysis model. The first three stages were needed to establish the existing condition. It is necessary to start the analysis without structures and horizontal ground level in order to establish appropriate initial stresses in the ground. Undrained conditions were modelled in terms of effective stress with a high bulk modulus to represent virtually incompressible pore water and obtain excess pore pressure values. In the final construction stage, drained conditions were restored by allowing excess pore pressures to dissipate to less than 1kPa in a consolidation analysis.

In Stage 4 the ground floor slab was installed with a 4m wide gap as shown in Figure 2.6 which is proposed to allow excavation of the basement. In Stage 5 a berm was formed of 5m width at the top coincident with the ground floor slab width and with a slope angle of 42° as shown in Figure 2.6.

	Stage description	Bagshot and Claygate drainage conditions
1	Establish in situ stresses: horizontal ground level at +112.6mOD, groundwater level at 7m below ground level in next stage with hydrostatic conditions, K_0 values and pre-consolidation stress as shown in Table 2.1.	Drained
2	Install existing structures.	Drained
3	Remove ground to create 1:10 slope with ground level of +108.4mOD at study site. Apply existing building loads.	Drained
4	Install proposed temporary raking struts to 3 Streatley Place retaining wall and foundation piling. Remove existing struts to retaining wall. Change 3 Streatley Place retaining wall properties to reinforced concrete from GF slab level to top of upstand. Install GF slab with gap for excavation. Reset datum point for displacements to zero at the start of this stage.	Undrained
5	Excavate to LGF level leaving a berm to 3 Streatley Place.	Undrained
6	Install underpinning to 3 Streatley Place to LGF level, LGF slab and remaining basement perimeter walls.	Undrained
7	Excavate remaining ground within basement volume.	Undrained
8	Apply building load. Complete GF slab installation.	Undrained
9	Long term consolidation of Bagshot and Claygate soils.	Drained (consolidation analysis)

Table 2.2: Construction sequence

The structures were "wished in place", i.e. without consideration of installation effects. The magnitude of ground movements resulting from installation effects depends on a complex interaction between the underpinning panel size, the safety margin on trench stability and the time taken to excavate the panel, pour concrete and for the concrete to set. Simulation of installations involving the formation of a 3D void in the ground, placement of fluid concrete and subsequent concrete setting are rarely attempted in practical analyses because of the complexity involved and because such methods are insufficiently validated to know whether they provide realistic predictions. The use of plate elements to represent the basement walls has an element of conservatism since the moment restoring effect of soil-wall friction on the zero-thickness elements is ignored so, overall, this approach tends to give deflection predictions that err on the conservative side.

Alternatively, simple rules of thumb can be used to estimate installation-induced ground movements and this was performed as described in Section 3.

In the FEA model, it was assumed that the contractor would use appropriate construction methods and good workmanship to minimise installation-induced ground movements while installing the underpinning in a hit-and-miss fashion. Consequently, underpinning elements were installed prior to soil excavation. Therefore, the FEA model predicted ground movements only resulting from the change in the magnitude and distribution of the loading, including removal of the weight of soil occupying the proposed basement volume, and not directly due to installation effects which are addressed in Section 3.

