Walsh Associates

Camden Lock Village, London

Basement Impact Assessment – Area D & E

Revision 2

March, 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 basement levels ranging in depth between approximately 4 metres below ground level (mbgl) and 16mbgl.

Card Geotechnics Limited (CGL) has been instructed by Walsh Associates (the structural engineers for the project) 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 the combined single storey basement below Building D and Building E only. The impacts of basement development for Building A and Building C have been assessed separately. 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, 4] 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. (2015) Camden Lock Village – *Geotechnical and Geoenvironmental Interpretative Report*. Ref: CG/18067A. January 2015.
[4] Card Geotechnics Limited. (2015) Camden Lock Village: Site E – *Geotechnical and Geoenvironmental Interpretative Report*. Ref: CG/18067A. February 2015.

2. SITE CONTEXT

2.1 Site location

The site is situated off Kentish Town Road in Camden, northwest London. The Ordnance Survey grid reference for the approximate centre of the site is 528908N, 184195E.

A site location plan is presented as Figure 1.

2.2 Site layout

The site is approximately triangular in shape and is bordered by a National Rail viaduct to the north, Kentish Town Road to the east, an area of open land, the *Grand Union Towpath* and *Regent's Canal* to the south and Camden Lock Village Market to the west. The northern region of the site (Building D site) area currently comprises office buildings with associated car parking. The southern region of the site (Building E site) comprises open land covered with grass and light vegetation.

Drawings and information provided by Walsh Associates indicates that the canal wall is a gravity concrete structure founded at approximately 3.5mbgl.

A site layout plan is presented within Figure 2. This Figure also shows the outline of the site within which the combined basement of Building D and Building E is proposed. Exploratory hole locations completed during the current site investigation across the entire Camden Lock Village development site are also presented on Figure 2.

LUL tunnels (Northern line) run below Kentish Town Road and National Grid cable infrastructure runs along the southern site boundary and follows the profile of the canal wall. These constraints are discussed in greater detail within Section 6.2 and are shown in Figure 5 and Figure 6.

2.3 Proposed development

The proposed development comprises a series of multi-storey buildings with a combined single-storey basement below the entire development footprint. The basement layout within the footprint of Site D is in line with the 2013 approved master plan for the site.

The upper floors of the proposed developments will comprise residential properties, with office space and a restaurant on the ground floor with parking and plant rooms in the basement level.

The proposed basement is typically 4.5m deep with its formation level at approximately 21.5mOD. It is currently proposed to construct the basement using bottom up construction methods within a contiguous piled box. Pile wall capping beam level is assumed to be at 26mOD and ground level across the site ranges from 25.6m in the east of the site to a maximum of 26.4mOD along the western boundary of the site. Contiguous piled walls of 0.6m diameter at 0.75m spacing are currently proposed to support the basement excavation. Sheet piles may also be adopted as a value engineering option, however for the purpose of this assessment a contiguous pile wall will be assumed to assess the worst case with regard to ground movements associated with installation of the piles.

The above information is taken from current detailed drawings provided by Walsh Associates and presented within Appendix A.

2.4 Historical Development

The historical development of the site was established by RPS in their October 2009[5] and November 2009[6] reports and is 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. No further significant changes were noted at the site until the construction of the current office buildings.

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[7] 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 D and Building E. Additionally, no substantial damage to the viaducts was noted.

[5] RPS (2009) Camden Lock Village London Borough of Camden. *Phase 1 – Environmental Risk Assessment*. Ref: HLEI4880/001R. October 2009

[6] RPS (2009) Camden Lock Village London Borough of Camden. *An Archaeological Desk Based Assessment*. Ref: JLK0617 RO1. November 2009

[7] 6 Alpha Associates Limited (2014) *Detailed Unexploded Ordnance (UXO) Risk Assessment*. Ref: P4063. September 2014

2.6 Anticipated ground conditions

2.6.1 Published and unpublished geology

The British Geological Survey (BGS) map sheet 256 (North London)[8] 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[9] 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.

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. The surface of the Lambeth Group was encountered at 42mbgl to 44.8mbgl and the Thanet Sand was encountered at 53.9mbgl underlain by the Chalk encountered between 64mbgl and 125mbgl.

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

[8] British Geological Survey. (1994) North London. Sheet 256. Solid and Drift Geology 1:50,000.
[9] www.bgs.ac.uk (accessed December 2014)

2.7 Hydrogeology

The Environment Agency[10] 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).

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. 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) including the *River Westbourne, Tyburn* and *River Fleet*. The map indicates that two tributaries of the *River Fleet* join near the viaduct on the eastern site boundary, where the river then trends south east along Camden Street. Historical mapping for the site (Survey of the Borough of St Marylebone 1834) provided by the client, indicates that before the river was culverted it passed through the site.

With reference to the Arup report[2], the site is approximately 2.2km southeast of the catchment for the pond chains on Hampstead Heath. Additionally, with reference to the EA website the site is not within a Flood Risk Zone.

Current flood mapping (Figure 15 CPG4) indicates that Kentish Town Road was affected by flooding in 1975. However, the road was not affected by the 2002 flooding in the region or by the serious national floods in 2007 and 2012. It is noted in the London Borough of Camden flood risk management strategy[11] and report of the Floods Scrutiny Panel[12] that the 1975 flood event was caused by the heaviest and most concentrated rainfall event recorded in this part of Camden. This 1 in 100 year event was preceded by a very dry summer and is therefore not considered to be representative of typical conditions in the area.

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[10] www.environment-agency.gov.uk (September 2014)
[11] London Borough of Camden (2014) Managing Flood Risk in Camden: The London Borough of Camden flood risk management strategy
[12] London Borough of Camden (2003) Floods in Camden: Report of the Floods Scrutiny Panel

Following the 2002 flood event, new infrastructure, including larger diameter sewers and a holding tank, was installed in the Borough to help mitigate the potential for future flooding. Additionally, the proposed development is positioned approximately 4m from Kentish Town Road. The site is not within an area identified by the Environment Agency to be at risk of surface water flooding.

3. SCREENING (STAGE 1)

3.1 Introduction

A screening assessment has been undertaken 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* forms the southern site boundary and is located minimum of approximately 3m from the proposed basement footprint. The *River Fleet* which is now culverted underground is located approximately 30m east of the site. Historical maps indicate that the River Fleet may have flowed through the site prior to being culverted.	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 generally covered by hardstanding and structures and underlain by impermeable London Clay.	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)?	Yes The majority of surface water is likely to be discharged to the sewer network through existing connections. It is understood that two new connections will be made. SUDS options are being considered.	Investigation and assessment
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'. However, flow rates are considered to be extremely slow within the effectively impermeable London Clay, and there is no water table or general flow that is likely to be affected by basement construction.

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.

Lost rivers of London maps indicates that two tributaries of the *River Fleet* join approximately 50m north of the site boundary where the river then trends south east along Camden Street. Historical mapping for the site (Survey of the Borough of St Marylebone 1834) provided by the client, indicates that before the river was culverted it passed through the site. Evidence of this will be assessed during the ground investigation.

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 The site is relatively flat	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 The topography of the surrounding region is relatively flat	None

Question	Response	Action required
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?	No Current drawings do not indicate the removal of any trees	None
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* forms the southern site boundary and is located minimum of approximately 3m from the proposed basement footprint. The *River Fleet* which is now culverted underground is located approximately 30m east of the site. Historical maps indicate that the River Fleet may have passed through the site prior to being culverted.	Investigation and assessment
9. Is the site within an area of previously worked ground?	No	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 Kentish Town Road, Network Rail viaduct and pedestrian walkway along Grand Union Canal.	Investigation and assessment
13. Will the proposed basement significantly increase the differential depth of foundations relative to neighbouring properties?	Yes The neighbouring Network Rail infrastructure is likely to have shallow foundations.	Investigation and assessment

Question	Response	Action required
14. Is the site over (or within the exclusion zone of) any tunnels?	No The site is not within the exclusion zone of the northern line tunnels running below Kentish Town Road. However the impact stress relief and ground movement on the tunnels due to basement excavation will be assessed.	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 site investigation will determine the presence and extent of the historical course of the River Fleet passing through the site. The founding level of the viaduct arch foundations will also be confirmed.

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 movements on the Network Rail viaduct, LUL tunnels and National Grid cable runs following the profile of the canal wall along the southern boundary of the site.

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?	Yes The majority of surface water is likely to be discharged to the sewer network through existing connections. It is understood that two new connections will be made. SUDS options are being considered.	Investigation and assessment

3. Will the proposed development result in a change in the proportion of hard surfaced/paved external areas?	No The site is generally covered by hardstanding and structures and underlain by 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?	Yes Current flood mapping (Figure 15 CPG4) indicates that Kentish Town Road was impacted by flooding in 1975. However, the road was not impacted by the 2002 flooding in the region and the site is not within an area identified to be at risk of surface water flooding. It is considered that drainage networks in the region have been considerably upgraded since the 1975 flood event to mitigate the potential for future flooding along Kentish Town Road.	Investigation and assessment

In summary, the proposed basement is to be constructed in areas of existing hardstanding and is therefore not anticipated to impact surface water flow. The site is underlain by the relatively impermeable London Clay Formation. Additionally, the site was not impacted by recent 2002 flooding events in the region and is not within an area identified to be at risk of surface water flooding.

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 forming the southern boundary
2.	Assess impact of the historical or current culverted course of the River Fleet 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
1	*Assess potential for flooding of basement due to historical flooding along Kentish Town Road*
	(The above is addressed in Section 9)

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 within the footprint of Building D and Building E by CGL between November 2014 and January 2015. 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 reports [3, 4] 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 was undertaken within the footprint of Building D and Building E between November 2014 and January 2015. The investigation comprised 7 window sampler boreholes to a maximum depth of 5.0 metres below ground level (mbgl) and one cable percussion borehole (BH7) to 30.5mbgl.

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. Full details of the site investigation are provided in the Geotechnical and Geoenvironmental Interpretative Report prepared by CGL[3].

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.

Table 6. Summary of ground conditions encountered in on-site ground investigations

Stratum	Top of stratum (mOD) [mbgl][a]	Typical thickness (m)
MADE GROUND Concrete overlying soft dark brown sandy gravelly silt. Sand is fine to coarse. Gravel is fine to coarse subrounded to subangular of brick, flint and occasional concrete.	25.79 [0.0]	1.5 to 3.0
Firm dark orange brown slightly silty CLAY with occasional fine selenite crystals [WEATHERED LONDON CLAY FORMATION].	22.79 to 24.29 [1.5 to 3.0]	7.4
Stiff closely fissured dark grey silty CLAY. Frequent fine selenite crystals noted. [LONDON CLAY FORMATION]	16.89 [7.4]	>21.6 (Base not encountered in boreholes)

a. mOD = metres above Ordnance Datum

Further details of the soils encountered are provided in the following sections. A plot of SPT 'N' versus level (mOD) is presented in Figure 3 and a plot of Undrained Shear Strength, cu (kPa) versus level (mOD) is presented in Figure 4.

Little evidence of this historical river course was noted during the site investigation and it is expected that it may have been removed during the construction of the historical/current developments and infrastructure onsite. All historical mapping provided which shows the *River Fleet* passing through the site are dated before any infrastructure or developments have been constructed adjacent to our within the site footprint. However, should the historical river course be encountered onsite during pile installation or basement excavation, the depth and extend of alluvial deposits should be carefully recorded and the impact of this change in expected ground conditions on foundations and basement design should be carefully assessed.

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 -4.71mOD. The upper 7.4m of the clay was found to consist of firm silty clay (Weathered London Clay Formation), becoming stiff (unweathered) from 16.89mOD. SPT 'N' values in this stratum ranged from 7 to 44 increasing with depth. Undrained shear strength values can be derived from these values using established Stroud correlations[13]. These values range from 31.5kPa to >198kPa.

Laboratory testing on samples of the London Clay Formation recorded undrained shear strength (c_u) values of 64kPa to 344kPa, increasing with depth.

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	24 to 34	65 to 79	25 to 29	38 to 50

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

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

5.3 Groundwater

No groundwater strikes were recorded in the cable percussion boreholes during drilling and boreholes remained dry when left open overnight. However, perched groundwater was encountered from 1.0mbgl in Window Sample borehole WS9, within the Made Ground.

Groundwater was noted 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 [surface 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**
BH7 [25.79]	18.34 [18.29]	18.55 [18.27]	18.78 [18.24]	NR	18.69 [18.26]
WS9 [25.79]	24.59 [22.86]	24.54 [23.01]	NR	24.63 [23.07]	24.71 [22.99]
	20/01/15	**26/01/15**	**06/01/15**	**10/02/15**	**16/02/15**
WS10 [26.0]	21.52 [21.0]	21.70 [21.0]	21.58 [21.0]	22.50 [21.0]	22.22 [21.0]
WS11B [25.8]	21.01 [20.8]	21.34 [20.8]	21.05 [20.8]	21.62 [20.8]	21.26 [20.8]
WS12 [25.9]	21.38 [21.0]	21.64 [21.0]	21.40 [21.0]	22.66 [20.9]	21.82 [21.0]

The monitoring records indicate that standing groundwater across the site is at approximately 1.1mbgl to 7.45mbgl. The groundwater level monitored in WS9 corresponds to the perched water level recorded within the Made Ground during drilling. It is considered that the groundwater in the CP boreholes and remaining WS holes 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.

Recharge tests undertaken during current monitoring visits indicate that the infiltration rate of perched water is negligible with water levels within the boreholes recovering by less than 50mm over a four hour monitoring period.

Based on the above and taking account of the close proximity of the canal, a long term design water level of 25mOD is recommended i.e. approximately 1mbgl.

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 [26.0]	18	30 [0]	26[d]	18[b] [13.5][c]
London Clay Formation	2 [24.0]	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.

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[2] (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.

2. Stress relief and heave movements due to excavation of the basement within the piled wall. This will be considered over the short and long term.

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 0.6m diameter, 0.75m 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 viaduct

The site is bordered to the north by a Network Rail viaduct. Current drawings indicate that the proposed basement will not be located closer than 3m from the viaduct.

With reference to findings of viaduct arch foundation inspection pits undertaken by CGL as part of the current site investigation, the viaduct arch footings are founded at approximately 2.5mbgl (23.5mOD) in the region.

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

6.2.2 Section adjacent to London Underground tube tunnels

Current drawings indicate that two Northern Line tunnels run below Kentish Town Road at the eastern boundary of the site. The crowns of the tunnels are at 12.3mOD (13.5mbgl) and 11.4mOD (14.4mbgl) and positioned approximately 4m from the plan location of the proposed basement. Based on this the LUL tunnels are considered to be within the zone of influence from ground movements due to the basement excavation and construction. The impact of this will be assessed within the ground movement analysis.

6.2.3 Section adjacent to National Grid cables and Grand Union Canal wall

A concrete encasement harbouring National Grid (NG) cable infrastructure is located along the southern site boundary and follows the profile of the canal wall. The impact of predicted ground movements on this infrastructure will be assessed. Current drawings indicate that the cable run and dock wall come within 1m to >3m of the proposed basement wall. It will be assumed within the assessment that the concrete encasement for the cable run is founded at 24.5mOD (1.5mbgl).

Through discussion with the client it is understood that the allowable deflection criteria for the cable run is 5mm of differential movement per 5m span i.e. angular distortion of 1:1000. This will be assessed for both lateral and vertical ground movement profiles.

The movement of the canal wall will also be assessed with a similar ground movement profile expected for that calculated for the National Grid infrastructure. Movements in the region of 10mm vertical and horizontal are considered tolerable for the canal wall.

6.3 Ground movements arising from basement excavation

The soils at basement formation level will be subject to stress relief during excavation, as some 4.5m 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. 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.

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 90kPa. This value assumes 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 21.5mOD i.e. basement formation level. 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. Due to the curvature of the National Grid (NG) infrastructure on plan, displacement points have been modelled at 2m centres (and at a level of 24.5mOD) along the centre line of the service run.

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.

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)	Viaduct [a] (mm)	LUL tunnels [a] (mm)	NG cable & dock wall [b] (mm)
Short term heave movement	10	1.5	2.0	2.5
Long term heave movement	24	5.5	5.0	8.5

a. Based on results of displacement line at level and plan location of constraint
b. Based on results of displacement points at 2m centres along line of cable run

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

It is proposed to adopt a bottom up construction sequence. Given the size of the wall, depth of excavation and position of the viaduct from the wall it is proposed to cantilever the wall along the western, northern and eastern site boundary in the temporary condition. Along the southern site boundary and where the canal and NG infrastructure comes within 5m of the wall it is proposed to construct a temporary berm and install high level propping to control ground movement. The typical construction sequence for this region of the site is summarised below.

1. Install contiguous piled wall, comprising 600mm diameter piles at 750mm spacing.
2. Excavate to 24.5mOD to allow sufficient space to construct capping beam and excavate a temporary berm. It is assumed that the berm will be 1m wide at the top and 4m wide at the base i.e. 45 degree batter. Top of capping beam is typically at 26.0mOD.
3. Install raking temporary propping at 25.5mOD prior to removing berm.
4. Excavate to formation level at 21.5mOD and construct basement slab.
5. Construct ground floor slab prior to removing temporary prop at 25.5mOD.

In total, three different wall analyses have been undertaken.

1. The first analysis models a viaduct surcharge of 200kPa at a distance of 3m from the wall and level of 23.5mOD. The wall is assumed to be cantilevered in the short term.
2. The second calculation models the presence of the canal with a nominal surcharge of 10kPa behind the wall. The piled wall is assumed to be propped in the short term.
3. The third calculation models the presence of a nominal 10kPa surcharge behind the wall and 20kPa surcharge at 4m from the wall (Kentish Town Road) along the eastern boundary. The piled wall is assumed to be cantilevered in the short term.

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	[26]	3.3	4.5
LUL tunnels	15	[26.0]	Negligible	Negligible
NG & canal wall	4	[23.5]	2.0	2.0

The analysis indicates that an embedment of 2.5m below formation level is sufficient to satisfy global stability for both the temporary propped and cantilevered wall sections. With reference to the pile working load profiles presented within the current site investigation report by CGL[3] and taking the worst case wall loading provided by Walsh (255kN/m or 190kN/pile), the pile wall embedment predicted within the WALLAP analysis is sufficient to support this load. Final detailed pile design will be undertaken by the piling contractor awarded the works.

Based on the above, it will be assumed for the purpose of this assessment that piles within the contiguous wall will be 7.0m 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 structures and infrastructure may be over-predicted by the WALLAP program.

6.5 Ground movement due to retaining wall installation

With reference to CIRIA C580[14], 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[15] 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 7.0m. This pile length would give rise to a predicted horizontal and vertical movement of 1.4mm 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	Ground movement (mm) [a]	Distance behind wall to negligible movement (m)	Deflection at viaduct (mm) [b]	Deflection at LUL tunnels (mm) [c]	Deflection at NG cable & dock wall (mm) [d]
Vertical movement	1.4	14.5	1.1mm	Negligible	1.2mm to negligible movement
Lateral movement	1.4	10.5	1mm	Negligible	1.1mm to negligible movement

[a] Ground movement immediately behind piled wall
[b] Viaduct located typically 3m from the basement wall
[c] LUL tunnels located typically 4m from the basement wall and 13.5mbgl
[d] NG cable & dock wall located between 2m and 35m from the basement wall

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

6.5.1 Ground movement due capping beam deflection

The potential ground movement due to the lateral deflection of the capping beam for the contiguous piled wall adjacent to the National Grid infrastructure has been assessed. This information will be used within the assessment of the horizontal displacement profile. An Indicative deflection value has been calculated using standard beam deflection formula for uniformly loaded sections.

Free span for the capping beam between temporary propping has been assumed to be 10m. Loading values (kN/m) have been obtained from the results of the WALLAP analysis. The size of the reinforced concrete capping beam has been taken from detailed drawings. The results of the deflection analysis are summarised in Table 13 below.

Table 13. Capping beam deflection

Critical section Reference	Capping beam size (mm)	Free span (m)	Load (kN/m)	Max. deflection (mm)
NG cable & dock wall	Reinforced concrete 900 x 1200	10	32.0	2

It will be assumed for the purpose of the damage assessment, as a worst case ground movement scenario, that the NG cable run will deflect laterally 1.5mm where it comes within 3m of the piled wall. The impact of capping beam deflection on the NG infrastructure is assumed to be negligible when the offset distance is greater than 5m (i.e. beyond 45 degree zone of influence).

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[16] and later supplemented by the work of Boscardin and Cording[17] has been used, as described in *CIRIA Special Publication 200*[18] and *CIRIA C580*.

General damage categories are summarised in Table 14 below:

Table 14. 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 viaduct. The movement of the NG cables and LUL tunnels will be assessed against assessment criteria provided and published limits. It is understood that movements in the region of 10mm are acceptable for the canal wall.

[16] 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
[17] Boscardin, M.D., and Cording, E.G., (1989). *Building response to excavation induced settlement*. J Geotech Eng, ASCE, 115 (1); pp 1-21.
[18] 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 viaduct

The results of the predicted ground movement below the viaduct due to the proposed basement development have been compiled to determine the overall lateral 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 foundation is predicted to be 11mm. The corresponding horizontal movement of the viaduct (located a minimum of 3m from the wall and at a level of approximately 23.5mOD) has been calculated to not exceed 3.7mm. This has been calculated assuming that the movement reduces linearly over a distance of 5.5m 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 summarised in Figure 10. The profile indicates that the movement below the viaduct ranges between 2mm of heave 3m from the piled wall reducing to negligible movement at the opposite end of the viaduct (13m from the basement wall). The corresponding maximum deflection beneath the viaduct is 1mm.

Table 15 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 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
Viaduct	3.7	1	0.037	0.01	0 – Negligible

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 0’ corresponding to negligible damage.

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

7.2 Impact Assessment – London Underground tube tunnels

The results of the VDISP analysis indicate that the proposed basement construction will cause a maximum stress reduction of 39kPa on the nearest LUL tunnel running below Kentish Town Road. This corresponds to a stress reduction of approximately 14% assuming the shallowest tunnel is at 13.5mbgl and the weight of the overburden material is 20kN/m^3. The maximum vertical deflection of the tunnel is predicted to be 7mm.

The predicted combined short and long term displacement (mm) and stress change profiles along the crown of the tunnel are presented within Figure 12.

With reference to LUL Track Dimensions and Tolerances Standard (2007)[19], the allowable longitudinal deflection of the tunnel/tracks is 5mm per 5m span. Additionally, the radius of curvature of the tunnel should not be less than 15km.

An assessment of the differential movement of the tunnel at 5m intervals (Figure 12) predicts a maximum deflection of 3mm which is below the assessment criteria. Additionally, the maximum radius of curvature of the tunnel at the maximum point of deflection is predicted to be 113km which is also within the assessment criteria.

7.3 Impact Assessment – National Grid cables and Grand Union Canal wall

To assess the impact of the proposed basement development on the NG infrastructure and canal wall the predicted lateral and horizontal movement profiles have been combined to determine the overall worst case movement.

The vertical movement profile along the line of the NG infrastructure and canal wall due to short and long term heave, settlement due to pile installation and deflection is presented within Figure 13. It should be noted that the variation in settlement due to pile installation and deflection takes account of the varying offset distance between the proposed piled wall and existing line of the NG infrastructure and canal wall. The corresponding differential movement at typically 5m centres (i.e. assessment criteria) has also been plotted.

The results indicate that the maximum combined vertical movement of the infrastructure is 8mm and the maximum differential movement over a 5m span is 3mm. The overall average differential movement is typically less than 1.0mm. The results indicate that the

[19] London Underground Limited. Track Dimensions and Tolerances. (2007)

vertical movement of the NG infrastructure and canal wall fall below the assessment criteria.

The lateral movement profile along the line of the NG infrastructure and canal due to pile installation, pile deflection due to excavation and capping beam deflection is presented within Figure 14. It should be noted that the variation in lateral deflection takes account of the varying offset distance between the proposed piled wall and existing line of the NG infrastructure and canal wall. The corresponding differential movement at typically 5m centres (i.e. assessment criteria) has also been plotted.

The results indicate that the maximum combined lateral movement of the infrastructure is 5mm and the maximum differential movement over a 5m span is 3.8mm. The overall average differential movement is typically less than 1.5mm.

The results of the assessment indicate that the vertical and lateral movement of the NG infrastructure and canal wall due to the proposed basement development fall below the assessment criteria.

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.

The 'Lost Rivers of London' map produced by Barton indicates that two tributaries of the River Fleet join approximately 50m north of the site boundary where the river then trends south east along Camden Street. Historical mapping for the site (Survey of the Borough of St Marylebone 1834) provided by the client, indicates that before the river was culverted it passed through the site. Little evidence of this historical river course was noted during the site investigation. However, it is expected that the river bed may have been removed during the construction of the historical and existing developments and infrastructure onsite.

With reference to the Arup report[2], the site is approximately 2.2km southeast of the catchment for the pond chains on Hampstead Heath. Additionally, with reference to the EA website the site is not within a Flood Risk Zone.

Current flood mapping (Figure 15 CPG4) indicates that Kentish Town Road (KTR) was impacted by flooding in 1975. However, the road was not impacted by the 2002 flooding in the region or by the serious national floods in 2007 and 2012. It is noted in the London Borough of Camden flood risk management strategy[20] and Report of the Floods Scrutiny Panel[21] that the 1975 flood event was caused by the heaviest and most concentrated rainfall event recorded in this part of Camden. This 1 in 100 year event was preceded by a very dry summer and is therefore not considered to be representative of typical conditions in the area. In addition, the site is not within an area identified by the Environment Agency to be at risk of surface water flooding. Following the 2002 flood event in the region (not impacting KTR), new infrastructure, including larger diameter sewers and a holding tank, was installed in the Borough to mitigate the potential for future flooding. Additionally, the proposed development is positioned approximately 4m from Kentish Town Road.

