



A2 Site Investigation

Brunel Walk

Geotechnical Design Report

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A2 Site Investigation Limited

One Westminster Bridge Rd
London, SE1 7XW

020 7021 0396
info@a2-si.com
www.a2-si.com

Prepared by

Finlay Campbell
MSci (Hons)

Geo-environmental Consultant

Checked by

G. Turner
MEng (Hons) CEng MICE ROGEP

Associate

Approved by

A. Fasano
Dott Ing CEng MICE

Director

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1. Introduction

A2 Site Investigation (A2SI) were instructed by Meinhardt (UK) Ltd, on behalf of London Borough of Haringey, to prepare a Geotechnical Design Report (GDR) for the proposed development at Brunel Walk, London N15 5HQ.

1.1. Study Aims and Objectives

The scope of this report comprises the following elements:

- Technical assessment and interpretation of ground investigation data carried out for geotechnical design parameters.
- Outline assessment of shallow and deep foundations (ULS and SLS performance, and groundwater considerations including uplift and heave mitigation).
- Earth retention system typology assessment.
- General buildability and earthworks considerations.
- Geo-environmental interpretation and Phase II Quantitative Risk Assessment including Generic Quantitative Risk Assessment (GQRA).

The geo-environmental assessment has been undertaken in general accordance with *Land Contamination Risk Management (LCRM)* guidance, published by the Environment Agency on the UK Government website. This GDR presents a Generic Quantitative Risk Assessment (GQRA) and updated Conceptual Site Model (CSM) for the proposed development in the context of the *National Planning Policy Framework (NPPF)* and *The Building Regulations 2010, Approved Document C - Site preparation and resistance to contaminants and moisture (2004 Edition incorporating 2010 and 2013 amendments)*. The GDR includes an assessment of whether there are any unacceptable risks (ref. *LCRM* guidance) in relation to the proposed development which need to be further addressed.

1.2. Information Sources

The principal sources of information provided by the project team, which have informed the preliminary assessment presented herein, include the following:

- A2SI, *Factual Report* (ref: 12621-A2SI-XX-XX-RP-X-0001-00), dated January 2022.
- Meinhardt, Brunel Walk Specification For Site Investigations (ref: 2844-MHT_ST-SP-00001), dated December 2020.
- Bell Phillips Architects, Brunel Walk Stage 1 report (ref:0682_Brunel Walk_Stage 1 Report), dated November 2020.
- Soil Consultants, Phase 1 Geo-Environmental Desk Study Report (ref: 10582 Brunel Walk Phase 1 Report Rev 0 Final), dated January 2021.



2. The Site and Proposed Development

2.1. Development Location and Current Site Use

The site is situated in the London borough of Haringey, at Brunel Walk, London, N15 5HQ. The site is roughly rectangular, located approximately 0.5km northeast of Seven Sisters Underground Station. The Ordnance Survey Grid reference for the centre of the site is TQ 33081 89026. The approximate surface elevation of the site is 12mOD in the east of the site, gradually rising to about 13.5mOD at the western end. The car park at the extreme east of site has a lower elevation, at approximately 10.5mOD.

The site is currently occupied by an existing estate comprising four two storey residential structures consisting of 35 single person flats, 1 family house and some communal facilities. The site is bordered by Turner Avenue estate (council owned) to the north, Kirkton Road to the east, Seaford Road to the south and Elmar Road to the west.



Figure 2-1 Location of the proposed development

2.2. Site History

The site has predominantly been undeveloped forming parts of residential gardens and open fields until (1894) in the north-west corner of the site garden sheds and trees were present. Site comprises a railway line (G.E.R. Alexandra Park Branch) on an embankment in the east and central area and a cutting in the west part (1894/1936). West part of the railway not showing to be within a cutting anymore (1954). Railway is subsequently shown to be dismantled (1970/73), and finally the site has been redeveloped into current configuration with embankment section no longer shown (1975/2003). No further changes to the site have recorded since early 2000's.

2.3. Proposed Scheme

The scheme for the proposed development comprises demolition of the existing flats, houses and communal facilities and the construction of four new blocks ranging between three to four storey, shown in Figure 2-2 and Figure 2-3. Block A is to be private whilst blocks B, C and D are to be used for social housing. The development has a total of 46 units.

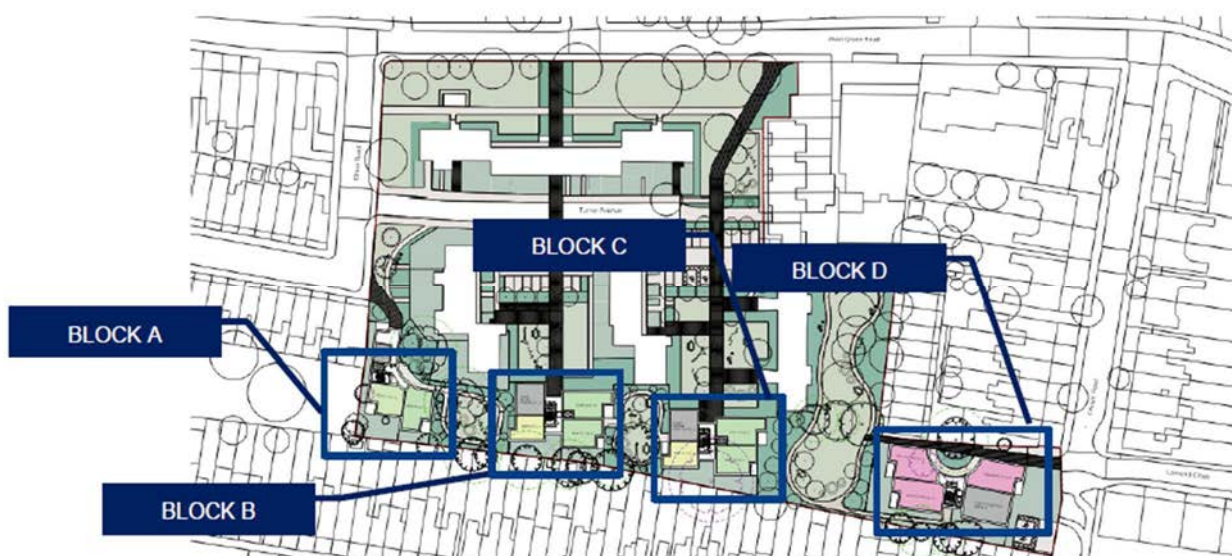


Figure 2-2 Proposed site plan



Figure 2-3 Propose site massing with block reference

2.4. Preliminary Risk Assessment (PRA)

Table 2.1 summarises the PRA presented in the Soil Consultants Phase I Desk Study Report and highlights the potentially complete contaminant linkages identified. Table 2.1 provides a baseline summary of site understanding prior to undertaking the site investigation and GQRA presented later in this GDR. Where 'low to moderate' risk or 'moderate' risk is indicated this is also categorised as



unacceptable risk in accordance with LCRM guidance. Unacceptable risks have undergone GQRA assessment in the later sections of this GDR.