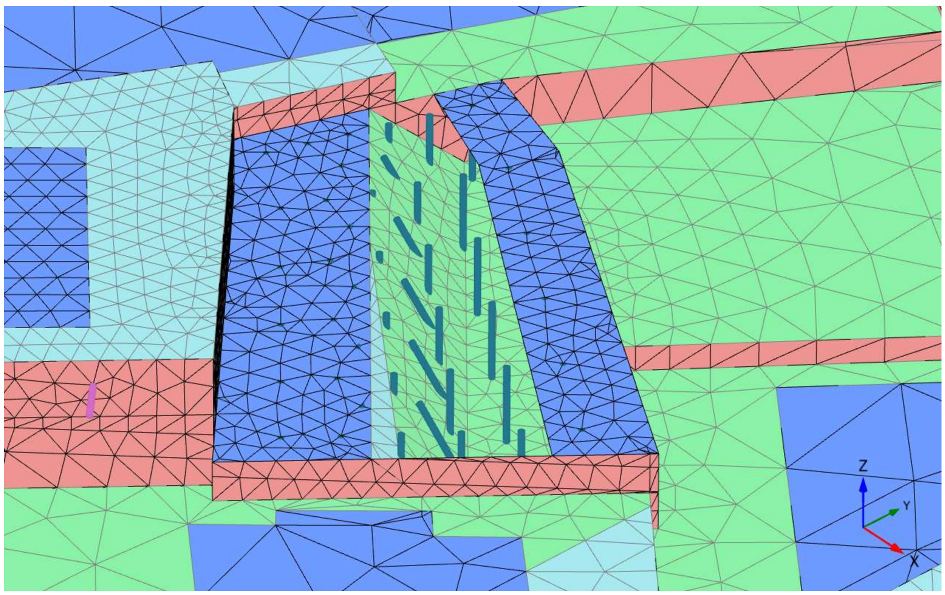

Figure 2.6: FEA model geometry showing berm excavation

2.1.6 Analysis results

Figures 2.7, 2.8 and 2.9 show the predicted short-term (undrained conditions) vertical ground movements (heave positive and settlement negative) at basement formation level and the surrounding ground surface resulting from excavation to form the berm, completion of basement excavation and the subsequent proposed loading respectively. A maximum heave of about 3.5mm was predicted in the proposed basement on the NE side due to the additional permanent heave occurring beyond the toe of the berm, and up to about 2.2mm on the SW side. Up to about 1mm heave was predicted at the nearest edge of the adjacent buildings. The predicted basement heave reduced to less than 2mm due to the rebalancing effect of the building load.

Up to 1mm settlement was predicted behind the retaining wall to 3 Streatley Place due to flexure of this unsupported wall, but it was localised around the wall and remote from the adjacent building. It is proposed to control deflection of this retaining wall with temporary struts but these were omitted conservatively from the FEA model.

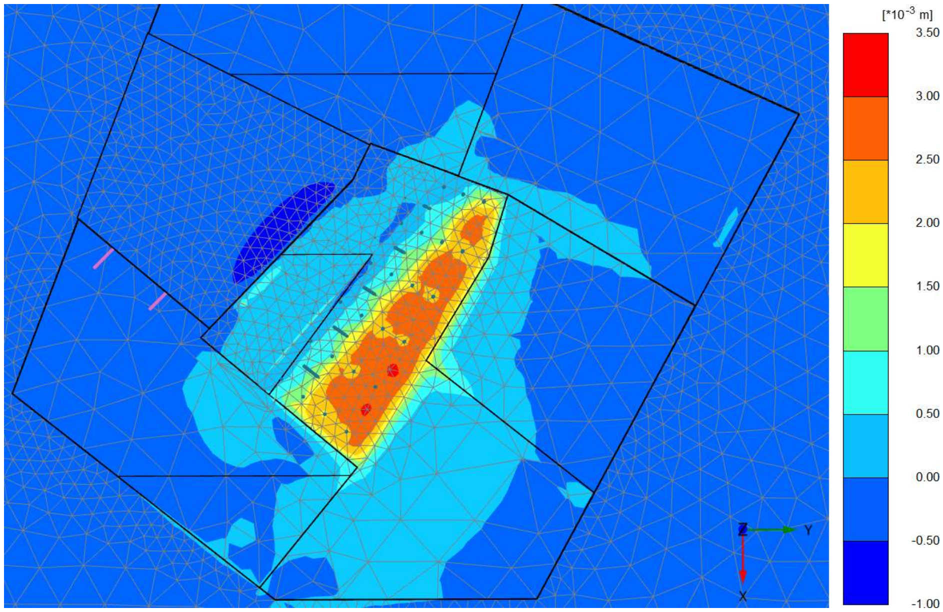

Figure 2.7: Predicted short-term vertical ground movement of basement formation level and surrounding ground surface following excavation to form berm

