Walsh Associates

Camden Lock Village, London

Basement Impact Assessment
Building C - Revision 1

May 2015

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1. INTRODUCTION

It is proposed to demolish the existing buildings and some of the road infrastructure at Camden Lock Village and replace them with a number of medium to high rise developments with basements levels ranging in depth between approximately 4m and 16m.

Card Geotechnics Limited (CGL) has been instructed by Walsh Associates to undertake a Basement Impact Assessment (BIA) for the proposed basement development blocks to assess the potential impact on surrounding buildings, infrastructure and hydrological features. Given the size of the site and proposed developments, the BIA will be undertaken in stages. This report assesses the impact of Building C only. The impact of basement development for Building A and Building D will be assessed within separate BIA reports.

Camden Guidance CPG4[1] requires Basement Impact Assessments to be undertaken for new basements in the borough and sets out 5 stages:

1. Screening
2. Scoping
3. Site investigation
4. Impact assessment
5. Review and decision making

This report is intended to address the screening, scoping and impact assessment processes set out in CPG4 and the Camden geological, hydrogeological, and hydrological study (CGHHS)[2]. It identifies key issues relating to land stability, hydrogeology and hydrology as part of the screening process (Stage 1). CGL has previously scoped and completed an extensive ground investigation[3] for the entire site (Stage 2 and Stage 3), and as such the scoping process comprises a summary of the findings of the current site investigation and derivation of an appropriate ground model and design parameters for the site to allow the ground movement and damage assessment calculations to be undertaken (Stage 4).

[1] Camden Planning Guidance, CPG4, Basements and Lightwells, September 2013.
[2] Ove Arup and Partners, Camden geological, hydrogeological, and hydrological study. Guidance for subterranean development, November 2010.
[3] Card Geotechnics Limited. Camden Lock Village – Geotechnical and Geoenvironmental Interpretative report. January 2015.

2. SITE CONTEXT

2.1 Site location

The site is situated off Castlehaven Road in Camden, northwest London. The Ordnance Survey grid reference for the approximate centre of the site is 528765N, 184210E.

A site location plan is presented as Figure 1.

2.2 Site layout

The site is triangular in shape covering an area of approximately 6000m^2. The site is bordered by National Rail viaducts to the north, south and east and by Castlehaven Road to the northwest. The *Hawley Arms* pub borders the site along the south west corner.

The site is occupied by an office building with associated car parking, a waste transfer depot, and vehicle maintenance and repair workshops, which are predominately situated in the arches beneath the railway viaducts.

LUL tunnels run below Chalk Farm Road 60m south west of the site and also below Kentish Town Road located approximately 100 east of the site. The Grand Union Canal runs in a east to west direction 40m south of the site.

The site layout plan is presented within Figure 2. This Figure also shows the outline of the site within which Building C is proposed. Exploratory hole locations completed during the current site investigation across the entire site are also presented on Figure 2.

2.3 Proposed development

The proposed development of Building C comprises two multi storey buildings with a two storey basement beneath the entire site footprint. The upper floors of the buildings will contain residential properties and the lower floors and basement levels will predominantly contain retail units, office space, leisure facilities and car parking.

The proposed basement across Building C is 16m deep with its formation level at approximately 11mOD. It is currently proposed to construct the basement using top down construction methods with a single intermediate floor level at approximately 16.5mOD. Pile cap level is assumed to be at 27.0mOD and ground level across the site is typically at 27.0mOD. Contiguous piled walls of 0.75m diameter at 0.9m spacing are currently proposed to support the basement excavation.

The above information is taken from current detailed drawings provided by Walsh Associates (structural engineer for the project) and presented within Appendix A.

The current drawings indicate that the proposed basement will not come within 3m to 4m of the network rail viaducts or neighbouring properties.

2.4 Historical Development

The historical development of the site was established by RPS in their November 2009[4] report and is briefly summarised below.

The site consisted of open fields until the *Regent's Canal* was constructed in the early 1800s, with residential properties constructed across the site. A number of these buildings were subsequently demolished during construction of the North London Overground Railway viaducts in the mid-1800s, with the remainder demolished in the mid-1970s for construction of the office building currently located on site.

2.5 Bomb damage and unexploded ordnance

The area experienced intensive bombing during the Second World War, with a number of properties being destroyed or damaged beyond repair.

A detailed unexploded ordnance (UXO) risk assessment[5] was undertaken by 6 Alpha Associates Limited in September 2014. The report notes that the risk posed by UXO at the site is 'low to medium' for basements and excavations within Building C. Additionally, no substantial damage to the viaducts were noted.

2.6 Published and unpublished geology

The British Geological Survey BGS map sheet 256 (North London)[6] indicates that the site is directly underlain by the London Clay Formation, which consists of stiff blue grey silty clay, weathering to brown silty clay near the surface.

The BGS[7] holds records of a number of historical ground investigations within 300m of the site. Selected logs are summarised in Table 1 and details are included in Appendix B.

[4] RPS (2009) Camden Lock Village London Borough of Camden. *An Archaeological Desk Based Assessment*. Ref: JLK0617 RO1. November 2009
[5] 6 Alpha Associates Limited (2014) *Detailed Unexploded Ordnance (UXO) Risk Assessment*. Ref: P4063. September 2014
[6] British Geological Survey. (1994) North London. Sheet 256. Solid and Drift Geology 1:50,000.
[7] www.bgs.ac.uk (accessed December 2014)

Table 1. Summary of BGS historical borehole records

BH record reference	Distance (m)	Direction	Base of BH (mbgl)	Ground water level (mbgl)	Depth to top of stratum (mbgl)				
					MG	London Clay Formation	Lambeth Group	Thanet Sand	Chalk
TQ28SE5	90	S	91.4	NR	-	0.0	42	NR	64
TQ28SE1203	300	SE	18.7	1.1	0.0	1.5	-	-	-
TQ28SE1204	300	SE	18.4	NR	0.0	0.9	-	-	-
TQ28SE1206	300	SE	9.6	1.1	0.0	2.1	-	-	-
TQ28SE1208	300	SE	9.4	NR	0.0	1.37	-	-	-
TQ28SE1239	180	NW	3.0	-	0.0	0.63	-	-	-
TQ28SE1240	180	NW	3.0	-	0.0	0.5	-	-	-
TQ28SE1241	180	NW	3.0	-	0.0	0.8	-	-	-
TQ28SE1242	180	NW	3.0	-	0.0	0.6	-	-	-
TQ28SE1491	190	SE	198.7	91.7	0.0	6.7	44.8	53.9	125.0
TQ28SE2272	257	SW	1.1	-	0.0	1.08	-	-	-

The historical borehole records generally recorded Made Ground ranging in thickness between 0.6m and 6.7m over the London Clay Formation. The surface of the Lambeth Ground was encountered at 42mbgl to 44.8mbgl and the Thanet Sand was encountered at 53.9mbgl underlain by Chalk encountered between 64mbgl and 125mbgl.

Generally shallow groundwater was not encountered or noted within the boreholes.

2.7 Hydrogeology

The Environment Agency[8] has produced an aquifer designation system consistent with the requirements of the Water Framework Directive. The designations have been set for superficial and bedrock geology and are based on the importance of aquifers for potable water supply and their role in supporting surface water bodies and wetland ecosystems.

The underlying London Clay Formation is classified as ‘Unproductive Strata’ and the site is not within a Groundwater Source Protection Zone (SPZ).

[8] www.environment-agency.gov.uk (September 2014)

2.8 Hydrology

Figure 11 of the Hampstead Heath Surface Water Catchments and Drainage of the Camden Geological, Hydrogeological and Hydrological report produced by Arup[2] presents a copy of the 'Lost Rivers of London' map produced by Barton. This shows that a number of springs outcrop at the base of the Bagshot Formation to the north, flowing through various drainage channels and in various directions into the watercourses of the district (most of which are now diverted underground), these includ the River *Westbourne*, *Tyburn* and *Fleet*. The map indicates that the *River Fleet* runs approximately 50m to the north of the site boundary and continues southeast where it links up with another tributary of the *River Fleet* and continues south at a distance of approximately 70m from the site towards the *River Thames*. Additionally, *Regent's Canal* is located approximately 40m south of the site, flowing in a west to east direction.

With reference to the Arup[2] report, the site is approximately 2.2km southeast of the catchment for the pond chains on Hampstead Heath.

With reference to the EA website, the site is not within a Flood Risk Zone.

Current mapping (Figure 15 CPG4) indicates that roads impacted by flooding in 1975 are located 40m north and 70m west of the site. The site is not within a region that was impacted by 2002 flooding or areas with potential to be at risk of surface water flooding.

3. SCREENING (STAGE 1)

3.1 Introduction

A screening process has been adopted in accordance with CPG4, based on the flowcharts presented in that document. Responses to the questions posed by the flowcharts are presented below, and where ‘yes’ or ‘unknown’ may be simply answered, with no analysis required, these answers have been provided.

3.2 Subterranean (Groundwater) flow

This section answers questions posed by Figure 1 of CPG4.

Table 2. Responses to Figure 1 of CPG4

Question	**Response**	**Action Required**
1a. Is the site located directly above an aquifer?	No The site is underlain by the London Clay Formation	None
1b. Will the proposed basement extend beneath the water table surface?	No The site is underlain by the impermeable London Clay Formation.	None
2. Is the site within 100m of a watercourse, well, or potential spring line?	Yes The *Regent’s Canal* is located approximately 40m south of the site. The *River Fleet* which is now culverted underground is located approximately 40m north of the site.	Investigation and assessment
3. Is the site within the catchment of the pond chains on Hampstead Heath?	No	None
4. Will the proposed basement development result in a change in the proportion of hard surfacing?	No The site is currently covered by hardstanding and is underlain by the relatively impermeable London Clay Formation.	None
5. As part of site drainage, will more surface water than at present be discharged to ground (e.g. via soakaways and/or SUDS)?	No All surface water will be discharged to the sewer network through existing and two new connections. Current drawings also suggest that SuDS measure will also be incorporated into the design.	None
6. Is the lowest point of the proposed excavation close to, or lower than, the mean water level in any local pond or spring lines?	No	None

In summary, the site is underlain by the relatively impermeable London Clay Formation. Regional groundwater flow is likely to be to the south towards the Regent's Canal and River Thames, evidenced by the spring lines shown on Barton's 'Lost Rivers of London'.

There is the potential for localised and small quantities of perched water within the Made Ground or within sandy/silty horizons in the London Clay Formation and groundwater seepage is likely between the Made Ground and London Clay Formation interface.

The proposed development will not increase the proportion of impermeable surfaces and as such there is likely to be no additional recharge to the ground above that of the existing hydrogeological regime.

3.3 Slope/land stability

This section answers questions posed by Figure 2 of CPG4.

Table 3. Responses to Figure 2 of CPG4

Question	**Response**	**Action required**
1. Does the site include slopes, natural or man-made, greater than about 1 in 8?	No	None
2. Will the proposed re-profiling of the landscaping at site change slopes at the property boundary to greater than about 1 in 8?	No	None
3. Does the development neighbour land including railway cuttings and the like with a slope greater than about 1 in 8?	No	None
4. Is the site within a wider hillside setting in which the general slope is greater than about 1 in 8?	No	None
5. Is the London Clay Formation the shallowest stratum on site?	Yes The London Clay Formation was encountered directly below a limited thickness of Made Ground during the current site investigation.	Investigation and assessment
6. Will any trees be felled as part of the proposed development and/or are any works proposed within any tree protection zones where trees are to be retained?	Yes, Some trees within the basement footprint will be removed as part of the basement excavation. However, it is understood that these are not within tree protection zones.	None

Question	Response	Action required
7. Is there a history of shrink/swell subsidence in the local area and/or evidence of such at the site?	Unknown The London Clay Formation is susceptible to seasonal shrink/swell movements and it is likely that these will occur, particularly in close proximity to high water demand trees. The impact of this on the proposed development and adjacent properties should be assessed.	Investigation and assessment
8. Is the site within 100m of a watercourse or a potential spring line?	Yes The *Regent's Canal* is located approximately 40m south of the site. The *River Fleet* which is now culverted underground is located approximately 40m north of the site.	Investigation and assessment
9. Is the site within an area of previously worked ground?	No Limited Made Ground was encountered on site during the current investigation, most associated with construction of the existing property and hardstanding.	None
10. Is the site within an aquifer?	No The London Clay Formation is classified as an 'Unproductive Strata'.	None
11. Is the site within 50m of the Hampstead Heath Ponds?	No The site is more than 2km downslope of the Hampstead Chain Catchment.	None
12. Is the site within 5m of a highway or pedestrian right of way?	Yes The site is adjacent to Castelhaven road and two Network Rail viaducts.	Investigation and assessment
13. Will the proposed basement significantly increase the differential depth of foundations relative to neighbouring properties?	Yes The neighbouring properties (*Hawley Arms*) and Network Rail infrastructure are likely to have shallow foundations.	Investigation and assessment
14. Is the site over (or within the exclusion zone of) any tunnels?	Yes The site is bordered to the south and north by Network Rail viaducts. It is understood that the proposed basement will not come within 3.5m of the viaducts.	Investigation and assessment

In summary, an investigation and impact assessment is required to confirm ground conditions and assess the magnitude of ground movements that may result from basement excavation and construction as these may affect adjacent structures and infrastructure.

The impact assessment will determine potential damage caused by ground movements to adjacent structures and infrastructure, and will recommend measures to mitigate potentially damaging movements.

The impact assessment will focus primarily on the impact of ground movement on the Network Rail Viaducts and neighbouring *Hawley Arms* Pub structure located at the south-western boundary of the site.

The impact of removal of any trees within the basement footprint is considered to be negligible considering the depth of soil removed to reach formation level.

3.4 Surface flow and flooding.

This section answers questions posed by Figure 3 of CPG4.

Table 4. Responses to Figure 3 of CPG4

Question	**Response**	**Action required**
1. Is the site within the catchment of the pond chains on Hampstead Heath?	No	None
2. As part of the proposed site drainage, will surface water flows (e.g. volume of rainfall and peak run-off), be materially changed from the existing route?	No All surface water will be discharged to the sewer network through existing and two new connections. Current drawings also suggest that SuDS measure will also be incorporated into the design.	None
3. Will the proposed development result in a change in the proportion of hard surfaced/paved external areas?	No The site is currently covered by hardstanding and is underlain by the relatively impermeable London Clay Formation.	None
4. Will the proposed basement result in a change to the profile of the inflows of surface water being received by adjacent properties or downstream watercourses?	No	None
5. Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream watercourses?	No	None
6. Is the site in an area known to be at risk from surface flooding… or is it at risk from flooding because the proposed basement is below the static water level of a nearby surface water feature?	No The site is not in a Flood Risk Zone and is not identified as a street that flooded in 1975 and 2002.	None

In summary, the proposed basement is to be constructed in areas of existing hardstanding and is therefore not anticipated to impact surface water flow. Additionally, the site is not known to be at risk from flooding and is underlain by the relatively impermeable London Clay Formation.

3.4.1 Conclusions

The items summarised below in Table 5 were identified as part of the Stage 1 screening process.

Table 5. Summary of Basement Impact Assessment requirements

Item	Description
	Subterranean (Groundwater flow)
1.	Assess the potential impact on the Regent Canal located 40m south of the site
2.	Assess impact of the historical course and current culverted course of the River Fleet (which comes within 40m of the site) on the proposed development
	Slope and land stability
1.	Assessment of potential movements associated with construction in the London Clay Formation, including short and long term heave movements, settlement associated with retaining wall deflections, and ground movements around the basement perimeter. Shrink/swell behaviour is a possibility.
2.	An assessment of the impact the proposed excavation and basement installation could have on neighbouring structures and their foundations.
	Surface flow and flooding
-	*No issues for scoping identified during screening.*

4. SCOPING (STAGE 2)

This section of the report provides the scoping process (Stage 2) of CPG4, which is used to identify potential impacts of the new basement as set out in the screening process in Section 3 of this report, and to recommend an appropriate investigation strategy.