Table 2.1 PRA Summary

Source	Pathway	Receptor	Assessed Risk and commentary/justification
On-site: Contaminated soil and groundwater	Ingestion, contact, inhalation	End user, construction workers and infrastructure	Moderate risk: The historical railway line presence (with associated earthworks/made ground) is the main potential source identified. Pre-2000 buildings and potential made ground giving potential asbestos risk
	Leaching from contaminated soils and migration in groundwater	Aquifer and surface water	Low risk: There is an unproductive aquifer beneath the site, which lies within a Type 2 Catchment Source Protection Zone. There are no nearby water abstraction points or surface water features The London Clay underlies the site and will provide protection to the main aquifer within the chalk, therefore the risk to the aquifer is expected to be low.
Off-site: Contaminated Soil	Lateral migration of contaminants to site in groundwater	End user	Low risk: Main potential source is the former works immediately north of the site, but very low permeability London Clay is present which would prevent any significant migration. Several electricity substations identified but the nearest was some 43m from the site and not considered to present a significant risk due to its distance and very low mobility of PCBs
On-site and off-site: Ground gas and vapours	Lateral and vertical migration of gas/vapour	End-user and buildings	Low to moderate risk: There will be a risk of ground gas/vapours associated with any significant Made Ground infill associated with the former railway line cutting if this contains significant putrescible waste. The site is not in a radon affected area.

Identified potential on-site sources of contamination included:

- Railway - Heavy metals and metalloids, acids / alkalis, PAHs, TPH and elevated sulphates.
- Made Ground – Heavy metals and metalloids, acids / alkalis, PAHs, asbestos, elevated sulphate and ground gases.
- Asbestos containing materials (ACMs) in the current building fabrics.

Identified potential off-site sources of contamination included:

- Electrical Substation 46m north of the site – Heavy metals and metalloids, acids / alkalis, PAHs, TPH and PCBs
- Historical industrial usage defined as 'works' immediately to the north – Heavy metals and metalloids, acids / alkalis, PAHs, TPHs, VOCs and elevated sulphates.

Notes

TPH – total petroleum hydrocarbons

PAH – polycyclic aromatic hydrocarbons

BTEX – benzene, toluene, ethylbenzene, xylenes

PCB – polychlorinated biphenyls

VOC – volatile organic compounds

Asbestos – potential free fibres, debris and / or fragments of asbestos containing material (ACM).

Ground gas – methane and carbon dioxide

It is also important to note the points below regarding the Soil Consultants Phase I Report which have been further considered in the A2SI quantitative assessments presented later:

- The PRA does not appear to consider off-site below ground structures as plausible receptors e.g. buildings and structures. However, A2SI believe that there is a potential contaminant linkage between the site and off-site buildings and structures due to possible ground gas generation, migration, accumulation and ignition, so this contaminant linkage is also assessed in the GQRA.



- The PRA does not appear to consider off-site human health is a plausible receptor in regard to soil ingestion, inhalation and dermal contact, and ground gas/soil vapour inhalation, due to potential on-site sources of contamination. However, A2SI believe that there are potentially complete contaminant linkages and therefore these contaminant linkages have been included in the GQRA.
- The Phase I does not state on site flora and fauna as a receptor. Due to the presence of soft landscaping in the proposed development, this receptor has been taken forward in the GQRA.
- Consumption of water from impacted water supply pipes installed as part of the proposed redevelopment has not been included as a potential pathway within the PRA. A water supply pipes assessment has been included as part of this GQRA anyway.

This GDR presents an updated risk assessment for the proposed development based on site investigation, as recommended by the Soil Consultants Phase I. The updated risk assessment includes GQRA in relation to human health (on and off-site), flora and fauna (on-site) and buildings and structures (on and off-site property).



3. Geological Setting

3.1. Regional Geological Overview

Published BGS geological maps indicate that the site is underlain by the London Clay Formation with no superficial deposits present. The Lambeth Beds, Thanet Formation and Chalk are present at greater depths.

The development site is located within the London Basin, which refers to an approximately triangular synclinal structure in which the sedimentary units underlying London and much of southeast England were deposited. The London Basin is comprised of the following formations, in order of decreasing depth:

- A deep (~200m thick) layer of Chalk, deposited throughout the Upper Cretaceous period, forms the base of the basin and is the principal aquifer of the region.
- The Thanet Beds, which comprise fine, silty glauconitic sands originating in shallow seas.
- The Lambeth Group, a depositionally and geographically complex unit which comprises layers of sands and gravels, shelly and mottled clays, minor limestones and lignites, and occasional sandstone and conglomerate.
- The London Clay Formation, a fine-grained silty clay which is the dominant Thames Group Deposit.
- River Terrace Gravels, deposited by the River Thames and its tributaries on top of the London Clay.

3.2. Site Geology and Anticipated Ground Conditions

Published BGS geological maps indicate that the site is underlain by the London Clay Formation with no superficial deposit present. The Lambeth Beds, Thanet Formation and Chalk are present at greater depths. From a review of available geological maps and memoirs, including the online British Geological Survey “Geology of Britain Viewer”, the following geological sequences were anticipated.

Table 3-1 Anticipated Geological Sequence

Unit	Depth(mbgl)	Thickness (m)	Description
Made Ground	0.00	2.80	Variable anthropogenic deposits.
London Clay	2.80	15.20 proven	Stiff grey/brown mottled fissured clay with pockets of silty sand.