[20] London Borough of Camden (2014) Managing Flood Risk in Camden: The London Borough of Camden flood risk management strategy
[21] London Borough of Camden (2003) Floods in Camden: Report of the Floods Scrutiny Panel

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 viaduct) generated by the assumed construction methods and sequence is likely to be (within Category 0) ‘negligible’. Additionally, movements predicted in the vicinity of the canal wall, NG and LUL infrastructure are 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.

The system should operate broadly in accordance with the ‘Observational Method’ as defined in CIRIA Report 185[22]. 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 piled wall and face of the adjacent 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 top of the NG cable run structure to provide ‘real time’ monitoring of this structure as excavation progresses.

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 piling works. It is also recommended that a condition survey be undertaken of the Northern line LUL tunnels.

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

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

10.1 Construction monitoring – Installation

Monitoring of adjacent structures/infrastructure 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 identified critical constraints.

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. Depending on the limits for the LUL tunnels electronic distance meter (EDM) surveys could be undertaken using robotic Total Station. Alternatively, Shape Accel Array (SAAs) could be installed along the crown and around the intrados of the tunnel to measure longitudinal and transvers deflections remotely and in 'real time'.

In addition, a pre-commencement condition survey of the constraints 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 marks) positioned outside the zone of influence of ground movement.

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.
- Little evidence of this historical river course of the Fleet was noted during the site investigation and it is expected that it may have been removed during the construction of the historical/current developments and infrastructure onsite. All historical mapping provided which shows the *River Fleet* passing through the site are dated before any infrastructure or developments were constructed in the region. However, should the historical river course be encountered onsite during pile installation or basement excavation, the depth and extent of alluvial deposits should be carefully recorded and the impact of this change in expected ground condition on foundations and basement design should be carefully assessed.
- 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 0’ (negligible damage) for the Network Rail viaduct.
- Combined vertical and horizontal ground movements predicted along the line of the LUL tunnel and NG infrastructure fall below current limits recommended. Additionally, the predicted movement of the canal wall is below the assessment criteria.
- 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. Additionally, if the historical course of the river fleet is encountered there may also

be water present within this material. 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

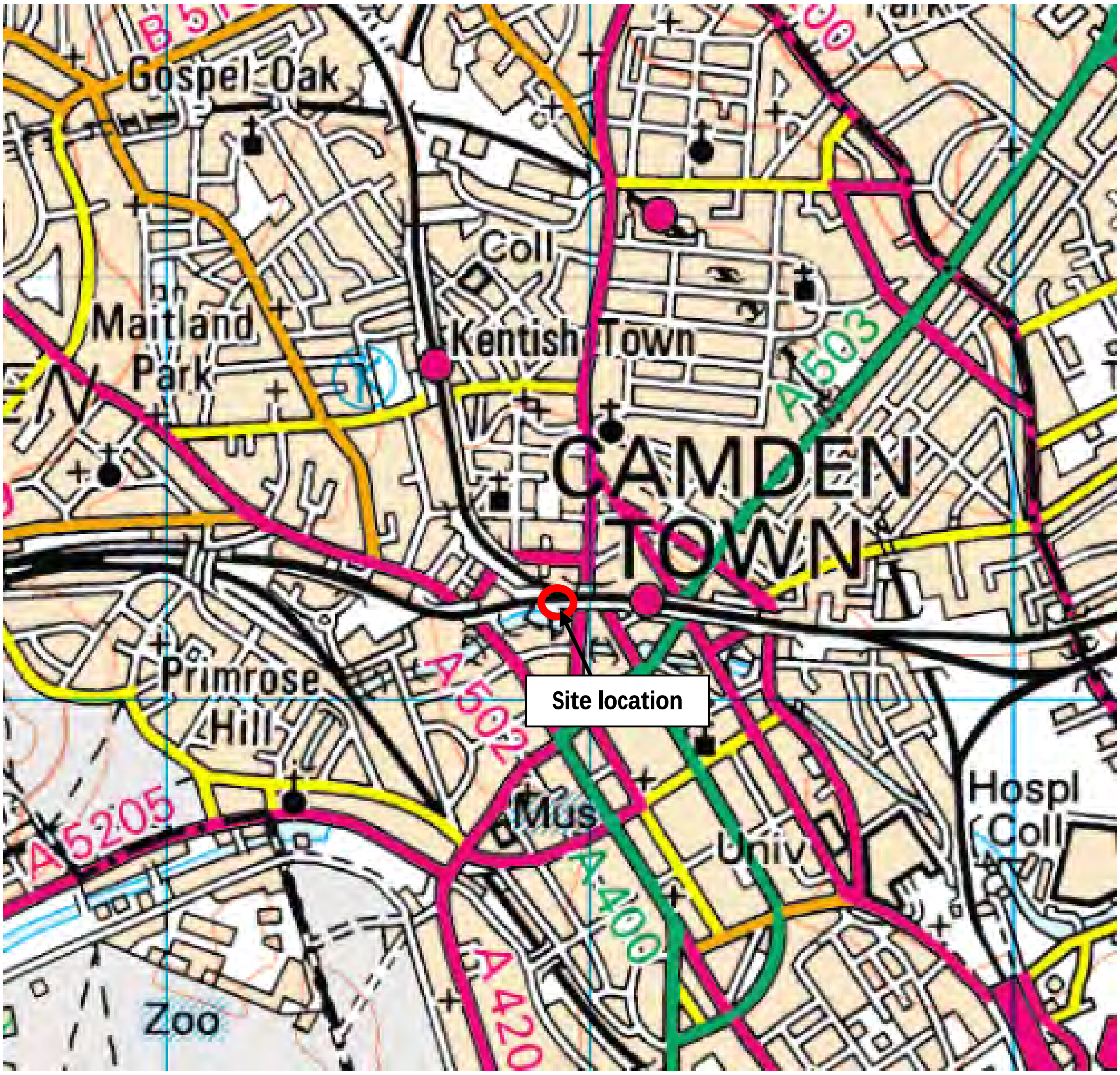
The ground movement and impact assessments have indicated that damage to neighbouring properties will be within allowable limits. Therefore, it is considered that there are no cumulative impacts in respect of ground or slope stability.

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.

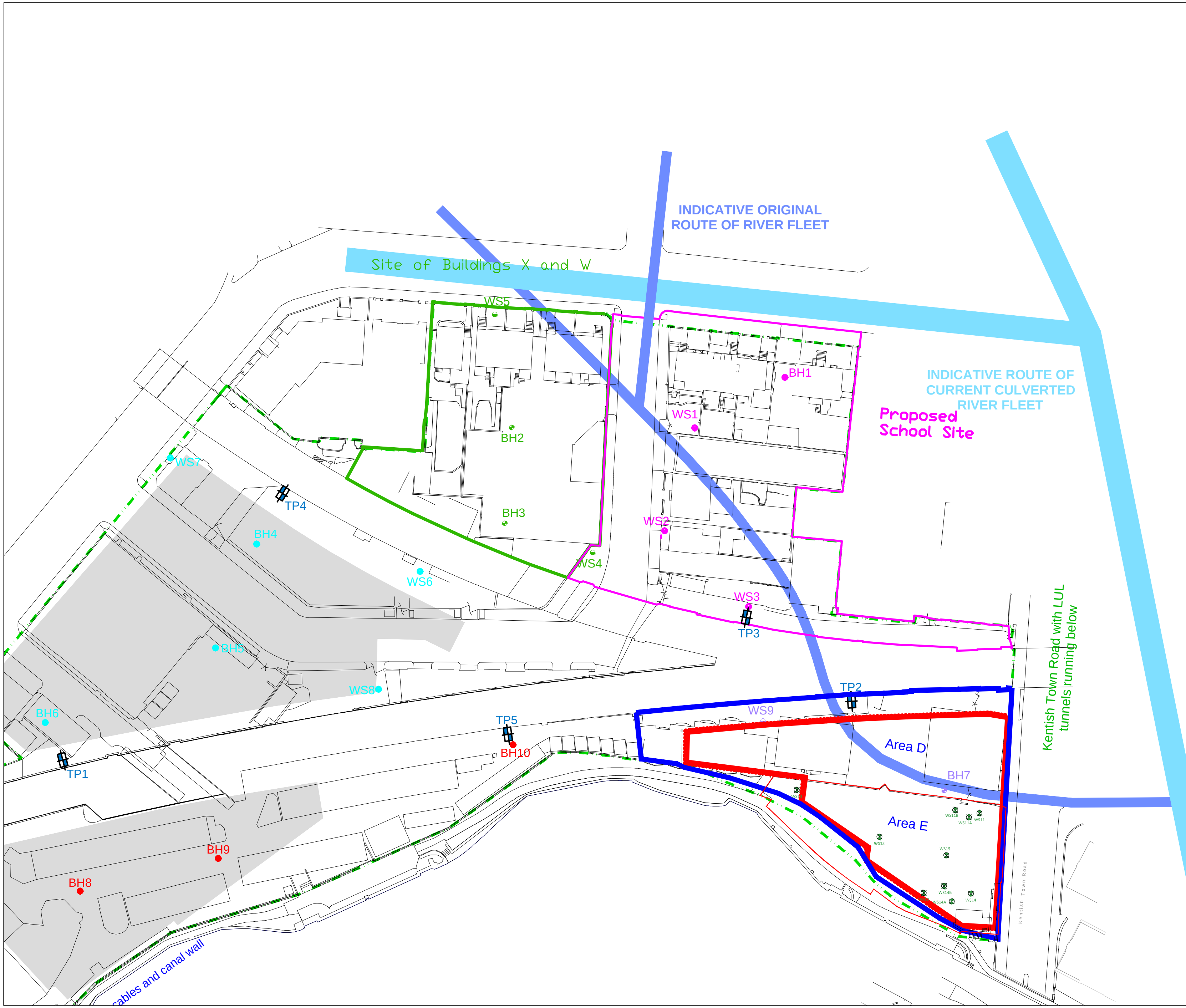
It is understood that surface waters will join the existing drainage infrastructure (albeit via basement pumping if a gravity fed solution is not feasible), with no significant changes in peak drainage outflows anticipated from the site. The site is currently covered by hardstanding and is underlain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

Based on the results of the ground movement assessment and taking account of the distance to the other proposed basement blocks, the cumulative impact of these basements and associated ground movements on the NR infrastructure will not change considerably compared to current predictions.

FIGURES



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

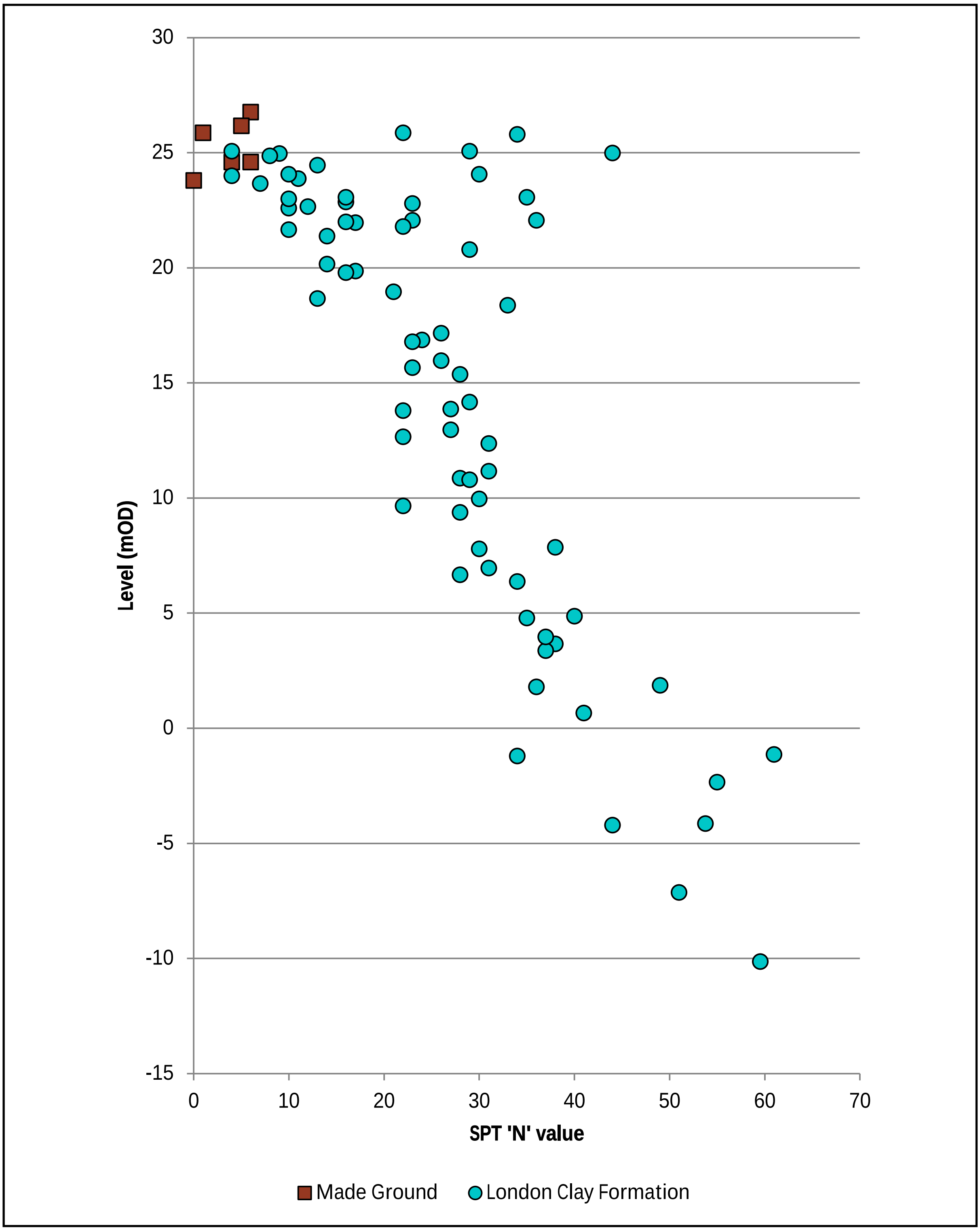


Borehole ID	SI Phase	Maximum depth (m)
BH1	1	15
WS1 - WS3	1	3.15
BH2	1	30
BH3	1	15
WS4 - WS5	1	5
BH4 & BH6	2	25
BH5	2	40
BH7	2	30
WS6 - WS9	2	5
BH8 & BH10	3	25
BH9	3	30

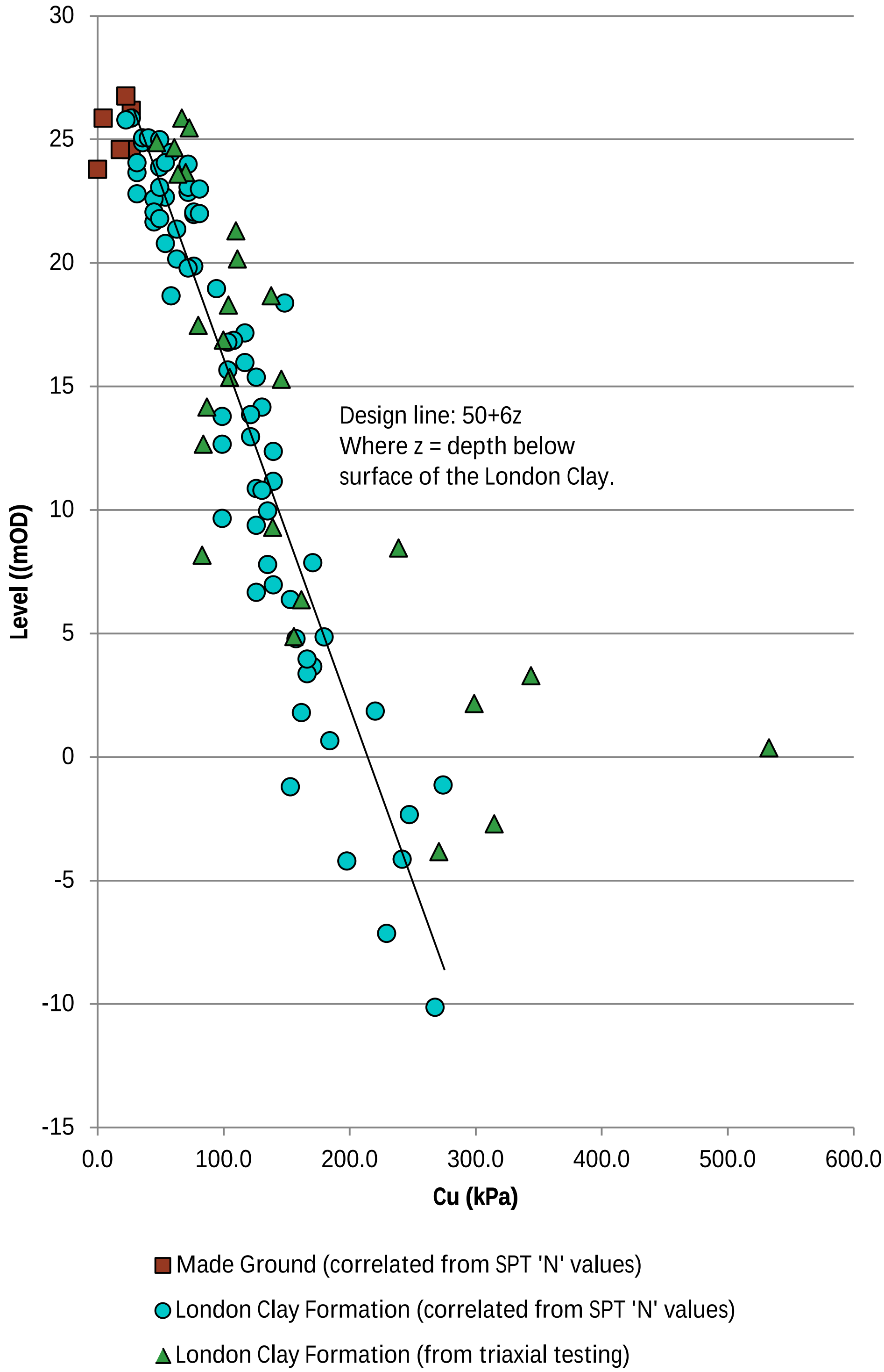
Rev.		Date:	

Card Geotechnics Ltd.
4 Godalming Business Centre
Woolsack Way
Godalming
Surrey
GU7 1XW
T: 01483 310600

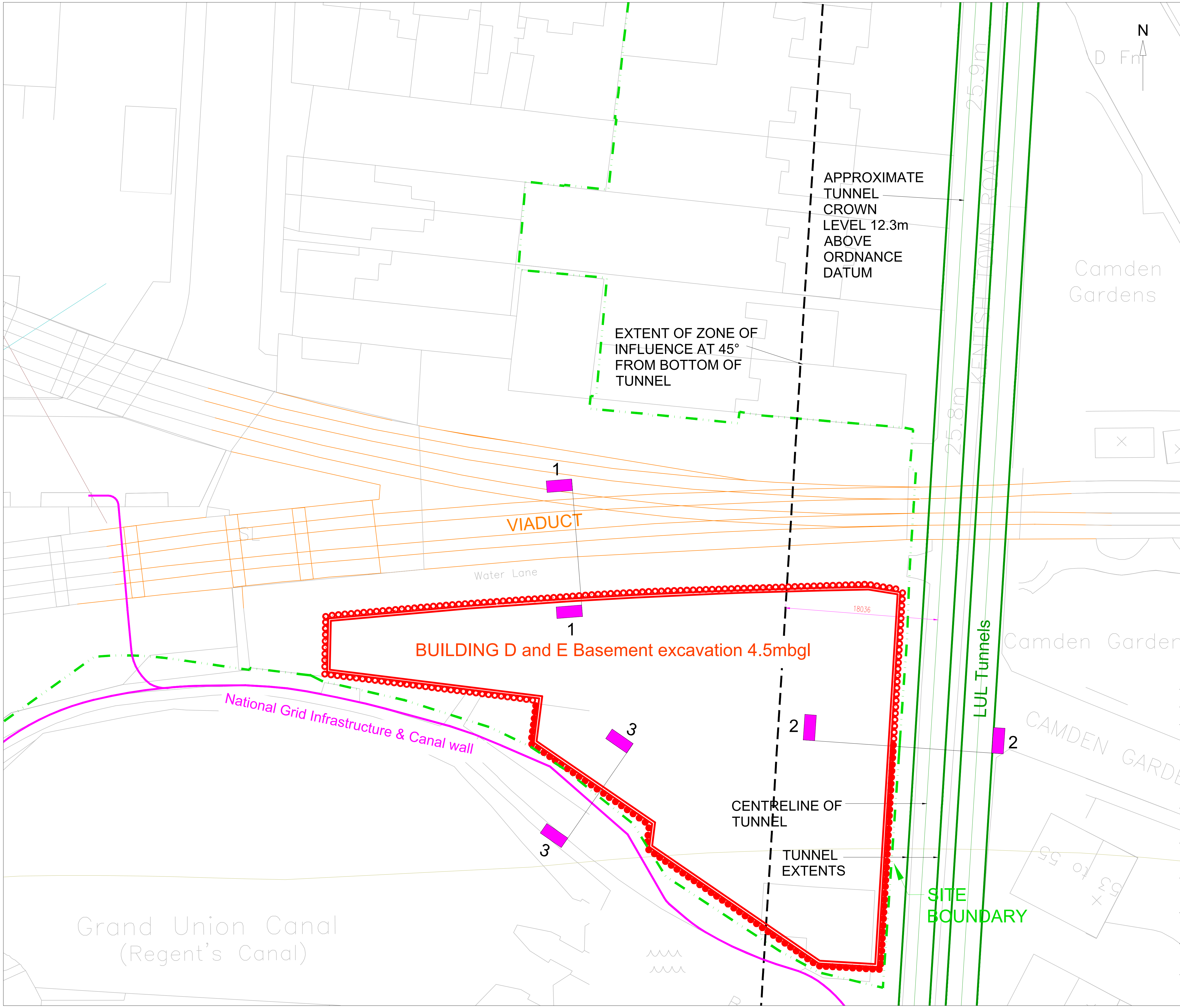
		Date:		
			Project: Camden Lock Village, London	Scale: NTS
Dwn.	JLA	21/1/15	Client: Walsh Associates	
Ckd.	JMS	21/1/15	Title: Figure 2 - Site layout and exploratory hole location plan	
Appr.	RJB	21/1/15		
Job No.: CG/18067A			Drw.no.: CG/18067A-002	Rev.: .