An intrusive investigation was undertaken by CGL between 10th and 18th November 2014. This investigation satisfies the requirements of the screening and scoping process and details of this investigation are summarised in Section 5. Reference should be made to the site investigation report[3] for detailed findings of the works. The conceptual site model is discussed within Section 5.5

5. GROUND INVESTIGATION (STAGE 3)

5.1 Introduction

An intrusive investigation within the footprint of Building C was undertaken between the 10th and 18th November 2014. The investigation within the footprint of Building C comprised three window sampler boreholes (WS6 to WS8) to a maximum depth of 5.0 metres below ground level (mbgl) and three cable percussion boreholes (BH4 to BH6) to a maximum depth of 40mbgl.

The borehole arisings were recorded and representatively sampled by a suitably qualified geotechnical engineer from CGL in order to obtain samples for laboratory testing, and to characterise the near surface ground conditions across the site. Soil samples were obtained for chemical and geotechnical laboratory analysis. Standpipes were installed in all boreholes to enable subsequent gas and groundwater monitoring to be undertaken.

The scope of the ground investigation is considered acceptable to satisfy the requirements of Stage 2 (Scoping and Investigation) of CPG4.

5.2 Summary

The ground conditions encountered during the intrusive investigation are summarised in Table 6. Reference should be made to the CGL site investigation report[3] for detailed findings of the current site investigation.

Table 6. Summary of ground conditions encountered

Stratum	Top of stratum (mOD) [mbgl][a]	Typical thickness (m)
MADE GROUND Concrete overlying brown sandy gravelly silt or gravelly silty clay. Sand is fine to coarse. Gravel is fine to coarse subrounded to subangular of brick, flint and occasional concrete.	26.99 to 27.96 [0.0]	0.64 to 1.8
Firm dark orange brown slightly silty CLAY with occasional fine selenite crystals [WEATHERED LONDON CLAY FORMATION]	25.26 to 26.72 [0.64 to 1.8]	9.16 to 9.8
Stiff becoming very stiff and hard at depth closely fissured dark grey silty CLAY. Frequent fine selenite crystals noted. [LONDON CLAY FORMATION]	16.17 to 17.56 [9.16 to 9.8]	Base not proven at 40mbgl

a. mOD = metres above Ordnance Datum

The ground conditions encountered during the site investigation generally correlated with the BGS mapping of the area, with a limited thickness of Made Ground directly overlying the London Clay Formation. The upper surface of the London Clay Formation was found to be relatively consistent across the majority of the site.

5.2.1 Made Ground

The Made Ground was found to be relatively consistent across the majority of the site and comprised concrete or paving slabs overlying brown sandy gravelly silt or gravelly silty clay. No visual or olfactory evidence of contamination was noted in the boreholes.

5.2.2 London Clay Formation

The London Clay Formation was proved to a maximum depth of 40mbgl. The upper 9.16m to 9.8m of the clay was found to consist of firm brown silty clay (Weathered London Clay Formation), becoming stiff grey (unweathered) from 17.56mOD to 16.17mOD. SPT 'N' values in this stratum ranged from 5 to >50 increasing with depth. Undrained shear strength values can be derived from these values using established correlations[9] were found to range from 22.5kPa to >225kPa.

Laboratory testing on samples of the London Clay Formation recorded undrained shear strength (c_u) values of 47kPa to 533kPa, increasing with depth. Plots of SPT 'N' values and undrained shear strength values (kPa) against level (mOD) are presented as Figure 3 and Figure 4 respectively. These data plots have been extracted from the site investigation report and include data from all exploratory hole locations across the entire site and not just those undertaken within the footprint of Building C.

Moisture content and Atterberg Limits recorded within the clay are summarised in Table 7.

Table 7. Summary of liquid limits and Atterberg limits

Strata	**Moisture content (%)**	**Liquid limit (%)**	**Plastic limit (%)**	**Modified plasticity index, I' (%)**
London Clay Formation	20 to 33	48 to 75	20 to 31	28 to 49

These indicate that the material at this site is a high to very high plasticity clay of medium to high volume change potential.

[9] Tomlinson, M.J. (2001) *Foundations Design and Construction (7th Ed.)*. Pearson Prentice Hall

5.3 Groundwater

No groundwater strikes were recorded in the cable percussion boreholes during drilling and the boreholes remained dry when left open overnight. However, groundwater was recorded in all boreholes during the subsequent monitoring visits as summarised in Table 8. Due to the nature of the site, some positions were not accessible during monitoring visits due to parked vehicles.

Table 8. Summary of groundwater monitoring undertaken to date

Borehole [Ground level mOD]	Groundwater level (mOD) [Level of base of well (mOD)]				
	19/11/14	01/12/14	18/12/14	08/01/15	13/01/15
BH4 [27.37]	NR	26.19 [18.39]	26.01 [19.17]	25.94 [19.95]	23.31 [19.15]
BH5 [27.36]	NR	22.57 [19.80]	23.78 [20.31]	25.78 [20.31]	24.71 [20.31]
BH6 [27.96]	NR	NR	19.86 [19.51]	20.06 [19.56]	20.26 [19.43]
WS6 [27.06]	26.44 [25.00]	26.41 [24.97]	26.51 [25.97]	26.44 [25.00]	19.36 [24.96]
WS7 [27.06]	NR	25.79 [24.98]	26.01 [25.03]	26.20 [24.99]	26.28 [24.96]
WS8 [26.99]	26.53 [24.93]	26.48 [24.93]	NR	NR	26.76 [24.96]

The monitoring records indicate that standing groundwater across the site is at approximately 0.55mbgl to 8.1mbgl. It is considered that the groundwater in the boreholes during monitoring is likely to be due to water seepage at the interface between the Made Ground and London Clay Formation and also potentially due to very slow seepage within the silty sandy layers/pockets within the upper weathered London Clay Formation.

Bailing of the boreholes during current monitoring visits confirm that that the infiltration rate of perched water is negligible with water levels recovering by less than 50mm four hours after bailing the boreholes dry.

5.4 Geotechnical Design Parameters

Geotechnical design parameters are recommended based on the information from the intrusive investigation and published data from the well-studied London geology. These are summarised in Table 9. The values are unfactored (Serviceability Limit State) parameters and are considered to be characteristic values for the local soils.

Table 9. Geotechnical design parameters

Stratum	**Depth (mbgl)** **Level [Mod]**	**Bulk Unit Weight γ_b (kN/m^3)**	**Undrained Cohesion c_u (kPa) [c']**	**Friction Angle ϕ' (°)**	**Young's Modulus E_u (MPa) [E']**
Made Ground	0 [27.5]	18	30 [0]	26[d]	18[b] [13.5][c]
London Clay Formation	1.5 [26.0]	20	50 + 6z[e] [5]	24[d]	30 + 3.6z[b] [22.5 + 2.7z][c]

a. *Burland et. al (Eds) (2001) Building response to tunnelling, CIRIA Special Publication 200, CIRIA*
b. *Based on 600c_u - Burland, Standing J.R., and Jardine F.M. (eds) (2001), Building response to tunnelling, case studies from construction of the Jubilee Line Extension London, CIRIA Special Publication 200. Increased to 1000 c_u for London Clay Formation within retaining wall deflection calculations.*
c. *Based on 0.75Eu - Burland, Standing J.R., and Jardine F.M. (eds) (2001), Building response to tunnelling, case studies from construction of the Jubilee Line Extension London, CIRIA Special Publication 200.*
d. *BS 8002:1994 Code of practice for Earth retaining structures, British Standards institution.*
e. *z = depth below surface of London Clay Formation.*

A long term design water level of approximately 2m above final formation level is recommended. This value has been chosen assuming that piles will be cast tight against the London Clay and prevents water seepage down the back of the wall. Additionally, it is understood that a drainage channel will be incorporated between the contiguous piled wall and basement liner wall so water pressures will not be allowed to build up behind the wall or below the basement slab over the long term.

5.5 Conceptual site model (Stage 3)

A conceptual site model (CSM) has been developed based on the available data and in accordance with the recommendations of the Camden geological, hydrogeological and hydrological study[10] (CGHHS) report.

A basement plan is shown in Figure 5 and Figure 6 which shows critical cross-sections through the identified critical constraints around the perimeter of the basement. The main construction activities causing movement of the neighbouring properties and infrastructure are summarised below;

1. Vertical and lateral ground movements due to contiguous piled wall installation.

[10] Ove Arup and Partners Limited (2010). *London Borough of Camden. Camden geological, hydrogeological and hydrological study. Guidance for subterranean development.* Issue 01, November 2010.

2. Stress relief and heave movements due to excavation of the basement within the piled wall. This should be considered over the short and long term. It is understood that piled foundation and suspended floor slabs will be adopted for the new development. Based on this, it will be assumed that there is little net reduction in stress relief below the basement slab in the long term. This is considered to model the worst case with regard to ground movement and corresponding impact on neighbouring structures and infrastructure.

3. Ground settlement due to piled wall deflection during excavation in front of the wall.

6. BASEMENT IMPACT ASSESSMENT – LAND STABILITY (STAGE 4)

6.1 Introduction

This section provides calculations to assess ground movements that may result from the construction of the proposed basement and how these may affect the adjacent structures and infrastructure. It is understood that a 0.75m diameter, 0.9m spacing contiguous piled wall will form the temporary and permanent support system for the excavation.

6.2 Critical sections for analysis

Based on discussion with Walsh Associates and with reference to current development drawings, a number of constraints have been identified (see Figure 5 and Figure 6) that will be considered within the ground movement analysis and damage assessment predictions sections of this report. The identified constraints and critical sections that will be assessed are summarised in the following sections.

6.2.1 Section adjacent to Network Rail viaducts

The site is bordered by two National Rail viaducts to the north and south. The viaducts converge along the eastern boundary of the site. Current drawings indicate that the proposed basement will not come closer than 3.5m from the viaducts.

With reference to CGLs archive and based on current drawings the viaduct arch footings are assumed to be founded at 2mbgl. This is a conservative assumption for retaining wall analysis.

A surcharge load of 200kPa for the viaducts has been provided by Walsh Associates for use within the ground movement calculations. The viaducts are approximately 10m wide perpendicular to the basement.

6.2.2 Section adjacent to neighbouring development (The Hawley Arms)

The *Hawley Arms* Pub is located along the south western boundary. The structure comprises three above ground storeys and a single basement level extending to approximately 3mbgl in the region (25.0mOD). Current drawings indicate the basement wall be offset approximately 1m from the party wall.

A surcharge load of 100kPa for the strip foundation of this property has been assumed for use within the ground movement calculations.

6.2.3 Section adjacent to existing oil tank

An existing UK Power Network (UKPN) oil tank has been identified onsite and it is understood that this tank and associated pipework are to remain in place with the proposed basement constructed around it.

Current drawings indicate that the basement will not be located closer than approximately 1.5m from the tank. Drawings also indicate that the tank is founded at approximately 3.5mbgl (23.5mOD).

It is understood through discussion with the client that movements in the region of 10mm horizontally and vertically are tolerable for the oil tank.

6.2.4 Section adjacent to Castlehaven Road

Castlehaven Road forms the western boundary of the site. Current drawings indicate that the road infrastructure will come within 1m of the piled basement wall. Ground movements in this region will be assessed to determine the potential impact of the basement construction on the road underlying service runs.

Movements in the region of 10mm vertical and horizontal are considered tolerable for such infrastructure.

6.3 Ground movements arising from basement excavation

The soils at basement formation level will be subject to stress relief during excavation, as some 16m of overburden is removed to form the new basement. This is likely to give rise to a degree of elastic heave over the short term and potential heave or settlement over the long term as pore pressures recover in the London Clay Formation.

The magnitude of these movements has been calculated using OASYS Limited VDISP (Vertical DISPlacement) analysis software. VDISP assumes that the ground behaves as an elastic material under loading, with movements calculated based on the applied loads and the soil stiffness (Eu and E’) for each stratum input.

The proposed bulk basement excavation gives rise to a net unloading of the underlying strata both during construction and over the long term. The excavation will unload the soils at the basement formation level by some 320kPa. These values assume a typical bulk unit weight of 20kN/m^3 for the excavated soils.

The combined effects of both the immediate undrained unloading and the long-term drained recovery of pore pressures have been analysed and the results are presented as displacement contour plots within Figure 7 and Figure 8 for the short and long term respectively. The ground movements within the contour plots are taken from a single displacement grid applied at 11.0mOD i.e. basement formation level.

The presence of stiff piles and pile caps in the soil below formation level have been ignored in the analysis. These elements will help to reduce heave movements further therefore heave movements, and values predicted in the analysis are likely to be greater than actual movements.

The presence of the contiguous piled wall around the perimeter of the excavation has also been ignored in the analysis. It is anticipated that the skin friction of the piled wall would further reduce heave movements around the perimeter of the basement.

The results of the analysis are summarised in Table 10 below for both short and long term. The VDISP output can be provided separately upon request.

Table 10. Summary of maximum heave movements within excavation and at constraint locations

Stage	Centre of excavation (mm)	Viaducts [a] (mm)	Hawley Arms [a] (mm)	UKPN Oil [a] tank (mm)	Castlehaven [a] Road (mm)
Short term heave movement	20	3.0	3.5	4.0	4.0
Long term heave movement	50	10	8	14	14

a. Based on results of displacement line at level and plan location of constraint

Displacement lines have been added to the VDISP models corresponding to the line of the critical sections identified. These displacement profiles will be used to illustrate the ground movement profile at these locations and to undertake a damage assessment for the relevant structure. Heave movements will be counteracted by ground settlement behind the piled wall due to pile installation and deflection, the effects of which are considered in subsequent report sections.

6.4 Ground movement due to retaining wall deflection

This section presents the results of retaining wall analysis to provide predictions of ground movement behind basement walls in the location of the critical sections.

The proposed construction methodology and sequence has been derived based on discussions with Walsh Associates and with reference to current drawings (Appendix A).

6.4.1 Proposed construction sequence

The proposed top-down construction sequence is summarised below.

1. Install contiguous piled wall, comprising 750mm diameter piles at 900m spacing.

2. Excavate to 26.0mOD to allow sufficient space for capping beam and ground floor slab to be constructed. Top of capping beam is typically at 27.0mOD.

3. Excavate to 16.0mOD and install intermediate basement slab (B1).

4. Excavate to 11.0mOD and install lower basement slab (B2)

In total, three different wall analyses have been undertaken;

1. The first calculation models a viaduct surcharge of 200kPa at a distance of 3.5m from the wall and at a level of 25.5mOD.

2. The second calculation models a surcharge of 100kPa for the *Hawley Arms*, applied at a distance of 3m from the piled wall and at a level of 25.0mOD.

3. The third and final section only models a nominal 10kPa surcharge behind the wall applied at capping beam level (27.0mOD). The ground movement profile from the analysis has been used to assess ground movements in the vicinity of Castlehaven Road and the UKPN oil tank.

6.4.2 Analysis results

Analysis of the retaining wall has been undertaken using WALLAP embedded retaining wall analysis software. Serviceability limit state (SLS) criteria have been used to determine wall deflections. Calculation sheets are provided within Appendix C and summarised within Table 11. The corresponding ground settlement at the critical sections is also provided.

The distance to negligible lateral movements behind the wall has been calculated assuming the ground movement occurs within a soil wedge based on a 45 degree load spread from the base of the excavation depth.