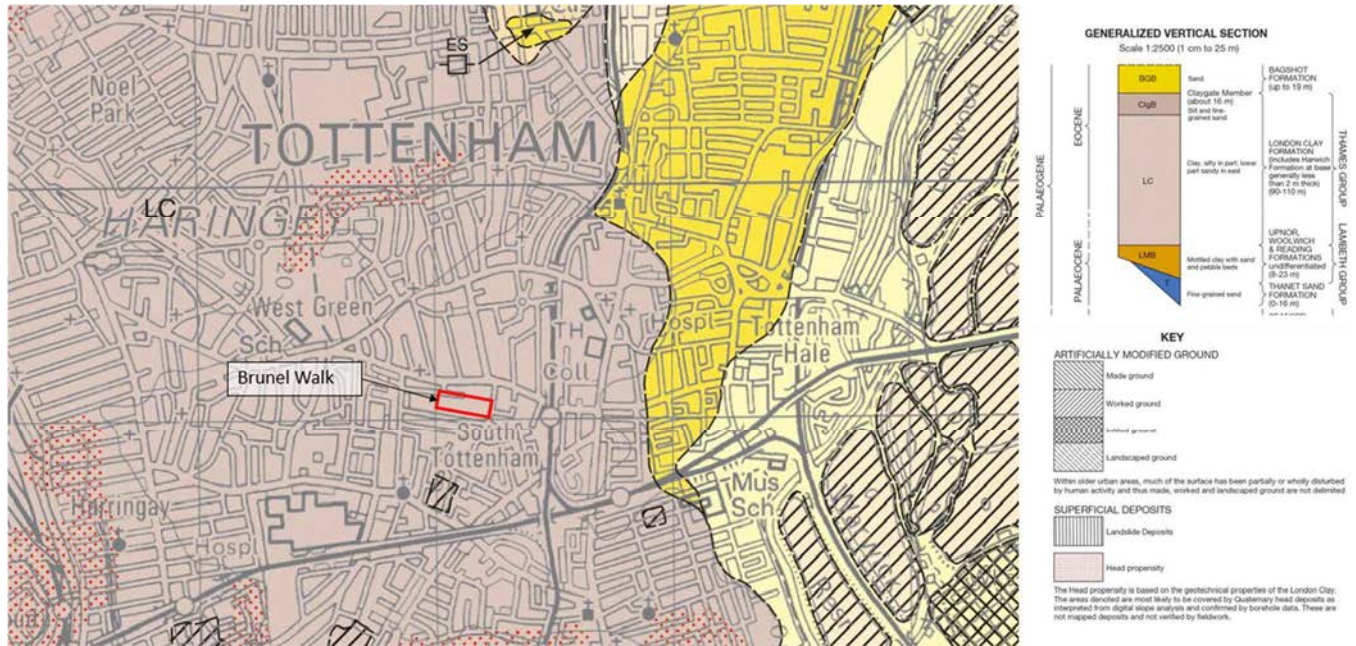


Figure 3-1 Geological context on site



4. Ground Investigation

A site-specific ground investigation was undertaken in November 2021 by A2SI. Details of the ground investigation findings are presented in the *Factual Report* (as referenced in Section 1), which is included as Appendix C.

The primary purpose of the ground investigation works was to manage and mitigate the geo-environmental and geotechnical risk profile associated with the proposed redevelopment of the site and to achieve the geotechnical objectives outlined in Section 1.1. The aims of the ground investigation were thus to:

- Develop and refine the existing ground model regarding the types and spatial extent of soils.
- Assess the engineering performance of the soil deposits to enable recommendations of suitable parameters for geotechnical design.
- Identify potential geotechnical hazards and evaluate risks with regards to both design and construction.
- Review the geo-environmental soil, groundwater and ground gas profile of the site.

The general scope of the investigation is summarised as follows:

- 3 no. restricted access cable percussion boreholes (BH01-BH03) to a maximum depth of 20m to facilitate the collection of soil samples and to install wells for gas and groundwater monitoring.
- 12 no. structural trial pits (TP01-TP12) excavated using hand dug techniques. Trial pits are to expose and record details of existing foundations.
- 8 no. window sample boreholes (WS01-WS08) to facilitate the collection of soil samples and to install wells for gas and groundwater monitoring.
- 6 no. return site visits to monitor gas and groundwater.

The exploratory hole plan of the investigation is presented in Appendix B. The borehole depth of 20m was chosen to capture the anticipated *zone of influence* of the proposed development and prove the primary strata of engineering significance.

Four standpipes were installed for groundwater and ground gas monitoring purposes, as summarised in Table 4-1.

Table 4-1 Groundwater and ground gas monitoring installations

Location	Installation Diameter (mm)	Type of Installation	Depth to Bottom of Response Zone (m)	Depth to Top of Response Zone (m)	Strata
BH02	50mm	SP/G	1.30m	1.00m	Made Ground
WS01	50mm	SP/G	1.30m	0.50m	Made Ground
WS05	50mm	SP/P	1.10m	0.50m	Made Ground
WS07	50mm	SP/G	1.10m	0.50m	Made Ground, London Clay

SP/P – Standpipe piezometer.

SP/G – Standpipe with gas monitoring valve.

A series of in-situ and laboratory geotechnical tests were performed as part of the investigative efforts, including the following:

In-Situ Testing

- 45no. standard penetration tests (SPTs).



Monitoring:

- Groundwater level gauging, and ground gas and soil vapour monitoring.

Geotechnical Laboratory Testing:

- 10no. moisture content tests.
- 10no. classification/index tests.
- 3no. particle size distribution tests.
- 8no. BRE SD1 Suite D tests.
- 13no. unconsolidated undrained (UU) triaxial tests.

Geo-environmental Laboratory Testing (Soil):

- 8no. Asbestos ID (with quantification of asbestos detected).
- 8no. Fraction Organic Carbon (FOC)
- 8no. Soil Organic Matter (SOM)
- 11no. Total Organic Carbon (TOC)
- 8no. Heavy metals and metalloids suite (including hexavalent chromium)
- 3no. Polychlorinated biphenyls (PCB) suite
- 8no. Polycyclic Aromatic Hydrocarbons (PAH).
- 8no. Total Petroleum Hydrocarbons Criteria Working Group (TPH).
- 8no. Water soluble sulphate
- 8no Total sulphur
- 8no. Acid soluble sulphate (SO_4)
- 16no. pH

4.1. Return Monitoring Visits

Six return monitoring visits were undertaken on the 16th and 23rd December 2021, 10th, 19th, 27th and 31st January 2022 as part of the A2SI investigation.

The return visits included ground gas monitoring of the installed boreholes BH02, WS01, WS05 and WS07 using a calibrated Gas Data GFM436 hand-held gas analyser and a calibrated MiniRae Lite ATEX PID. The data collected included ground gas concentrations and ground gas flow rates.

Each return visit also included groundwater level gauging of each of the installed monitoring wells using an oil-water interface probe.



5. Ground Conditions

5.1. Ground Model

A summary of the ground conditions encountered during the intrusive investigation is presented in Table 5-1 below.

Table 5-1 Summary of the encountered geological profile

Unit	Maximum Level Top (mOD)	Minimum Level Top (mOD)	Minimum Depth Top (mbgl)	Maximum Depth Top (mbgl)	Thickness (m)	Description
Made Ground	13.23	10.08	0.00	1.70	0.2 to 1.7	Medium dense orangish brown clayey sandy GRAVEL.
						Soft brown sandy gravelly CLAY with rare inclusions of subangular, fine to coarse gravel sized calcareous nodules.
						Soft to firm orangish brown locally sandy silty gravelly CLAY with occasional decayed rootlets.
London Clay	12.95	10.38	0.2	1.70	>18.5	Firm brownish grey mottled brown fissured silty CLAY with rare flecks of mica and selenite crystal
						Firm to stiff brown mottled grey silty CLAY with frequent thin orangish brown laminae of silt and fine sand, frequent selenite crystals up to 8mm.
						Firm grey mottled orangish brown silty CLAY with frequent pockets (5x5x5mm) to (15x10x5mm) of orangish brown locally yellowish brown fine sand.
						Stiff brownish grey silty sandy CLAY with frequent partings of fine white SAND and abundant locally thin laminae of white SAND

1. Base of London Clay Formation not proven.

Water strike is encountered in only one Trial Pit (TP06) approximately 0.50mbgl, within the London Clay which are likely to be perched water on top of the London Clay within granular lenses of the Made Ground. No water strikes are recorded within any other boreholes, WS and TPs.

5.1.1. Made Ground

The Made Ground is a heterogeneous mixture of natural soils and deleterious materials of anthropogenic origin. Made Ground was encountered in the trial pits, window samples and boreholes to depths of up to 1.7m.