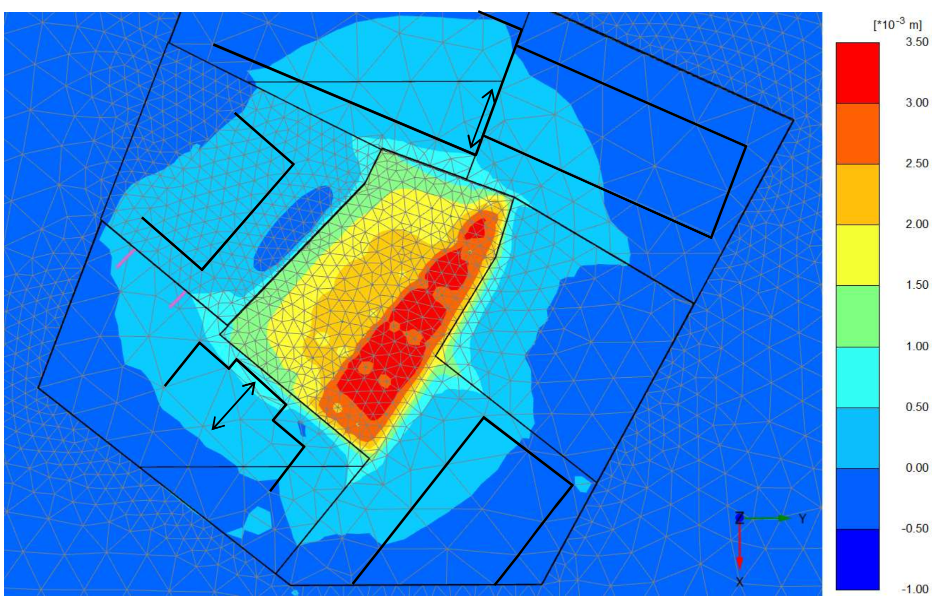

Figure 2.8: Predicted short-term vertical ground movement of basement formation level and surrounding ground surface following excavation of proposed basement

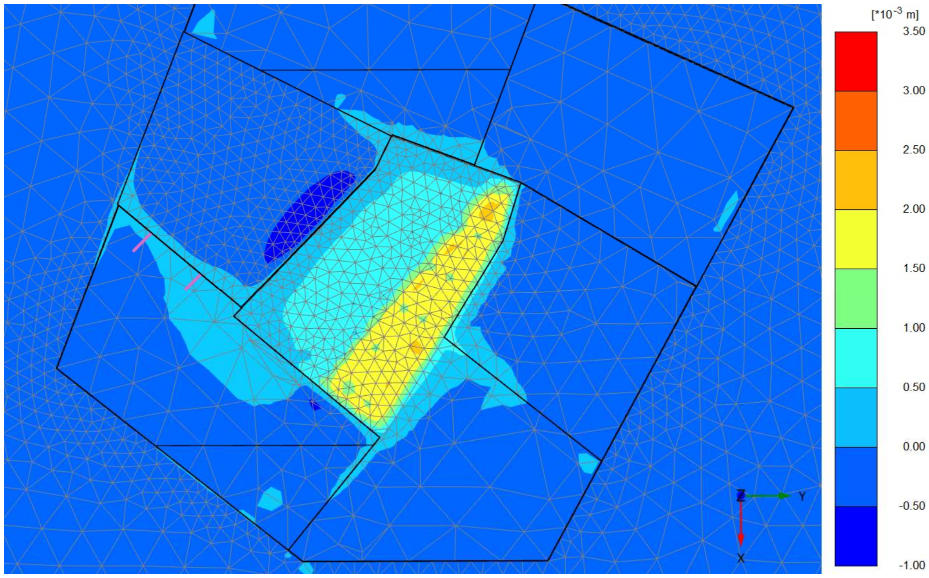

Figure 2.9: Predicted short-term vertical ground movement of basement formation level and surrounding ground surface following loading of proposed basement

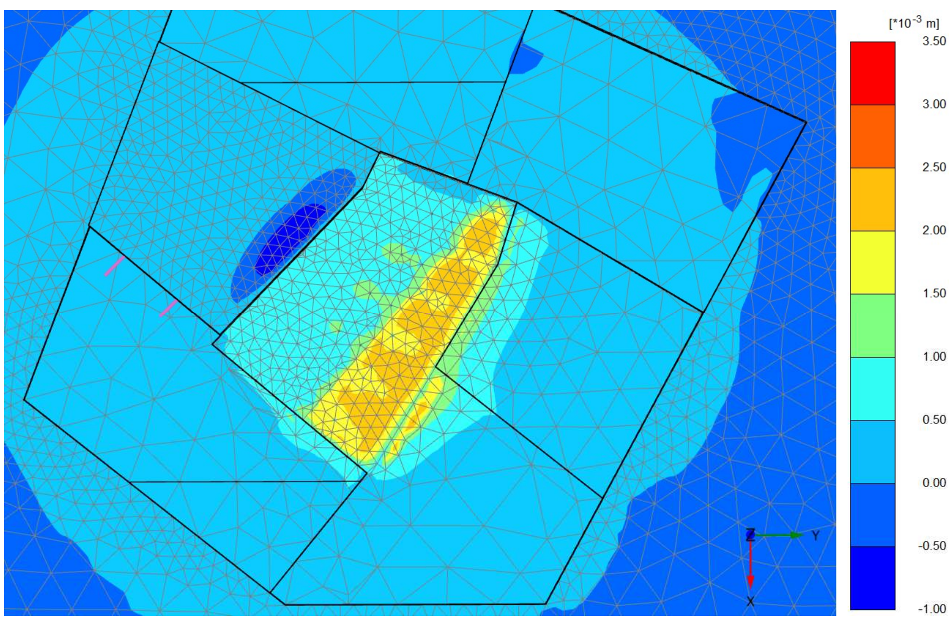

Figure 2.10: Predicted long-term vertical ground movement of basement formation level and surrounding ground surface due to basement construction