Vertical ground movement has been calculated by taking 50% of the displacement profile predicted from WALLAP. This is in line with the results of finite element analysis reported within *CIRIA C580 – Embedded retaining wall design 2003.*

Table 11: Results of WALLAP analysis

Section	Maximum wall deflection (mm)	Level of maximum deflection [mOD]	Lateral deflection at location/level of constraint (mm)	Vertical settlement below location of constraints (mm)
Railway Viaduct	15	[20.0]	4.5	5.0 to 7.5
Hawley Arms	16	[20.0]	3.5	2 to 8
UKPN oil tank	16	[20.0]	11.0	3.0 to 7.0
Castlehaven Road	16	[20.0]	6.0	3.0 to 8.0

The analysis indicates that an embedment of 2m below formation level is sufficient to satisfy global stability, assuming top down construction throughout. It should be noted that where the basement wall is required to carry vertical columns, the pile embedment will be governed by these loads. Final detailed pile design will be undertaken by the piling contractor.

Based on the above, it will be assumed for the purpose of this assessment that piles within the contiguous wall will be 18m long.

In regard to indicative wall displacements that may be expected during excavation, it should be noted that WALLAP uses a Winkler spring analysis to determine the wall displacements. In a Winkler medium, springs are used to represent a continuum and there is no transfer of shear stresses between the springs. In general, the application of this concept leads to an overestimation of structural deformations; hence the resulting wall displacements and corresponding impact on the nearby building and infrastructure may be over-predicted by the WALLAP program.

6.5 Ground movement due to retaining wall installation

With reference to CIRIA C580[11], vertical and horizontal surface movements due to installation of a contiguous piled wall are generally in the region of 0.04% of the wall depth assuming a good standard of workmanship. The distance behind the wall to negligible movement is 1.5 times wall depth for horizontal movements and 2 times wall depth for vertical movements.

Based on the ground conditions, CGL's experience with similar projects[12] and by adopting a 'hit and miss' pile installation sequence onsite, the maximum ground movement due to piled wall installation are likely to be in the region of 0.02% of the wall depth. The value of 0.02% will be adopted for this assessment.

The WALLAP analysis indicates that the pile length will be in the region of 18m. This pile length would give rise to a predicted horizontal and vertical movement of less than 4mm immediately adjacent to the piled wall.

Predicted installation movements are summarised in Table 12. The corresponding ground movement at the location of adjacent constraints is summarised below.

Table 12. Vertical movement due to pile installation

Section 1	Pile type	Movement at wall (% wall depth)	Ground movement (mm)	Distance behind wall to negligible movement (m)	Deflection at 3m[a] from the piled wall (mm)
Vertical movement	Contiguous	0.02	3.6	36	3.3mm
Lateral movement	Contiguous	0.02	3.6	27	3.2mm[a]

[a] Typically distance from piled wall to identified constraints

[11] CIRIA C580 (2003) *Embedded Retaining Walls – guidance for economic design*
[12] Ground Engineering (September 2014). Prediction of party wall movements using Ciria Report C580

7. DAMAGE ASSESSMENT

The calculated ground movements have been used to assess potential 'damage categories' that may apply to neighbouring structures and infrastructure due to the proposed basement construction method and assumed construction sequence. The methodology proposed by Burland and Wroth[13] and later supplemented by the work of Boscardin and Cording[14] has been used, as described in *CIRIA Special Publication 200*[15] and *CIRIA C580*.

General damage categories are summarised in Table 13 below:

Table 13. Classification of damage visible to walls (reproduction of Table 2.5, CIRIA C580)

Category	Description
0 (Negligible)	Negligible – hairline cracks
1 (Very slight)	Fine cracks that can easily be treated during normal decoration (crack width <1mm)
2 (Slight)	Cracks easily filled, redecoration probably required. Some repointing may be required externally (crack width <5mm).
3 (Moderate)	The cracks require some opening up and can be patched by a mason. Recurrent cracks can be masked by suitable linings. Repointing of external brickwork and possibly a small amount of brickwork to be replaced (crack width 5 to 15mm or a number of cracks > 3mm).
4 (Severe)	Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows (crack width 15mm to 25mm but also depends on number of cracks).
5 (Very Severe)	This requires a major repair involving partial or complete re-building (crack width usually >25mm but depends on number of cracks).

The above assessment criteria are primarily relevant for assessing masonry structures founded on shallow footings. Therefore, this methodology will be adopted within the damage assessment for the NR viaducts and *Hawley Arms*.

[13] Burland, J.B., and Wroth, C.P. (1974). *Settlement of buildings and associated damage*, State of the art review. Conf on Settlement of Structures, Cambridge, Pentech Press, London, pp611-654

[14] Boscardin, M.D., and Cording, E.G., (1989). *Building response to excavation induced settlement*. J Geotech Eng, ASCE, 115 (1); pp 1-21.

[15] Burland, Standing J.R., and Jardine F.M. (eds) (2001), *Building response to tunnelling, case studies from construction of the Jubilee Line Extension London*, CIRIA Special Publication 200.

7.1 Impact Assessment – Network Rail viaducts

The results of the predicted ground movement below the viaduct due to the proposed basement development have been compiled to determine the overall lateral deflection and vertical deflection of the viaduct.

Figure 9 shows the combined lateral movement of the piled wall due to pile installation and deflection. The maximum deflection of the wall at the level of the viaduct is predicted to be 8mm. The corresponding horizontal movement of the viaduct (located a minimum of 3.5m from the wall and at a level of 25.5mOD) has been calculated to not exceed 6mm. This has been calculated assuming that the movement reduces linearly over a distance of 16m behind the wall i.e. within a 45 degree soil wedge spread from formation level.

Combined vertical movement profiles below the viaduct, including short and long term heave due to excavation, retaining wall installation and wall deflection due to excavation are presented within Figure 10. The profile indicates that the movement below the viaduct ranges between 4mm of heave 3.5m from the piled wall to 4mm of settlement at the opposite end of the viaduct (13.5m from the wall). The corresponding maximum deflection beneath the viaduct is 3mm.

Table 14 incorporates a summary of the maximum lateral and vertical deflection (mm) for the viaduct and prediction of the corresponding horizontal strain and vertical deflection ratio. The width of the viaduct has been assumed from development drawings to be approximately 10m.

Table 14. Summary of ground movements and corresponding damage category

Constraint	Horizontal movements (mm)	Maximum deflection (mm)	Horizontal Strain ε_h[b] (%)	Deflection ratio Δ/L[a] (%)	Damage category
Viaduct	6	3	0.06	0.03	1 – very slight

a. *See Figure 2.18 (a) CIRIA C580 (2003) Embedded retaining walls guidance for economic design. (L = length of adjacent structure in metres, perpendicular to basement; Δ = relative deflection)*

b. *See Box 2.5 (v) CIRIA C580 (2003) Embedded retaining walls guidance for economic design. (δ_h = horizontal movement in metres).*

Based on the above and assuming a good standard of workmanship and adopting a ‘hit and miss’ contiguous pile wall installation method, the estimated maximum damage category imposed on the Network Rail viaduct will be ‘Category 1’ corresponding to very slight damage.

The structure interaction chart for the adjacent properties is presented in Figure 11.

Additionally, with reference to published data for brickwork set in mortar[16,17] the maximum deflection ratio of the structure should not exceed 0.075% to ensure Category 1 damage is not exceeded. The predicted ground movements fall within this criterion.

7.2 Impact Assessment – Neighbouring property (*Hawley Arms*)

The results of the predicted ground movement below the *Hawley Arms* due to the proposed basement development have been compiled to determine the overall lateral deflection and vertical deflection of the structure.

Figure 12 shows the combined lateral movement of the piled wall due to pile installation and deflection adjacent to the *Hawley Arms*. The maximum deflection of the piled wall at the level of the *Hawley Arms* basement is predicted to be 11mm. The corresponding horizontal movement of the structure (located at approximately 1.0m from the wall and at a level of 25.0mOD) has been calculated to not exceed 10mm. However, lateral movement at the opposite end of the structure located approximately 9m from the wall is approximately 5.5mm resulting in a net extension of some 4.5mm. The differential lateral movement can be taken as the difference between the lateral movement predicted at the near wall (10mm) minus the lateral movement predicted at the far wall (5.5mm), which is 4.5mm.

Combined vertical movement profiles below the structure, including short and long term heave due to excavation, retaining wall installation and wall deflection due to excavation are presented within Figure 13. The profile indicates that the movement below the *Hawley Arms* ranges between 10mm of heave 1m from the wall to 6mm of settlement at the opposite end of the structure (9m from the wall). The corresponding maximum deflection is 5mm. The maximum differential movement of the neighbouring property is 16mm which corresponds to an angular distortion of 1/500 assuming the width of the property perpendicular to the basement is 8m. The predicted angular distortion is within published limits[18, 19] for preventing excess cracking and damage to loading bearing walls and partitions.

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[16] Ali Q., Badrashi I. R., Ahmad N., Alam B., Rehman S., Banori S. A. F., *Experimental investigation on the characterization of solid clay brick masonry for lateral shear strength evaluation*. Internal journal of earth science and engineering. ISSN 0974-5904, Volume 05, No.04 August 2012

[17] Milosevic J., Gago S. A., Lopes M., Bento R., Rubble stone masonry walls – Evaluation of shear strength by diagonal compression tests. 8th international confrerence on structural analysis of historical constructions.

[18] Skempton , A. W. & Mac Donald, D. H. (1956). The Allowable settlement of buildings. Proc. Instn Civ, Engrs, Part 3, No. 5, pp 727-784.

[19] Polshin, D. E. & Tokar, R. A. (1957). Maximum allowable non-uniform settlement of structures. Proc. 4th Int. Conf. SM&FE, Wiesbaden, No. 1, pp. 285.

Table 15 incorporates a summary of the maximum lateral and vertical deflection (mm) of the *Hawley Arms* and prediction of the corresponding horizontal strain and vertical deflection ratio. The width of the structure perpendicular to the piled wall has been taken from survey drawings to be approximately 8m.

Table 15. Summary of ground movements and corresponding damage category

Constraint	Horizontal movements (mm)	Maximum deflection (mm)	Horizontal Strain ε_h [b] (%)	Deflection ratio Δ/L [a] (%)	Damage category
Hawley Arms	4.5	5	0.056	0.063	2 – Slight

a. *See Figure 2.18 (a) CIRIA C580 (2003) Embedded retaining walls guidance for economic design. (L = length of adjacent structure in metres, perpendicular to basement; Δ = relative deflection)*
b. *See Box 2.5 (v) CIRIA C580 (2003) Embedded retaining walls guidance for economic design. (δ_h = horizontal movement in metres).*

Based on the above and assuming a good standard of workmanship and adopting a 'hit and miss' contiguous pile wall installation method, the estimated maximum damage category imposed on the *Hawley Arms* is 'Category 2' corresponding to 'slight' damage, or cracks of up to 5mm in width.

The building interaction plot for the property is presented in Figure 11.

7.3 Impact assessment – Existing UKPN oil tank

The results of the predicted ground movement in the vicinity of the UKPN oil tank due to the proposed basement development have been compiled to determine the overall lateral and vertical deflection of the tank.

Figure 14 shows the combined lateral movement of the piled wall due to pile installation and deflection adjacent to the UKPN oil tank. The maximum deflection of the wall at the level of the oil tank is predicted to be 14mm. The corresponding horizontal movement of the tank (located at approximately 1.5m from the wall and at a level of 23.5mOD) has been calculated to not exceed 13mm. However, the lateral movement of the opposite end of the tank located approximately 4.5m from the wall is approximately 10mm, resulting in a net extension of some 3mm. The differential lateral movement can be taken as the difference between the lateral movement predicted at the nearest tank location (13mm) minus the lateral movement predicted at the opposite far end of the tank (10mm), which is 3mm.

The predicted lateral movement across the tank corresponds to a horizontal stain of 0.1%.

Combined vertical movement profiles below the tank, including short and long term heave due to excavation, retaining wall installation and wall deflection due to excavation are presented within Figure 15. The profile indicates that the combined movement below the tank ranges between 12.5mm of heave 1.5m from the wall to 4mm of heave at the opposite end of the tank (4.5m from the wall). The combined profile indicates that the corresponding maximum deflection of the tank is negligible and the angular distortion is approximately 1:350.

Based on the ground movement profiles, differential movement between the tank and perimeter pipe connections is typically less than 5mm i.e. 50% of the maximum vertical and horizontal deflection.

Overall, the assessment indicates that ground movements predicted in the vicinity of the oil tank are within allowable limits of 10mm. However, other relevant assessment criterial should be confirmed by UKPN i.e. allowable differential movement between connections and maximum strain.

7.4 Impact assessment - Castlehaven Road

The results of the predicted ground movement in the vicinity of a typical section perpendicular to Castlehaven Road have been compiled to determine the overall lateral and vertical deflection in this region.

Figure 16 shows the combined lateral movement of the piled wall due to pile installation and deflection adjacent to Castlehaven Road (Section 4-4 on CSM). The maximum deflection of the wall at the assumed formation level of the road and potential services running below the road is predicted to be 7.5mm. The corresponding horizontal movement of the road/services (located at approximately 1.0m from the wall) has been calculated to not exceed 7mm.

Combined vertical movement profiles below the road, including short and long term heave due to excavation, retaining wall installation and wall deflection due to excavation are presented within Figure 17. The profile indicates that the movement below the road ranges between 18mm of heave 1m from the wall to 2mm of settlement at the opposite end of the road (approximately 11m from the wall).

With reference to the ground movement contour plots (Figure 7 and Figure 8) the differential movement along the road at approximately 1m from the basement wall is less than 5mm for both short and long term predictions. It should be noted that these movements are predicted from a displacement grid at formation level so the magnitude of movements at the assumed formation level of the road and potential services (26mOD) would be lower than values shown. Additionally, these movements are counteracted by ground settlement due to pile wall installation and wall deflection as discussed previously and shown in Figure 17. The magnitude of global long term heave is likely to be less than predicted due the presence of heavily loaded piles and pile caps across the formation and also the perimeter piled wall providing skin friction resistance to heave pressures.

Overall, the assessment indicates that the ground deflections predicted and corresponding differential movements in the vicinity of the Castlehaven Road and potential services located 1m and spanning parallel to the basement wall are within allowable limited. The presence of critical service runs in this region of the site should be confirmed and a more detailed assessment undertaken if the identified infrastructure is sensitive to the predicted ground movements. Appropriate assessment criteria should also be provided for further detailed assessment.

8. SUBTERRANEAN (GROUNDWATER) FLOW

8.1 Introduction

This section addresses outstanding issues raised by the screening process regarding groundwater flow.

8.2 Impact on groundwater flow

Based on the groundwater observations from the boreholes on and off site, site monitoring data and CGL's experience of groundwater conditions in the area, it is anticipated that little or no groundwater will be encountered during the basement excavation and any seepage that may be encountered will be limited and likely to be encountered at the interface of the London Clay Formation and Made Ground and potentially within sandy layers and pockets within the near surface Weathered London Clay Formation. This should be controllable by adopting localised pump and sump systems.

The hydrogeological regime is typical of London conditions, with the London Clay Formation providing an effectively impermeable barrier to vertical flow in the ground, leaving any lateral flows to occur within the Made Ground. Given the topography of the area, it is likely that hydraulic gradients will be relatively flat and consequent groundwater flow rates will be minimal.

Based on the above, it is considered that the new development will have little impact on localised groundwater flows and generally have a negligible impact on the local groundwater regime.

8.3 Recommendations for groundwater control

It is anticipated that due to the low permeability of the London Clay Formation and presence of a contiguous piled wall around the perimeter, it is likely that inflows during the construction will be relatively minor and generally dewatering will not be required. However isolated and limited perched water may be encountered in the shallow Made Ground or within more sandy partings of the upper layers of the Weathered London Clay Formation. Observations on groundwater should be recorded during excavation and appropriate mitigation strategies put in place if water is encountered.