The Made Ground varies across the site. Greater thicknesses of Made Ground were encountered in selected window sample and borehole locations, likely associated with historical foundation construction and associated earthworks.

It is noted that the Made Ground is an uncontrolled fill and is inherently variable in terms of its consistency, characteristics, and engineering properties. This stratum is not recommended to be relied upon to support any engineered structures.

Due to the variability with regards to source and deposition, the assessment of its engineering performance must be undertaken with appropriate caution. The limited thickness of the layer means that the majority will be removed during excavation for substructure components (i.e., slabs, pile caps, foundations, etc.)



5.1.2. London Clay

London clay was encountered in all boreholes, window samples and in four of trial pits (TP03, TP06, TP10, and ,TP11) at minimum depth of 0.2m bgl. The thickness of this material was proven to be up to 18.5m, as reported in Table 5-1. Encountered London Clay Formation was mostly characterised by firm to stiff brownish grey silty clay with pockets of fine sands, more details are present in Table 5-1. Geotechnical in-situ testing on this stratum consisted of 39no. SPT tests, with N_{60} values between 9 and 20 blows/300mm, generally increasing with depth.

Moisture content and Atterberg limit testing has been undertaken on samples of the London Clay. The moisture content for the London Clay ranges from 15% to 38%, liquid limit ranging between 32% to 64% and the plasticity index ranges from 20% to 35%. One sample recorded a value 68%, which is considered an outlier, with soil found to be in the high plasticity range.

13no. unconsolidated undrained triaxial tests were conducted on 100mm-diameter undisturbed London Clay samples. These gave undrained shear strengths ranging from 69kPa to 170kPa. The average dry and bulk density of the samples were approximately 1.5Mg/m^3 and 1.9Mg/m^3 . The bulk and dry density appear to be relatively consistent over the depth of the stratum.

The response to loading of the London Clay can be assessed using *undrained* parameters in the short term and *drained* parameters in the long term, following dissipation of excess pore water pressures.

The London Clay is well known to be heavily overconsolidated as a result of erosion of significant depths of material following deposition. A coefficient of horizontal earth pressure at rest, K_0 , of 1.0 to 1.5 is considered suitable for routine design work, based on past experience and extensive published data. The design of any substructure elements may consider the impact/effect of any given stress paths and installation effects, as appropriate (it is noted that the in-situ *undisturbed* K_0 may be greater than the range given – however it is common to account for disturbance, installation effects and reduction in in-situ pressure at rest conditions as part of retaining wall design, for example).

The London Clay Formation is a competent stratum and is often used to found permanent structural foundations including shallow footings, rafts, piled rafts and pile foundations in London. The formation is considered to provide good conditions for construction, not requiring the use of dewatering systems to enable dry excavation due to its low permeability, and not requiring the use of wet construction methods when carrying out bored piling. Notwithstanding, upper layers of cohesionless superficial deposits will generally require temporary casing.

5.2. Characteristic Geotechnical Parameters

The purpose of this section of the GDR is to describe the salient physical properties of the main geological units that were encountered during the ground investigation works. The information reviewed in this chapter includes stratigraphy, in conjunction with basic physical characteristics (particle grading, density, moisture content and consistency), as evaluated from laboratory testing.

Additionally, this section aims to provide an understanding of the basic characteristics of the various soils deposited at the development site, from which a more detailed understanding of their engineering behaviour and associated risks can be derived.

The geotechnical design of the proposed development will be performed in accordance with the requirements of BS EN 1997 Eurocode 7: Part 1 Geotechnical Design. The selection of geotechnical properties for design should thus represent *characteristic* values, which is defined as that which *represents a cautious estimate of the value affecting the occurrence of the limit state* (BS EN 1997-1 §2.4.5.2(2)P). This definition of the *characteristic* value differs from that for other Eurocodes, which define the characteristic value as being based on a statistical estimate of the 95% probability of occurrence.

The use of limit states thus invokes subtleties into design that must be appreciated by the Geotechnical Designer, not least that a particular soil property (e.g. unit weight), may have multiple *characteristic* values, depending on the structure type and limit state under consideration. For example, when assessing the ultimate bearing capacity of a pad footing, the characteristic value (cautious



estimate) of the unit weight of soil above the founding level may represent a *lower bound* of the measured values. However, in the evaluation of structural forces within an embedded retaining wall an *upper bound* of the measured values may represent a cautious estimate for that particular limit state.

The characteristic values presented herein represent those that are assessed to be most relevant to the types of routine calculations that may be performed, e.g. cautious (lower bound) estimates for strength and stiffness, as they are likely to relate to the design of piled foundations and embedded retaining structures. Notwithstanding this, the Designer may need to evaluate alternative *characteristic* values from the presented to facilitate design in accordance with the Eurocodes.

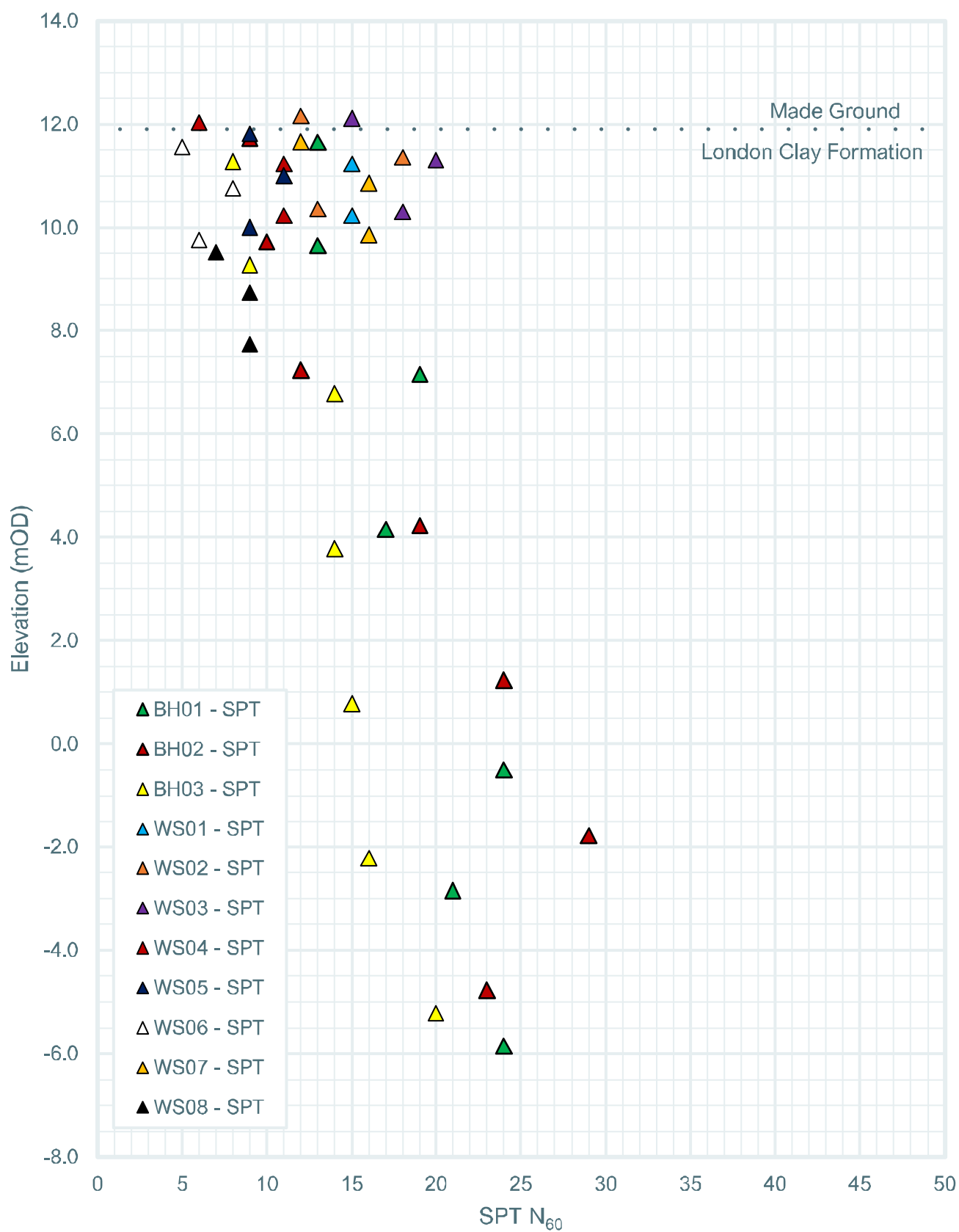
The characteristic geotechnical parameters determined for the main geological units are shown in Table 5-2.