Client **Walsh Associates**	Project **Camden Lock Village, London**	Job No **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
*	*	*

CGL
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4 Godalming Business Centre
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Godalming
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Project: **Camden Lock Village Phase 2, London**

Client: **Walsh Associates**

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

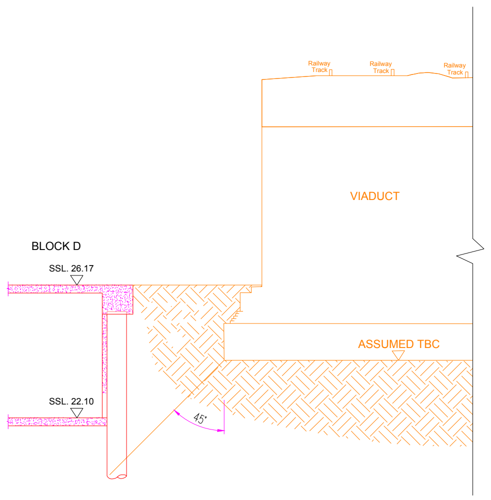
Scale(s): **NTS**

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

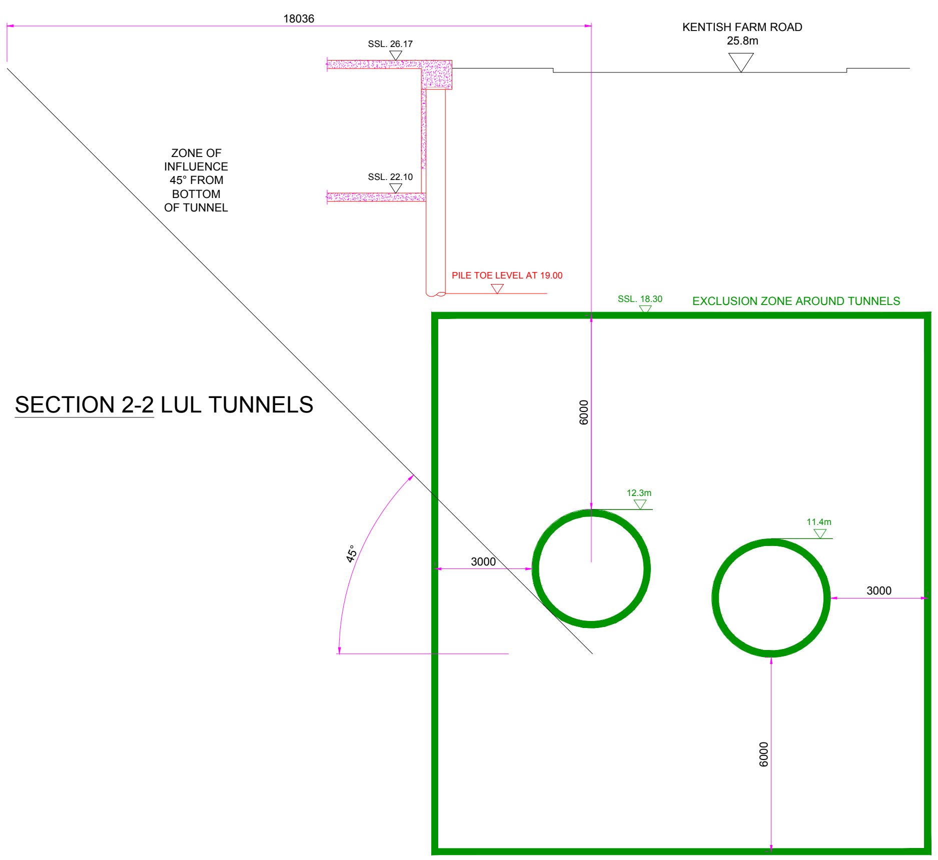
Drawn	JLA	21/01/15
Checked	JMS	21/01/15
Approved	RJB	21/01/15

Dwg No.: **CG/18067a-101**

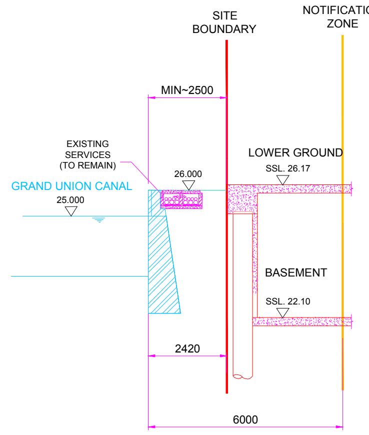
Rev.: *



SECTION 1-1 NR VIADUCT



SECTION 2-2 LUL TUNNELS



SECTION 3-3 NG INFRASTRUCTURE & CANAL WALL

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 6 Conceptual site model - constraint sections**

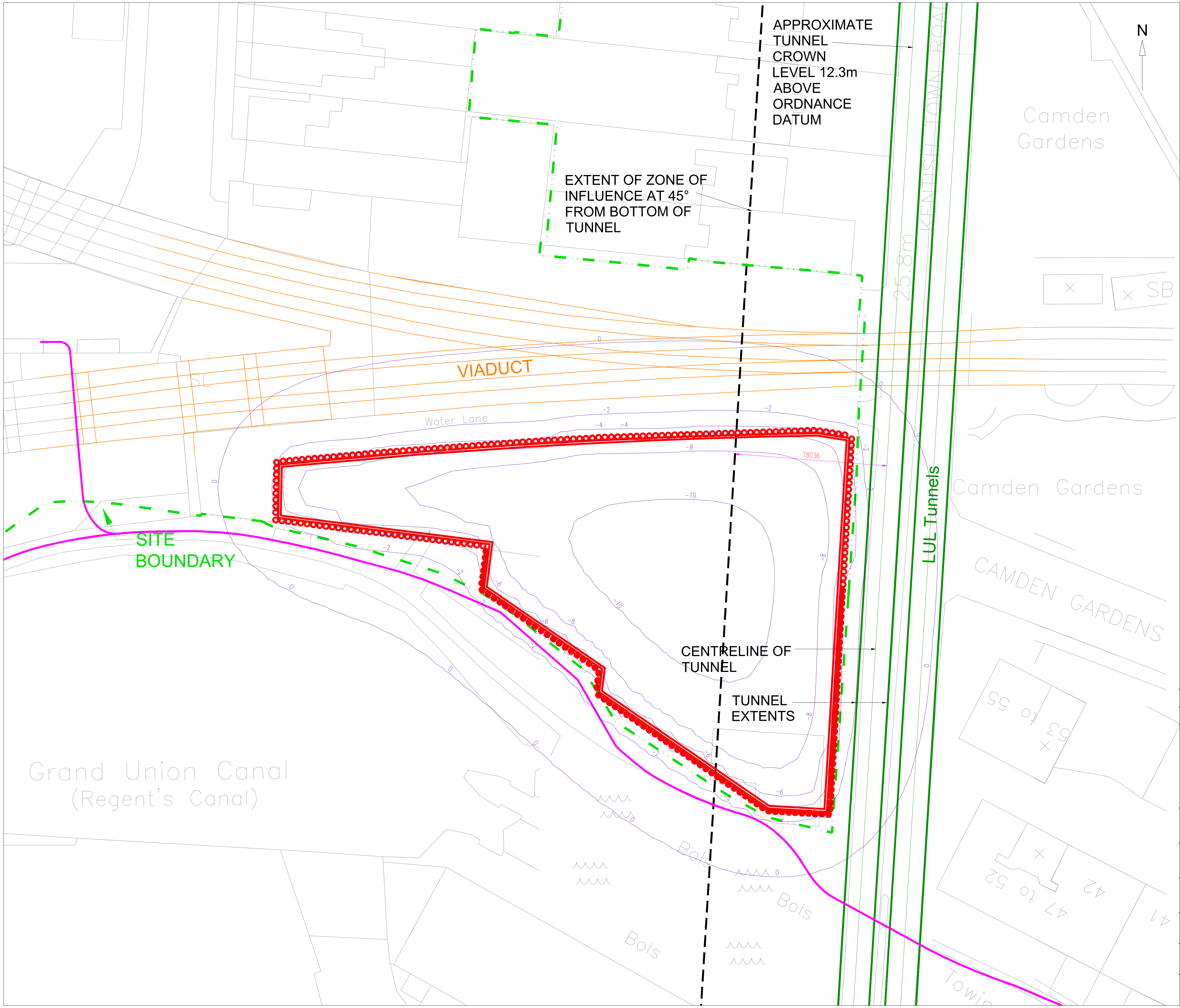
Scale(s): **NTS**

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

Drawn	JLA	21/01/15
Checked	JMS	21/01/15
Approved	RJB	21/01/15

Dwg No.: **CG/18067a-102**

Rev.: *



NOTE:
Negative values indicate heave

*	*	*
Rev	Date	Comments

CGL

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4 Godalming Business Centre
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Surrey
GU7 1XW
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Project: **Camden Lock Village Phase 2, London**

Client: **Walsh Associates**

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

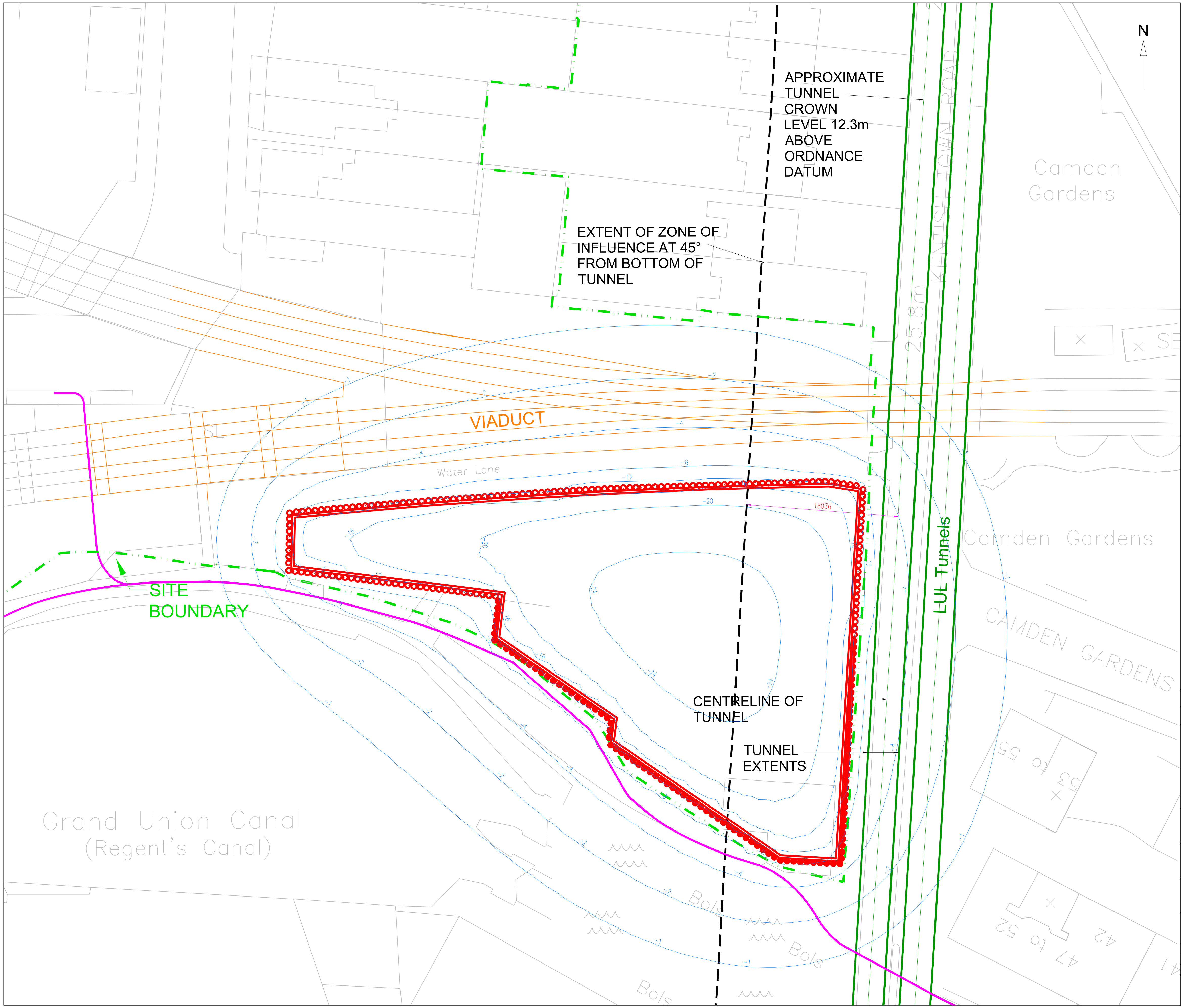
Scale(s): **NTS**

Job No. **CG/18067a**

Drawn	JLA	21/01/15
Checked	JMS	21/01/15
Approved	RJB	21/01/15

Dwg No. **CG/18067a-103**

Rev. *



NOTE:
Negative values indicate heave

Rev	Date	Comments
*	*	*

Card Geotechnics Ltd
4 Godalming Business Centre
Woolsack Way
Godalming
Surrey
GU7 1XW
T: 01483 310600

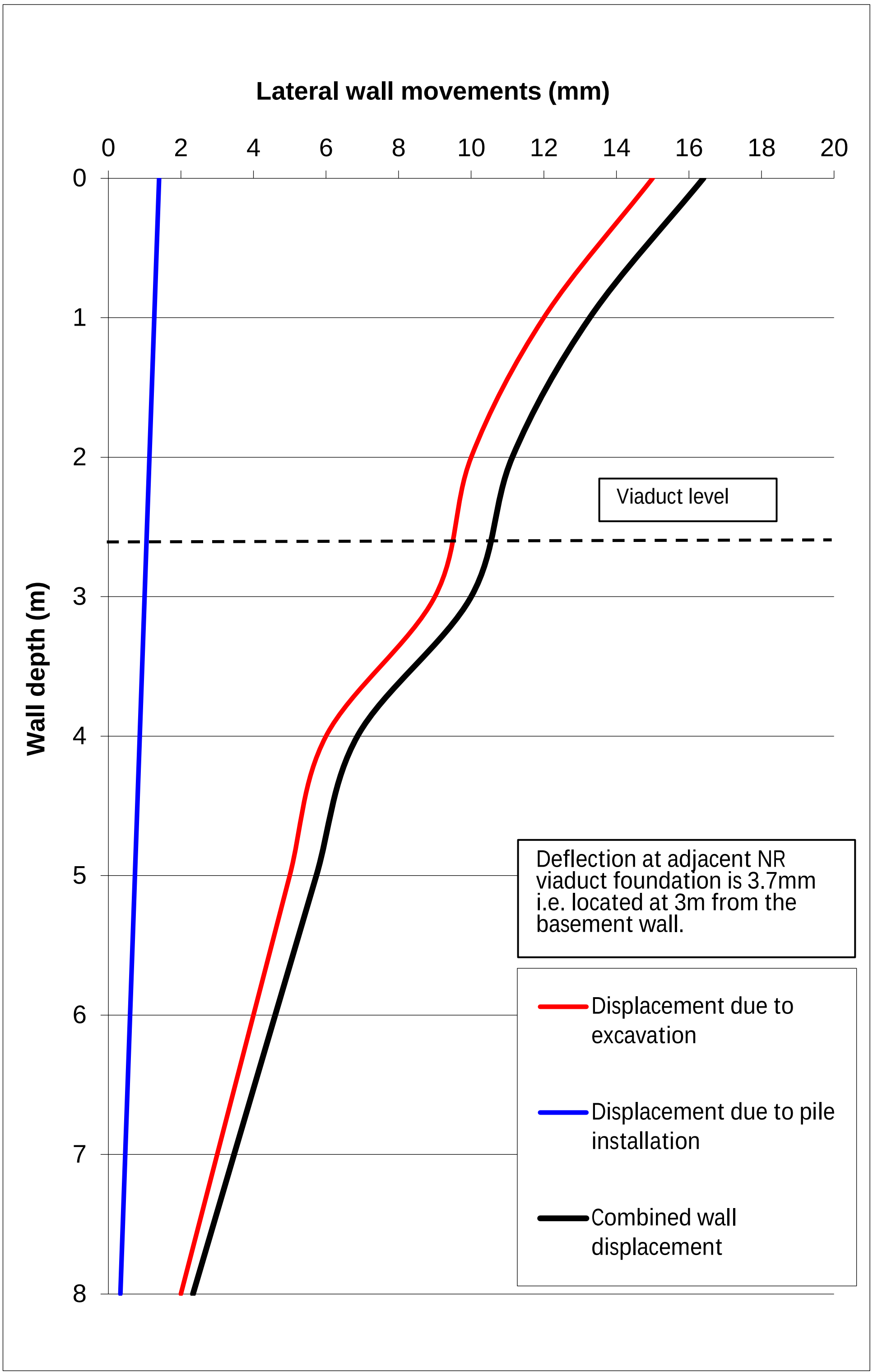
Project: **Camden Lock Village Phase 2, London**

Client: **Walsh Associates**

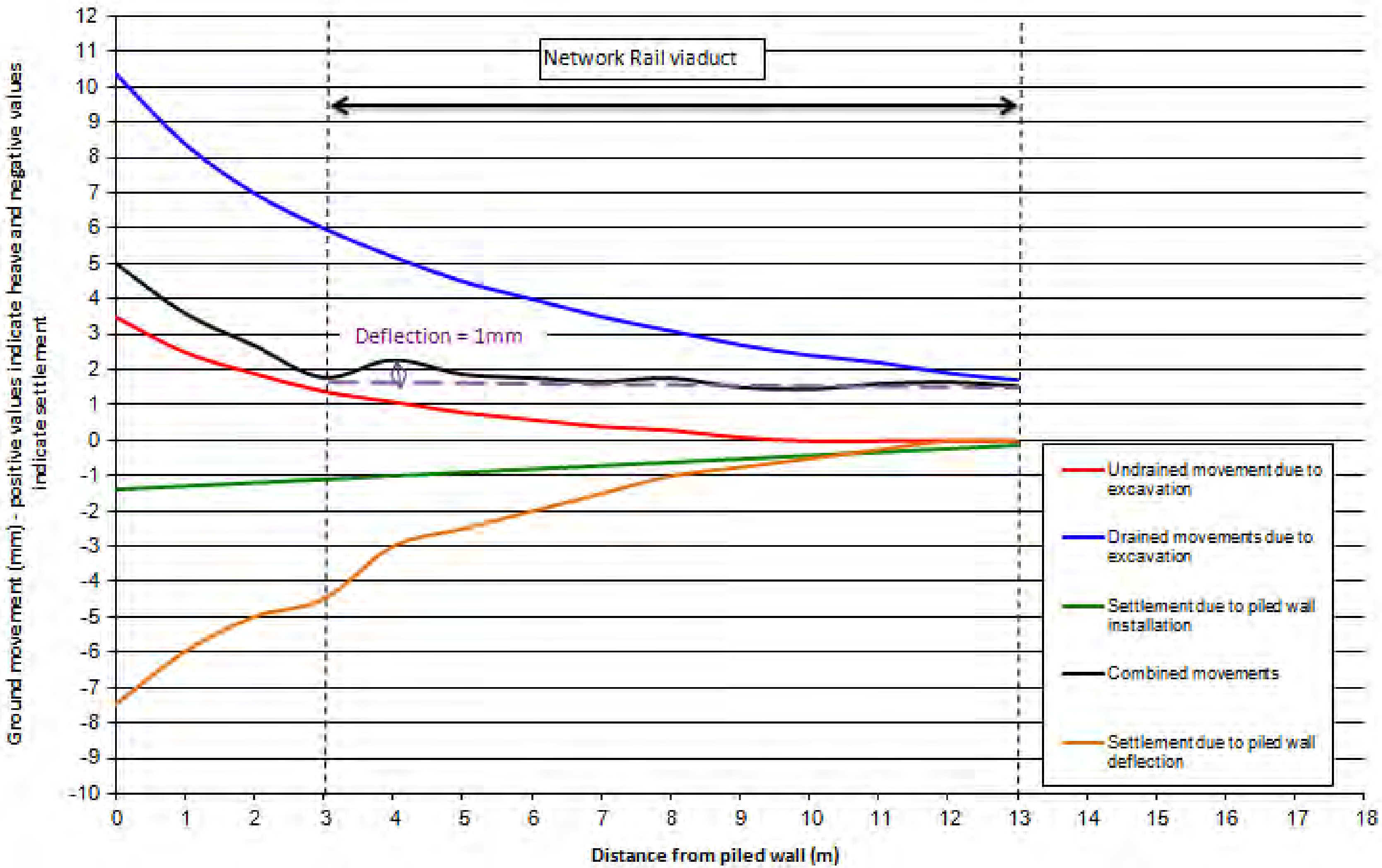
Drawing title: **Figure 8 Long term ground movement contour plot**

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

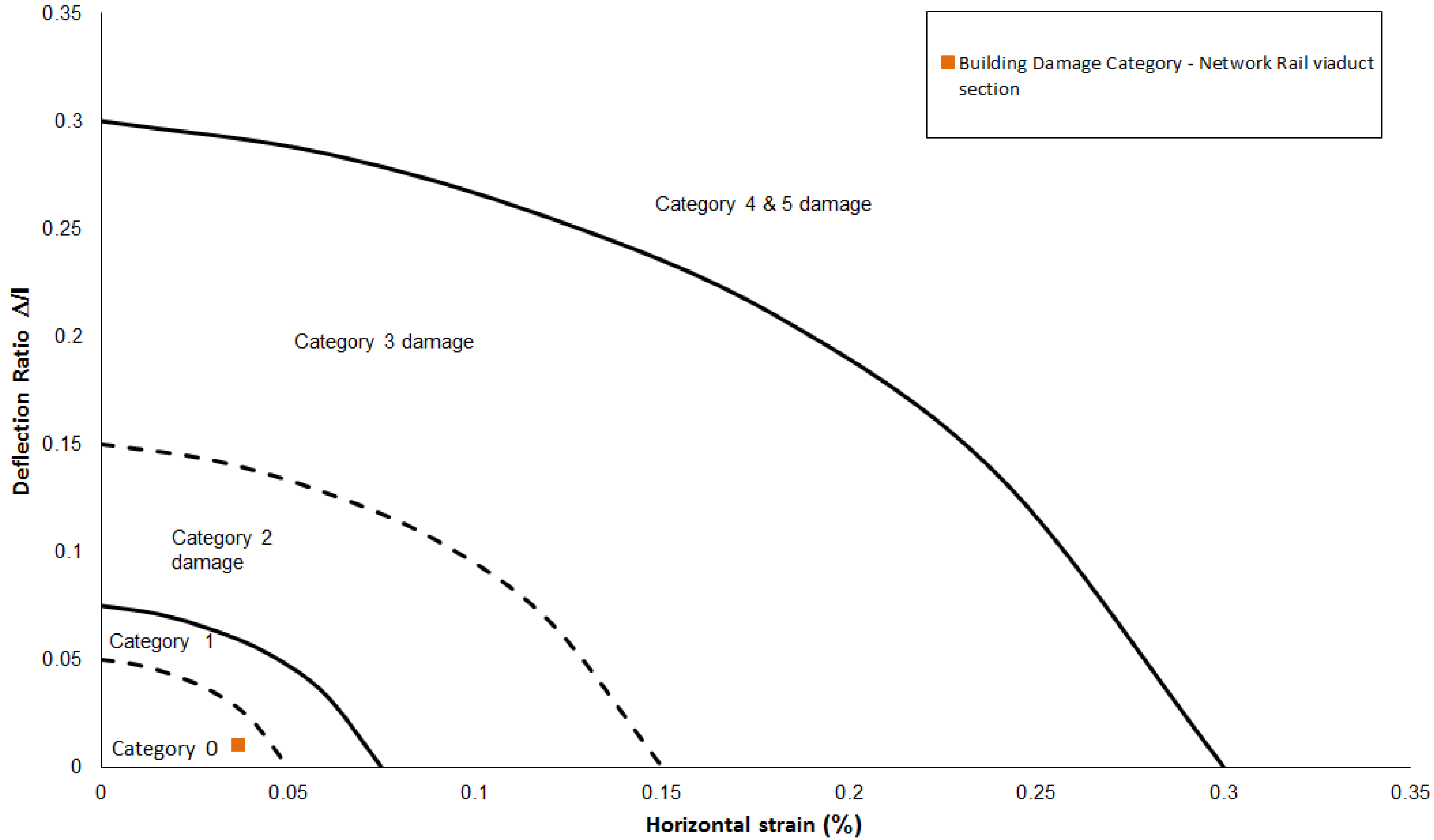
			Dwg No.	Rev.
Drawn	JLA	21/01/15	CG/18067a-104	*
Checked	JMS	21/01/15		
Approved	RJB	21/01/15		



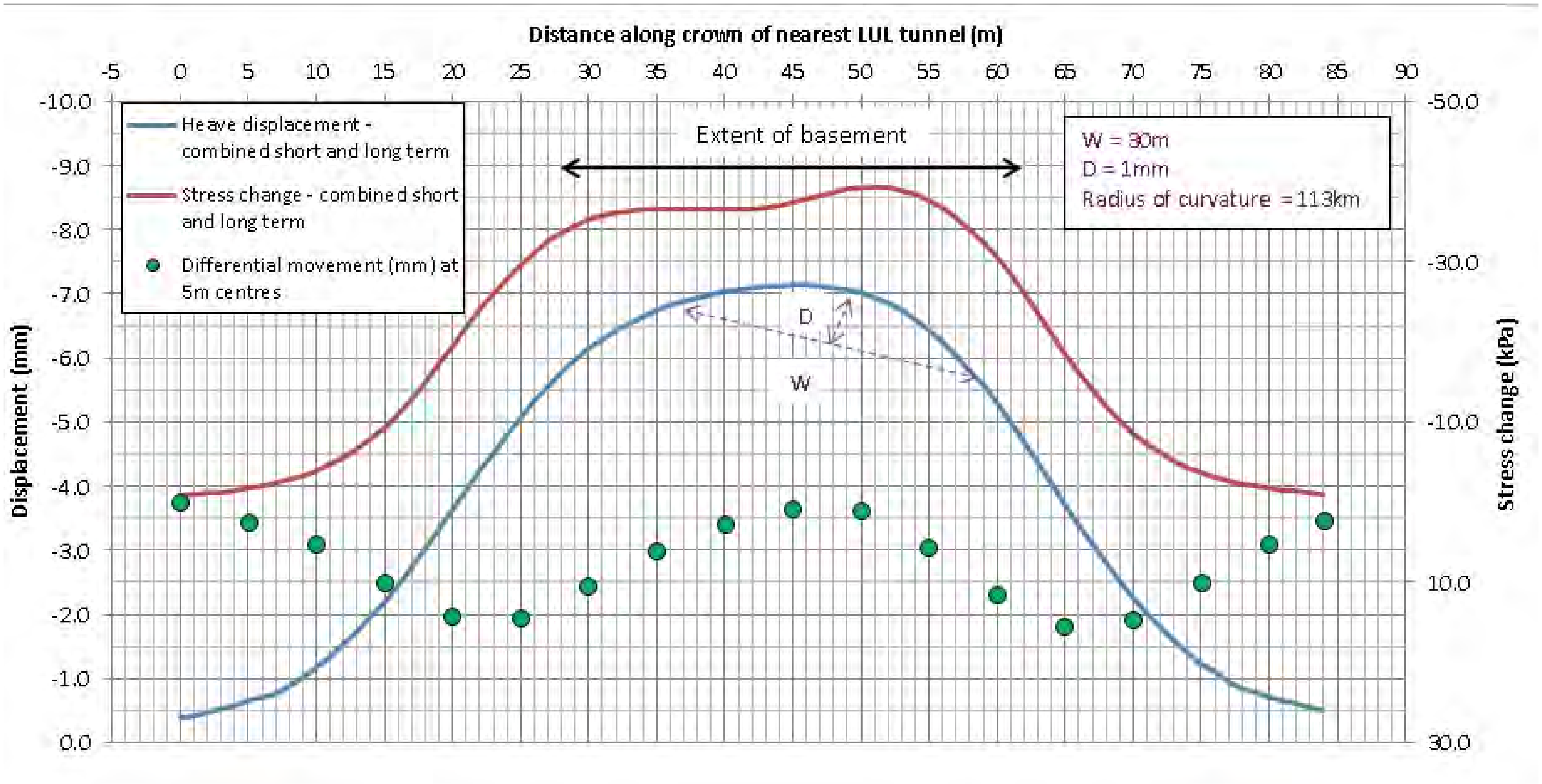
Client	Project	Job No
Walsh Associates	**Camden Lock Village, London**	**CG/18067a**
CGL	Title **Combined lateral movement profile –piled wall adjacent to NR viaduct**	**Figure 9**