9. SURFACE FLOW AND FLOODING

It is understood that surface waters will join the existing drainage infrastructure (albeit via basement pumping if a gravity fed solution is not feasible), with no significant changes in peak drainage outflows anticipated from the site.

As already identified, the site lies outside any EA designated Flood Zone and is not highlighted as being on a street that flooded in the 1975 and 2002 events. Current mapping (Figure 15 CPG4) indicates that roads impacted by flooding in 1975 are located 40m north and 70m west of the site. The site is not within a region that was impacted by 2002 flooding or areas with potential to be at risk of surface water flooding.

Based on the above, it is considered that the development will have a negligible impact on surface water flow and flooding. In addition, the basement is likely to provide enhanced attenuation given its requirement to be drained in accordance with Building Regulations.

10. CONSTRUCTION MONITORING

The results of the ground movement analysis suggest that with good construction control, damage to adjacent structures (NR viaducts and *Hawley Arms*) generated by the assumed construction methods and sequence are likely to be (within Category 1 to Category 2) 'very slight' to 'slight'. Additionally movements predicted in the vicinity of the UKPN oil tank and along Castlehaven Road are generally within allowable limits, subject to confirmation from relevant stakeholders.

A formal monitoring strategy should be implemented across the site and especially in the regions identified as being critical and analysed within this assessment in order to observe and control ground movements during construction, and in particular movements of adjacent NR arches, *Hawley Arms* structure and UKPN oil tank.

The system should operate broadly in accordance with the 'Observational Method' as defined in CIRIA Report 185[20]. Monitoring can be undertaken by using vertical inclinometers installed within selected contiguous piles to determine wall displacement as excavation and construction progresses, while further use of survey targets affixed to the top of the wall and face of the adjacent buildings/infrastructure can determine if any horizontal translation of the piled wall or tilt/settlement of the neighbouring structure is occurring. Alternatively, remote tilt beams can be connected to the façade of the viaducts and *Hawley Arms* structure to provide 'real time' monitoring of this structure as excavation progresses. A similar approach can be adopted for the UKPN oil tank.

Precise levelling can be undertaken at regular intervals around the perimeter of the excavation and in the region between the basement and identified critical constraints to give an early and accurate indication of deviating ground movements. It is recommended that a specialised monitoring contractor is employed to install and monitor the instrumentation on site.

It is recommended that vibration monitoring also be considered during the demolition of the existing building onsite and during the piling works.

Monitoring data should be checked against predefined trigger limits and can also be further analysed to assess and manage the damage category of the adjacent buildings as

[20] Nicholson, D., Tse, Che-Ming., Penny, C., The Observational Method in ground engineering: principles and applications, CIRIA report R185, 1999.

construction progresses. The data could also potentially be used to undertake back analysis calculations and value engineer certain elements of the construction.

10.1 Construction monitoring – Installation

Monitoring of adjacent structures should commence a minimum of two weeks prior to piling beginning on site, and incorporate a 'baseline' data set taken prior to any excavation works. Monitoring should be continued regularly throughout pile installation with the data reviewed continuously to update the empirical assumptions made to date. Monitoring points should be established on capping beams, neighbouring properties/infrastructure and the ground between the excavation and neighbouring properties.

10.2 Construction monitoring – Excavation

The monitoring data obtained during pile installation should be reviewed prior to excavation and used to calibrate 'trigger limits'. Table 16 shows typical trigger level divisions and appropriate actions to take within each division. Trigger values can be provided based upon a review of the ground movements once the design and construction method/sequence is finalised.

Inclinometers should be installed in critical piles at an appropriate spacing for the length of the retained wall.

Reference targets should be installed on capping beams and on neighbouring property/infrastructure where appropriate, with precise levelling points installed along the ground behind the wall to correlate with values from the inclinometers (in the basement walls) and survey targets (on the face of critical neighbouring structures). By adopting this approach the movement of the wall, ground behind and neighbouring property can be compared to that of the VDISP/WALLAP analysis and damage category assessment plots. The presence of remotely read tilt beams will provide early warning signs of movement trends.

In addition, a pre-commencement condition survey of neighbouring structures is recommended with strain/crack gauges applied to any existing defects to monitor changes brought about by construction activities.

Data from building targets and precise levels should be referred back to an appropriate datum (bench mark) positioned outside the zone of influence of ground movement outside the basement.

Table 16: Proposed trigger limits – basement construction

Trigger Limit based on absolute movement	Outcome	Action
Trigger limits can be determined based on finalised design, construction method and sequence	GREEN	Works can proceed as normal and monitoring to continue at regular intervals as specified
Trigger limits can be determined based on finalised design, construction method and sequence	AMBER 1	Increase monitoring frequency, review data and check instrumentation calibration and accuracy
Trigger limits can be determined based on finalised design, construction method and sequence	AMBER 2	Reduce excavation works in the area, increase monitoring frequency, review data/trends and carry out visual inspections
Trigger limits can be confirmed based on finalised design, construction method and sequence	RED	Stop all works in the area, implement contingency plan and a detailed review of the available monitoring data should be undertaken by an engineering review panel

11. NON-TECHNICAL SUMMARY

11.1 General

The findings of this Basement Impact Assessment are informed by site investigation data, information regarding construction methods provided by the client and assumed construction sequence and detail.

- From the available information, it is considered that the proposed basement construction will have a negligible effect on groundwater, surface water and flooding at this site.
- The construction of the basement will generate ground movements due to a variety of causes including; heave due to excavation and ground settlement due to pile installation and deflection during excavation.
- An assessment of the results of the ground movement analysis and displacement profiles indicate that these movements will give rise to a damage category within 'Category 1' (very slight damage) for the Network Rail viaducts and damage category within 'Category 2' (slight damage) for the *Hawley Arms* structure.
- Combined vertical and horizontal ground movements predicted within the vicinity of the UKPN oil tank onsite and potential services along Castlehaven Road fall within current limits recommended.
- There is the potential for localised perched water within the shallow Made Ground, but this is likely to be very limited and underlain by impermeable clay. Observations on groundwater should be carefully recorded during excavation. Should perched groundwater be encountered, a temporary pumping strategy will need to be implemented to ensure the excavation and formation levels are kept dry prior to blinding. This could be achieved by the use of, for example, a localised sump and pump system.
- It is recommended that an appropriate monitoring regime is adopted to manage risk and potential damage to the identified neighbouring constraints.

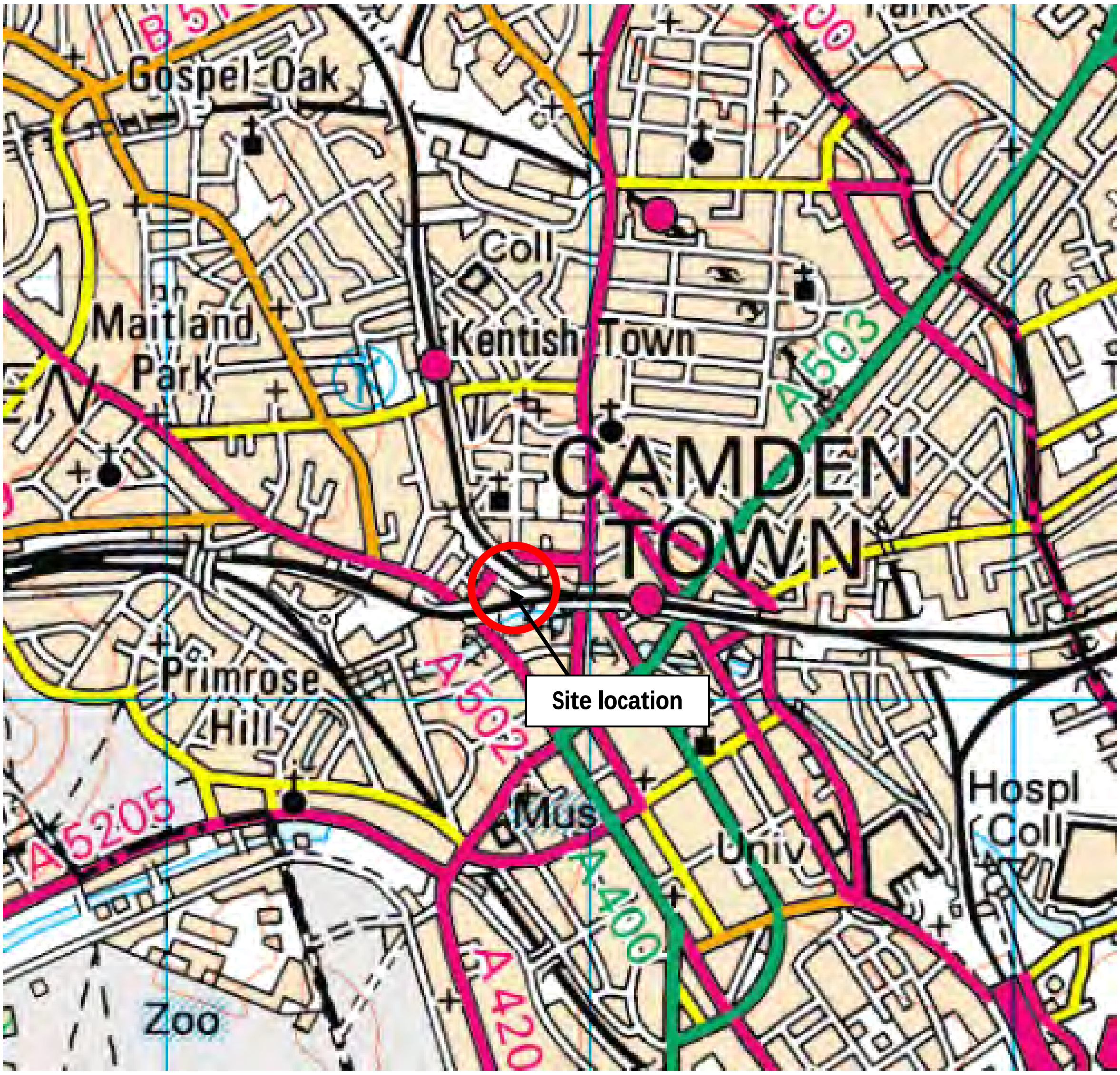
11.2 Cumulative impacts

The ground movement and building damage assessment have indicated that damage to neighbouring properties will be within allowable limits. Therefore, it is considered that there are no cumulative impacts in respect of ground or slope stability.

Groundwater was not encountered during the investigation and boreholes generally remained dry when left open overnight. Although groundwater was noted in the boreholes during subsequent monitoring, it is considered that the groundwater in the boreholes is due to water seepage at the interface between the Made Ground and London Clay Formation and also potentially due to very slow seepage within the silty sandy layers/pockets within the upper weathered London Clay Formation. Additionally, bailing of the boreholes during current monitoring visits confirm that that the infiltration rate of perched water is negligible. It is assumed based on the above that the development will have no significant impact on the flow of ground water in the region and would not contribute further to any cumulative effects.

It is understood that surface waters will join the existing drainage infrastructure (albeit via basement pumping if a gravity fed solution is not feasible), with no significant changes in peak drainage outflows anticipated from the site. The site is currently covered by hardstanding and is underlain by the relatively impermeable London Clay Formation. On this basis, the development is not considered to contribute to any significant cumulative impact with regard to surface flow or flooding.

FIGURES



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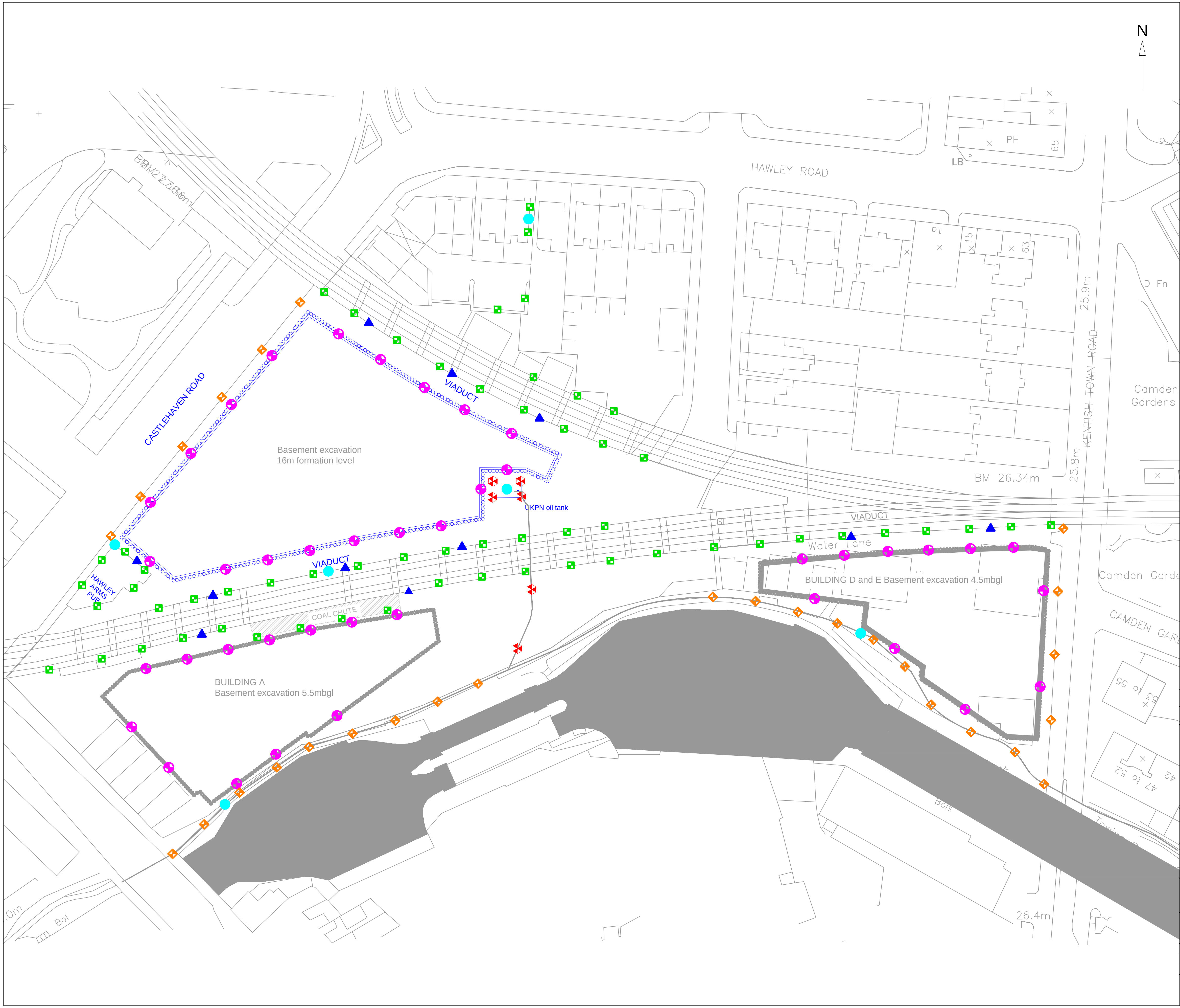
Client	Project	Job No
Walsh Associates	**Camden Lock Village, London**	**CG/18067a**
CGL	Title **Site location plan**	**Figure 1**



KEY

INC	MANUAL INCLINOMETER WITH 3D SURVEY POINT ON TOP
WTM	WIRELESS TILT METER
SP	3D SURVEY POINT
PLP	PRECISE LEVELLING POINT
SP&LP	COMBINED 3D AND SETTLEMENT POINT
VMP	VIBRATION MONITORING POINT

Note 1
It is recommended that settlement plates are also installed in the ground between the NR viaduct and basements.

Note 2
Capping beams and tops of all inclinometers should be monitored by the contractor.

Note 3
Base station for precise leveling should be located outside the zone of influence from ground movements.

Note 4
Refer to monitoring report for trigger levels for all recommended instrumentation.