Table 5-2 Characteristic geotechnical parameters adopted for design

Stratum	Top of strata (mOD)	γ_{bulk} (kN/m ³) ^[2]	ϕ'_k (°)	c'_k (kPa)	$c_{u,k}$ (kPa)	E' (MPa)	E_u (MPa)	ν	K_0 ^[11]
Made Ground ^[3]	13.23	18	30	0	-	10.0	-	$\nu' = 0.2$	0.5
London Clay	11.73	19	23 ^[4]	0	$43 + 5.7z^{[10]}$	V: $17.2 + 2.28z^{[10]}$ H: $34.4 + 4.56z^{[10]}$	V: $21.5 + 2.85z^{[10]}$ H: $43 + 5.7z^{[10]}$	$\nu' = 0.2$ $\nu_u = 0.5$ ^[10]	1.2

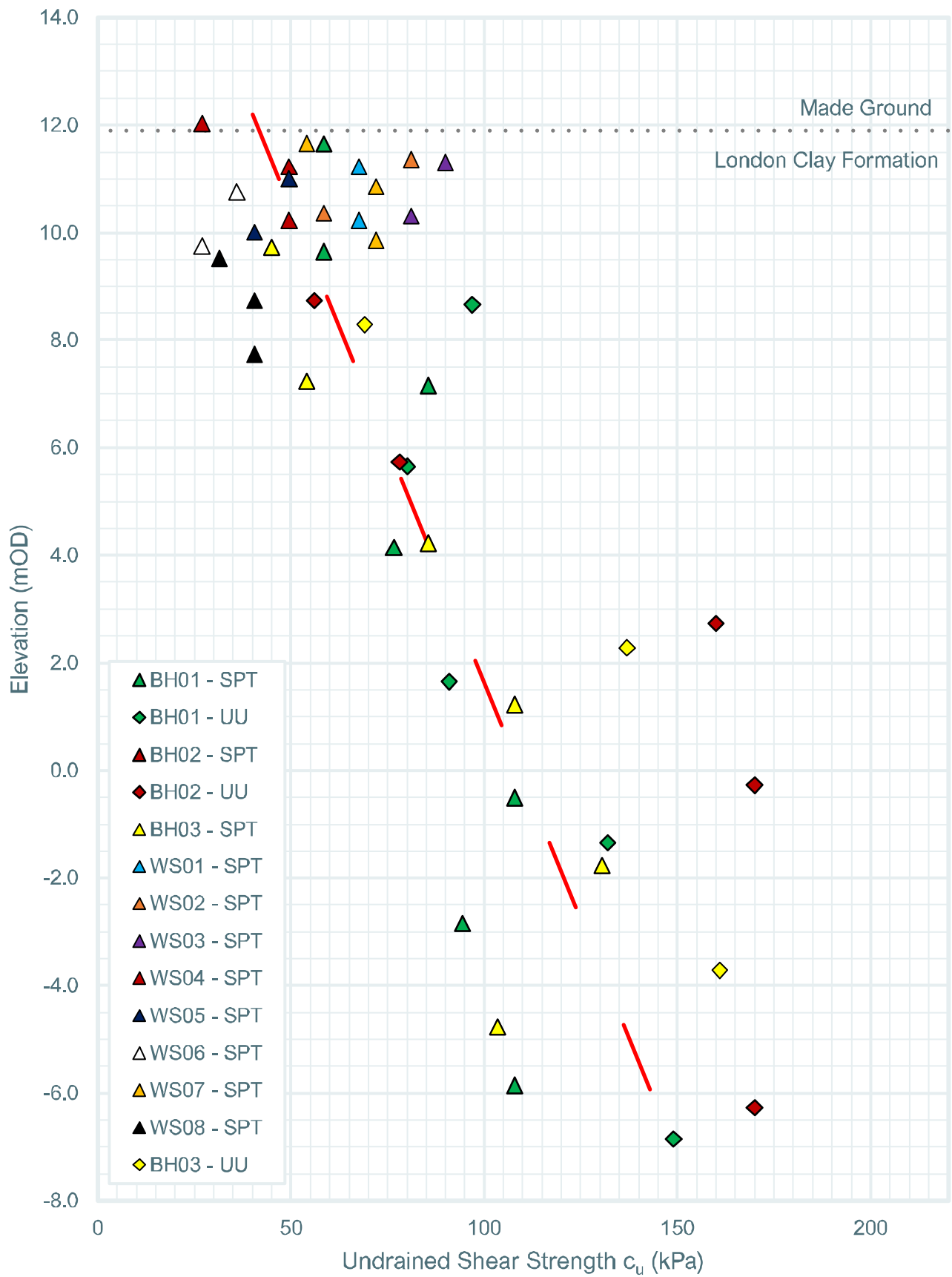
$\gamma_{b,k}$: bulk unit weight ϕ'_k : effective angle of shearing resistance c'_k : effective cohesion $c_{u,k}$: undrained shear strength E' : drained Young's Modulus
 E_u : undrained Young's Modulus ν : Poisson's Ratio K_0 : in-situ lateral earth pressure coefficient V: Vertical H: Horizontal