The predicted long-term (drained conditions) vertical ground movements at basement formation level and the surrounding ground surface are shown in

Figure 2.10. A maximum value of 2.4mm was predicted along the NE side of the proposed basement.

The most onerous conditions on adjacent buildings were predicted to occur in the short-term following basement excavation but prior to building load. This output is shown in Figure 2.8 together with the adjacent building footprints. The two most onerous cases in terms of differential settlement are shown by the arrows at 7 Lakis Close and 64 Heath Street. The profiles of predicted short-term vertical deflection along the arrows are plotted against horizontal distance from the basement perimeter in Figures 2.11 and 2.12. The maximum relative deflection Δ at 7 Lakis Close was predicted to be 0.11mm giving a deflection ratio Δ/L of 1/47,000, while the maximum Δ at 64 Heath Street was predicted to be 0.09mm giving Δ/L of only about 1/106,000. The predicted horizontal tensile strain in these areas was less than 0.05%.

A deflection ratio Δ/L of 1/47,000 with a tensile strain of 0.05% would be expected to cause up to *very slight* damage on the Burland scale comprising less than 1mm wide cracks that are easily treated during normal redecoration and is acceptable on most projects.

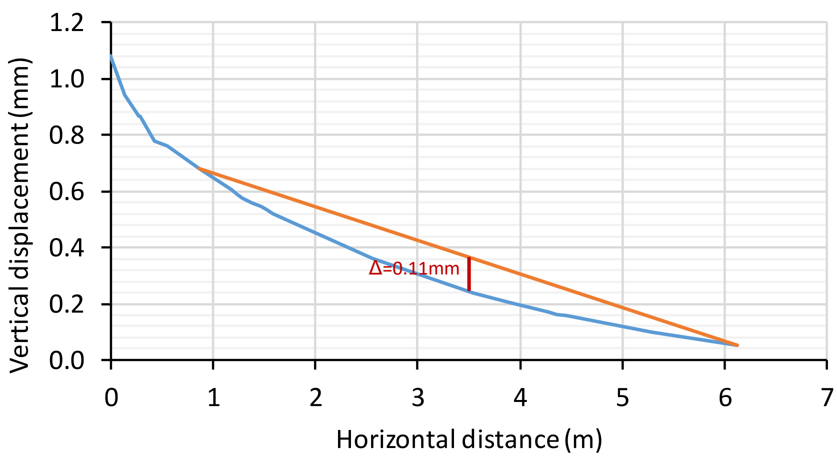

Figure 2.11: Predicted short-term vertical deflection at 7 Lakis Close

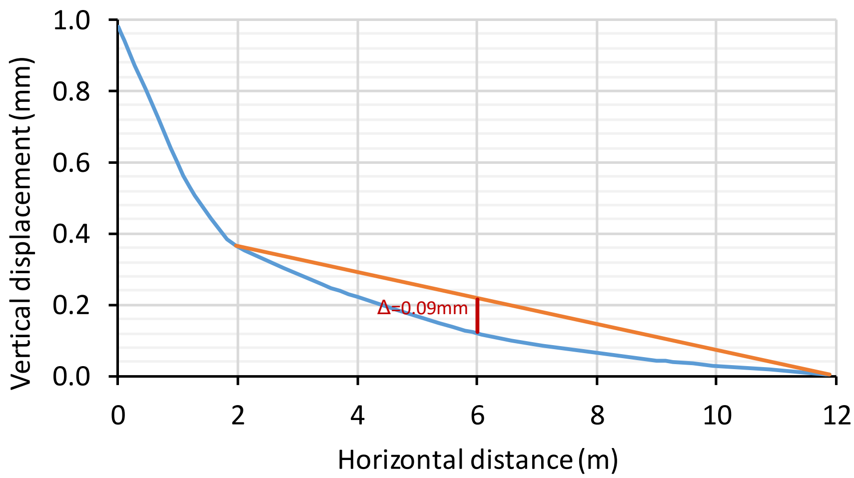

Figure 2.12: Predicted short-term vertical deflection at 64 Heath Street

3 INSTALLATION-INDUCED GROUND MOVEMENTS

The empirical charts for diaphragm wall installation have been used in this instance to estimate the installation-induced ground movements (CIRIA C760 Fig 6.9) which may occur as a result of construction of new retaining walls and the south-western boundary wall underpinning. It is understood that underpinning will be constructed in a controlled 'hit-and-miss' fashion with multiple lifts where necessary. Temporary backfilling should be completed as appropriate to maintain support to the new retaining structure, prior to the subsequent basement excavation.