Note 5
instrumentation on coal chute should be reinstalled on viaduct once demolition of the coal chute is complete.

Note 6
Location of vibration monitoring points are subject to change.

Rev	Date	Comments
*	*	*

CGL

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Project: **Camden Lock Village Phase 2, London**

Client: **Walsh Associates**

Drawing title: **Figure 2 - Conceptual monitoring strategy plan**

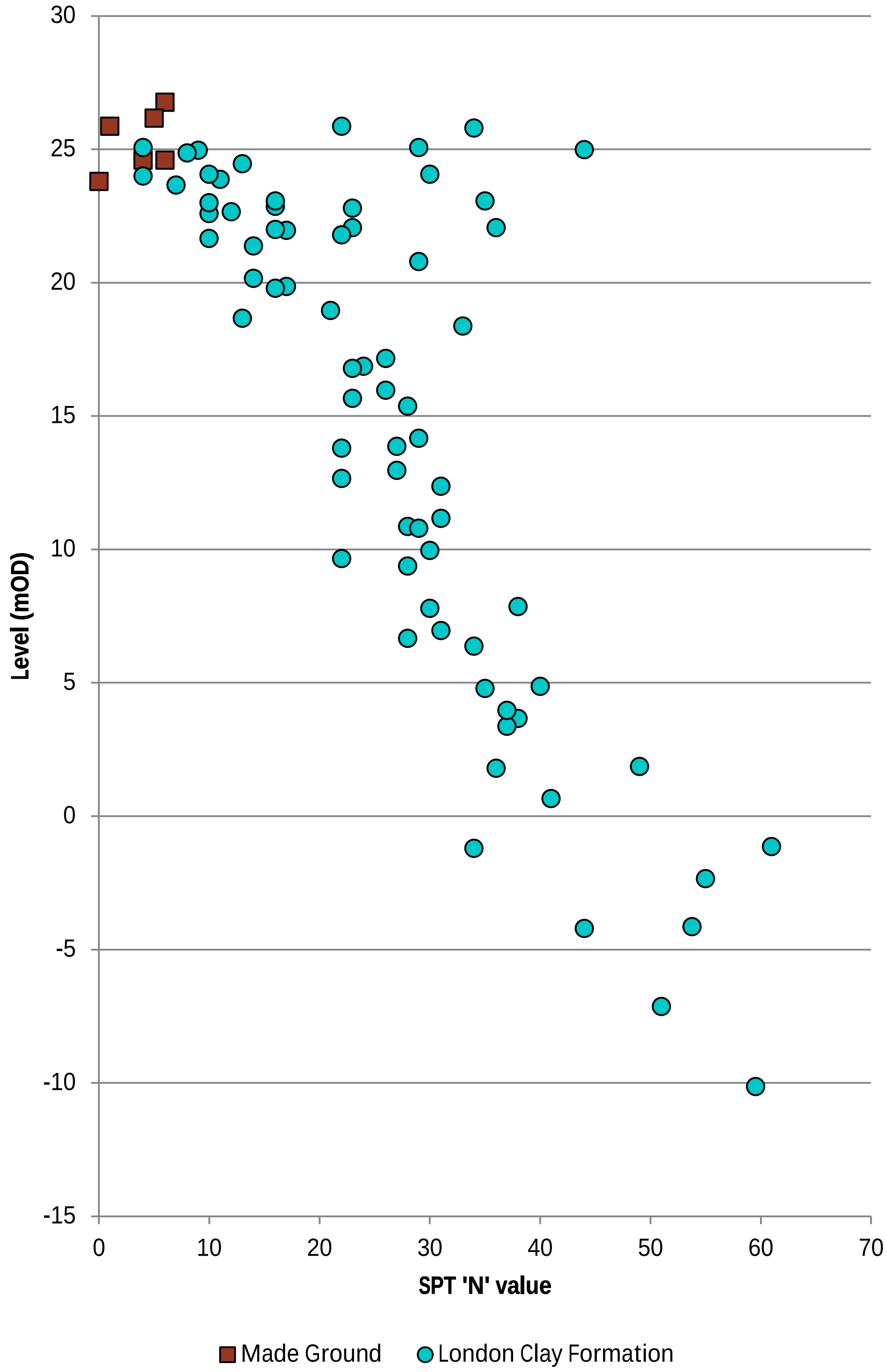
Scale(s): **NTS**

Job No.: **CG/18067a**

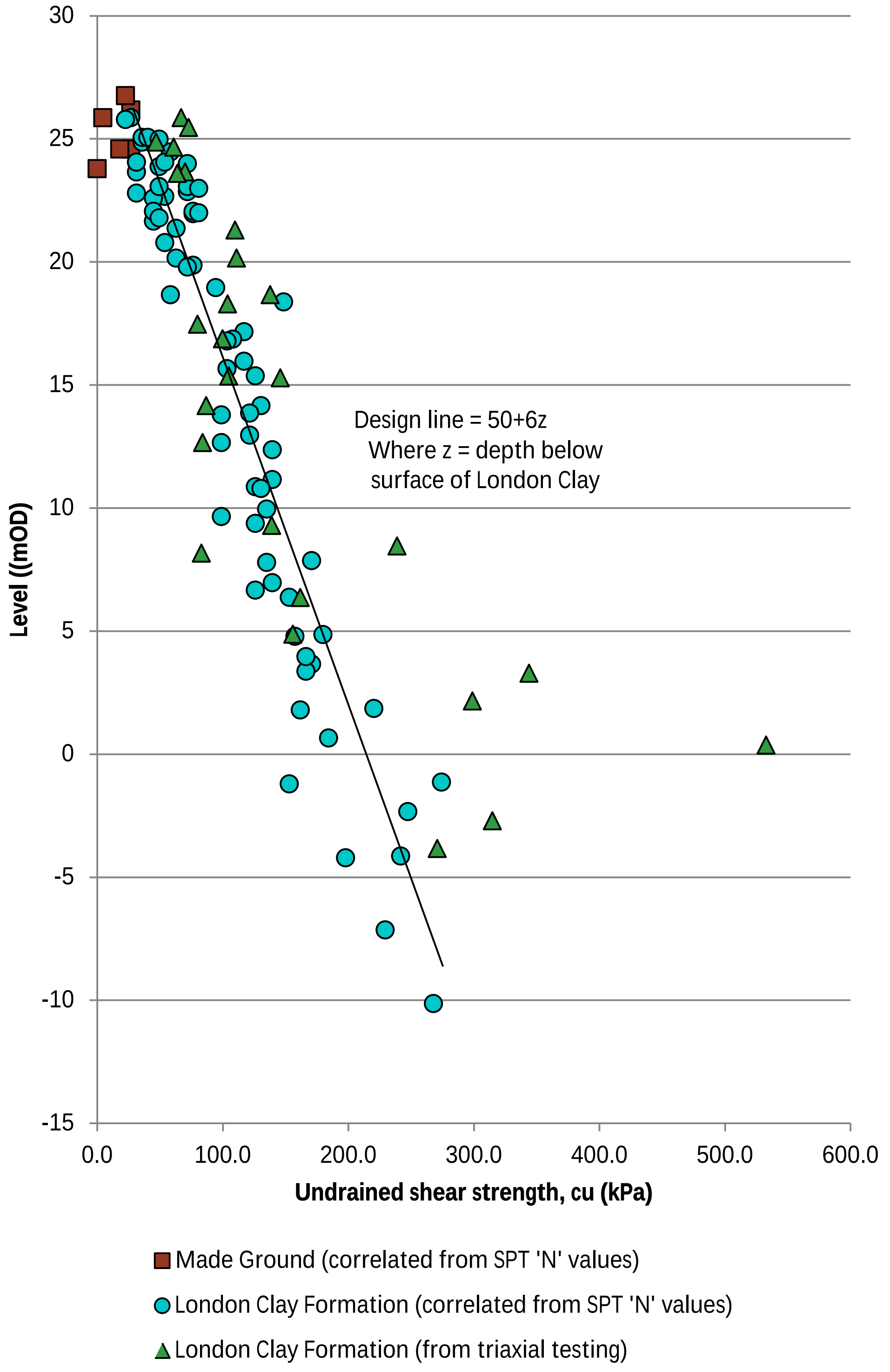
Drawn	TSB	02/04/15
Checked	JMS	02/04/15
Approved	RJB	02/04/15

Dwg No.: **CG/18067a-001**

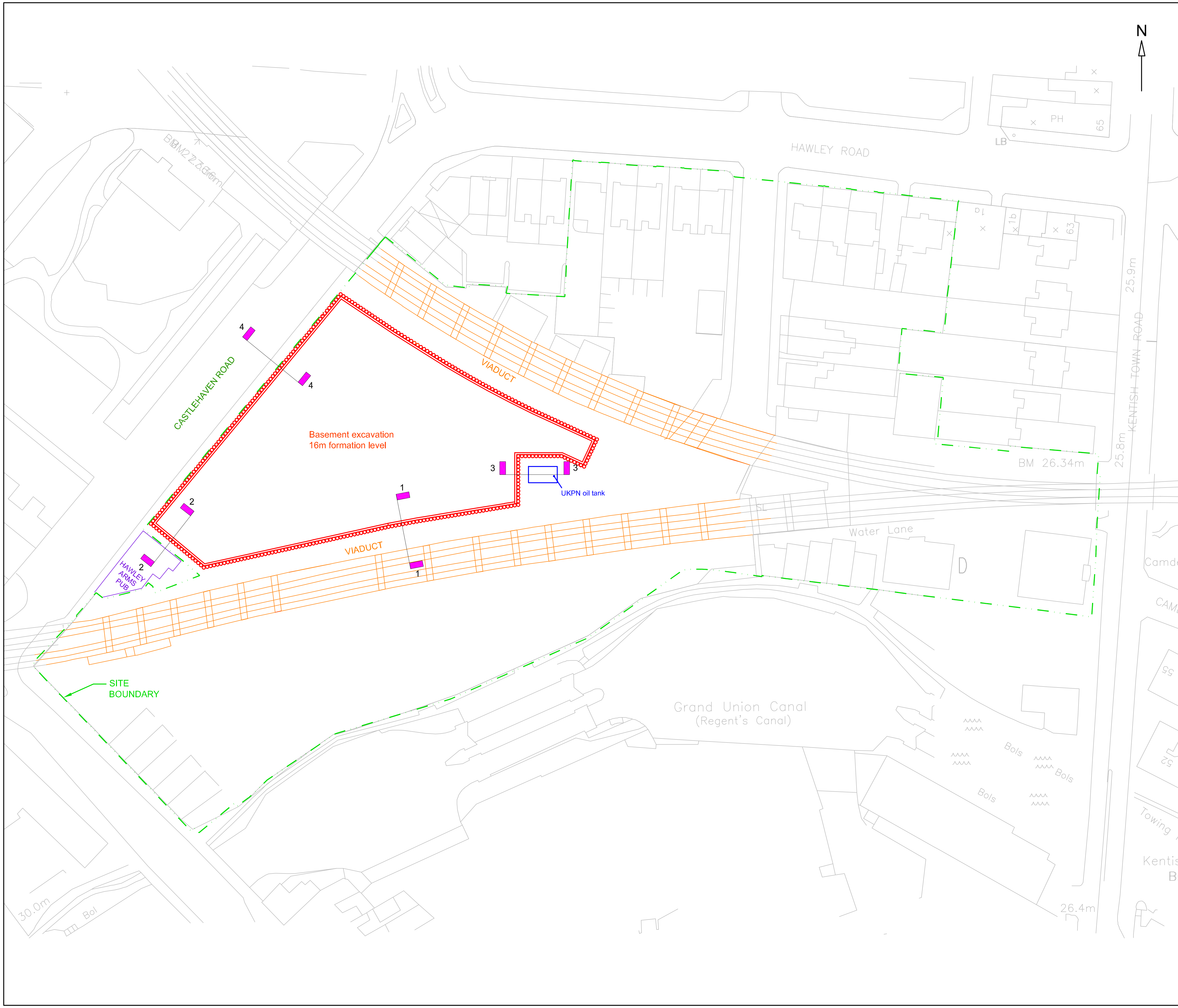
Rev.: *



Client	Project	Job No
Walsh Associates	**Camden Lock Village, London**	**CG/18067a**
CGL	Title **Plot of SPT ‘N’ values against level**	**Figure 3**



Client	Project	Job No
Walsh Associates	**Camden Lock Village, London**	**CG/18067a**
CGL	Title **Plot of c_u against level**	**Figure 4**



NOTE:
Reproduced from drawings and information provided by the client

Rev	Date	Comments
*	*	*

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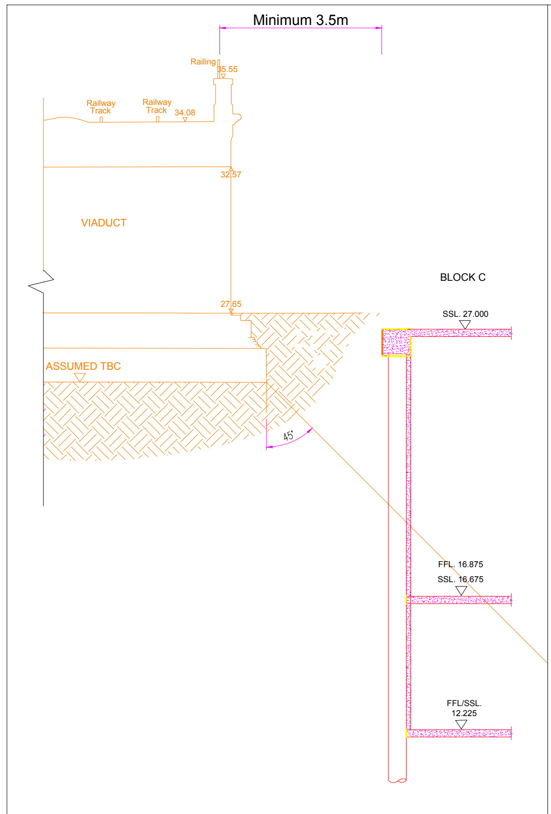
Project: **Camden Lock Village Phase 2, London**

Client: **Walsh Associates**

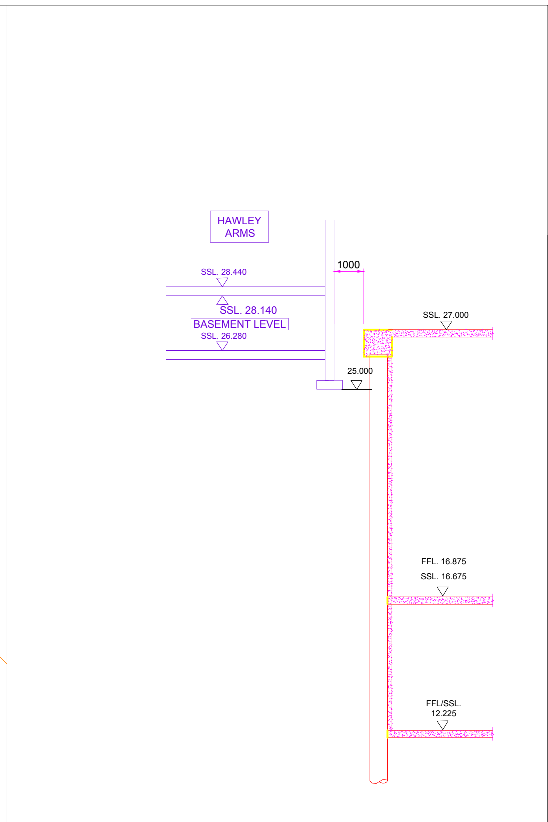
Drawing title: **Figure 5 Conceptual site model - basement plan & constraints**