1. z refers to the depth in metres below the top of the stratum.
2. Bulk unit weights are based on material descriptions and dry and bulk density testing.
3. Moderately conservative geotechnical parameters representative of the variable nature of the Made Ground have been provided based on the material description.
4. Undrained shear strengths, $c_{u,k}$, of the London Clay has been estimated from SPT N_{60} values and unconsolidated undrained (UU) triaxial tests. SPT N_{60} and c_u have been correlated using the ratio $c_u/N_{60} = f_1 = 4.5$ per CIRIA C143. The SPT N_{60} plot and the c_u plot with the adopted design lines are presented in Figure 5-1 and Figure 5-2, respectively.
5. The effective angle of shearing resistance for the London Clay has been calculated from their plasticity index testing results using Equation 7 from BS 8002:2015 Code of practice for earth retaining structures. Per BS 8002:2015 §4.3.1.4.9 c'_k is taken as 0kPa.
6. Drained vertical and horizontal stiffnesses of the London Clay has been estimated using the relationship $E_u/E' = 0.8$.
7. Undrained vertical stiffnesses of the London Clay has been estimated using the relationship $E_u/c_u = 500$.
8. Undrained horizontal stiffnesses of the London Clay has been estimated using the relationship $E_u/c_u = 1000$.
9. Where no horizontal stiffness of a material is provided, the soil is assumed to be isotropic.
10. ν_u is the undrained Poisson's Ratio (no volume change undrained condition).
11. K_0 calculated from $1 - \sin\phi'$ for normally consolidated and lightly overconsolidated materials. K_0 for the London Clay, on overconsolidated nature of the materials.



Approximate boundaries between strata marked by dashed lines.

Figure 5-1 SPT N_{60} results



Strata design lines shown in red.

Figure 5-2 Undrained shear strength, c_u (kPa)



5.3. Groundwater

At the time of writing, groundwater level gauging has been carried out on six occasions on the 16th and 23rd of December 2021, and on the 10th, 19th, 27th and 31st of January 2022. Groundwater was encountered during the monitoring visit within the Made Ground and London Clay Formation stratum. A summary of the groundwater monitoring results is shown below in Table 5-3.

Table 5-3 Groundwater monitoring results

Location	Date	Well base depth (m)	Water depth (m bgl)	Water elevation (mOD)
BH02	16-12-2021	1.30	1.65	11.58
	23-12-2021	1.30	1.64	11.59
	10-01-2022	1.30	1.51	11.72
	19-01-2022	1.30	1.55	11.68
	27-01-2022	1.30	1.54	11.69
	31-01-2022	1.30	1.57	11.66
WS01	16-12-2021	1.30	0.82	12.41
	23-12-2021	1.30	0.86	12.37
	10-01-2022	1.30	0.81	12.42
	19-01-2022		No access	
	27-01-2022		No access	
	31-01-2022	1.30	0.87	12.36
WS05	16-12-2021	1.10	0.97	12.04
	10-01-2022	1.10	0.98	12.17
	16-12-2021	1.10	0.84	12.03
	19-01-2022	1.10	0.86	12.15
	27-01-2022	1.10	0.87	12.14
	31-01-2022	1.10	0.89	12.12
WS07	16-12-2021	1.10	0.83	12.01
	23-12-2021	1.10	0.85	12.12
	10-01-2022	1.10	0.74	11.58
	19-01-2022	1.10	0.76	12.10
	27-01-2022	1.10	0.79	12.07
	31-01-2022	1.10	0.80	12.06



The groundwater table is anticipated to be perched water on top of the London Clay within granular lenses of the Made Ground. It is likely to vary seasonally over the design life of the proposed development, as a result of human induced phenomena and hydrological/hydrogeological features in proximity of the site.

It is recommended that design water table of 1.0mbgl is adopted for foundation and substructure design.

5.4. Visual and Olfactory Evidence of Contamination

During the intrusive investigations, soil sample VOC head-space testing was undertaken using a PID. The PID registered readings did not exceed 1 parts per million (ppm).

Visual and/or olfactory evidence of contamination in soil recorded during the investigations as summarised in Table 5.4.

Table 5.4 Summary of Visual / Olfactory Evidence of Contamination

Borehole	Depth (m)	Strata	Observation	Underwent laboratory analysis
TP08	0.06-1.34	Made Ground	Clinker and Suspected ACM (chrysotile)	Yes
TP09	0-0.75	Made Ground	Clinker	Yes
TP11	0.25-1.0	Made Ground	Clinker	No
WS01	0.2-1.32	Made Ground	Clinker	Yes
WS02	0.25-1.1	Made Ground	Clinker	Yes
WS06	0-0.7	Made Ground	Clinker	No
WS08	0.2-0.3	Made Ground	Clinker	No

It is considered that sufficient soil samples containing the clinker have undergone laboratory analysis for the potential chemical impacts to be captured in the laboratory results.



6. Geotechnical Engineering Design

The design of temporary and permanent structures shall conform with BS EN 1997 Eurocode 7: Part 1 – Geotechnical Design (EC7). EC7 adopts a limit state approach to design, whereupon the safety of the structure is assessed under Ultimate Limit States (ULS) and performance under Serviceability Limit States (SLS) and. Any element of geotechnical design should also consider relevant guidance and industry best practice to supplement compliance with codes.

ULS conditions shall be evaluated such that the Design Resistance, R_d , of the structure/element is equal to or greater than the Design Effect of Actions, E_d .

The Design Resistance is determined in accordance with the requirements of the particular limit state under consideration as defined in EC7. The Design Resistance is evaluated from the Characteristic Resistance of the design element, which has been reduced as specified by the code to allow for uncertainty in the estimation of soil properties and the means and methods of evaluating the ultimate strength.

With regards to geotechnical design of the proposed development, the following ultimate limit states should be verified:

Verification of Strength

- Geotechnical (GEO): GEO assesses the ultimate geotechnical capacity of a design element as it interfaces with and relies upon the ground to maintain stability, for example, the geotechnical capacity of piles subject to axial loading.
- Structural (STR): STR assesses the integrity of structural elements to withstand the internal stresses generated from the application of external loads. With regards to typical geotechnical design works, an example may be reinforcing requirements for an embedded wall.

Verification of Stability

- Uplift (UPL): UPL relates to the assessment of the loss of static equilibrium due to buoyancy effects.
- Hydraulic (HYD): HYD relates to the loss of stability resulting from internal seepage forces such as at the toe of a retaining wall partially embedded in a saturated, permeable stratum.
- Equilibrium (EQU): EQU relates to the global stability of the structure and its equilibrium.
- Other soil-structure interaction and stability mechanisms applicable to the particular structure or engineering challenge under consideration.

Verification of Serviceability

Design of geotechnical and structural elements for the verification of serviceability should conform to the requirements of the British National Annex to Eurocode 7, adopting Design Approach 1. Checks of serviceability should include movements, both absolute and differential, of proposed development itself as well as the impact of ground movements associated with the construction works on surrounding buildings and infrastructure.

6.1. Geotechnical Category

EC7 defines three Geotechnical Categories that relate to the risk associated with a structure (or portion of that structure). Figure 6-1 provides a summary description of the three Geotechnical Categories.



Table 6-1 provides a summary of geotechnical design elements that are anticipated to require consideration as part of the final scheme and their associated Geotechnical Category.

It is assessed that most geotechnical design elements will fall within Geotechnical Category 2 and will thus be suitable for design via routine methods, with appropriate consideration of the site-specific constraints.