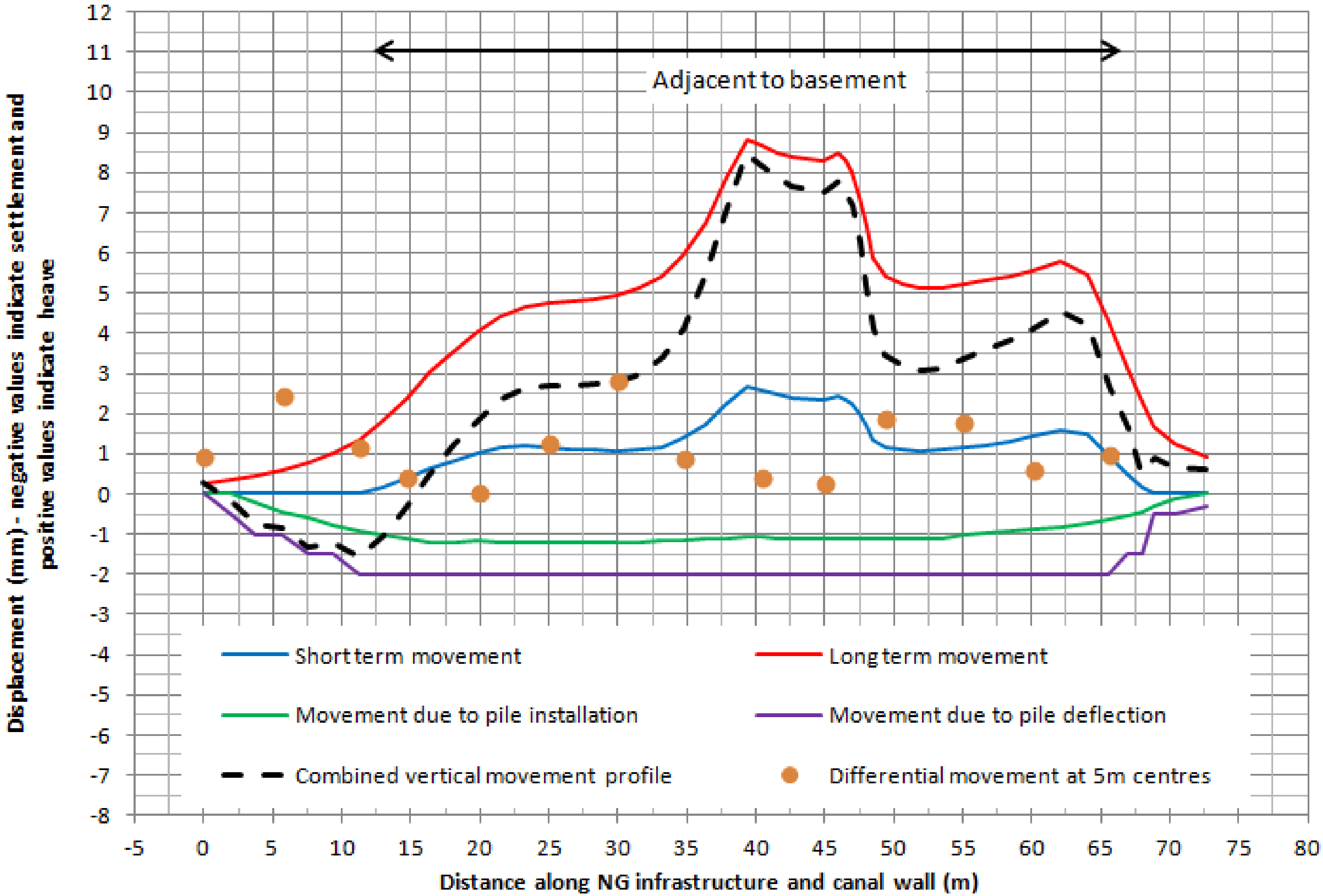
Client	Project	Job No
Walsh Associates	**Camden Lock Village, London**	**CG/18067a**
CGL	Title **Combined vertical movement profile –below the NR viaducts**	**Figure 10**



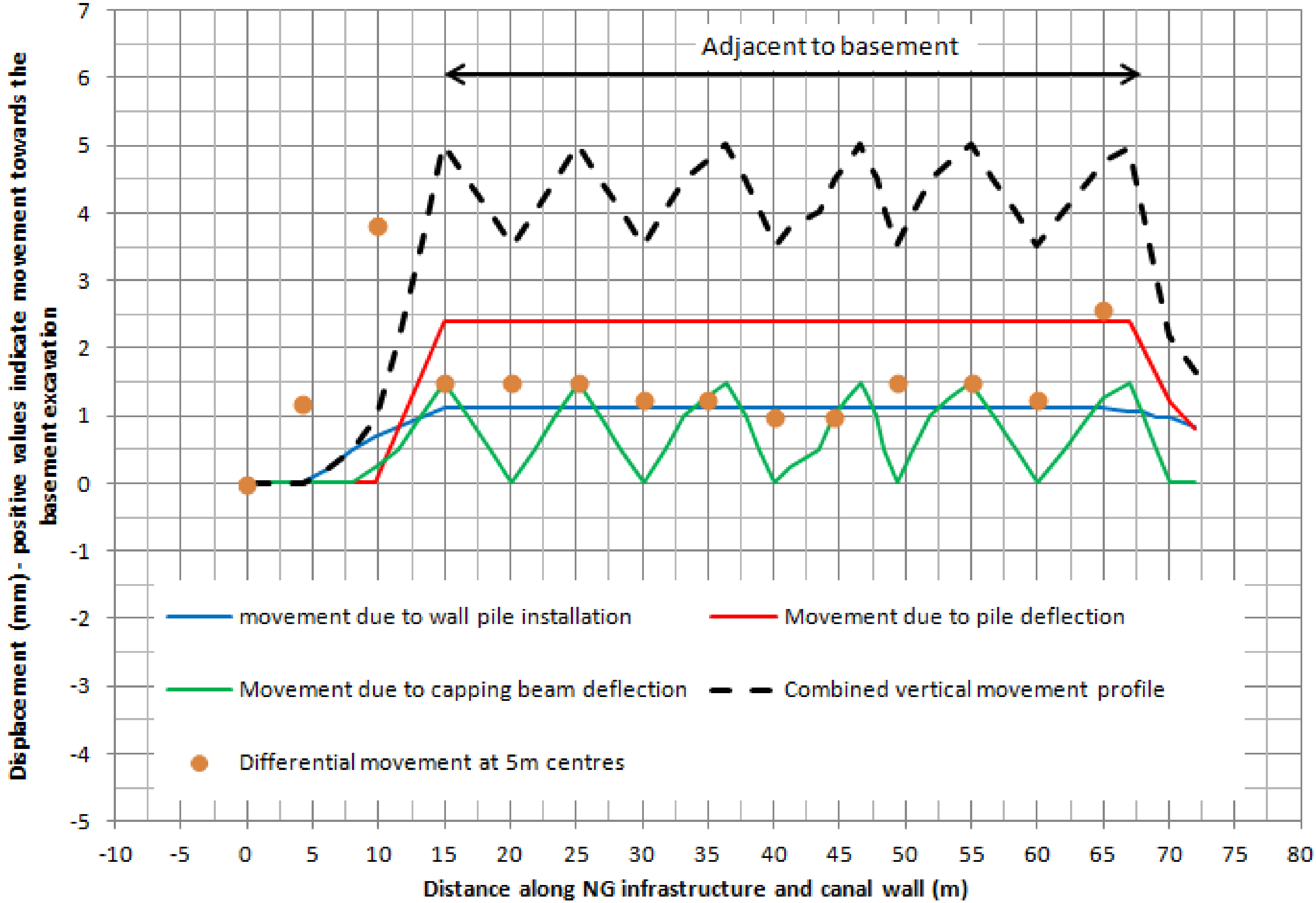
Client	Project	Job No
Walsh Associates	**Camden Lock Village, London**	**CG/18067a**
CGL	Title: **Structure interaction plot**	**Figure 11**



Client	Project	Job No
Walsh Associates	**Camden Lock Village, London**	**CG/18067a**
CGL	Title **Displacement and stress change along crown of LUL tunnel**	**Figure 12**



Client	Project	Job No
Walsh Associates	**Camden Lock Village, London**	**CG/18067a**
CGL	Title **Vertical movement profile – NG infrastructure and canal wall**	**Figure 13**



Client	Project	Job No
Walsh Associates	**Camden Lock Village, London**	**CG/18067a**
CGL	Title **lateral movement profile – NG infrastructure and canal wall**	**Figure 14**

APPENDIX A

Development drawings

Plans

Ground + Basement Levels
ONGOING DESIGN DEVELOPMENT

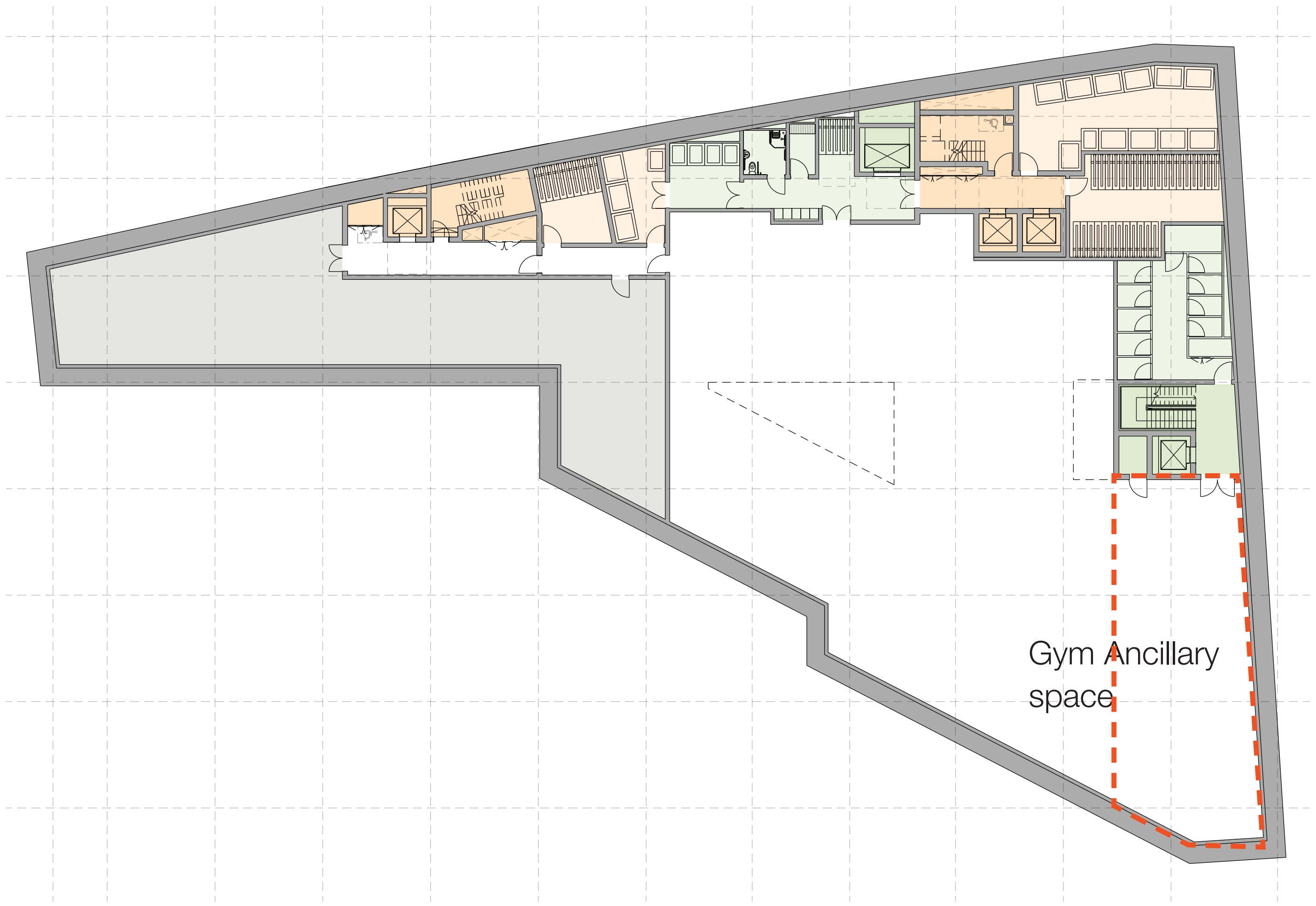
INCLUSION OF PUBLIC/PRIVATE GYM

Brief development and considerations;

- Only in building E demise as D has a protected use from planning.
- Locate around canal side elevations.
- Possibly locate on 2 levels

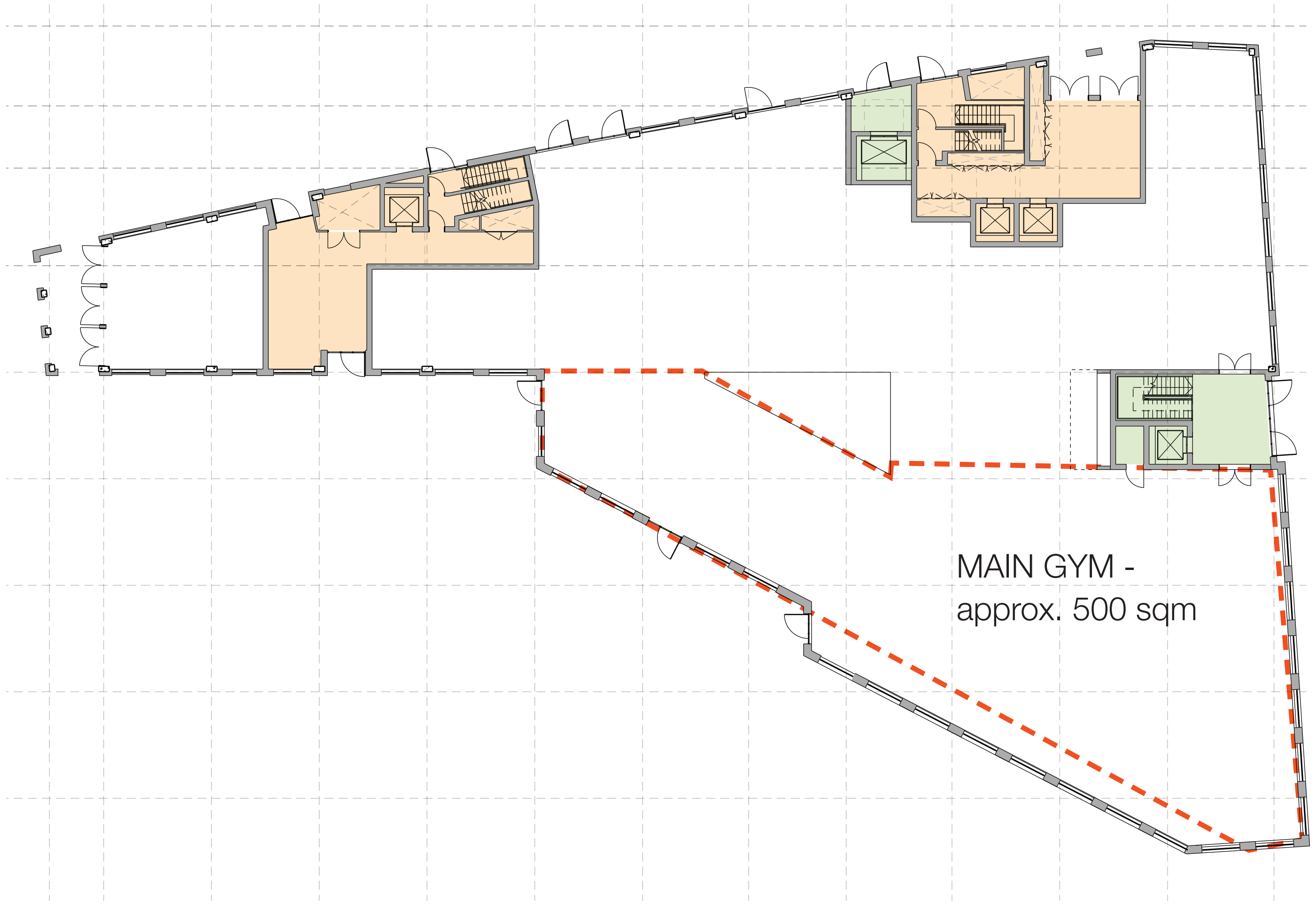
Basement

Not To Scale



Ground Floor

Not To Scale

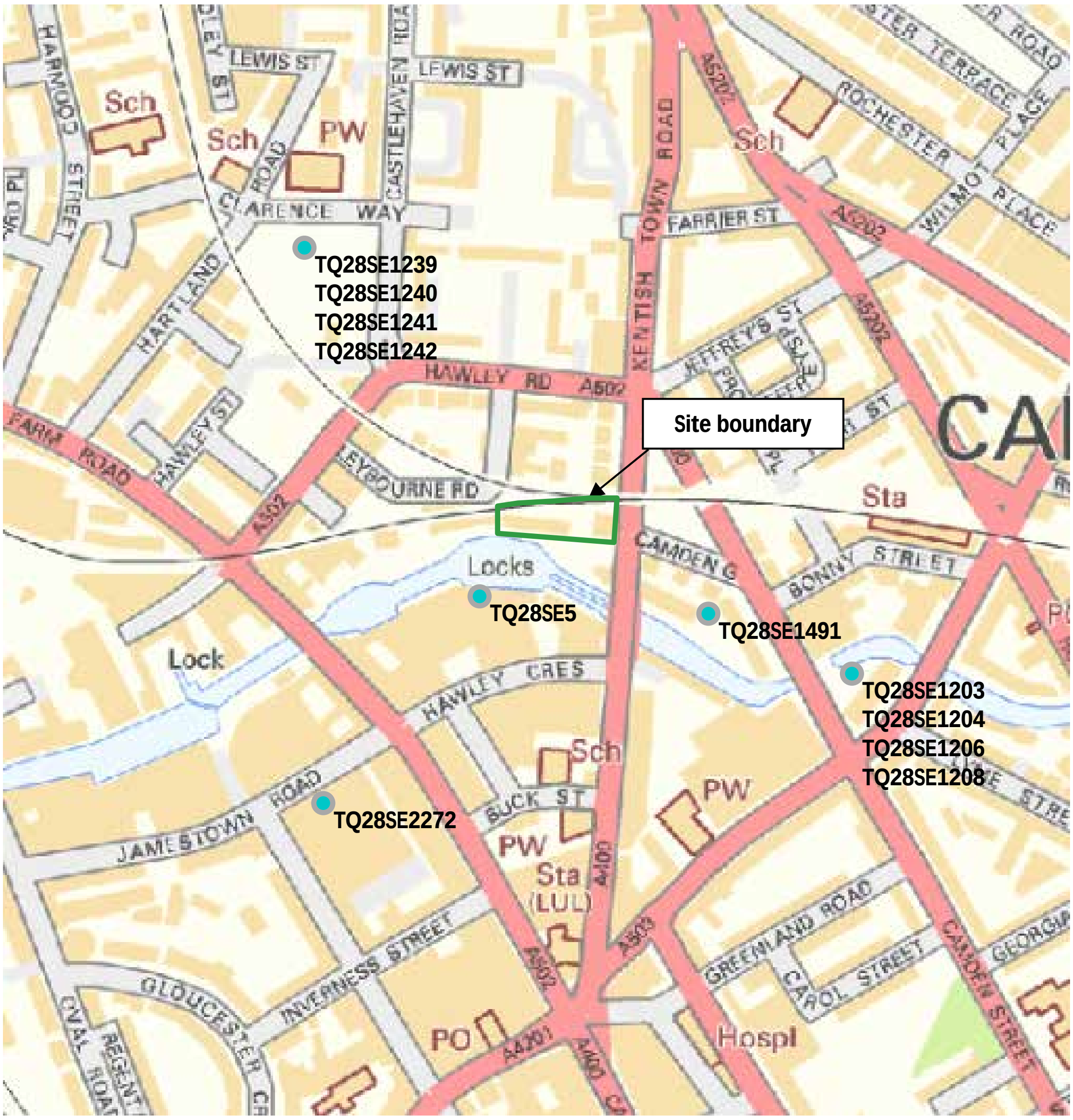


Information required for development:

- Do SSL have an operator in mind?
- What will be the likely catchment area - no. of occupants?

APPENDIX B

BGS borehole logs



N

Base figure taken from BGS online
Not to scale

Client	Project	Job No
Walsh Group	**Camden Lock Village, London**	**CG/18067A**
CGL	Title **BGS borehole location plan**	**Appendix B**

Project	ARLINGTON HOUSE, 220 ARLINGTON ROAD, CAMDEN, LONDON	Client	[redacted]	Trial Pit Excavation Methods	BRADFORD WATTS HAND PIT	Hole No.	TH8A
		Engineer	[redacted]	Pit Dimensions:	Length - 1.80 m Width - 1.40 m	Sheet	1 of 1
Ground Level	25.33 m.A.O.D.	Coordinates	m.E. m.N.	Orientation:	Length -	Job No	10482

WATER		STRATA				SAMPLING/IN SITU TEST			LAB TESTING				OTHER TESTS AND NOTES
Date/Time at Depth	Depth to Water m	Description	Legend	Level m.A.O.D.	Depth m	Depth m	Type & No.	Test Result	% <425	W %	Wp %	WL %	
		Made Ground (Brickwork wall)				0.20	D1						TH8A logged from north west face of Trial hole CLEA screen with speciated polyaromatic hydrocarbons (D1)
						0.50	D2		100	34	27	75	No groundwater recorded during fieldwork
		Made Ground (Concrete)		24.60	0.73								
30/10/06	DRY C			24.25	1.08								Water in hole from Diamond Drilling corehole in wall above pit
													Trial pit complete at 1.09m

Pit Stability, Shoring, etc.

No collapse of sides of trial pit

Water Level observations during digging, depths below GL.

Strike	Depth Obs.	Depth after 5min	10 min	15 min	20 min

WATER
- 1 First Strike
- 2 Subsequent Strike
- N - Overnight Depth
- C - Completion Depth
- S Seepage not rising

SAMPLE AND TEST KEY
- D Small disturbed sample
- B Bulk disturbed sample
- W Water sample
- U Undisturbed sample
- K Percolation Test
- PP Perth Penetrometer Test
- HV Hand shear vane test
- SRD Sand replacement density test
- CBR In situ CBR test
- PB Plate Bearing Test

TEST RESULT
- Np = Np Value
- V = Average Hand Shear Vane Strength - kN/m^2
- BD = In-Situ Bulk Density - Mg/m^3
- CBR = California Bearing Ratio - %

Fieldwork By	GJB
Dates	30/10/03
Log	GJB

Sheet 1 of 1 TH8A

RSA GEOTECHNICS LTD (01449) 723723

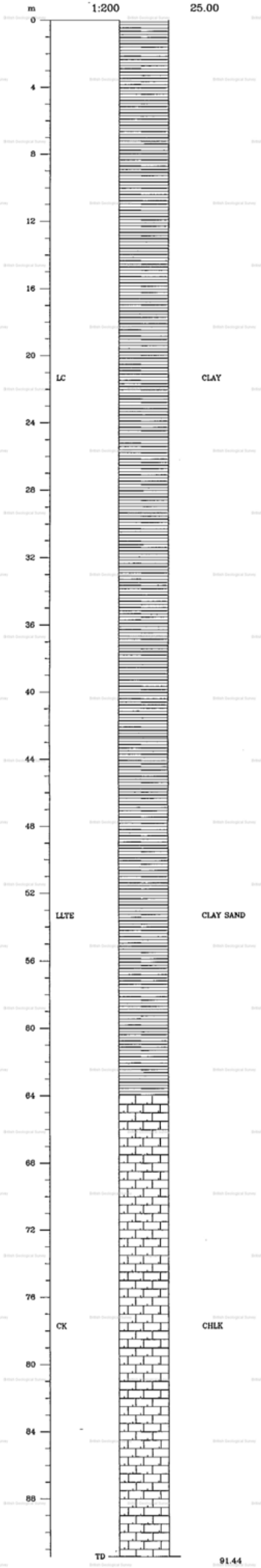
5 TQ28SE 5

N–WHITAKERS BREWERY HAMPSTEAD

Grid Reference: 28850 84138

Scale: 1:200

Ordnance Datum: 25.00



TERRESEARCH LIMITED

BOREHOLE NO. 1 TQ28SE 1203

Contract Name: Camden Town
Report No.: S. 808/15
Client: S. Deltis Ltd.
Site Address: Corner of Camden Street and Camden Road
Engineers: Leonard & Partners, 344 - 360 South Lambeth Rd, London S.W.8.

2908, 8410

Standing Water Level: 55'0" 17.6.65. 30'0" 21.6.65.
Diameter: 8"
Water Struck: 3'6"
Method of Boring: Shell/Auger
Ground Level: 78.49
Start: 14.6.65. Finish: 16.6.65.

Remarks:

Description of Strata	Thickness	Depth	Disturbed Samples	'U' Cores and 'N' P. Test
Made ground (sand, bricks stones etc.)	1'0"	1'0"	J2101 0'6"	
Soft brown mottled clay	2'6"	3'6"	J2102 2'6"	
Brown sandy clay with gravel	5'0"	8'6"	B2103 5'0" J2104 7'6"	5'0" N=14
Stiff brown mottled clay with layers of silt and sulphate crystals	8'0"	16'6"	J2106 12'6"	U2105 10'0" U2107 14'0"
Stiff fissured brown clay with sulphate crystals	5'6"	22'0"	J2108 17'6"	U2109 19'0"
Hard fissured grey silty clay with traces of organic material	6'0"	28'0"	J2110 22'6" J2112 27'6"	U2111 24'0"
Hard fissured silty grey clay	10'0"	38'0"	J2114 32'6" J2116 37'6"	U2113 29'0" U2115 34'0"
Hard fissured grey clay with layers of silt and occasional sulphate crystals	23'6"	61'6"	J2118 42'6" J2120 47'6" J2122 52'6" J2124 57'6"	U2117 39'0" J2119 45'0" U2121 49'0" U2123 54'0" U2125 60'0"
			W2126	
TOTALS	61'6"	61'6"		

NOTES: 1. Descriptions are given in accordance with the B.S. Civil Engineering Code of Practice C.P.2001 "Site Investigations"
2. J indicates Jar Samples.
B „ Bulk Samples.
W „ Water Samples.
U „ Undisturbed Core Samples. These are nominal 4 in. diam. and 18 in. long. Depths shown are top of sample.
N „ Number of blows per ft. penetration with Standard Penetration Tests.