Scale(s)	Job No.
NTS	**CG/18067a**

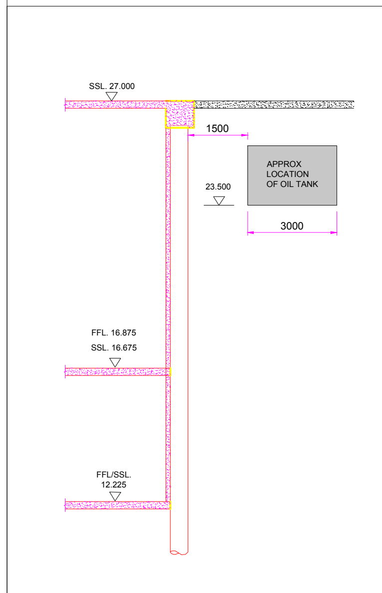
			Dwg No.	Rev.
Drawn	JLA	12/01/15	**CG/18067a-001**	*
Checked	JMS	12/01/15		
Approved	RJB	12/01/15		



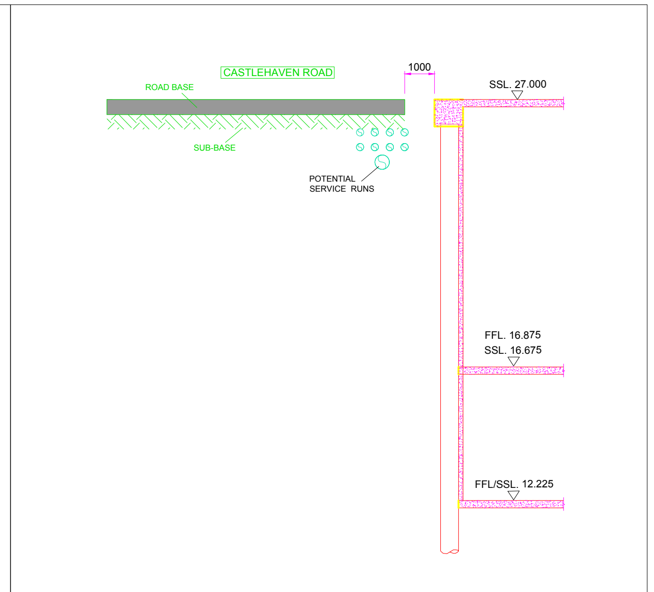
SECTION 1-1 NR VIADUCT



SECTION 2-2 HAWLEY ARMS



SECTION 3-3 UKPN OIL TANK



SECTION 4-4 CASTLEHAVEN ROAD

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Project: **Camden Lock Village Phase 2, London**

Client: **Walsh Associates**

Drawing title: **Figure 6 Conceptual site model - constraint sections**

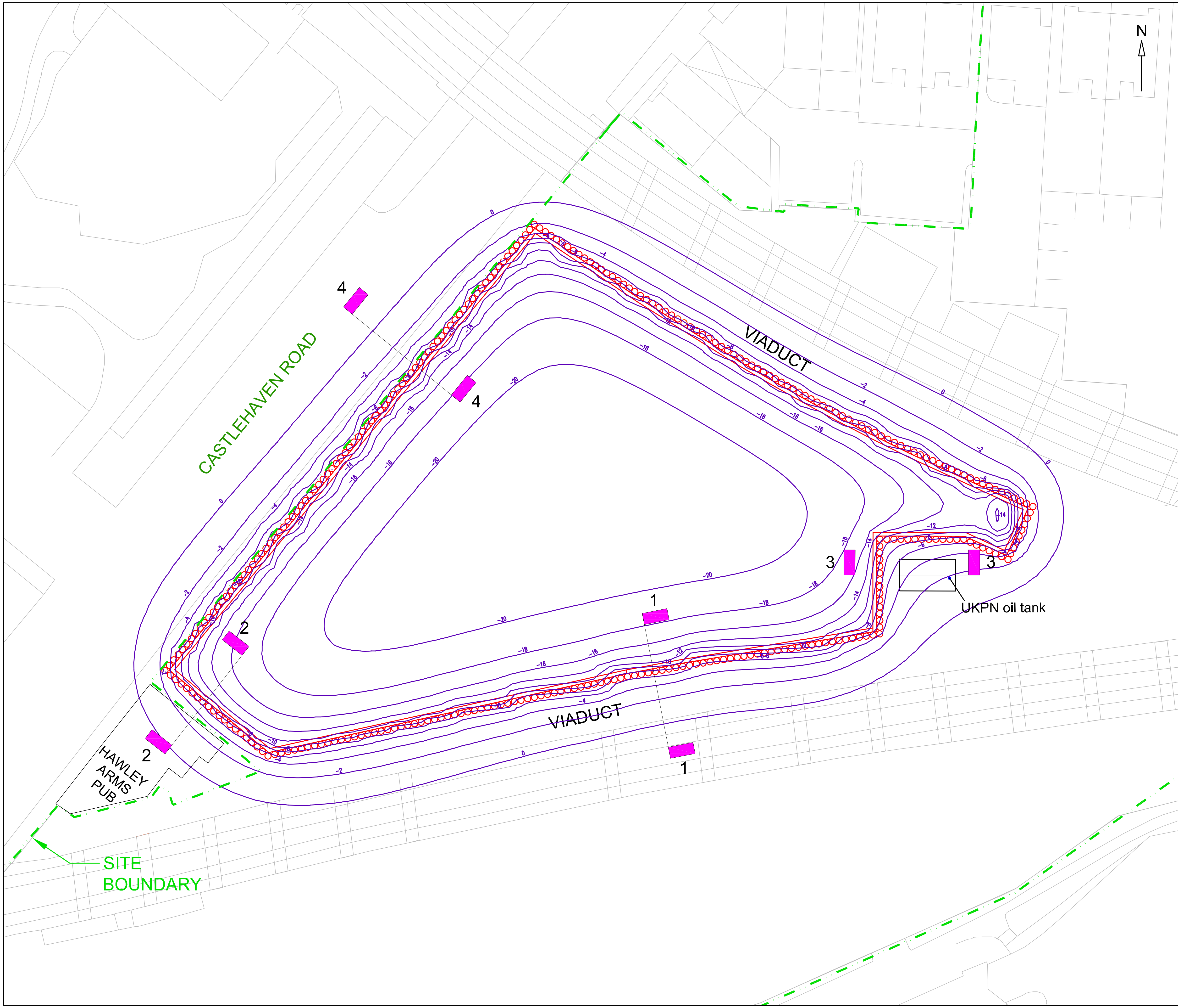
Scale(s): **NTS**

Job No.: **CG/18067a**

Drawn	JLA	12/01/15
Checked	JMS	12/01/15
Approved	RJB	12/01/15

Dwg No.: **CG/18067a-002**

Rev.: *



NOTE:
Negative values indicate heave

Rev	Date	Comments
*	*	*

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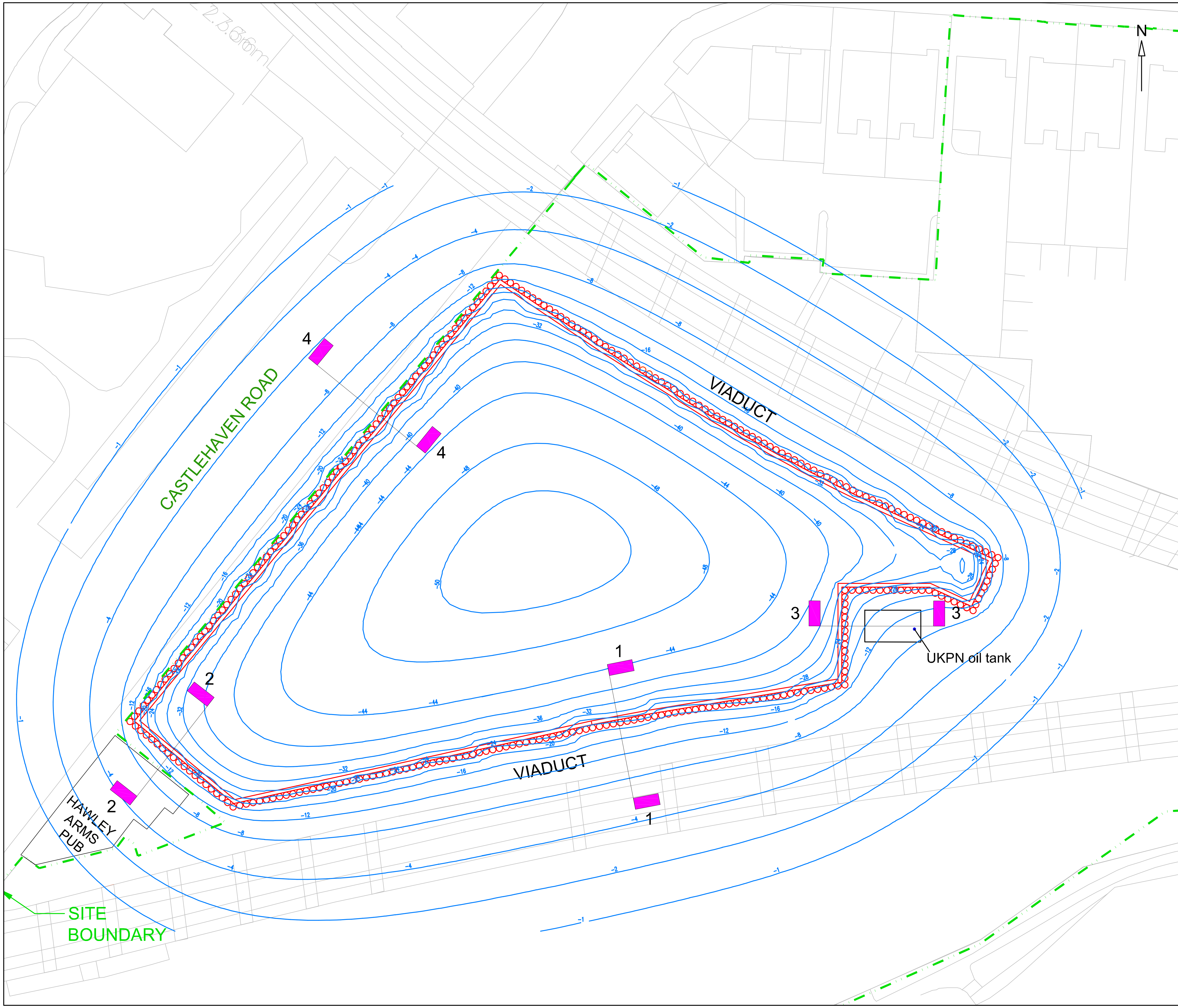
Project: **Camden Lock Village Phase 2, London**

Client: **Walsh Associates**

Drawing title: **Figure 7 Short term ground movement contour plot**

Scale(s)	Job No.
NTS	**CG/18067a**

			Dwg No.	Rev.
Drawn	JLA	12/01/15	**CG/18067a-003**	*
Checked	JMS	12/01/15		
Approved	RJB	12/01/15		



NOTE:
Negative values indicate heave

Rev	Date	Comments
*	*	*

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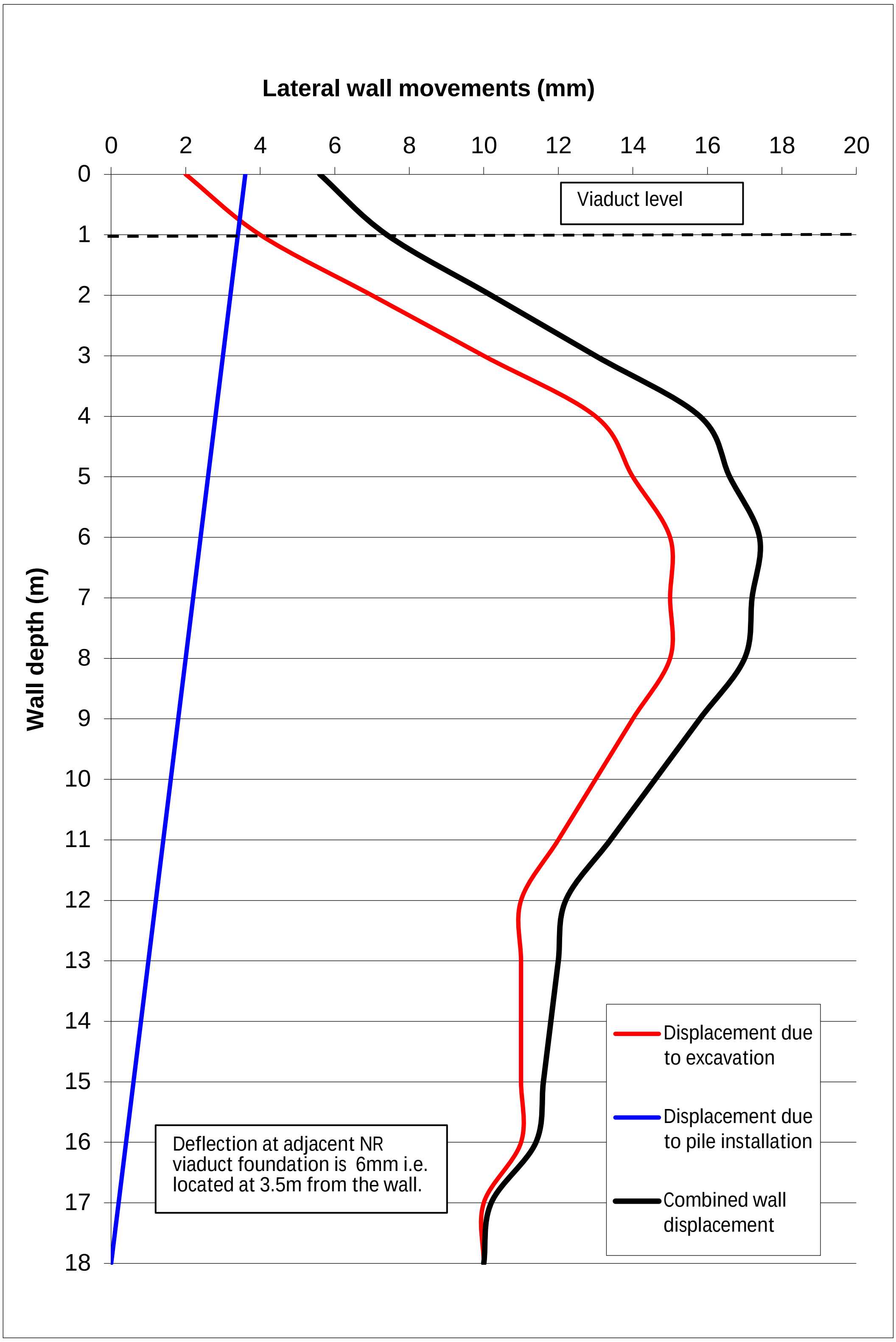
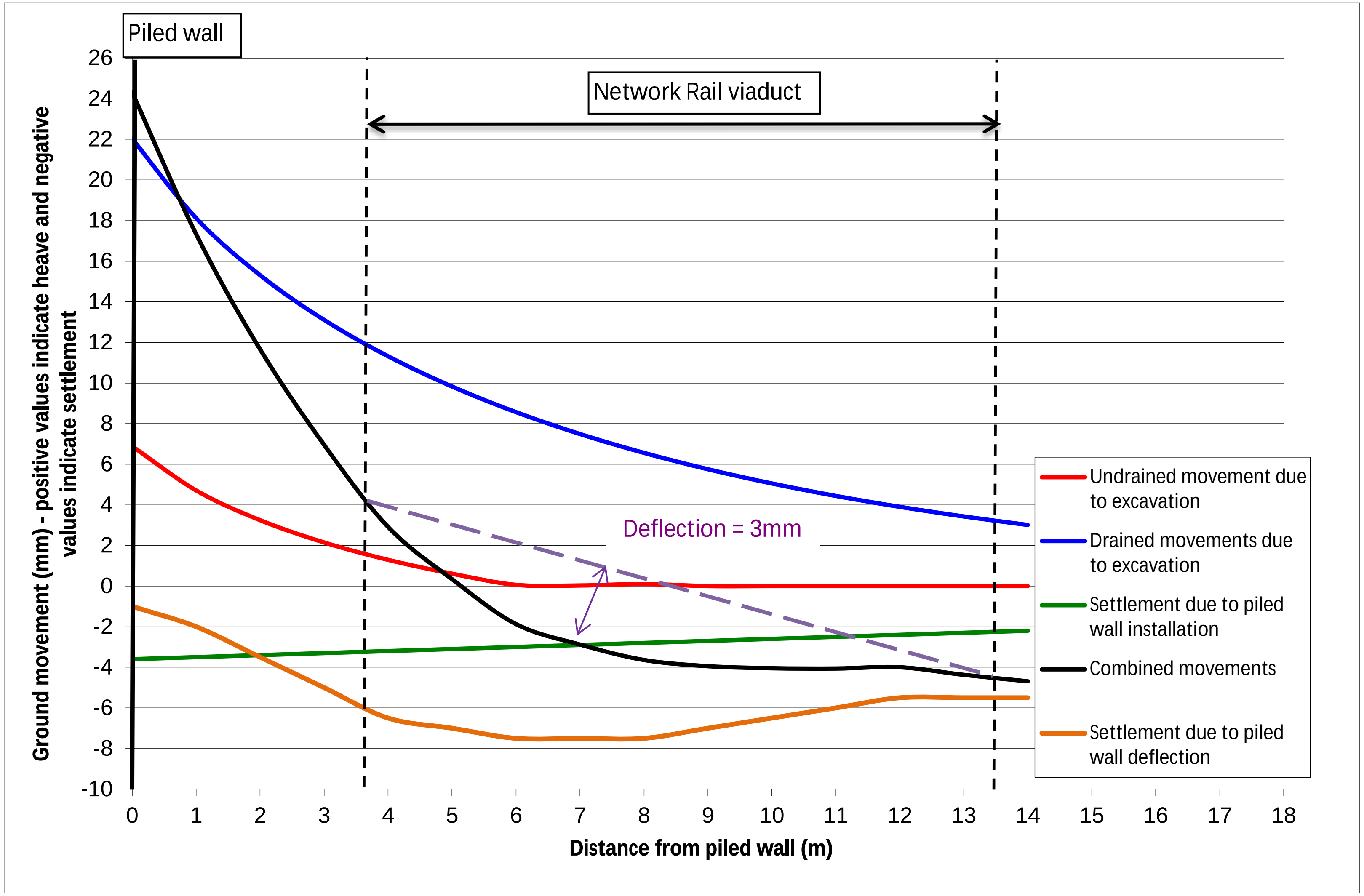
Project	**Camden Lock Village Phase 2, London**
Client	**Walsh Associates**
Drawing title	**Figure 8 Long term ground movement contour plot**
Scale(s) **NTS**	Job No. **CG/18067a**