GC	Includes ...	Design requirements	Design procedure
1	Small and relatively simple structures ... with negligible risk	Negligible risk of instability or ground movements; ground conditions are 'straightforward'; no excavation below water table (or such excavation is 'straightforward')	Routine design and construction (i.e. execution) methods
No examples given in EN 1997-1			
2	Conventional types of structure and foundation with no exceptional risk or difficult soil or loading conditions	Quantitative geotechnical data and analysis to ensure fundamental requirements are satisfied	Routine field and lab testing Routine design and execution
Examples: spread, raft, and pile foundations; walls and other structures retaining or supporting soil or water; excavations; bridge piers and abutments; embankments and earthworks; ground anchors and other tie-back systems; tunnels in hard, non-fractured rock, not subject to special water-tightness or other requirements			
3	Structures or parts of structures not covered above	Include alternative provisions and rules to those in Eurocode 7	

Reproduced from Bond and Harris, 2008

Figure 6-1 Geotechnical categories

Table 6-1 Geotechnical categories of geotechnical design elements

Geotechnical Design Element	Geotechnical Category
Spread or raft foundations founded on the London Clay	2
Pile design in the London Clay	2

6.2. Codes of Practice for Geotechnical Design

In addition to the Principles and Recommendations described in EC7, the following codes of practice provide non-conflicting (with EC7) guidance regarding the routine design of geotechnical elements (as deemed relevant for the proposed development):

- BS 6031:2015 – Code of practice for earthworks.
- BS 8002:2015 – Code of practice for earth retaining structures.
- BS 8004:2015 – Code of practice for foundations.

In addition to the aforementioned codes of practice, it is recommended that the following guidance document is referred to:

- CIRIA C760, 2017 – Guidance on embedded retaining wall design.



6.3. Temporary Works Design

The design of temporary works shall comply with:

- BS EN 1997: Part 1 – Geotechnical Design.
- PAS8811 (2017): Temporary Works.
- PAS8812 (2017): Temporary Works. Application of European Standards in Design.

6.4. Design Life

A 50-year design life has been assumed.

6.5. Proposed Development and Review of Key Constraints and Risks

The proposed scheme intends to demolish the existing flats, family houses and communal facilities to replace them with four new blocks ranging between three to four storeys. The primary substructure engineering constraints and risks that have been reviewed as part of this assessment are as follows:

1. Interaction of the proposed structures (comprising both temporary and permanent works considerations) with the adjacent properties which fall within the zone of influence of the construction works.
2. Groundwater considerations during construction where seasonal variations in groundwater may occur.
3. Construction programme and assessment of time-dependent movements within cohesive strata (i.e. applicability of undrained/drained assumptions in design).

The following sections provide an overview of potential soil retention and foundation options for the development taking into account the current proposals, site constraints and geological conditions.

6.6. Excavation Works and Retention

6.6.1. Groundwater Cut-off

The groundwater monitoring from the site-specific investigation works indicate that perched water is present approximately 1mbgl. Given that no below-ground space is proposed, groundwater is not expected to be encountered above the formation level of any pile caps or other local excavations. Based on this, it is unlikely that significant dewatering operations will be required to facilitate any earthworks. However, it is recommended that a provision for local sumping or pumping is provided in the instance that finite volumes of groundwater isolated within granular deposits of Made Ground.

6.6.2. Net Loading Overview

6.6.2.1. Heave

The demolition of the existing superstructure elements will introduce an equivalent unloading pressure of approximately 10-12kPa/storey. This will cause the soil within the zone of influence of any existing foundations to heave, resulting in upwards ground movements. The unloading pressures associated with the demolition works will generate negative excess pore pressures in the underlying cohesive materials as they respond to the unloading. The excess pore water pressures will dissipate with time, resulting in long-term heave. Given the heights of the existing buildings are relatively low, the long-term impact of demolition is anticipated to be negligible.

Any local excavations associated with the works required to form foundations elements, pile caps, lift pits, any MEP substructures or similar will introduce further unloading pressures, however given their limited depths and zones of influence, it is unlikely that substantial long-term heave pressures will be induced.



6.6.2.2. Uplift

Given that no basement space is proposed, it is unlikely that UPL mechanisms will be a governing limit state for design.

6.6.3. Earth Retention

6.6.3.1. Overview

Key considerations have been presented herein in order to cater for any substructure design development or future desire for below ground space. The presented considerations may also apply to excavations required to form foundations elements, pile caps, lift pits, any MEP substructures or similar.

6.6.3.2. Proposed Earth Retention Systems

For any deeper basement excavations required, embedded retaining walls may be considered. Typical embedded wall lengths would be of the order of 1.5 to 2 times the retained height, assuming no significant axial loads are applied, and the earth retention system is propped/braced in an appropriate fashion. Propping will likely be required to minimise lateral movements to acceptable levels.

Appropriate care would be required during construction to ensure that the installation effects do not introduce excessive ground movements and potential impact on surrounding structures and assets. The installation effects and construction tolerances will also need to be considered accordingly when specifying this system.

If the embedded retaining wall is required to resist significant axial loading, the length of the secondary piles local to any given column position may be increased to generate the required resistance. The capping beam provision and design will govern load distribution and the number of piles which may be mobilised to resist any axial loading.

Sheet piles may be considered as an alternative means of forming the basement excavation. Predrilling may be required in order to achieve any significant penetration into the London Clay. Jetting should not be permitted to prevent any impact on the London Clay. The viability of sheet piles installation would need to be reviewed with caution on the basis of the basement depth requirements, disturbance of sensitive in-situ materials and considering installation limitations and constraints within congested urban settings.

6.6.3.3. Excavation and General Earthworks Considerations

Where buried obstructions are not encountered, it is anticipated that the proposed excavation may be carried out using standard means and methods. Berms providing support for retaining walls during construction should undergo detailed design to ensure that the berm geometry is safe and does not lead to excessive deformation of the retaining wall.

6.6.3.4. Retaining Wall Design Earth Pressures

Active and passive earth pressures acting on normal to the face of a vertical retaining wall may be calculated using the equations below from BS EN 1997-1:2004 Annex C. The effects of porewater pressure are added to the earth pressures evaluated from the equations:

Active Limit State	$\sigma'_a(z) = K_a \sigma'_v - 2c'\sqrt{K_a}$
Passive Limit State	$\sigma'_p(z) = K_p \sigma'_v + 2c'\sqrt{K_p}$

The vertical effective stress σ'_v acting at a depth z below the ground surface shall include the effects of soil weight and, where appropriate, surcharge loading. Recommended values of active and passive earth pressure coefficients are summarised in Table 6-2 below. Because of the use of partial factors under EC7, earth pressure coefficients are not constant for each limit state. The retaining wall must deflect sufficiently to mobilise the limiting active and passive earth pressures. It is reasonable to assume such conditions



will exist at the ultimate limit state. However, earth pressures under serviceability conditions may represent intermediate conditions between the at-rest condition (K_0) and ultimate limit state.