TERRESEARCH LIMITED

BOREHOLE NO. 2 TQ28SE

Contract Name: Camden Town
Report No.: S. 808/15 1204

Client: W. Daltin Ltd.
Site Address: Corner of Camden Street, and Camden Road, London N.W.1.

Engineers: Leonard & Partners, 344 - 360 South Lambeth Rd., London S.W.8.

2910, 8406

Standing Water Level: None
Diameter: 8"

Water Struck: None
Method of Boring: Shell/Auger

Ground Level: 78.23
Start: 19.6.65. Finish: 21.6.65.

Remarks:

Description of Strata	Thickness	Depth	Disturbed Samples	'U' Cores and 'N' F. Test
Made ground (concrete, grey silty clay with bricks)	3'0"	3'0"	J3724 2'6"	
Brown sandy clay with gravel	2'6"	5'6"	B3725 5'0"	
Stiff fissured mottled brown clay with occasional sulphate crystals and layers of silt	17'6"	23'0"	J3727 8'6" J3729 12'6" J3731 17'6" J3733 22'6"	U3726 6'0" U3728 10'0" U3730 14'0" U2732 19'0"
Hard silty mottled grey clay with sulphate crystals	5'0"	28'0"	J3735 27'6"	U3734 24'0"
Stiff to hard fissured grey silty clay with layers of light grey silt. Small crystalline aggregates of pyrites towards the base	32'6"	60'6"	J3737 32'6" J3739 37'6" J3741 42'6" J3743 47'6" J3745 52'6" J3747 57'6"	U3736 29'0" U3738 34'0" U3740 39'0" U3742 44'0" U3744 49'0" U3746 54'0" U3748 59'0"
TOTALS	60'6"	60'6"		

NOTES: 1. Descriptions are given in accordance with the B.S. Civil Engineering Code of Practice C.P.2001 "Site Investigations"

2. J indicates Jar Samples.
B " Bulk Samples.
W " Water Samples.
U " Undisturbed Core Samples. These are nominal 4 in. diam. and 18 in. long. Depths shown are top of sample.
N " Number of blows per ft. penetration with Standard Penetration Tests.

TERRESEARCH LIMITED

BOREHOLE NO. 4 TQ28SE

Contract Name Camden Town

Report No. S. 808/15 1206

Client Leo. S. Baltis Ltd.

Site Address Corner of Camden Street, and Camden Road

Engineers: Leonard & Partners. 344-360 South Lambeth Rd. London. S.W.8.

2910, 8410

Standing Water Level 25'0" 17.6.65. 25'9" 21.6.65.

Diameter 8"

Water Struck 3'6"

Method of Boring Shell/Auger

Ground Level 79.60

Start 16.6.65 Finish 16.6.65

Remarks:

Description of Strata		Thickness	Depth	Disturbed Samples	'U' Cores and 'N' P. Test
MADE	Sand bricks and stones etc.	0'9"	0'9"		
GROUND	Brown sandy clay with bricks and stones	2'9"	3'6"	J2127 2'6"	
Grey silty clay		7'0"	10'6"	B2128 5'0" J2129 7'6"	U2130 9'0"
Brown mottled clay		12'6"	23'0"	J2131 12'6" J2133 17'6" J2135 22'6"	U2132 14'0" U2134 19'0"
Grey clay		8'6"	31'6"	J3127 27'6"	U2136 24'0" U2138 30'0"
				W2139	
TOTALS		31'6"	31'6"		

Notes: 1. Descriptions are given in accordance with the B.S. Civil Engineering Code of Practice C.P.2001 "Site Investigations"

2. J indicates Jar Samples.
B " Bulk Samples.
W " Water Samples.
U " Undisturbed Core Samples. These are nominal 4 in. diam. and 18 in. long. Depths shown are top of sample.
N " Number of blows per ft. penetration with Standard Penetration Tests.

TERRESEARCH LIMITED

BOREHOLE NO. 6 TQ28 SE 1208

Contract Name: Camden Town

Report No.: S. 808/15

Client: [illegible] Ltd.

Site Address: Cornerof Camden Street, and Camden Road

Engineer: [illegible] and Partners, 344-360 South Lambeth Rd. London, S.W.8.

2913, 8411

Standing Water Level:

Diameter: 8"

Water Struck: None

Method of Boring: Shell/Auger

Ground Level: 76.27

Start: 17.6.65 Finish: 17.6.65

Remarks: 2 [illegible] breaking out concrete from ground level to 6" and pitting to 1'6".

Description of Strata		Thickness	Depth	Disturbed Samples	'U' Cores and 'N' P. Test
MADE GROUND	Concrete	0'6"	0'6"		
	Cobble stones	1'0"	1'6"		
Brown mottled silty clay		4'6"	6'0"	J3712 2'6" J3713 5'0"	
Mottled brown clay		14'0"	20'0"	J3714 7'6" J3716 12'6" J3718 17'6"	U3715 9'0" U3717 15'0" U3719 19'6"
Grey clay		4'0"	24'0"	J2720 22'6"	
Grey clay with layers of silt		7'0"	31'0"	J3722 27'6"	U3721 25'0" U3723 29'6"
	TOTALS	31'0"	31'0"		

NOTES: 1. Descriptions are given in accordance with the B.S. Civil Engineering Code of Practice C.P.2001 "Site Investigations"

2. J indicates Jar Samples.

B " Bulk Samples.

W " Water Samples.

U " Undisturbed Core Samples. These are nominal 4 in. diam. and 18 in. long. Depths shown are top of sample.

N " Number of blows per ft. penetration with Standard Penetration Tests.

Terresearch

Contract: Hawley Road, Camden
Client: Materials Science Consultants Ltd

Borehole No. 1
Sheet No. 1 Of 1.
Depth 0 to 5 metres.

Equipment and Methods
Hand Auger 100mm diameter

Orientation : Vertical

Ground Level : m.O.D.

Coordinates :

287, 843

Job Number : S91/191

Location : TQ28SE 1239

Dates : 20/11/91

Daily Prog.	Water Levels	Remarks	In Situ Tests	Samples Taken	Depth (Thick)	Reduced Level	Description	Legend
					0.00			
					0.05		MADE GROUND (tarmac)	
					0.15		MADE GROUND (concrete)	
					(0.48)		Firm greyish brown silty CLAY with scattered gravel traces	
				J 12				
				J 13	0.63			
				U 14			Firm to stiff brown slightly silty CLAY with occasional blue-grey reduction zones and traces of selenite crystals	
				J 15				
					(2.37)			
				U 16				
				J 17				
				J 18				
				U 19				
20/11					3.00		End of Borehole	

Operator
NF

Scale
5m/sheet

General Remarks:

Appendix
1

Sheet No.
1

Terresearch Limited, Taywood Road, Northolt, Middlesex UB5 6RL. Tel: 081 575 4901 Fax: 081 575 4462

Terresearch

Contract: Hawley Road, Camden Client: Materials Science Consultants Ltd		Borehole No 2 Sheet No. 1 Of 1. Depth 0 to 5 metres.
Equipment and Methods Hand Auger 100mm diameter Orientation : Vertical	Ground Level : m.O.D. Coordinates : 287,483	Job Number : S91/191 Location : TQ28SE 1240 Dates : 20/11/91

Daily Prog.	Water Levels	Remarks	In Situ Tests	Samples Taken	Depth (Thick)	Reduced Level	Description	Legend
					0.00			
					0.05		MADE GROUND (tarmac)	
					0.15		MADE GROUND (concrete)	
					(0.35)		Firm greyish brown silty CLAY	
				J 20	0.50			
							Firm brown silty CLAY with frequent blue-grey reduction zones, occasional pockets of orange-brown sandy clay and traces of selenite crystals becoming more abundant with depth	
				J 21				
				U*				
				U 22				
				J 23				
					(2.50)			
				U 24				
				J 25				
				J 26				
				U 27				
20/11					3.00		End of Borehole	

Operator NF	General Remarks:	Appendix 1
Scale 5m/sheet		Sheet No. 2

Terresearch Limited, Taywood Road, Northolt, Middlesex UB5 6RL. Tel: 081 575 4901 Fax: 081 575 4462

Contract: Hawley Road, Camden
Client: Materials Science Consultants Ltd

Borehole No 3
Sheet No. 1 Of 1.
Depth 0 to 5 metres.

Equipment and Methods
Hand Auger 100mm diameter

Orientation : Vertical

Ground Level : m.O.D.

Coordinates :

287, 843

Job Number : S91/191
TQ21SE
Location : 1241
Dates : 19/11/91

Daily Prog.	Water Levels	Remarks	In Situ Tests	Samples Taken	Depth (Thick)	Reduced Level	Description	Legend
					0.00			
					0.05		MADE GROUND (tarmac)	
				J 1	0.15		MADE GROUND (concrete)	
				J 2	(0.45) 0.60		MADE GROUND (dark grey clayey sand with bricks and stones)	
	19/11			J 3, W 11	(0.20) 0.80		MADE GROUND (ash with bricks and stones)	
19/11				J 4, U 5, J 6, U 7, J 8, J 9, U 10	(2.20) 3.00		Firm brown silty CLAY with occasional blue-grey reduction zones	
							End of Borehole	

General Remarks:

Operator
NF

Scale
5m/sheet

Appendix
1

Sheet No.
3

Terresearch

Contract: Hawley Road, Camden Client: Materials Science Consultants Ltd	Borehole No. 4 Sheet No. 1 Of 1. Depth 0 to 5 metres.
Equipment and Methods: Hand Auger 100mm diameter Orientation : Vertical	Ground Level : m.O.D. Coordinates : 287,843
	Job Number : S91/191 Location : TQ28SE 1242 Dates : 20/11/91

Daily Prog.	Water Levels	Remarks	In Situ Tests	Samples Taken	Depth (Thick)	Reduced Level	Description	Legend
					0.00			
					0.05		MADE GROUND (tarmac)	
					0.15		MADE GROUND (concrete)	
				J 28	(0.45)		MADE GROUND (soft silty sandy brown clay with occasional gravel and brick traces)	
				J 29	0.60			
				U 30 J 31	(0.55)		Soft to firm dark brownish grey silty CLAY with organic traces	
					1.15			
				J 32 U 33 J 34 J 35 U 36	(1.85)		Firm to stiff brown silty CLAY with some blue-grey reduction zones and occasional organic traces	
20/11					3.00		End of Borehole	

Operator NF	General Remarks:	Appendix 1
Scale 5m/sheet		Sheet No. 4

RECORD of WELL or BORING

2 398 C16

Survey No. 256
1" N.S.
1" O.S.

at (or near) Camden Road B.N.W.1.
Town Camden Town NW St. Pancras County London
Six-inch map n 5 NW

Exact site (unless a tracing from a map is supplied, give distance and direction from parish church, cross-roads, or other object shown on maps). A little S the SW from Camden Town Stn. vide sheet 16 Vol I
Popular edition (Sheet of one-inch map. (Square) TQ 28/50

Surface level of ground c 65 85 ft. above Ordnance Datum. Well or Bore commenced at ... ft. below surface level of ground.
Sunk 4 ft., diameter 4'x4' ft. Bored ... ft.; diameter of boring: at top ... in., at bottom ... in.
Details of lining tubes (internal diameters preferred) 34'2" of 16 in. Top. 3'4" b/o
197'6" .. 12 .. .. 2'6" ..

Water struck at depths of (feet) 301', 315', 333'. NGR TQ 2902 8412
Rest-level of water ~~above~~ below top of well or bore 278 ft. Pumping level ?278 ft. Time of recovery ... hours.
Suction at 598 ft. depth. Yield: (i) on test 7000/8000 galls. per hour, (ii) normal ... galls. per ...
Quality (attach copy of analysis if available) Standards Perm. 0° Temp 4° Total 4°
Made by LE GRAND, SUTCLIFF & GELL, LD. for Aerated Bread Co. Ltd Date of boring Aug/Oct, 1934
Information from LE GRAND, SUTCLIFF & GELL, LD. S134/p. 673.

(For Survey use only). GEOLOGICAL CLASSIFICATION.	NATURE OF STRATA. (and any additional remarks)	THICKNESS. Feet.	THICKNESS. Inches.	DEPTH. Feet.	DEPTH. Inches.
Made 2	Made ground	2	.	2	.
L.C. 106	Brown clay	20	.	22	.
	Blue clay	28	.	50	.
	Blue clay & stone	25	.	75	.
	Blue clay	33	.	108	.
W.R.B. 50	Mottled clay	39	.	147	.
	Conglomerate	6	.	153	.
	Green loamy sand	5	.	158	.
T.S. 20	Thanet Sand	19	.	177	.
	Green flints	1	.	178	.
CK. 474	Chalk & flints	232	.	410	.
	Hard grey Chalk	242	.	652	.

DB
26:10:35.

Site visited 30th July 1946.
Pumping controlled by demand &
Well top in basement 10' below ground level.

Confidential Water very soft. - have to harden it.

2 July 1946

~~Rest L.~~ P.W.L. 300 Yield 10,328 Nov. 1937

For Survey use only.

Date received.	G.S.M.	M. of H. notified.	Site marked on 1" map.
Oct 1935.			

(11969B) Wt 10256/0175 2,500 9/32 H, J, R & L, Ld

APPENDIX C

WALLAP output

Units: kN,m

INPUT DATA

SOIL PROFILE

Stratum no.	Elevation of top of stratum	Soil types: Active side	Soil types: Passive side
1	26.00	1 Made Ground	1 Made Ground
2	24.00	2 London Clay	2 London Clay

SOIL PROPERTIES

No.	Soil type Description (Datum elev.)	Bulk density kN/m3	Young's Modulus Eh,kN/m2 (dEh/dy)	At rest coeff. Ko (dKo/dy)	Consol state. NC/OC (Nu)	Active limit Ka (Kac)	Passive limit Kp (Kpc)	Cohesion kN/m2 (dc/dy)
1	Made Ground	18.00	18000	0.561	NC	1.000	1.000	30.00u
					(0.490)	(2.389)	(2.390)	
2	London Clay	20.00	50000	1.000	OC	1.000	1.000	50.00u
	(24.00)		(6000)		(0.490)	(2.475)	(2.476)	(6.000)
3	London Cl..	20.00	37500	0.593	OC	0.367	3.241	5.000d
	(24.00)		(4500)		(0.200)	(1.423)	(5.034)	
4	Made Ground DR	18.00	13500	0.561	NC	0.346	3.442	30.00d
					(0.200)	(1.340)	(5.007)	

Additional soil parameters associated with Ka and Kp

No.	Soil type Description	Ka: Soil friction angle	Ka: Wall adhesion coeff.	Ka: Back-fill angle	Kp: Soil friction angle	Kp: Wall adhesion coeff.	Kp: Back-fill angle
1	Made Ground	0.00	0.500	0.00	0.00	0.500	0.00
2	London Clay	0.00	0.667	0.00	0.00	0.667	0.00
3	London Clay DR	24.00	0.667	0.00	24.00	0.667	0.00
4	Made Ground DR	26.00	0.500	0.00	26.00	0.500	0.00

GROUND WATER CONDITIONS

Density of water = 10.00 kN/m3

	Active side	Passive side
Initial water table elevation	25.00	25.00

Automatic water pressure balancing at toe of wall : No

Water press. profile no.	Active: Point no.	Active: Elev. m	Active: Piezo elev. m	Active: Water press. kN/m2	Passive: Point no.	Passive: Elev. m	Passive: Piezo elev. m	Passive: Water press. kN/m2	
1	1	25.00	25.00	0.0	1	21.50	21.50	0.0	MC+WC

WALL PROPERTIES

Type of structure = Fully Embedded Wall
Elevation of toe of wall = 19.00
Maximum finite element length = 0.40 m
Youngs modulus of wall E = 3.0000E+07 kN/m2
Moment of inertia of wall I = 8.4823E-03 m4/m run
E.I = 254469 kN.m2/m run
Yield Moment of wall = Not defined

STRUTS and ANCHORS

Strut/ anchor no.	Elev.	Strut spacing m	X-section area of strut sq.m	Youngs modulus kN/m2	Free length m	Inclin -ation (degs)	Pre-stress /strut kN	Tension allowed
1	25.80	1.00	0.300000	3.000E+07	30.00	0.00	0	Yes
2	22.00	1.00	0.300000	3.000E+07	30.00	0.00	0	Yes
3	25.50	5.00	0.010000	2.000E+08	20.00	0.00	0	No

SURCHARGE LOADS

Surch -arge no.	Elev.	Distance from wall	Length parallel to wall	Width perpend. to wall	Surcharge kN/m2 Near edge	Surcharge kN/m2 Far edge	Equiv. soil type	Partial factor/ Category
1	26.00	0.00(A)	30.00	3.00	10.00	=	N/A	1.00 -
2	26.00	3.00(A)	30.00	10.00	20.00	=	N/A	1.00 -

Note: A = Active side, P = Passive side
Limit State Categories P/U = Permanent Unfavourable
P/F = Permanent Favourable
Var = Variable (unfavourable)

CONSTRUCTION STAGES

Construction stage no.	Stage description
1	Apply surcharge no.1 at elevation 26.00
2	Apply surcharge no.2 at elevation 26.00 No analysis at this stage
3	Change EI of wall to 254469 kN.m2/m run Yield moment not defined Reset wall displacements to zero at this stage
4	Apply water pressure profile no.1 (Mod. Conserv.)
5	Excavate to elevation 21.50 on PASSIVE side
6	Install strut or anchor no.2 at elevation 22.00
7	Install strut or anchor no.1 at elevation 25.80
8	Change properties of soil type 2 to soil type 3 Ko pressures will not be reset
9	Change properties of soil type 1 to soil type 4 Ko pressures will not be reset

FACTORS OF SAFETY and ANALYSIS OPTIONS

Limit State options: Serviceability Limit State
All loads and soil strengths are unfactored

Stability analysis:
Method of analysis - Strength Factor method
Factor on soil strength for calculating wall depth = 1.25

Parameters for undrained strata:
Minimum equivalent fluid density = 5.00 kN/m3
Maximum depth of water filled tension crack = 0.00 m

Bending moment and displacement calculation:
Method - Subgrade reaction model using Influence Coefficients
Open Tension Crack analysis? - No
Non-linear Modulus Parameter (L) = 0 m

Boundary conditions:
Length of wall (normal to plane of analysis) = 1000.00 m

Width of excavation on active side of wall = 20.00 m
Width of excavation on passive side of wall = 20.00 m

Distance to rigid boundary on active side = 20.00 m
Distance to rigid boundary on passive side = 20.00 m

OUTPUT OPTIONS

Stage no.	Stage description	Displacement Bending mom. Shear force	Active, Passive pressures	Graph. output
1	Apply surcharge no.1 at elev. 26.00	Yes	Yes	Yes
2	Apply surcharge no.2 at elev. 26.00	No	No	No
3	Change EI of wall to 254469kN.m2/m run	No	No	No
4	Apply water pressure profile no.1	No	No	No
5	Excav. to elev. 21.50 on PASSIVE side	No	No	No
6	Install strut no.2 at elev. 22.00	No	No	No
7	Install strut no.1 at elev. 25.80	No	No	No
8	Change soil type 2 to soil type 3	No	No	No
9	Change soil type 1 to soil type 4	No	No	No
*	Summary output	Yes	-	Yes

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CARD GEOTECHNICS LIMITED                                      | Sheet No.
Program: WALLAP  Version 6.05  Revision A41.B56.R46           | Job No.  18067a
                             Licensed from GEOSOLVE           | Made by :   JMS
Data filename/Run ID: Block D - KTR_SLS                       |
Camden Lock                                                   | Date:20-01-2015
Block D - KTR                                                 | Checked :
-----------------------------------------------------------------------------
                                                 Units: kN,m
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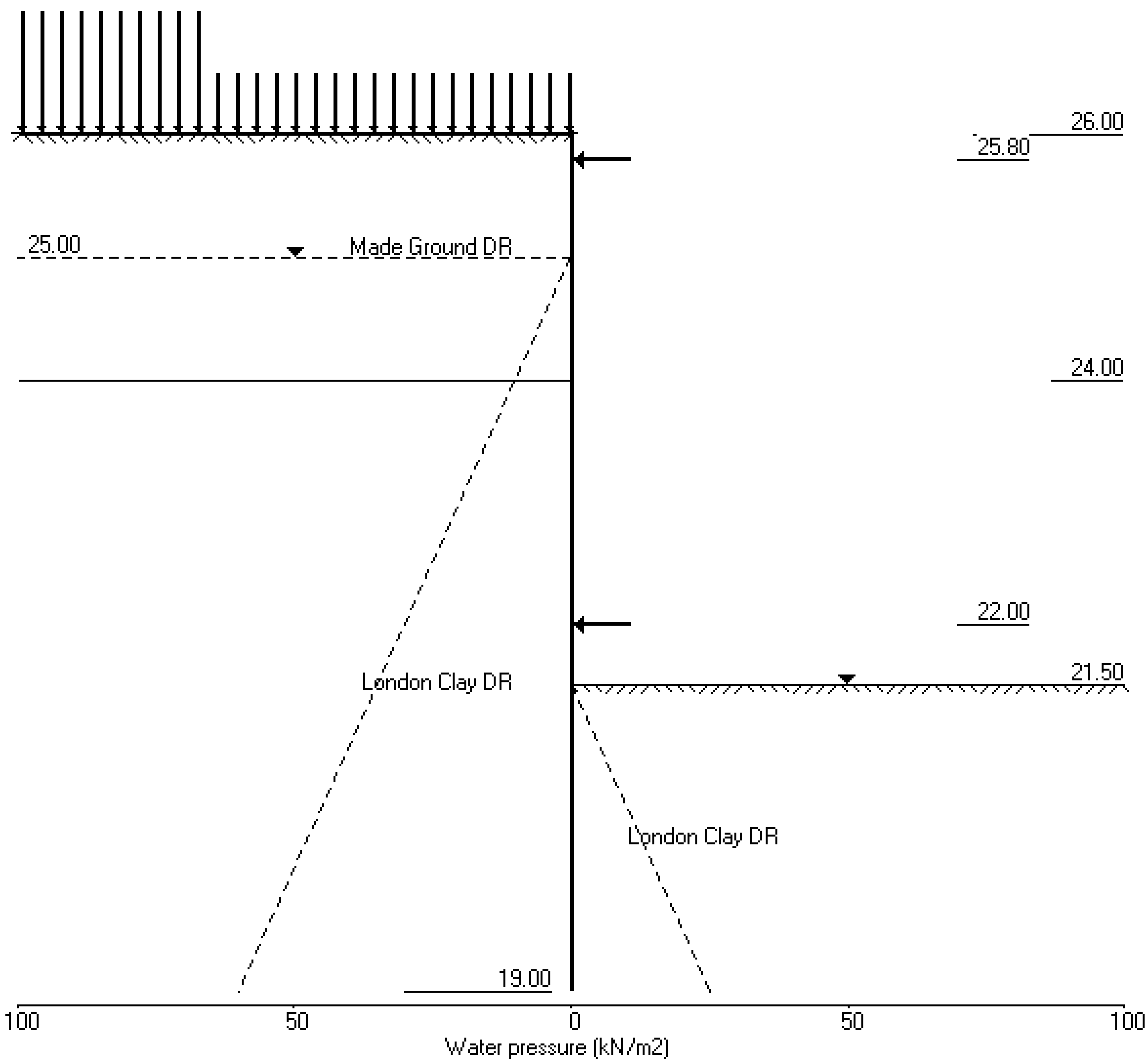
Stage No.9   Change soil type 1 to soil type 4



Units: kN,m

Stage No. 9 Change properties of soil type 1 to soil type 4
Ko pressures will not be reset

STABILITY ANALYSIS of Fully Embedded Wall according to Strength Factor method
Factor of safety on soil strength

Stage No.	--- G.L. --- Act.	Pass.	Strut Elev.	FoS for toe elev. = 19.00: Factor of Safety	Moment equilib. at elev.	Toe elev. for FoS = 1.250: Toe elev.	Wall Penetr -ation
9	26.00	21.50		More than one strut			

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall
Analysis options
Length of wall perpendicular to section = 1000.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Active side 20.00 from wall
Passive side 20.00 from wall

*** Wall displacements reset to zero at stage 3

Node no.	Y coord	Nett pressure kN/m2	Wall disp. m	Wall rotation rad.	Shear force kN/m	Bending moment kN.m/m	Strut forces kN/m
1	26.00	0.91	0.015	2.28E-03	0.0	-0.0	
2	25.80	1.57	0.015	2.28E-03	0.2	0.0	-6.1
		1.57	0.015	2.28E-03	6.3	0.0	
3	25.40	3.00	0.014	2.28E-03	7.3	2.7	
4	25.00	5.00	0.013	2.27E-03	8.9	5.9	
5	24.70	6.50	0.012	2.27E-03	10.6	8.8	
6	24.40	8.00	0.011	2.26E-03	12.8	12.3	
7	24.00	10.00	0.010	2.24E-03	16.4	18.1	
		16.36	0.010	2.24E-03	16.4	18.1	
8	23.60	21.96	0.010	2.21E-03	24.0	26.1	
9	23.20	27.56	0.009	2.17E-03	33.9	37.6	
10	22.80	33.17	0.008	2.11E-03	46.1	53.5	
11	22.40	38.76	0.007	2.03E-03	60.5	74.7	
12	22.00	44.35	0.006	1.90E-03	77.1	102.1	140.8
		44.35	0.006	1.90E-03	-63.7	102.1	
13	21.75	47.83	0.006	1.81E-03	-52.1	87.6	
14	21.50	51.31	0.005	1.74E-03	-39.8	76.1	
		26.14	0.005	1.74E-03	-39.8	76.1	
15	21.15	16.15	0.005	1.65E-03	-32.4	64.5	
16	20.80	6.15	0.004	1.57E-03	-28.5	53.3	
17	20.40	-5.30	0.004	1.50E-03	-28.3	41.2	
18	20.00	-8.41	0.003	1.45E-03	-31.0	28.7	
19	19.60	4.41	0.002	1.42E-03	-31.8	15.4	
20	19.30	31.08	0.002	1.41E-03	-26.5	6.1	
21	19.00	145.56	0.002	1.40E-03	-0.0	-0.0	