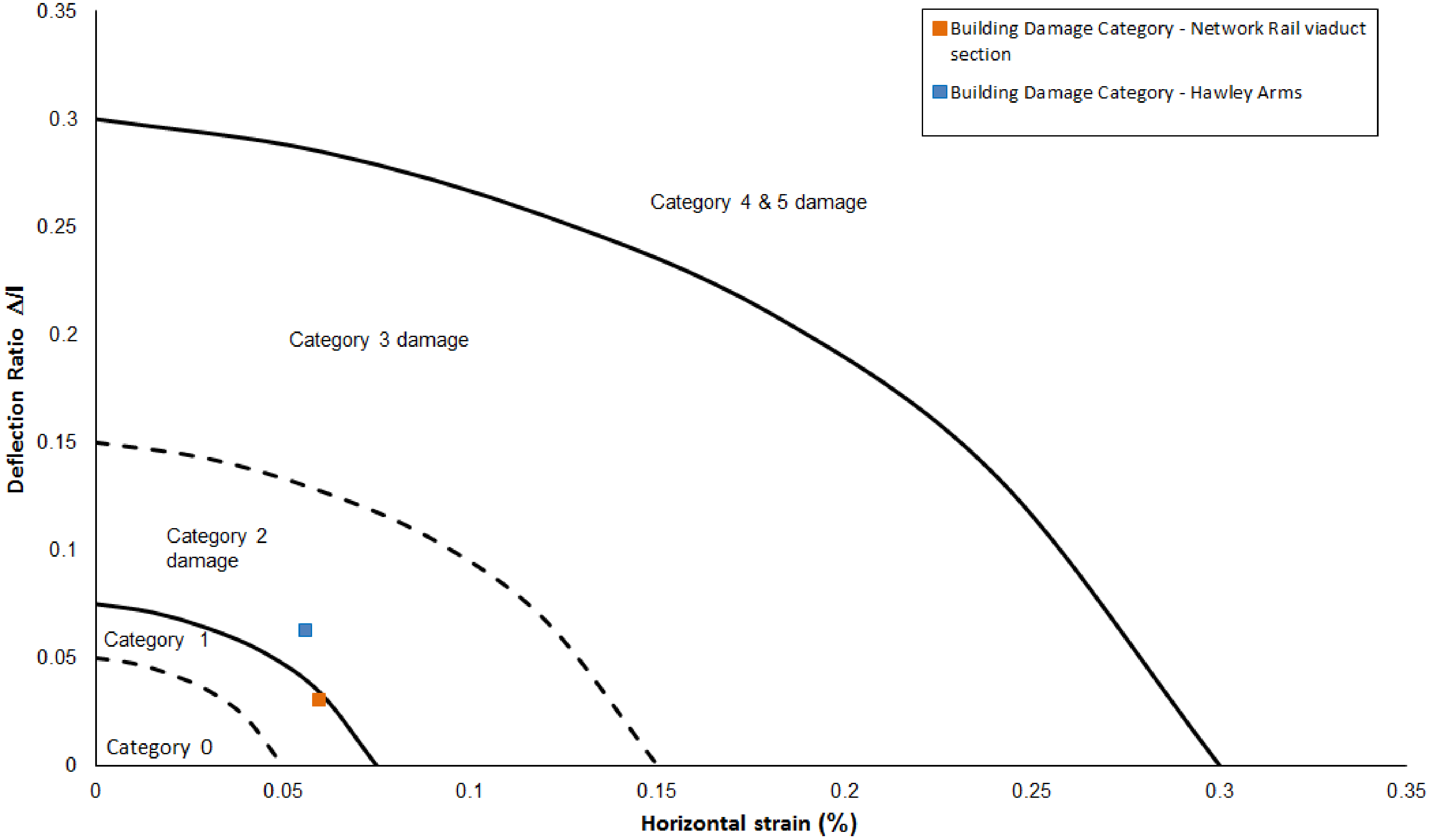
			Dwg No.	Rev.
Drawn	JLA	12/01/15	**CG/18067a-004**	*
Checked	JMS	12/01/15		
Approved	RJB	12/01/15		



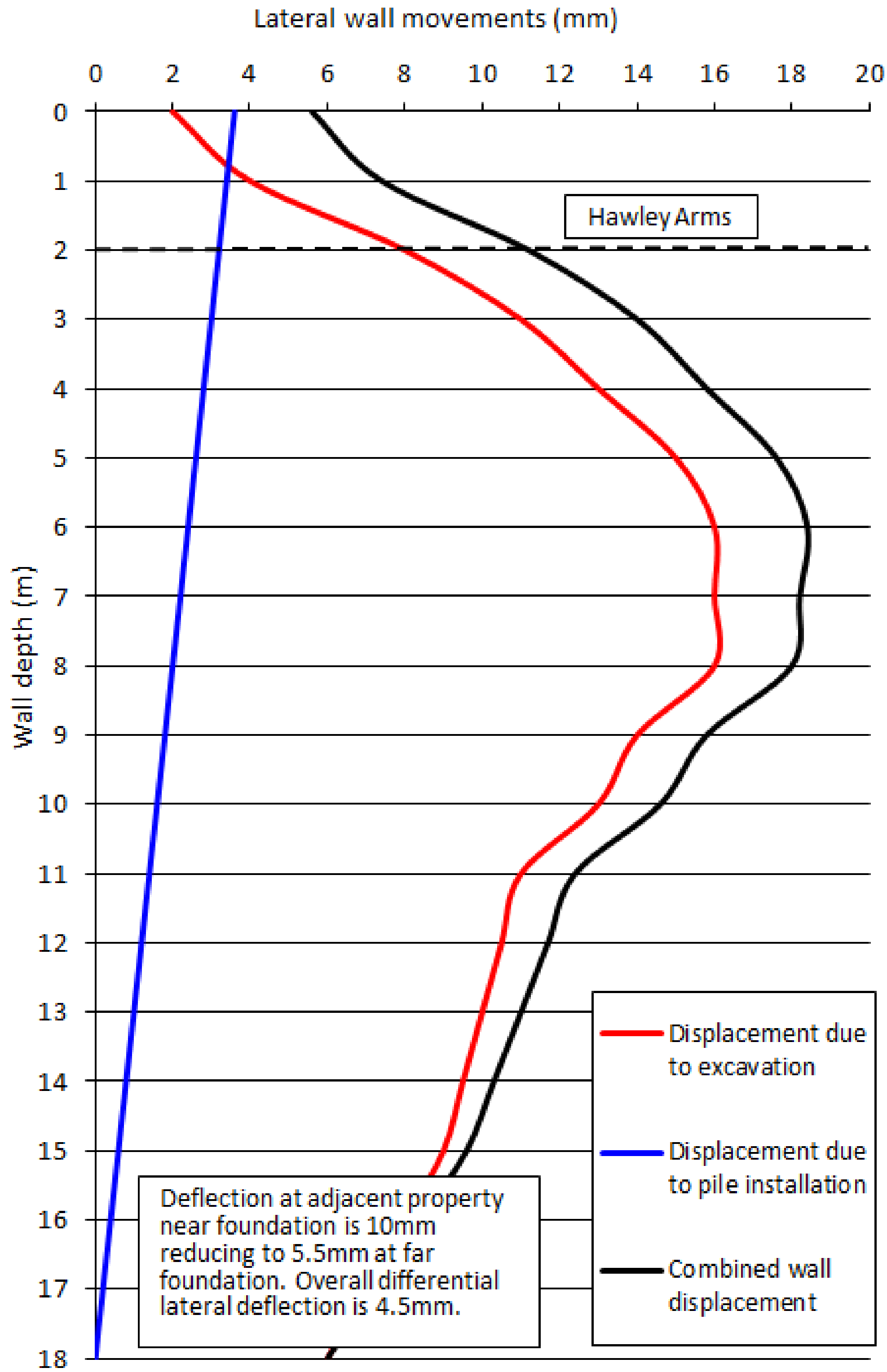
Client	Project	Job No
Walsh Associates	**Camden Lock Village, London**	**CG/18067a**
CGL	Title **Combined lateral movement profile –piled wall adjacent to NR viaducts**	**Figure 9**



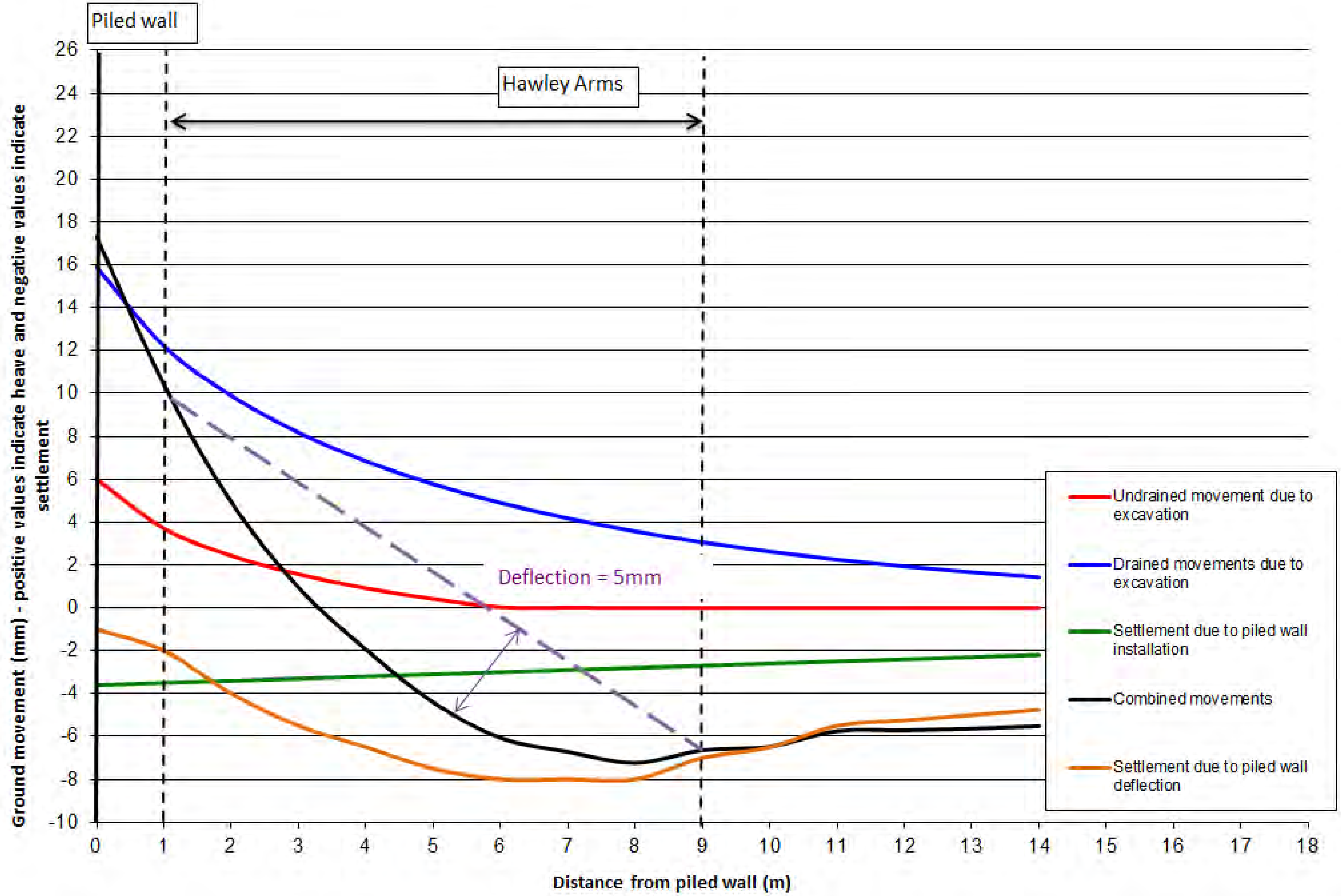
Client	Project	Job No
Walsh Associates	**Camden Lock Village, London**	**CG/18067a**
CGL	Title **Combined vertical movement profile –below the NR viaducts**	**Figure 10**