SCL have used the empirical CIRIA charts to estimate vertical and horizontal ground movements beneath the adjacent structures and the assessed damage categories due to installation are summarised in Table 3.1.

Building	Deflection Ratio Δ/L	Horizontal strain ε_h	Damage Category
3 Streatley Place	0.01%	0.02%	0
Heath St/Streatley Flats	0.01%	0.02%	0
Lakis Close	0.02%	0.025%	0

Table 3.1: Predicted damage categories in surrounding buildings due to installation induced ground movements

4 CONCLUSIONS AND RECOMMENDATIONS

Geofem has performed a ground movement analysis using 3D finite element analysis (FEA) of the proposed basement for 6 Streatley Place.

Construction of the proposed basement was assumed to be undertaken using appropriate construction methods and good workmanship to minimise installation-induced ground movements while installing the underpinning in a hit-and-miss fashion. Therefore, the FEA model predicted ground movements only resulting from the change in the magnitude and distribution of the loading, including removal of the weight of soil occupying the proposed basement volume.

The formation level of the proposed basement was predicted to heave up to about 3.5mm in the short-term on the NE side, decreasing to about 2.2mm on the SW side. The heave was predicted to reduce somewhat due to subsequent loading from the building and long-term consolidation. In the short-term case, the maximum deflection ratio Δ/L was predicted to be 1/47,000 and the maximum horizontal tensile strain 0.05%.

A deflection ratio Δ/L of 1/47,000 with a tensile strain of 0.05% would be expected to cause up to *very slight* damage on the Burland scale comprising less than 1mm wide cracks that are easily treated during normal redecoration and is acceptable on most projects.

Installation-induced ground movements were predicted by SCL using empirical relationships to be negligible.

Appendix 1

Glossary of material model parameters

Symbol	Description and typical ranges
E'	Drained linear elastic stiffness (Young's modulus).
v	Poisson's ratio.
φ'	Drained friction angle.
c'	Drained cohesion.
ψ	Dilation angle.
K_0	In situ coefficient of earth pressure (or stress ratio).
γ	Weight density.
E_{50}^{ref}	Contiguous stiffness in standard drained triaxial test at reference confining stress p^{ref}.
E_{ur}^{ref}	Elastic unloading/reloading stiffness in drained triaxial test at reference confining stress p^{ref}.
E_{oed}^{ref}	Tangent stiffness for primary oedometer loading at reference vertical stress p^{ref}.
m	Power for stress dependency of stiffness.
v_{ur}	Drained Poisson's ratio during elastic unloading/reloading.
p^{ref}	Reference stress for stiffness values.
K_0^{nc}	In situ coefficient of earth pressure (or stress ratio) for normal consolidation.
R_f	Failure ratio q_f/q_a (curve-fitting parameter)
p_p	Isotropic pre-consolidation stress.
G_0^{ref}	Shear modulus at very small strains at reference confining stress.
$\gamma_{0.7}$	Shear strain at which $G_{sec} = 0.722G_0$.

Assessment of G_0^{ref} value:

Reference minor principle effective stress p^{ref} occurs on site at about 6.5m depth (assuming K_0=0.8 for Claygate Member). At this level s_u≈80kPa. Using a rule of thumb for foundation settlement calculation that $E_u=500s_u$, gives E_u≈40MPa. Using rule of thumb that stiffness at very small strain $E_{u,0}\approx 5E_u$, gives $E_{u,0}$≈200MPa. Using elastic relationships to convert to shear modulus G_0:

$$G_0 = \frac{E_{u,0}}{2(1+v_u)} = E_{u,0}/3 = 67\text{MPa}.$$

G_0^{ref} conservatively assessed as 53MPa for the Claygate Member.

From site investigation data, s_u of Bagshot Formation about 20% lower than Claygate Member at equivalent confining stress, so G_0^{ref} conservatively assessed as 35MPa for Bagshot Formation.