Strut force at elev. 25.80 = -6.10 kN/m run = -6.10 kN/strut
Strut force at elev. 22.00 = 140.75 kN/m run = 140.75 kN/strut

(continued)

Stage No.9 Change properties of soil type 1 to soil type 4
Ko pressures will not be reset

ACTIVE side

Node no.	Y coord	Water press. kN/m2	Effective stresses: Vertic -al kN/m2	Effective stresses: Active limit kN/m2	Effective stresses: Passive limit kN/m2	Effective stresses: Earth pressure kN/m2	Total earth pressure kN/m2	Soil stiffness coeff. kN/m3
1	26.00	0.00	10.00	0.00	184.64	0.91	0.91	2746
2	25.80	0.00	13.60	0.00	197.04	1.57	1.57	2746
3	25.40	0.00	20.83	0.00	221.92	3.00	3.00	2746
4	25.00	0.00	28.13	0.00	247.06	5.00	5.00	2746
5	24.70	3.00	30.67	0.00	255.79	3.50	6.50	2746
6	24.40	6.00	33.25	0.00	264.69	2.00	8.00	2746
7	24.00	10.00	36.76	0.00	276.77	0.00	10.00a	2746
		10.00	36.76	6.36	144.33	6.36	16.36a	7628
8	23.60	14.00	41.12	7.96	158.45	7.96	21.96a	7994
9	23.20	18.00	45.49	9.56	172.63	9.56	27.56a	8361
10	22.80	22.00	49.86	11.17	186.80	11.17	33.17a	8727
11	22.40	26.00	54.22	12.76	200.91	12.76	38.76a	14104
12	22.00	30.00	58.54	14.35	214.91	14.35	44.35a	14672
13	21.75	32.50	61.22	15.33	223.61	15.33	47.83a	15027
14	21.50	35.00	63.89	16.31	232.26	16.31	51.31a	15382
15	21.15	38.50	67.60	17.67	244.28	17.67	56.17a	15879
16	20.80	42.00	71.28	19.02	256.21	19.02	61.02a	16376
17	20.40	46.00	75.44	20.55	269.72	20.55	66.55a	16944
18	20.00	50.00	79.57	22.06	283.10	30.43	80.43	17512
19	19.60	54.00	83.67	23.56	296.37	45.49	99.49	18080
20	19.30	57.00	86.72	24.68	306.26	57.12	114.12	18506
21	19.00	60.00	89.75	25.79	316.08	126.44	186.44	18932

PASSIVE side

Node no.	Y coord	Water press. kN/m2	Effective stresses: Vertic -al kN/m2	Effective stresses: Active limit kN/m2	Effective stresses: Passive limit kN/m2	Effective stresses: Earth pressure kN/m2	Total earth pressure kN/m2	Soil stiffness coeff. kN/m3
1	26.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
2	25.80	0.00	0.00	0.00	0.00	0.00	0.00	0.0
3	25.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
4	25.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
5	24.70	0.00	0.00	0.00	0.00	0.00	0.00	0.0
6	24.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
7	24.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
8	23.60	0.00	0.00	0.00	0.00	0.00	0.00	0.0
9	23.20	0.00	0.00	0.00	0.00	0.00	0.00	0.0
10	22.80	0.00	0.00	0.00	0.00	0.00	0.00	0.0
11	22.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
12	22.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
13	21.75	0.00	0.00	0.00	0.00	0.00	0.00	0.0
14	21.50	0.00	0.00	0.00	0.00	0.00	0.00	0.0
		0.00	0.00	0.00	25.17	25.17	25.17p	15382
15	21.15	3.50	3.50	0.00	36.52	36.52	40.02p	15879
16	20.80	7.00	7.00	0.00	47.87	47.87	54.87p	16376
17	20.40	11.00	11.01	0.00	60.85	60.85	71.85p	16944
18	20.00	15.00	15.02	0.00	73.84	73.84	88.84p	17512
19	19.60	19.00	19.03	0.00	86.86	76.08	95.08	18080
20	19.30	22.00	22.05	0.97	96.64	61.04	83.04	18506
21	19.00	25.00	25.07	2.08	106.43	15.87	40.87	18932

Note: 66.55a Soil pressure at active limit
88.84p Soil pressure at passive limit

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CARD GEOTECHNICS LIMITED                                      | Sheet No.
Program: WALLAP  Version 6.05  Revision A41.B56.R46           | Job No.  18067a
                             Licensed from GEOSOLVE           | Made by :   JMS
Data filename/Run ID: Block D - KTR_SLS                       |
Camden Lock                                                   | Date:20-01-2015
Block D - KTR                                                 | Checked :
----------------------------------------------------------------------------
                                                       Units: kN,m
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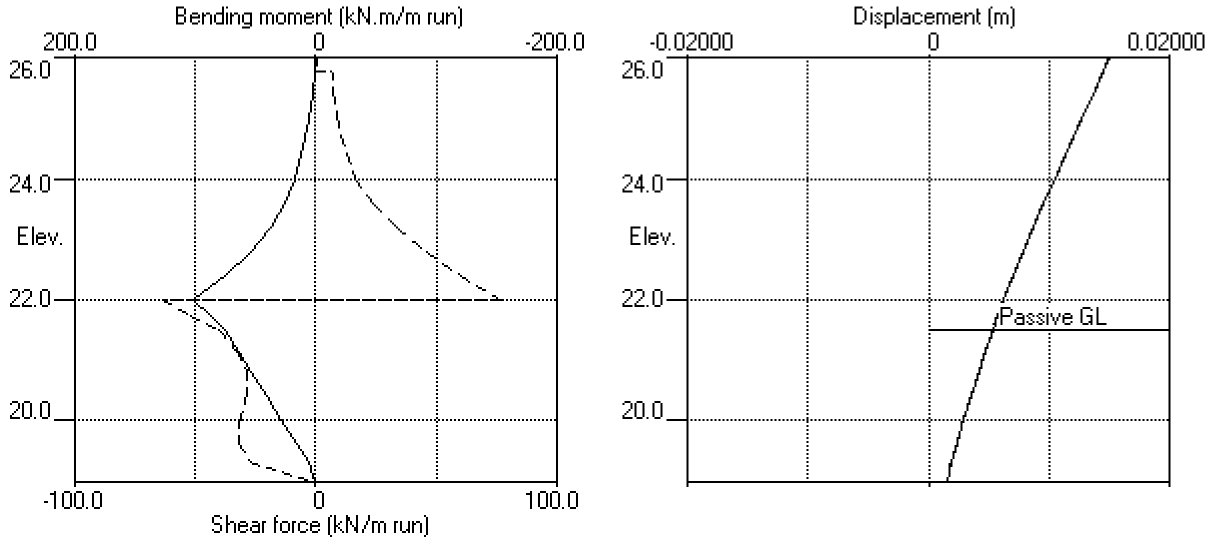
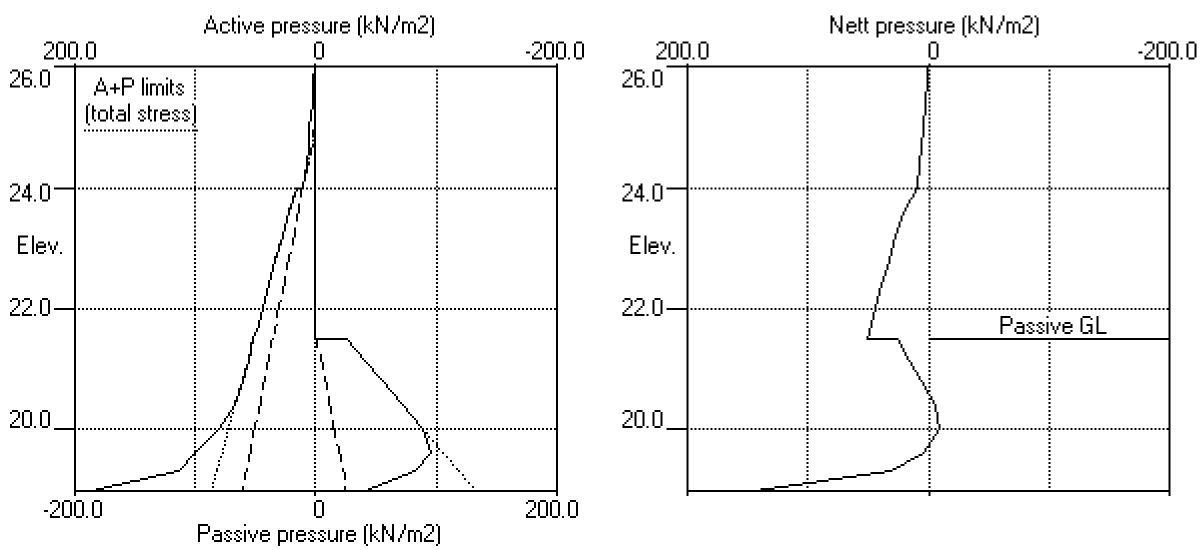
Stage No.9 Change soil type 1 to soil type 4



Stage No.9 Change soil type 1 to soil type 4



Units: kN,m

Summary of results

LIMIT STATE PARAMETERS

Limit State: Serviceability Limit State

All loads and soil strengths are unfactored

STABILITY ANALYSIS of Fully Embedded Wall according to Strength Factor method

Factor of safety on soil strength

Stage No.	G.L. Act.	G.L. Pass.	Strut Elev.	FoS for toe elev. = 19.00: Factor of Safety	FoS for toe elev. = 19.00: Moment equilib. at elev.	Toe elev. for FoS = 1.250: Toe elev.	Toe elev. for FoS = 1.250: Wall Penetr -ation
1	26.00	26.00	Cant.	Conditions not suitable for FoS calc.			
2	26.00	26.00		No analysis at this stage			
3	26.00	26.00		No analysis at this stage			
4	26.00	26.00	Cant.	Conditions not suitable for FoS calc.			
5	26.00	21.50	Cant.	1.611	19.57	19.34	2.16
6	26.00	21.50		No analysis at this stage			

All remaining stages have more than one strut - FoS calculation n/a

Units: kN,m

Summary of results

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

Length of wall perpendicular to section = 1000.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Active side 20.00 from wall
Passive side 20.00 from wall

Bending moment, shear force and displacement envelopes

Node no.	Y coord	Displacement maximum m	Displacement minimum m	Bending moment maximum kN.m/m	Bending moment minimum kN.m/m	Shear force maximum kN/m	Shear force minimum kN/m
1	26.00	0.015	0.000	0.0	-0.0	0.0	0.0
2	25.80	0.015	0.000	0.0	0.0	6.3	0.0
3	25.40	0.014	0.000	2.7	0.0	7.3	0.0
4	25.00	0.013	0.000	5.9	0.0	8.9	0.0
5	24.70	0.012	0.000	8.8	0.0	10.6	0.0
6	24.40	0.011	0.000	12.3	0.0	12.8	0.0
7	24.00	0.010	0.000	18.1	0.0	16.4	0.0
8	23.60	0.010	0.000	26.1	0.0	24.0	0.0
9	23.20	0.009	0.000	37.6	0.0	33.9	0.0
10	22.80	0.008	0.000	53.5	0.0	46.1	-0.3
11	22.40	0.007	0.000	74.7	0.0	60.5	-1.4
12	22.00	0.006	0.000	102.1	0.0	77.1	-63.7
13	21.75	0.006	0.000	87.6	0.0	45.2	-52.1
14	21.50	0.005	0.000	76.1	0.0	50.6	-39.8
15	21.15	0.005	0.000	86.7	0.0	8.3	-32.4
16	20.80	0.004	0.000	83.0	0.0	0.0	-28.5
17	20.40	0.004	0.000	67.3	0.0	0.0	-47.6
18	20.00	0.003	0.000	45.0	0.0	0.0	-56.8
19	19.60	0.002	0.000	21.9	0.0	0.0	-51.5
20	19.30	0.002	0.000	7.9	0.0	0.0	-37.6
21	19.00	0.002	0.000	0.0	-0.0	0.0	-0.0

Maximum and minimum bending moment and shear force at each stage

Stage no.	Bending moment maximum kN.m/m	elev.	Bending moment minimum kN.m/m	elev.	Shear force maximum kN/m	elev.	Shear force minimum kN/m	elev.
1	7.9	22.80	0.0	26.00	5.7	24.00	-3.1	20.80
2	No calculation at this stage							
3	No calculation at this stage							
4	6.9	22.80	0.0	26.00	5.5	24.00	-2.6	21.15
5	86.7	21.15	-0.0	26.00	50.6	21.50	-56.8	20.00
6	No calculation at this stage							
7	No calculation at this stage							
8	102.1	22.00	-0.0	26.00	77.1	22.00	-63.7	22.00
9	102.1	22.00	-0.0	26.00	77.1	22.00	-63.7	22.00

Summary of results (continued)

Maximum and minimum displacement at each stage

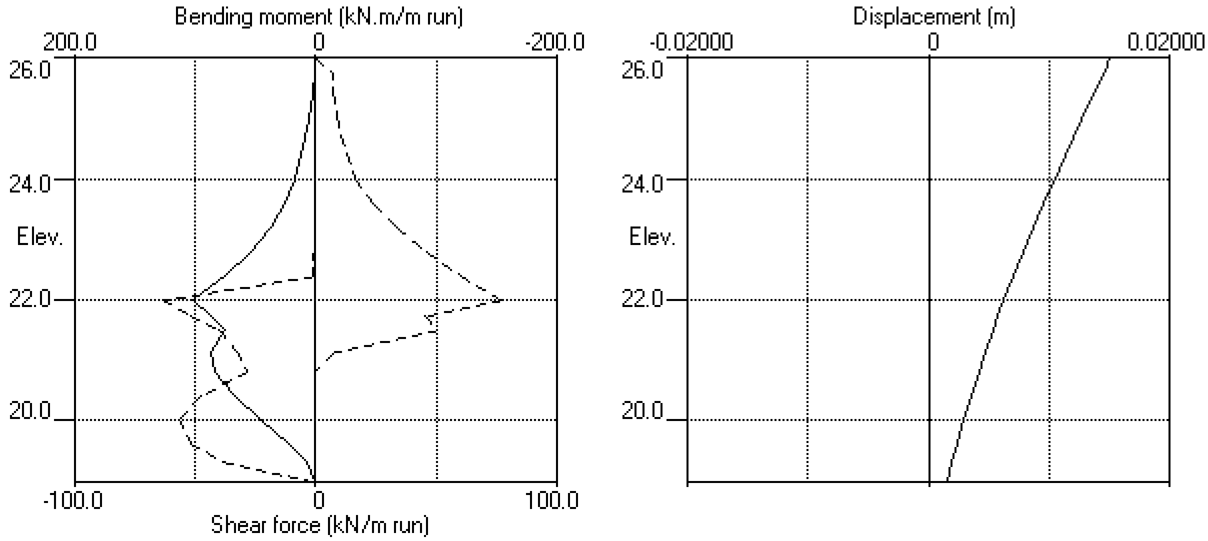
Stage no.	Displacement maximum (m)	elev.	minimum (m)	elev.	Stage description
1	0.001	26.00	0.000	26.00	Apply surcharge no.1 at elev. 26.00
2	No calculation at this stage				Apply surcharge no.2 at elev. 26.00
3	Wall displacements reset to zero				Change EI of wall to 254469kN.m2/m run
4	0.000	19.00	0.000	26.00	Apply water pressure profile no.1
5	0.015	26.00	0.000	26.00	Excav. to elev. 21.50 on PASSIVE side
6	No calculation at this stage				Install strut no.2 at elev. 22.00
7	No calculation at this stage				Install strut no.1 at elev. 25.80
8	0.015	26.00	0.000	26.00	Change soil type 2 to soil type 3
9	0.015	26.00	0.000	26.00	Change soil type 1 to soil type 4

Strut forces at each stage (horizontal components)

Stage no.	Strut no. 1 at elev. 25.80 kN/m run	kN/strut	Strut no. 2 at elev. 22.00 kN/m run	kN/strut
8	-6.10	-6.10	140.75	140.75
9	-6.10	-6.10	140.75	140.75

Units: kN,m

Bending moment, shear force, displacement envelopes



Units: kN,m

INPUT DATA

SOIL PROFILE

Stratum no.	Elevation of top of stratum	Soil types: Active side	Soil types: Passive side
1	26.00	1 Made Ground	1 Made Ground
2	24.00	2 London Clay	2 London Clay

SOIL PROPERTIES

-- Soil type -- No. Description (Datum elev.)	Bulk density kN/m3	Young's Modulus Eh,kN/m2 (dEh/dy)	At rest coeff. Ko (dKo/dy)	Consol state. NC/OC (Nu)	Active limit Ka (Kac)	Passive limit Kp (Kpc)	Cohesion kN/m2 (dc/dy)
1 Made Ground	18.00	18000	0.561	NC	1.000	1.000	30.00u
				(0.490)	(2.389)	(2.390)	
2 London Clay	20.00	50000	1.000	OC	1.000	1.000	50.00u
(24.00)		(6000)		(0.490)	(2.475)	(2.476)	(6.000)
3 London Cl..	20.00	37500	0.593	OC	0.367	3.241	5.000d
(24.00)		(4500)		(0.200)	(1.423)	(5.034)	
4 Made Ground	18.00	13500	0.561	NC	0.346	3.442	30.00d
DR				(0.200)	(1.340)	(5.007)	

Additional soil parameters associated with Ka and Kp

------- Soil type ------- No. Description	Ka: Soil friction angle	Ka: Wall adhesion coeff.	Ka: Back-fill angle	Kp: Soil friction angle	Kp: Wall adhesion coeff.	Kp: Back-fill angle
1 Made Ground	0.00	0.500	0.00	0.00	0.500	0.00
2 London Clay	0.00	0.667	0.00	0.00	0.667	0.00
3 London Clay DR	24.00	0.667	0.00	24.00	0.667	0.00
4 Made Ground DR	26.00	0.500	0.00	26.00	0.500	0.00

GROUND WATER CONDITIONS

Density of water = 10.00 kN/m3

	Active side	Passive side
Initial water table elevation	25.00	25.00

Automatic water pressure balancing at toe of wall : No

Water press. profile no.	Active: Point no.	Active: Elev. m	Active: Piezo elev. m	Active: Water press. kN/m2	Passive: Point no.	Passive: Elev. m	Passive: Piezo elev. m	Passive: Water press. kN/m2	
1	1	25.00	25.00	0.0	1	21.50	21.50	0.0	MC+WC

WALL PROPERTIES

Type of structure = Fully Embedded Wall
Elevation of toe of wall = 19.00
Maximum finite element length = 0.40 m
Youngs modulus of wall E = 3.0000E+07 kN/m2
Moment of inertia of wall I = 8.4823E-03 m4/m run
E.I = 254469 kN.m2/m run
Yield Moment of wall = Not defined

STRUTS and ANCHORS

Strut/ anchor no.	Elev.	Strut spacing m	X-section area of strut sq.m	Youngs modulus kN/m2	Free length m	Inclin -ation (degs)	Pre-stress /strut kN	Tension allowed
1	25.80	1.00	0.300000	3.000E+07	30.00	0.00	0	Yes
2	22.00	1.00	0.300000	3.000E+07	30.00	0.00	0	Yes
3	25.50	5.00	0.010000	2.000E+08	20.00	0.00	0	No

SURCHARGE LOADS

Surch -arge no.	Elev.	Distance from wall	Length parallel to wall	Width perpend. to wall	Surcharge kN/m2 Near edge	Surcharge kN/m2 Far edge	Equiv. soil type	Partial factor/ Category
1	26.00	0.00(A)	30.00	3.00	10.00	=	N/A	1.00 -

Note: A = Active side, P = Passive side
Limit State Categories P/U = Permanent Unfavourable
P/F = Permanent Favourable
Var = Variable (unfavourable)

CONSTRUCTION STAGES

Construction stage no.	Stage description
1	Apply surcharge no.1 at elevation 26.00
2	Excavate to elevation 26.00 on ACTIVE side Toe of berm at elevation 24.00 Width of top of berm = 3.00 Width of toe of berm = 3.01
3	Change EI of wall to 254469 kN.m2/m run Yield moment not defined Reset wall displacements to zero at this stage
4	Apply water pressure profile no.1 (Mod. Conserv.)
5	Excavate to elevation 24.50 on PASSIVE side Toe of berm at elevation 21.50 Width of top of berm = 1.00 Width of toe of berm = 4.00
6	Install strut or anchor no.3 at elevation 25.50
7	Excavate to elevation 21.50 on PASSIVE side
8	Install strut or anchor no.2 at elevation 22.00
9	Install strut or anchor no.1 at elevation 25.80
10	Remove strut or anchor no.3 at elevation 25.50
11	Change properties of soil type 2 to soil type 3 Ko pressures will not be reset
12	Change properties of soil type 1 to soil type 4 Ko pressures will not be reset

FACTORS OF SAFETY and ANALYSIS OPTIONS

Limit State options: Serviceability Limit State
All loads and soil strengths are unfactored

Stability analysis:
Method of analysis - Strength Factor method
Factor on soil strength for calculating wall depth = 1.25

Parameters for undrained strata:
Minimum equivalent fluid density = 5.00 kN/m3
Maximum depth of water filled tension crack = 0.00 m

Bending moment and displacement calculation:
Method - Subgrade reaction model using Influence Coefficients
Open Tension Crack analysis? - No
Non-linear Modulus Parameter (L) = 0 m

Boundary conditions:
Length of wall (normal to plane of analysis) = 1000.00 m

Width of excavation on active side of wall = 20.00 m
Width of excavation on passive side of wall = 20.00 m

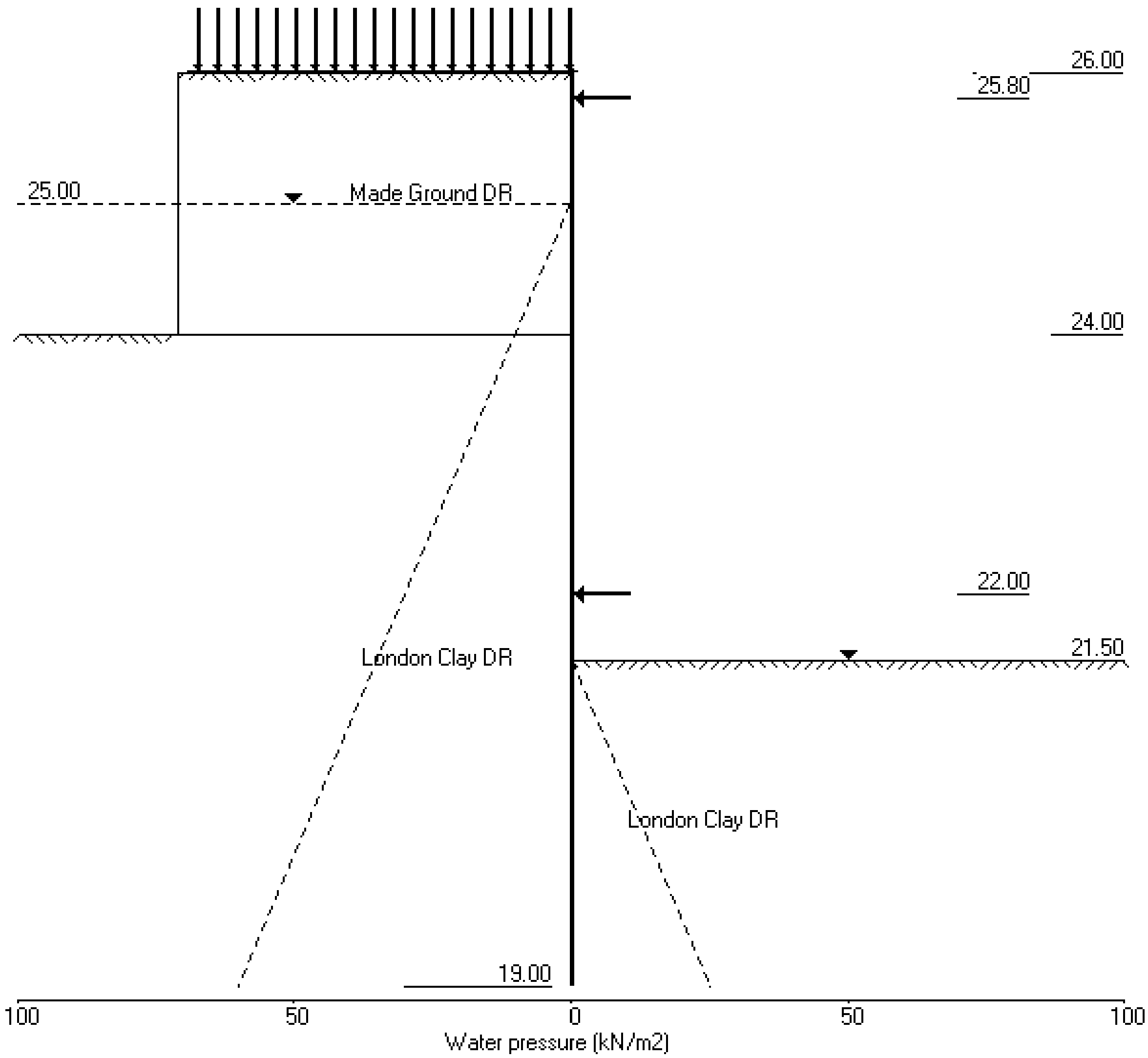
Distance to rigid boundary on active side = 20.00 m
Distance to rigid boundary on passive side = 20.00 m