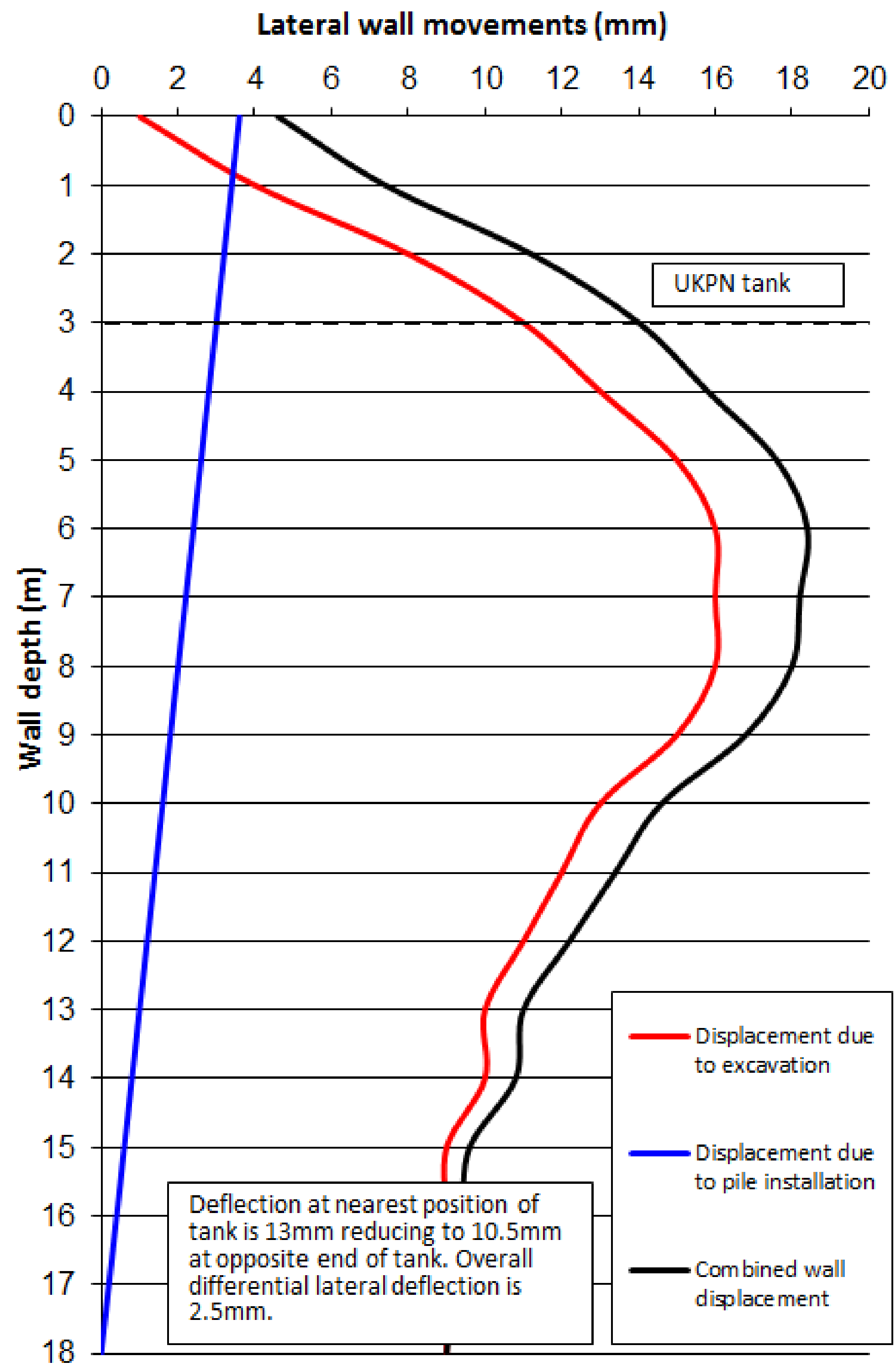
Client	Project	Job No
Walsh Associates	**Camden Lock Village, London**	**CG/18067a**
CGL	Title **Structure interaction plot**	**Figure 11**



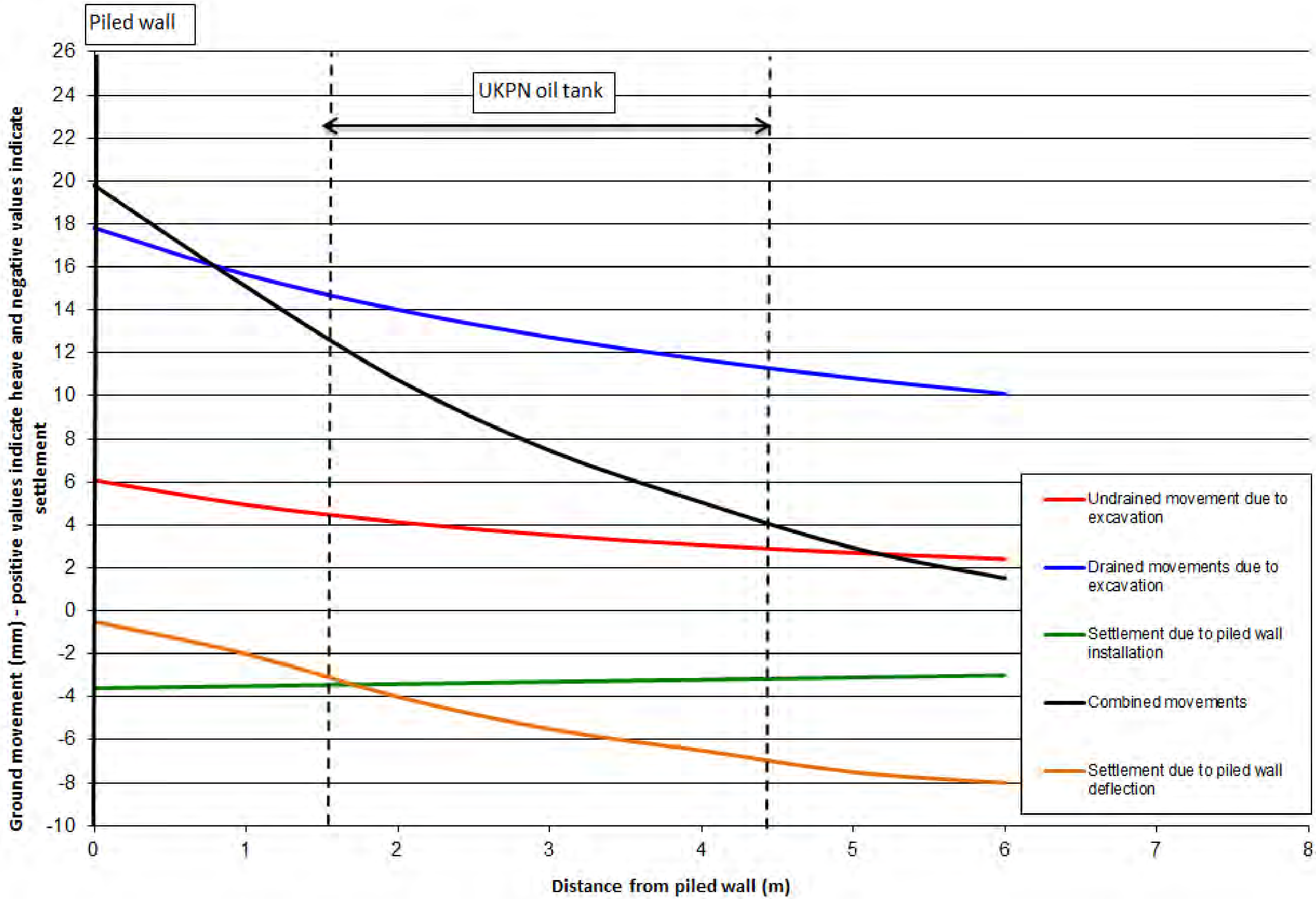
Client	Project	Job No
Walsh Associates	**Camden Lock Village, London**	**CG/18067a**
CGL	Title **Combined lateral movement profile – piled wall adjacent to Hawley Arms**	**Figure 12**



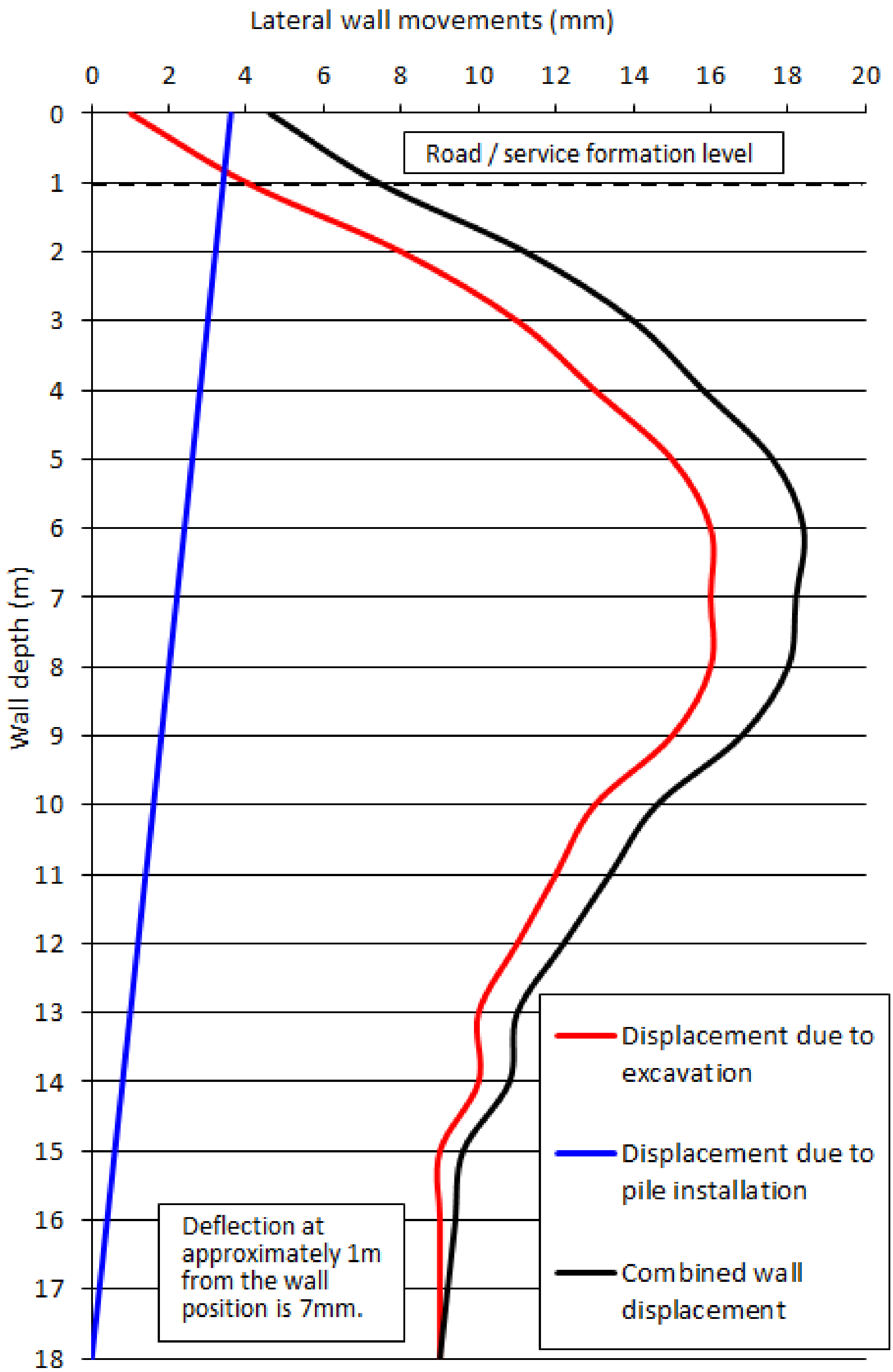
Client	Project	Job No
Walsh Associates	**Camden Lock Village, London**	**CG/18067a**
	Title **Combined vertical movement profile – below Hawley Arms**	**Figure 13**



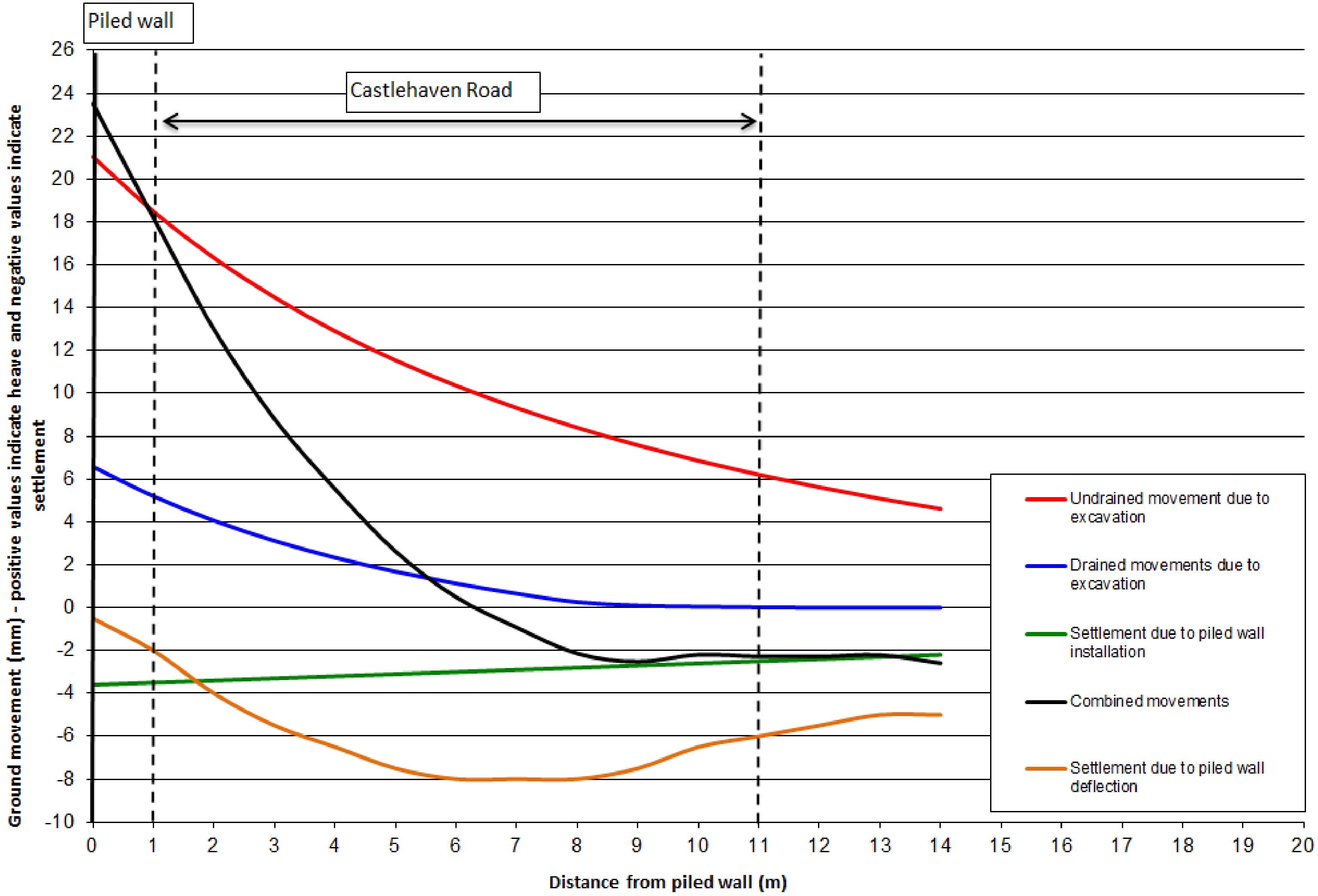
Client	Project	Job No
Walsh Associates	**Camden Lock Village, London**	**CG/18067a**
CGL	Title **Combined lateral movement profile – piled wall adjacent to UKPN oil tank**	**Figure 14**



Client	Project	Job No
Walsh Associates	**Camden Lock Village, London**	**CG/18067a**
CGL	Title **Combined vertical movement profile – below the UKPN oil tank**	**Figure 15**



Client	Project	Job No
Walsh Associates	**Camden Lock Village, London**	**CG/18067a**
CGL	Title **Combined lateral movement profile – piled wall adjacent to Castlehaven Road**	**Figure 16**



Client	Project	Job No
Walsh Associates	**Camden Lock Village, London**	**CG/18067a**
CGL	Title **Combined vertical movement profile – below Castlehaven Road**	**Figure 17**