OUTPUT OPTIONS

Stage no.	Stage description	Output options: Displacement Bending mom. Shear force	Active, Passive pressures	Graph. output
1	Apply surcharge no.1 at elev. 26.00	Yes	Yes	Yes
2	Excav. to elev. 26.00 on ACTIVE side	No	No	No
3	Change EI of wall to 254469kN.m2/m run	No	No	No
4	Apply water pressure profile no.1	No	No	No
5	Excav. to elev. 24.50 on PASSIVE side	No	No	No
6	Install strut no.3 at elev. 25.50	No	No	No
7	Excav. to elev. 21.50 on PASSIVE side	No	No	No
8	Install strut no.2 at elev. 22.00	No	No	No
9	Install strut no.1 at elev. 25.80	No	No	No
10	Remove strut no.3 at elev. 25.50	No	No	No
11	Change soil type 2 to soil type 3	No	No	No
12	Change soil type 1 to soil type 4	No	No	No
*	Summary output	Yes	-	Yes

Units: kN,m

Stage No.12 Change soil type 1 to soil type 4



Units: kN,m

Stage No. 12 Change properties of soil type 1 to soil type 4
Ko pressures will not be reset

STABILITY ANALYSIS of Fully Embedded Wall according to Strength Factor method
Factor of safety on soil strength

Stage No.	--- G.L. --- Act.	Pass.	Strut Elev.	FoS for toe elev. = 19.00: Factor of Safety	Moment equilib. at elev.	Toe elev. for FoS = 1.250: Toe elev.	Wall Penetr -ation
12	26.00	21.50		More than one strut			

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

Length of wall perpendicular to section = 1000.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Active side 20.00 from wall
Passive side 20.00 from wall

*** Wall displacements reset to zero at stage 3

Node no.	Y coord	Nett pressure kN/m2	Wall disp. m	Wall rotation rad.	Shear force kN/m	Bending moment kN.m/m	Strut forces kN/m
1	26.00	0.40	0.004	-3.35E-04	0.0	-0.0	
2	25.80	1.00	0.004	-3.35E-04	0.1	0.0	28.7
		1.00	0.004	-3.35E-04	-28.5	0.0	
3	25.50	2.50	0.004	-3.30E-04	-28.0	-8.4	
4	25.25	3.75	0.004	-3.18E-04	-27.2	-15.3	
5	25.00	5.00	0.004	-2.99E-04	-26.1	-22.0	
6	24.75	6.42	0.004	-2.72E-04	-24.7	-28.3	
		6.25	0.004	-2.72E-04	-24.7	-28.3	
7	24.50	7.50	0.004	-2.39E-04	-23.0	-34.2	
		8.64	0.004	-2.39E-04	-23.0	-34.2	
8	24.25	11.36	0.004	-2.00E-04	-20.5	-39.6	
		13.49	0.004	-2.00E-04	-20.5	-39.6	
9	24.00	17.02	0.004	-1.55E-04	-16.7	-44.3	
		15.79	0.004	-1.55E-04	-16.7	-44.3	
10	23.60	21.11	0.004	-7.28E-05	-9.3	-49.6	
11	23.20	26.43	0.004	1.67E-05	0.2	-51.6	
12	22.80	31.74	0.004	1.06E-04	11.8	-48.6	
13	22.40	37.05	0.004	1.87E-04	25.6	-41.2	
14	22.00	42.37	0.004	2.51E-04	41.5	-27.9	56.3
		42.37	0.004	2.51E-04	-14.8	-27.9	
15	21.75	45.69	0.004	2.86E-04	-3.8	-30.3	
16	21.50	49.03	0.004	3.21E-04	8.0	-29.8	
		23.85	0.004	3.21E-04	8.0	-29.8	
17	21.15	13.68	0.004	3.65E-04	14.6	-25.8	
18	20.80	3.59	0.004	4.02E-04	17.6	-20.0	
19	20.40	-4.39	0.003	4.32E-04	17.4	-13.0	
20	20.00	-12.14	0.003	4.50E-04	14.1	-6.7	
21	19.60	-17.24	0.003	4.58E-04	8.3	-2.2	
22	19.30	-13.85	0.003	4.60E-04	3.6	-0.5	
23	19.00	-10.16	0.003	4.60E-04	0.0	0.0	

Strut force at elev. 25.80 = 28.69 kN/m run = 28.69 kN/strut
Strut force at elev. 22.00 = 56.30 kN/m run = 56.30 kN/strut

(continued)

Stage No.12 Change properties of soil type 1 to soil type 4
Ko pressures will not be reset

Node no.	Y coord	Water press. kN/m2	Vertic -al kN/m2	Active limit kN/m2	Passive limit kN/m2	Earth pressure kN/m2	Total earth pressure kN/m2	Soil stiffness coeff. kN/m3
		ACTIVE side		Effective stresses				
1	26.00	0.00	10.00	0.00	2.27b	0.40	0.40	2748
2	25.80	0.00	13.60	0.00	2.42b	1.00	1.00	2748
		0.00	13.60	0.00	7.85b	1.00	1.00	2748
3	25.50	0.00	18.98	0.00	8.59b	2.50	2.50	2748
		0.00	18.98	0.00	14.17b	2.50	2.50	2748
4	25.25	0.00	23.44	0.00	15.18b	3.75	3.75	2748
		0.00	23.44	0.00	19.89b	3.75	3.75	2748
5	25.00	0.00	27.86	0.00	21.20b	5.00	5.00	2748
		0.00	27.86	0.00	26.07b	5.00	5.00	2748
6	24.75	2.50	29.75	0.00	26.76b	3.92	6.42	2748
		2.50	29.75	0.00	31.88b	3.75	6.25	2748
7	24.50	5.00	31.59	0.00	32.68b	2.50	7.50	2748
		5.00	31.59	0.00	37.70b	3.64	8.64	2748
8	24.25	7.50	33.41	0.00	38.61b	3.86	11.36	2748
		7.50	33.41	0.00	43.52b	5.99	13.49	2748
9	24.00	10.00	35.19	0.00	44.53b	7.02	17.02	2748
		10.00	35.19	5.79	49.57b	5.79	15.79a	7634
10	23.60	14.00	38.81	7.11	53.74b	7.11	21.11a	8001
		14.00	38.81	7.11	54.04b	7.11	21.11a	8001
11	23.20	18.00	42.39	8.43	58.19b	8.43	26.43a	8367
		18.00	42.39	8.43	65.38b	8.43	26.43a	8367
12	22.80	22.00	45.96	9.74	70.04b	9.74	31.74a	8734
		22.00	45.96	9.74	76.75b	9.74	31.74a	8734
13	22.40	26.00	49.54	11.05	81.86b	11.05	37.05a	14019
		26.00	49.54	11.05	88.17b	11.05	37.05a	14019
14	22.00	30.00	53.14	12.37	93.70b	12.37	42.37a	14584
		30.00	53.14	12.37	98.59b	12.37	42.37a	14584
15	21.75	32.50	55.39	13.19	102.24b	13.19	45.69a	14936
		32.50	55.39	13.19	105.85b	13.19	45.69a	14936
16	21.50	35.00	57.66	14.03	109.65b	14.03	49.03a	15289
		35.00	57.66	14.03	113.81b	14.03	49.03a	15289
17	21.15	38.50	60.85	15.19	119.36b	15.19	53.69a	15783
		38.50	60.85	15.19	124.05b	15.19	53.69a	15783
18	20.80	42.00	64.05	16.37	129.85b	16.46	58.46	16277
		42.00	64.05	16.37	134.67b	16.46	58.46	16277
19	20.40	46.00	67.74	17.72	141.58b	21.45	67.45	16842
		46.00	67.74	17.72	146.51b	21.45	67.45	16842
20	20.00	50.00	71.45	19.08	153.71b	26.70	76.70	17406
		50.00	71.45	19.08	158.44b	26.70	76.70	17406
21	19.60	54.00	75.18	20.45	165.90b	32.19	86.19	17971
		54.00	75.18	20.45	169.89b	32.19	86.19	17971
22	19.30	57.00	77.99	21.48	175.65b	36.47	93.47	18394
		57.00	77.99	21.48	178.96b	36.47	93.47	18394
23	19.00	60.00	80.81	22.51	184.84b	40.87	100.87	18817

(continued)

Stage No.12 Change properties of soil type 1 to soil type 4
Ko pressures will not be reset

Node no.	Y coord	Water press. kN/m2	Vertic -al kN/m2	Active limit kN/m2	Passive limit kN/m2	Earth pressure kN/m2	Total earth pressure kN/m2	Soil stiffness coeff. kN/m3
		PASSIVE side						
			Effective stresses					
1	26.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
2	25.80	0.00	0.00	0.00	0.00	0.00	0.00	0.0
3	25.50	0.00	0.00	0.00	0.00	0.00	0.00	0.0
4	25.25	0.00	0.00	0.00	0.00	0.00	0.00	0.0
5	25.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
6	24.75	0.00	0.00	0.00	0.00	0.00	0.00	0.0
7	24.50	0.00	0.00	0.00	0.00	0.00	0.00	0.0
8	24.25	0.00	0.00	0.00	0.00	0.00	0.00	0.0
9	24.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
10	23.60	0.00	0.00	0.00	0.00	0.00	0.00	0.0
11	23.20	0.00	0.00	0.00	0.00	0.00	0.00	0.0
12	22.80	0.00	0.00	0.00	0.00	0.00	0.00	0.0
13	22.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
14	22.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
15	21.75	0.00	0.00	0.00	0.00	0.00	0.00	0.0
16	21.50	0.00	0.00	0.00	0.00	0.00	0.00	0.0
		0.00	0.00	0.00	25.17	25.17	25.17p	15289
17	21.15	3.50	3.50	0.00	36.52	36.52	40.02p	15783
18	20.80	7.00	7.00	0.00	47.87	47.87	54.87p	16277
19	20.40	11.00	11.01	0.00	60.85	60.85	71.85p	16842
20	20.00	15.00	15.02	0.00	73.84	73.84	88.84p	17406
21	19.60	19.00	19.03	0.00	86.86	84.43	103.43	17971
22	19.30	22.00	22.05	0.97	96.64	85.31	107.31	18394
23	19.00	25.00	25.07	2.08	106.43	86.03	111.03	18817

Note: 53.69a Soil pressure at active limit
88.84p Soil pressure at passive limit
184.84b Passive limit reduced because of berm

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CARD GEOTECHNICS LIMITED                                              | Sheet No.
Program: WALLAP  Version 6.05  Revision A41.B56.R46                   | Job No.  18067a
                             Licensed from GEOSOLVE                   | Made by :   JMS
Data filename/Run ID: Block D - NG cables and canal_SLS               |
Camden Lock                                                           | Date:20-01-2015
Block D - NG cable and canal                                          | Checked :
-----------------------------------------------------------------------------
                                                                 Units: kN,m
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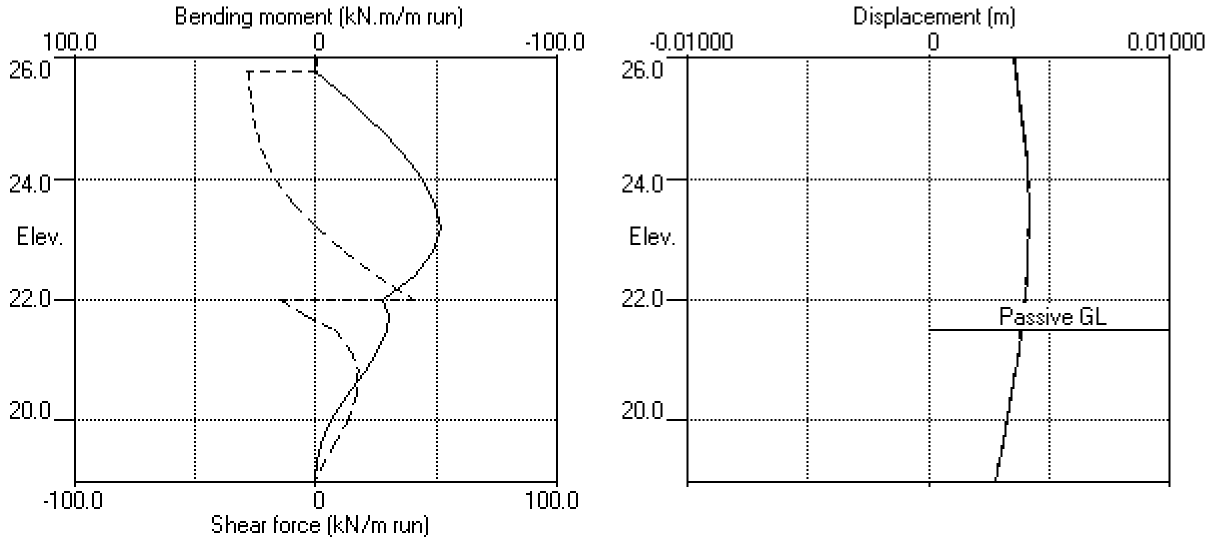
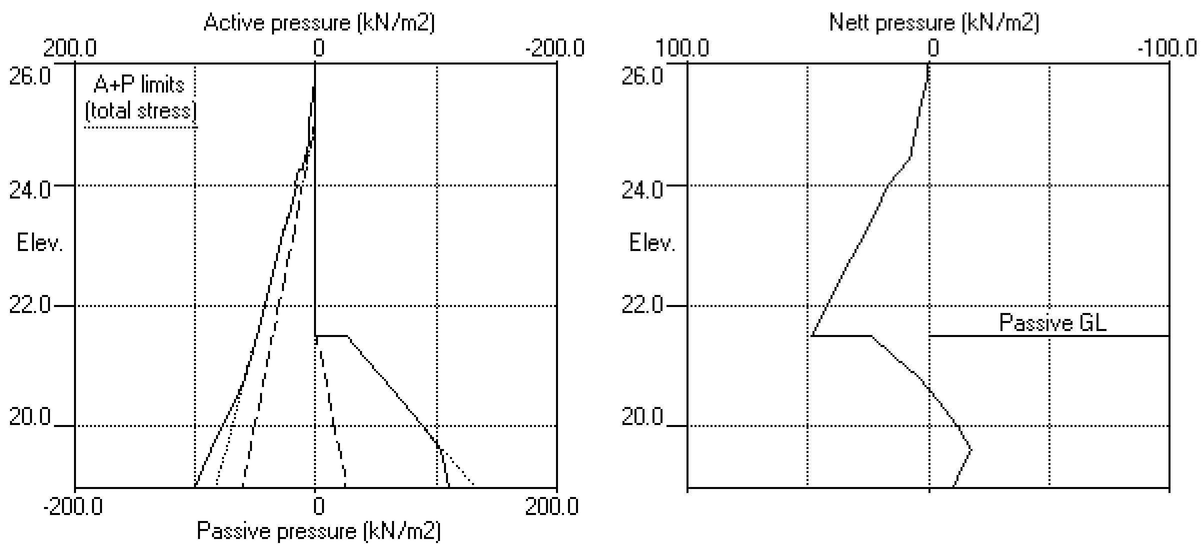
Stage No.12 Change soil type 1 to soil type 4



Stage No.12 Change soil type 1 to soil type 4



Units: kN,m

Summary of results

LIMIT STATE PARAMETERS

Limit State: Serviceability Limit State

All loads and soil strengths are unfactored

STABILITY ANALYSIS of Fully Embedded Wall according to Strength Factor method

Factor of safety on soil strength

Stage No.	G.L. Act.	G.L. Pass.	Strut Elev.	FoS for toe elev. = 19.00: Factor of Safety	FoS for toe elev. = 19.00: Moment equilib. at elev.	Toe elev. for FoS = 1.250: Toe elev.	Toe elev. for FoS = 1.250: Wall Penetr -ation
1	26.00	26.00	Cant.	Conditions not suitable for FoS calc.			
2	26.00	26.00	Cant.	Conditions not suitable for FoS calc.			
3	26.00	26.00		No analysis at this stage			
4	26.00	26.00	Cant.	Conditions not suitable for FoS calc.			
5	26.00	24.50	Cant.	3.451	19.25	23.89	0.61
6	26.00	24.50		No analysis at this stage			
7	26.00	21.50	25.50	3.345	n/a	21.23	0.27
8	26.00	21.50		No analysis at this stage			

All remaining stages have more than one strut - FoS calculation n/a

Units: kN,m

Summary of results

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

Length of wall perpendicular to section = 1000.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Active side 20.00 from wall
Passive side 20.00 from wall

Bending moment, shear force and displacement envelopes

Node no.	Y coord	Displacement maximum m	Displacement minimum m	Bending moment maximum kN.m/m	Bending moment minimum kN.m/m	Shear force maximum kN/m	Shear force minimum kN/m
1	26.00	0.004	0.000	0.0	-0.0	0.0	0.0
2	25.80	0.004	0.000	0.0	0.0	0.4	-28.5
3	25.50	0.004	0.000	0.3	-8.4	1.2	-31.5
4	25.25	0.004	0.000	0.7	-15.3	2.0	-30.7
5	25.00	0.004	0.000	1.3	-22.0	3.9	-29.6
6	24.75	0.004	0.000	2.5	-28.3	6.7	-28.1
7	24.50	0.004	0.000	4.5	-34.2	8.6	-26.4
8	24.25	0.004	0.000	6.9	-39.6	11.2	-23.7
9	24.00	0.004	0.000	10.2	-44.3	15.2	-19.7
10	23.60	0.004	0.000	15.1	-49.6	9.7	-15.3
11	23.20	0.004	0.000	18.0	-53.2	5.0	-10.1
12	22.80	0.004	0.000	19.1	-55.1	11.8	-3.2
13	22.40	0.004	0.000	18.8	-54.6	25.6	-2.2
14	22.00	0.004	0.000	17.3	-49.7	41.5	-14.8
15	21.75	0.004	0.000	16.0	-43.8	28.4	-5.8
16	21.50	0.004	0.000	14.4	-35.4	39.2	-6.6
17	21.15	0.004	0.000	11.9	-25.8	30.4	-7.3
18	20.80	0.004	0.000	9.3	-20.0	22.4	-7.5
19	20.40	0.003	0.000	6.2	-13.0	17.4	-7.1
20	20.00	0.003	0.000	3.5	-6.7	14.1	-6.0
21	19.60	0.003	0.000	1.4	-2.2	8.3	-4.2
22	19.30	0.003	0.000	0.4	-0.5	3.6	-2.3
23	19.00	0.003	0.000	0.0	-0.0	0.0	0.0

Maximum and minimum bending moment and shear force at each stage

Stage no.	Bending moment maximum kN.m/m	elev.	Bending moment minimum kN.m/m	elev.	Shear force maximum kN/m	elev.	Shear force minimum kN/m	elev.
1	7.9	22.80	0.0	26.00	5.7	24.00	-3.1	20.80
2	7.0	22.80	0.0	26.00	4.4	24.00	-2.6	20.80
3	No calculation at this stage							
4	6.6	22.80	0.0	26.00	4.2	24.00	-2.4	21.15
5	19.1	22.80	-0.0	26.00	15.2	24.00	-7.5	20.80
6	No calculation at this stage							
7	0.2	25.50	-55.1	22.80	39.2	21.50	-31.5	25.50
8	No calculation at this stage							
9	No calculation at this stage							
10	0.0	19.30	-53.9	22.80	37.6	21.50	-28.3	25.80
11	0.0	25.80	-51.6	23.20	41.5	22.00	-28.5	25.80
12	0.0	25.80	-51.6	23.20	41.5	22.00	-28.5	25.80

Summary of results (continued)

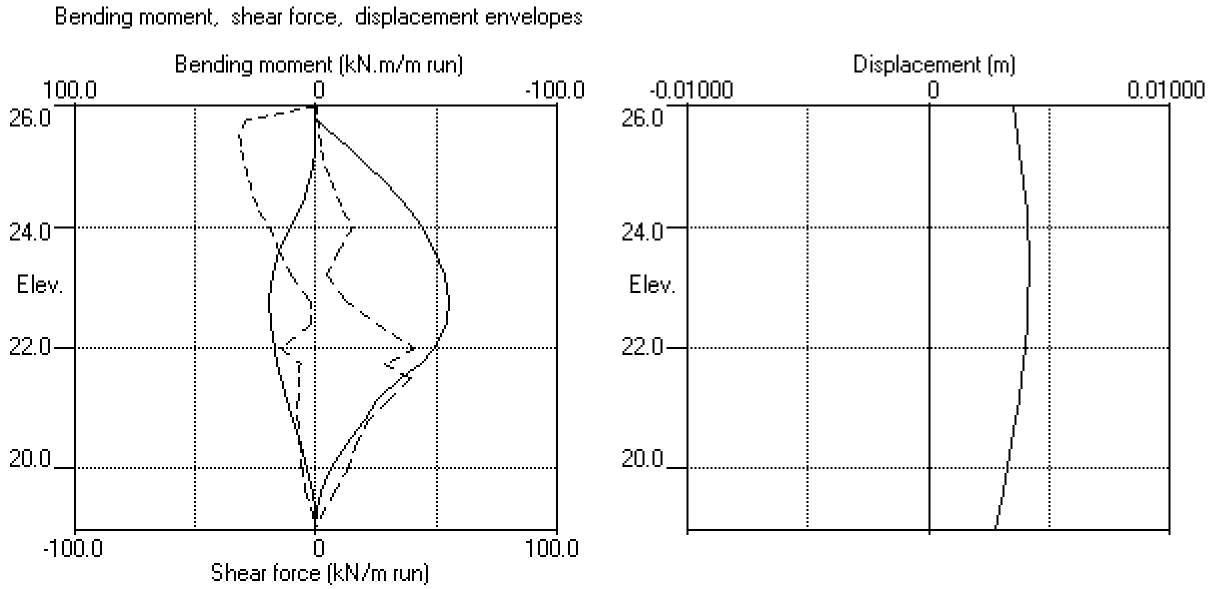
Maximum and minimum displacement at each stage

Stage no.	Displacement maximum m	elev.	Displacement minimum m	elev.	Stage description
1	0.001	26.00	0.000	26.00	Apply surcharge no.1 at elev. 26.00
2	0.001	26.00	0.000	26.00	Excav. to elev. 26.00 on ACTIVE side
3	Wall displacements reset to zero				Change EI of wall to 254469kN.m2/m run
4	0.000	19.00	0.000	26.00	Apply water pressure profile no.1
5	0.002	26.00	0.000	26.00	Excav. to elev. 24.50 on PASSIVE side
6	No calculation at this stage				Install strut no.3 at elev. 25.50
7	0.004	23.20	0.000	26.00	Excav. to elev. 21.50 on PASSIVE side
8	No calculation at this stage				Install strut no.2 at elev. 22.00
9	No calculation at this stage				Install strut no.1 at elev. 25.80
10	0.004	23.60	0.000	26.00	Remove strut no.3 at elev. 25.50
11	0.004	23.20	0.000	26.00	Change soil type 2 to soil type 3
12	0.004	23.20	0.000	26.00	Change soil type 1 to soil type 4

Strut forces at each stage (horizontal components)

Stage no.	Strut no. 1 at elev. 25.80 kN/m run	kN/strut	Strut no. 2 at elev. 22.00 kN/m run	kN/strut	Strut no. 3 at elev. 25.50 kN/m run	kN/strut
7	---	---	---	---	32.10	160.52
10	28.36	28.36	4.51	4.51	---	---
11	28.69	28.69	56.30	56.30	---	---
12	28.69	28.69	56.30	56.30	---	---

```
CARD GEOTECHNICS LIMITED                                        | Sheet No.
Program: WALLAP  Version 6.05  Revision A41.B56.R46             | Job No.  18067a
                             Licensed from GEOSOLVE             | Made by :   JMS
Data filename/Run ID: Block D - NG cables and canal_SLS         |
Camden Lock                                                     | Date:20-01-2015
Block D - NG cable and canal                                    | Checked :
-----------------------------------------------------------------------------
                                                       Units: kN,m
```



Units: kN,m

INPUT DATA

SOIL PROFILE

Stratum no.	Elevation of top of stratum	Soil types: Active side	Soil types: Passive side
1	26.00	1 Made Ground	1 Made Ground
2	24.00	2 London Clay	2 London Clay

SOIL PROPERTIES

-- Soil type -- No. Description (Datum elev.)	Bulk density kN/m3	Young's Modulus Eh,kN/m2 (dEh/dy)	At rest coeff. Ko (dKo/dy)	Consol state. NC/OC (Nu)	Active limit Ka (Kac)	Passive limit Kp (Kpc)	Cohesion kN/m2 (dc/dy)
1 Made Ground	18.00	18000	0.561	NC	1.000	1.000	30.00u
				(0.490)	(2.389)	(2.390)	
2 London Clay	20.00	50000	1.000	OC	1.000	1.000	50.00u
(24.00)		(6000)		(0.490)	(2.475)	(2.476)	(6.000)
3 London Cl..	20.00	37500	0.593	OC	0.367	3.241	5.000d
(24.00)		(4500)		(0.200)	(1.423)	(5.034)	
4 Made Ground	18.00	13500	0.561	NC	0.346	3.442	30.00d
DR				(0.200)	(1.340)	(5.007)	

Additional soil parameters associated with Ka and Kp

------- Soil type ------- No. Description	--- parameters for Ka --- Soil friction angle	Wall adhesion coeff.	Back-fill angle	--- parameters for Kp --- Soil friction angle	Wall adhesion coeff.	Back-fill angle
1 Made Ground	0.00	0.500	0.00	0.00	0.500	0.00
2 London Clay	0.00	0.667	0.00	0.00	0.667	0.00
3 London Clay DR	24.00	0.667	0.00	24.00	0.667	0.00
4 Made Ground DR	26.00	0.500	0.00	26.00	0.500	0.00

GROUND WATER CONDITIONS

Density of water = 10.00 kN/m3

	Active side	Passive side
Initial water table elevation	25.00	25.00

Automatic water pressure balancing at toe of wall : No

Water press. profile no.	Active side: Point no.	Elev. m	Piezo elev. m	Water press. kN/m2	Passive side: Point no.	Elev. m	Piezo elev. m	Water press. kN/m2	
1	1	25.00	25.00	0.0	1	21.50	21.50	0.0	MC+WC

WALL PROPERTIES

Type of structure = Fully Embedded Wall
Elevation of toe of wall = 19.00
Maximum finite element length = 0.40 m
Youngs modulus of wall E = 3.0000E+07 kN/m2
Moment of inertia of wall I = 8.4823E-03 m4/m run
E.I = 254469 kN.m2/m run
Yield Moment of wall = Not defined

STRUTS and ANCHORS

Strut/ anchor no.	Elev.	Strut spacing m	X-section area of strut sq.m	Youngs modulus kN/m2	Free length m	Inclin -ation (degs)	Pre-stress /strut kN	Tension allowed
1	25.80	1.00	0.300000	3.000E+07	30.00	0.00	0	Yes
2	22.00	1.00	0.300000	3.000E+07	30.00	0.00	0	Yes
3	25.50	5.00	0.010000	2.000E+08	20.00	0.00	0	No

SURCHARGE LOADS

Surch -arge no.	Elev.	Distance from wall	Length parallel to wall	Width perpend. to wall	Surcharge kN/m2 Near edge	Surcharge kN/m2 Far edge	Equiv. soil type	Partial factor/ Category
1	26.00	0.00(A)	30.00	3.00	10.00	=	N/A	1.00 -
2	23.50	3.00(A)	30.00	10.00	200.00	=	N/A	1.00 -

Note: A = Active side, P = Passive side
Limit State Categories P/U = Permanent Unfavourable
P/F = Permanent Favourable
Var = Variable (unfavourable)

CONSTRUCTION STAGES

Construction stage no.	Stage description
1	Apply surcharge no.1 at elevation 26.00
2	Apply surcharge no.2 at elevation 23.50 No analysis at this stage
3	Change EI of wall to 254469 kN.m2/m run Yield moment not defined Reset wall displacements to zero at this stage
4	Apply water pressure profile no.1 (Mod. Conserv.)
5	Excavate to elevation 21.50 on PASSIVE side
6	Install strut or anchor no.2 at elevation 22.00
7	Install strut or anchor no.1 at elevation 25.80
8	Change properties of soil type 2 to soil type 3 Ko pressures will not be reset
9	Change properties of soil type 1 to soil type 4 Ko pressures will not be reset

FACTORS OF SAFETY and ANALYSIS OPTIONS

Limit State options: Serviceability Limit State
All loads and soil strengths are unfactored

Stability analysis:
Method of analysis - Strength Factor method
Factor on soil strength for calculating wall depth = 1.25

Parameters for undrained strata:
Minimum equivalent fluid density = 5.00 kN/m3
Maximum depth of water filled tension crack = 0.00 m

Bending moment and displacement calculation:
Method - Subgrade reaction model using Influence Coefficients
Open Tension Crack analysis? - No
Non-linear Modulus Parameter (L) = 0 m

Boundary conditions:
Length of wall (normal to plane of analysis) = 1000.00 m

Width of excavation on active side of wall = 20.00 m
Width of excavation on passive side of wall = 20.00 m

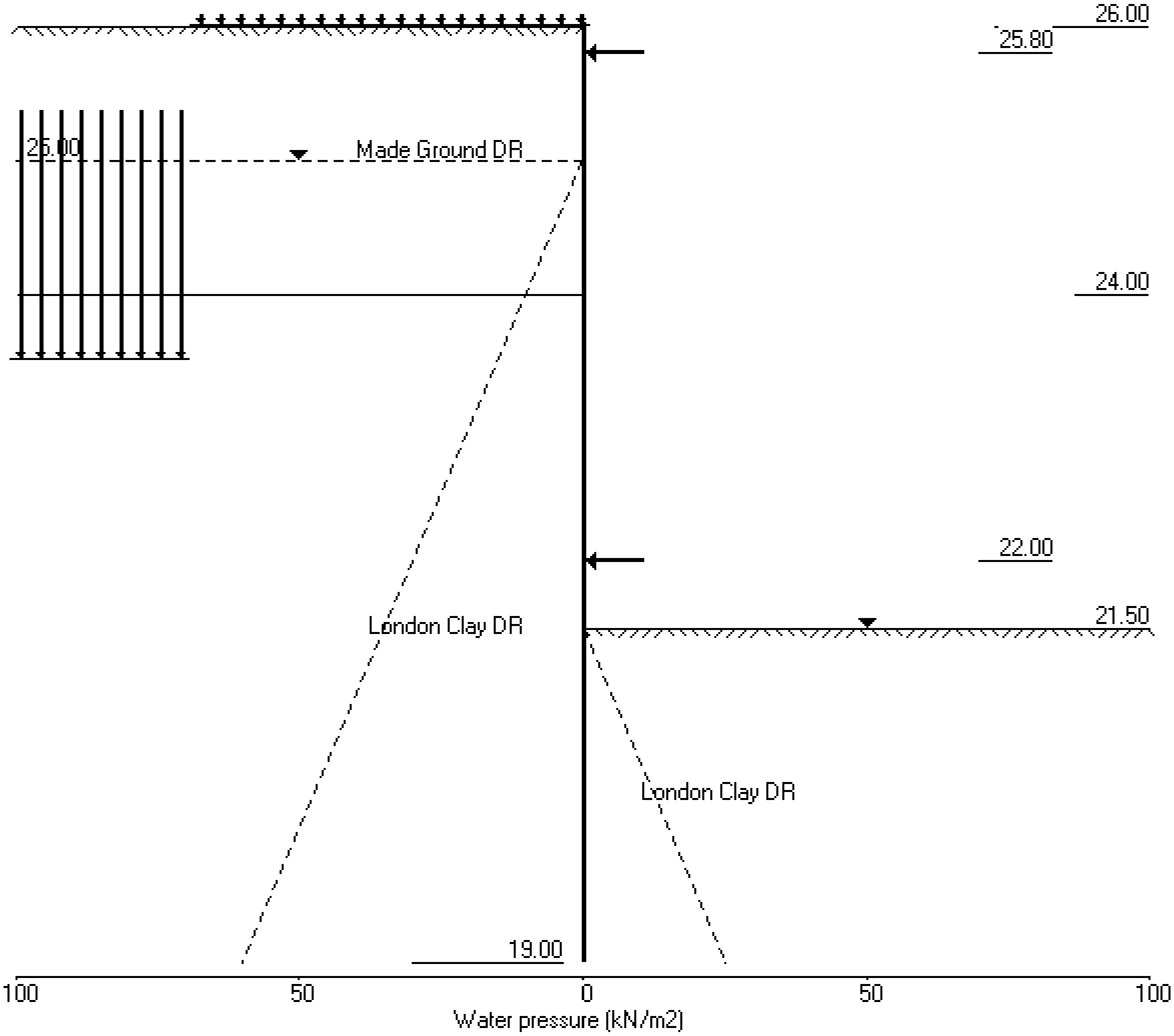
Distance to rigid boundary on active side = 20.00 m
Distance to rigid boundary on passive side = 20.00 m

OUTPUT OPTIONS

Stage no.	Stage description	Displacement Bending mom. Shear force	Active, Passive pressures	Graph. output
1	Apply surcharge no.1 at elev. 26.00	Yes	Yes	Yes
2	Apply surcharge no.2 at elev. 23.50	No	No	No
3	Change EI of wall to 254469kN.m2/m run	No	No	No
4	Apply water pressure profile no.1	No	No	No
5	Excav. to elev. 21.50 on PASSIVE side	No	No	No
6	Install strut no.2 at elev. 22.00	No	No	No
7	Install strut no.1 at elev. 25.80	No	No	No
8	Change soil type 2 to soil type 3	No	No	No
9	Change soil type 1 to soil type 4	No	No	No
*	Summary output	Yes	-	Yes

Units: kN,m

Stage No.9 Change soil type 1 to soil type 4



Units: kN,m

Stage No. 9 Change properties of soil type 1 to soil type 4
Ko pressures will not be reset

STABILITY ANALYSIS of Fully Embedded Wall according to Strength Factor method
Factor of safety on soil strength

Stage No.	--- G.L. --- Act.	Pass.	Strut Elev.	FoS for toe elev. = 19.00 Factor of Safety	Moment equilib. at elev.	Toe elev. for FoS = 1.250 Toe elev.	Wall Penetr -ation
9	26.00	21.50		More than one strut			

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall
Analysis options
Length of wall perpendicular to section = 1000.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Active side 20.00 from wall
Passive side 20.00 from wall

*** Wall displacements reset to zero at stage 3

Node no.	Y coord	Nett pressure kN/m2	Wall disp. m	Wall rotation rad.	Shear force kN/m	Bending moment kN.m/m	Strut forces kN/m
1	26.00	0.70	0.014	2.12E-03	0.0	-0.0	
2	25.80	1.57	0.014	2.12E-03	0.2	0.0	-10.5
		1.57	0.014	2.12E-03	10.7	0.0	
3	25.40	3.30	0.013	2.12E-03	11.7	4.5	
4	25.00	5.00	0.012	2.11E-03	13.3	9.5	
5	24.70	6.50	0.012	2.10E-03	15.0	13.7	
6	24.40	8.00	0.011	2.08E-03	17.2	18.5	
7	24.00	10.00	0.010	2.05E-03	20.8	26.1	
		15.79	0.010	2.05E-03	20.8	26.1	
8	23.75	19.12	0.010	2.03E-03	25.2	31.9	
9	23.50	22.44	0.009	2.00E-03	30.4	38.8	
10	23.15	27.14	0.009	1.95E-03	39.0	50.9	
11	22.80	32.10	0.008	1.88E-03	49.4	66.4	
12	22.40	38.34	0.007	1.77E-03	63.5	88.9	
13	22.00	45.28	0.006	1.62E-03	80.2	117.7	159.1
		45.28	0.006	1.62E-03	-78.8	117.7	
14	21.75	49.93	0.006	1.52E-03	-66.9	99.4	
15	21.50	54.78	0.006	1.43E-03	-53.9	84.3	
		29.61	0.006	1.43E-03	-53.9	84.3	
16	21.15	21.80	0.005	1.34E-03	-44.9	71.1	
17	20.80	14.15	0.005	1.25E-03	-38.6	56.1	
18	20.40	5.45	0.004	1.18E-03	-34.6	40.7	
19	20.00	6.39	0.004	1.13E-03	-32.3	26.9	
20	19.60	11.49	0.003	1.10E-03	-28.7	13.9	
21	19.30	23.56	0.003	1.09E-03	-23.4	6.0	
22	19.00	132.75	0.003	1.09E-03	-0.0	0.0	

Strut force at elev. 25.80 = -10.45 kN/m run = -10.45 kN/strut
Strut force at elev. 22.00 = 159.07 kN/m run = 159.07 kN/strut

(continued)

Stage No.9 Change properties of soil type 1 to soil type 4
Ko pressures will not be reset

ACTIVE side

Node no.	Y coord	Water press. kN/m2	Effective stresses: Vertic -al kN/m2	Effective stresses: Active limit kN/m2	Effective stresses: Passive limit kN/m2	Effective stresses: Earth pressure kN/m2	Total earth pressure kN/m2	Soil stiffness coeff. kN/m3
1	26.00	0.00	10.00	0.00	184.64	0.70	0.70	9014
2	25.80	0.00	13.60	0.00	197.03	1.57	1.57	9014
3	25.40	0.00	20.77	0.00	221.71	3.30	3.30	9014
4	25.00	0.00	27.86	0.00	246.12	5.00	5.00	3322
5	24.70	3.00	30.12	0.00	253.89	3.50	6.50	3322
6	24.40	6.00	32.32	0.00	261.48	2.00	8.00	3322
7	24.00	10.00	35.19	0.00	271.36	0.00	10.00a	3322
		10.00	35.19	5.79	139.24	5.79	15.79a	9228
8	23.75	12.50	37.46	6.62	146.58	6.62	19.12a	9505
9	23.50	15.00	39.70	7.44	153.87	7.44	22.44a	9782
10	23.15	18.50	42.97	8.64	164.44	8.64	27.14a	10170
11	22.80	22.00	46.96	10.10	177.38	10.10	32.10a	10557
12	22.40	26.00	53.07	12.34	197.19	12.34	38.34a	11000
13	22.00	30.00	61.07	15.28	223.12	15.28	45.28a	11443
14	21.75	32.50	66.95	17.43	242.17	17.43	49.93a	11720
15	21.50	35.00	73.36	19.78	262.97	19.78	54.78a	11997
16	21.15	38.50	83.01	23.32	294.22	23.32	61.82a	12384
17	20.80	42.00	93.09	27.01	326.92	27.01	69.01a	12772
18	20.40	46.00	104.77	31.30	364.78	31.30	77.30a	13215
19	20.00	50.00	116.29	35.52	402.12	45.23	95.23	13658
20	19.60	54.00	127.42	39.60	438.19	63.34	117.34	14101
21	19.30	57.00	135.43	42.53	464.14	76.84	133.84	14433
22	19.00	60.00	143.10	45.35	489.01	120.43	180.43	14765

PASSIVE side

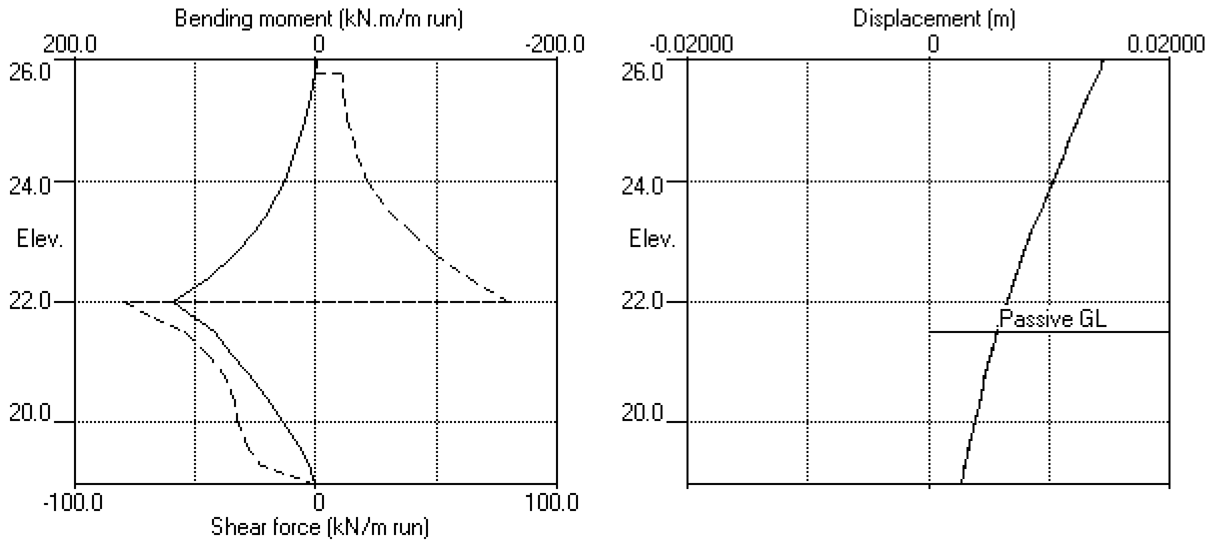
Node no.	Y coord	Water press. kN/m2	Effective stresses: Vertic -al kN/m2	Effective stresses: Active limit kN/m2	Effective stresses: Passive limit kN/m2	Effective stresses: Earth pressure kN/m2	Total earth pressure kN/m2	Soil stiffness coeff. kN/m3
1	26.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
2	25.80	0.00	0.00	0.00	0.00	0.00	0.00	0.0
3	25.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
4	25.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
5	24.70	0.00	0.00	0.00	0.00	0.00	0.00	0.0
6	24.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
7	24.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
8	23.75	0.00	0.00	0.00	0.00	0.00	0.00	0.0
9	23.50	0.00	0.00	0.00	0.00	0.00	0.00	0.0
10	23.15	0.00	0.00	0.00	0.00	0.00	0.00	0.0
11	22.80	0.00	0.00	0.00	0.00	0.00	0.00	0.0
12	22.40	0.00	0.00	0.00	0.00	0.00	0.00	0.0
13	22.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0
14	21.75	0.00	0.00	0.00	0.00	0.00	0.00	0.0
15	21.50	0.00	0.00	0.00	0.00	0.00	0.00	0.0
		0.00	0.00	0.00	25.17	25.17	25.17p	11997
16	21.15	3.50	3.50	0.00	36.52	36.52	40.02p	12384
17	20.80	7.00	7.00	0.00	47.87	47.87	54.87p	12772
18	20.40	11.00	11.01	0.00	60.85	60.85	71.85p	13215
19	20.00	15.00	15.02	0.00	73.84	73.84	88.84p	13658
20	19.60	19.00	19.03	0.00	86.86	86.86	105.86p	14101
21	19.30	22.00	22.05	0.97	96.64	88.28	110.28	14433
22	19.00	25.00	25.07	2.08	106.43	22.68	47.68	14765

(continued)

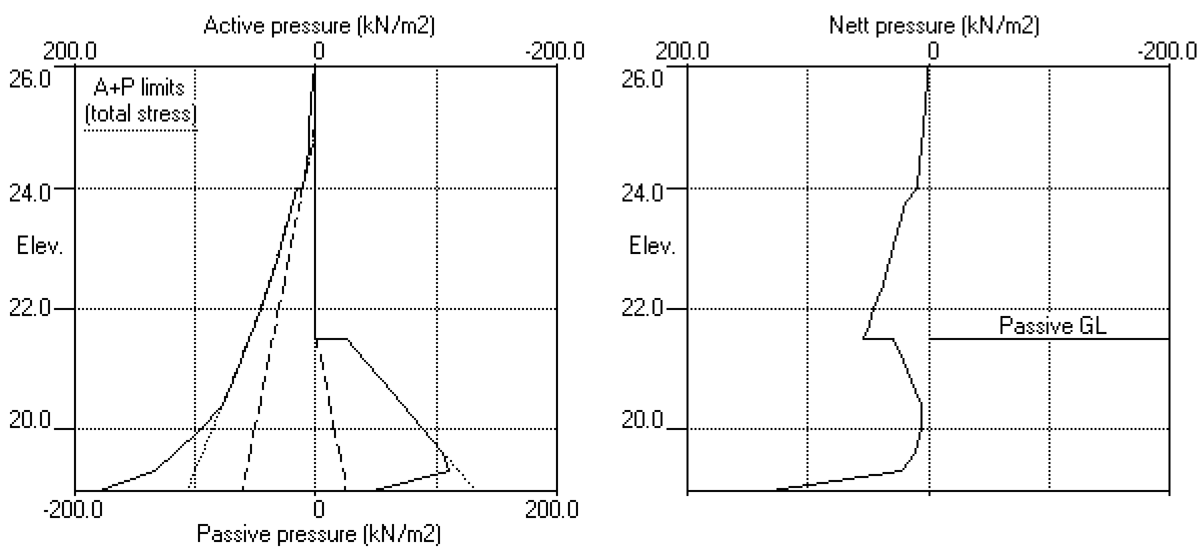
```
Stage No.9   Change properties of soil type 1 to soil type 4
              Ko pressures will not be reset
Note:     77.30a  Soil pressure at active limit
         105.86p  Soil pressure at passive limit
```

Units: kN,m

Stage No.9 Change soil type 1 to soil type 4



Stage No.9 Change soil type 1 to soil type 4



Units: kN,m

Summary of results

LIMIT STATE PARAMETERS

Limit State: Serviceability Limit State

All loads and soil strengths are unfactored

STABILITY ANALYSIS of Fully Embedded Wall according to Strength Factor method

Factor of safety on soil strength

Stage No.	G.L. Act.	G.L. Pass.	Strut Elev.	FoS for toe elev. = 19.00: Factor of Safety	FoS for toe elev. = 19.00: Moment equilib. at elev.	Toe elev. for FoS = 1.250: Toe elev.	Toe elev. for FoS = 1.250: Wall Penetr -ation
1	26.00	26.00	Cant.	Conditions not suitable for FoS calc.			
2	26.00	26.00		No analysis at this stage			
3	26.00	26.00		No analysis at this stage			
4	26.00	26.00	Cant.	Conditions not suitable for FoS calc.			
5	26.00	21.50	Cant.	1.655	19.42	19.42	2.08
6	26.00	21.50		No analysis at this stage			

All remaining stages have more than one strut - FoS calculation n/a

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Units: kN,m

Summary of results

BENDING MOMENT and DISPLACEMENT ANALYSIS of Fully Embedded Wall

Analysis options

Length of wall perpendicular to section = 1000.00m
Subgrade reaction model - Boussinesq Influence coefficients
Soil deformations are elastic until the active or passive limit is reached
Open Tension Crack analysis - No

Rigid boundaries: Active side 20.00 from wall
Passive side 20.00 from wall

Bending moment, shear force and displacement envelopes

Node no.	Y coord	Displacement maximum m	Displacement minimum m	Bending moment maximum kN.m/m	Bending moment minimum kN.m/m	Shear force maximum kN/m	Shear force minimum kN/m
1	26.00	0.015	-0.000	0.0	-0.0	0.0	0.0
2	25.80	0.014	-0.000	0.1	0.0	10.7	0.0
3	25.40	0.013	-0.000	4.5	0.0	11.7	0.0
4	25.00	0.012	-0.000	9.5	0.0	13.3	0.0
5	24.70	0.012	-0.000	13.7	0.0	15.0	0.0
6	24.40	0.011	-0.000	18.5	0.0	17.2	0.0
7	24.00	0.010	0.000	26.1	0.0	20.8	0.0
8	23.75	0.010	0.000	31.9	0.0	25.2	0.0
9	23.50	0.009	0.000	38.8	0.0	30.4	0.0
10	23.15	0.009	0.000	50.9	0.0	39.0	0.0
11	22.80	0.008	0.000	66.4	0.0	49.4	0.0
12	22.40	0.007	0.000	88.9	0.0	63.5	-1.5
13	22.00	0.006	0.000	117.7	0.0	80.2	-78.8
14	21.75	0.006	0.000	99.4	0.0	45.2	-66.9
15	21.50	0.006	0.000	84.3	0.0	50.6	-53.9
16	21.15	0.005	0.000	90.0	0.0	6.8	-44.9
17	20.80	0.005	0.000	85.6	0.0	0.0	-38.6
18	20.40	0.004	0.000	68.9	0.0	0.0	-50.4
19	20.00	0.004	0.000	45.7	0.0	0.0	-59.0
20	19.60	0.003	0.000	22.1	0.0	0.0	-52.3
21	19.30	0.003	0.000	8.1	0.0	0.0	-37.1
22	19.00	0.003	0.000	0.0	-0.0	0.0	-0.0

Maximum and minimum bending moment and shear force at each stage

Stage no.	Bending moment maximum kN.m/m	elev.	Bending moment minimum kN.m/m	elev.	Shear force maximum kN/m	elev.	Shear force minimum kN/m	elev.
1	7.9	22.80	-0.0	19.00	5.7	24.00	-3.1	20.80
2	No calculation at this stage							
3	No calculation at this stage							
4	19.2	22.40	-0.0	19.00	10.4	24.00	-8.8	20.80
5	90.0	21.15	-0.0	26.00	50.6	21.50	-59.0	20.00
6	No calculation at this stage							
7	No calculation at this stage							
8	117.7	22.00	-0.0	26.00	80.2	22.00	-78.8	22.00
9	117.7	22.00	-0.0	26.00	80.2	22.00	-78.8	22.00

Summary of results (continued)

Maximum and minimum displacement at each stage

Stage no.	Displacement maximum m	elev.	minimum m	elev.	Stage description
1	0.001	26.00	0.000	26.00	Apply surcharge no.1 at elev. 26.00
2	No calculation at this stage				Apply surcharge no.2 at elev. 23.50
3	Wall displacements reset to zero				Change EI of wall to 254469kN.m2/m run
4	0.001	19.00	-0.000	26.00	Apply water pressure profile no.1
5	0.015	26.00	0.000	26.00	Excav. to elev. 21.50 on PASSIVE side
6	No calculation at this stage				Install strut no.2 at elev. 22.00
7	No calculation at this stage				Install strut no.1 at elev. 25.80
8	0.014	26.00	0.000	26.00	Change soil type 2 to soil type 3
9	0.014	26.00	0.000	26.00	Change soil type 1 to soil type 4

Strut forces at each stage (horizontal components)

Stage no.	Strut no. 1 at elev. 25.80 kN/m run	kN/strut	Strut no. 2 at elev. 22.00 kN/m run	kN/strut
8	-10.45	-10.45	159.07	159.07
9	-10.45	-10.45	159.07	159.07

```
CARD GEOTECHNICS LIMITED                                    | Sheet No.
Program: WALLAP  Version 6.05  Revision A41.B56.R46         | Job No.  18067a
                             Licensed from GEOSOLVE         | Made by :   JMS
Data filename/Run ID: Block D - viaduct_SLS                 |
Camden Lock                                                 | Date:20-01-2015
Block D - NR viaduct                                        | Checked :
-----------------------------------------------------------------------------
                                                     Units: kN,m
```