Table 6-2 Recommended ULS and SLS earth pressure coefficients for retaining wall design

Stratum	ϕ'_k (°)	c'_k (kPa)	δ/ϕ'_k	K_0	EC7 DA1 Combination 1 & SLS		EC7 DA1 Combination 2	
					K_a	K_p	K_a	K_p
Made Ground	30	0	0.67	0.5	0.30	6.10	0.35	4.27
London Clay Formation	23	0	0.67	1.0	0.39	3.52	0.45	2.79

Based on EC7 Annex C.

6.6.3.5. Ultimate Limit State

Any retaining walls should be designed for geotechnical (GEO) and structural (STR) limit states using the Design Approach 1 Combination 1 & 2 partial factors.

Evaluation of the GEO and STR limit states shall consider the effect of over excavation on global stability and structural forces. The overdig allowance should be taken as the lesser of 10% of the retained height, or 0.5m. The design over-excavation depth may be reduced if appropriate construction controls are incorporated into the work method statements for the basement excavation.

For any proposed embedded retention system, the embedded length should be sufficient to mobilise a large enough stabilising passive earth pressure to resist the destabilising active earth pressure. The groundwater level data obtained from the groundwater monitoring visits should be adopted for the design of the embedded retaining walls.

In the long-term, the combination of the concrete basement box liner walls and retaining walls should be checked against the horizontal earth pressure and water table at 1.0m bgl. The ground floor slab can be taken as a “prop” acting on the top of the wall where present.

It is recommended that a minimum surcharge of 10kPa is applied to the retained ground surface for both ULS and SLS analyses. A higher surcharge may be warranted if the retained ground in proximity of the wall:

- Will be used to provide storage/laydown or provide staging areas for heavy plant.
- If there will be a low level of control over the placement of materials.
- Specific surcharging requirements are required such as adjacent neighbouring building and party wall.

6.6.3.6. Serviceability Limit State

Deflection limits should be set by the engineer within the retaining wall design and construction specifications. Deflection limits may be required to ensure:

- Construction tolerances on cavity wall/liner walls are not adversely affected as a result of excessive deformation.
- That the risk of damaging existing structural elements and surrounding third party assets/infrastructure is mitigated.

The deflection limits should be set to define:

- Maximum ground displacements that may occur during installation of the retaining wall.
- Maximum horizontal displacement of the retaining wall during construction and operation.
- If required as part of a ground movement assessment, maximum displacements/rotations of neighbouring building facades, hardstanding and/or significant utilities.



6.6.3.7. Other Considerations

- The horizontal earth pressure coefficients shown in Table 6-2 are dependent on the stiffness of any retention system and the earth pressure coefficient adopted for structural design purposes should take cognisance of the compaction effort and relative stiffness of the wall. The deformation of the system allows the mobilisation of the active earth pressures. High stiffness retention systems do not allow as much mobilisation of active earth pressure as softer systems, leading to horizontal earth pressures between K_0 (no mobilisation or stress relief) and K_a (full mobilisation) and higher design bending moments and shear forces. However softer systems will allow more horizontal ground movements as they deform, leading to changes in the design bending moment and shear force profiles.
- The overall lateral wall movements and deformation of the ground caused by any excavation works may be reduced through additional propping and a staged excavation sequence. Any props will require appropriate design to resist induced axial forces, and the additional stiffening of any retention system will likely increase the overall design bending moments and shear forces.
- The design guidance provided above is based on the groundwater levels from the monitoring results from the ground investigation. However, it is recommended that the water level is checked, and any retention system design updated if required, prior to the commencement of on-site works as the groundwater level can vary seasonally.
- The vertical loads acting on any retaining walls will cause the retention system to settle, which will influence the wall friction on the active side of the wall. Consideration should be given to how vertical loads on the wall are carried, as reduced wall friction may result in an increase in the horizontal earth pressure coefficient.
- Specialist contractors should be consulted in relation to construction means and methods (including site logistics constraints) considering the urban setting and congested environ.

6.7. Pile Foundations

6.7.1. Ultimate Limit State

The geotechnical capacity of a single pile has been evaluated by the Method of Calculation, as defined in EC7. The design geotechnical pile resistance, R_d , may be assessed as the sum of the characteristic shaft ($R_{s,k}$) and base ($R_{b,k}$) resistance reduced by appropriate partial factors (γ), as shown in the equation below:

$$R_d = R_{s,d} + R_{b,k} = \frac{R_{s,k}}{\gamma_s \gamma_{Rd}} + \frac{R_{b,k}}{\gamma_s \gamma_{Rd}}$$

The evaluation of unit shaft resistance, f_s , and unit base resistance, f_b , is based on the following equations:

Cohesive Soils (α -method)	$f_s = \alpha C_u$ $f_b = N_c C_u$
---------------------------------------	---------------------------------------

α : refers to the soil adhesion factor.

K_s : earth pressure coefficient for pile interface friction, δ : angle of interface friction.

N_c refer to bearing coefficients.

Equivalent safe working loads piles in general accordance with EC7 and BS 8004:2015 have been provided as part of this GDR. The methodology adopted to determine the safe working loads is presented in Appendix A.

Table 6-3 summarises the recommended pile design parameters to evaluate unit shaft and base resistance of the anticipated materials.



Table 6-3 Recommended pile design parameters

Stratum	Method	α	K_s	δ/φ'	N_c	N_q
Made Ground			Not used			
London Clay	α -method	0.5	-	-	9.0	-

Table 6-4 below provides an overview of compressive and tensile equivalent safe working loads for piles at varying depths below the basement formation level, calculated in accordance with EC7 and non-conflicting guidance.

Table 6-4 Pile equivalent safe working loads (kN) in axial compression

Length (m)		5.0	10.0	15.0
Pile Diameter (mm)	300	50	130	245
	450	85	210	385
	600	125	300	540

1. Pile capacities calculated using EC7 NA Design Approach 1 Combination 2 partial factors.
2. Serviceability limit state design partial factors in accordance with BS 8004:2015.
3. Length taken from the top of the pile at ground level (13,23mOD).
4. Diameters are tool diameters.
5. Lateral pile loading has not been considered in the provided capacities.
6. Long-term water table adopted.
7. GEO evaluation only. STR verification to be completed in accordance with BS EN 1992.
8. A 60% and 40% split for Dead Load and Live Load respectively has been assumed for the pile capacities calculation using EC7.

6.7.2. Serviceability Limit State

The performance of individual piles under working load tests should be defined as part of the piling works specification. The settlement of individual piles under working loads are typically limited to 0.5% to 1.0% of the pile diameter. Group effects between any potential closely spaced pile foundations would result in greater settlements.

6.8. Shallow Foundations

The viability of both ground-bearing raft and discrete pad/strip footing foundations systems have been reviewed on the basis of ultimate limit states (ULS) and serviceability limit states (SLS). The design of shallow foundations is generally governed by serviceability considerations, such as limiting differential settlements between loading positions, to avoid excessive distortions of the superstructure frame and damage to surrounding structures. This is of particular relevance where significant overburden is removed as a result of basement excavation.

The bearing pressure capacity of a pad foundation has been evaluated on guidance from EC7. ULS Bearing capacity is computed from three different pad sizes and their relative settlements. The reported Bearing Capacities indicates Safe Working Load (SWL) capacities based on an assumed 60/40 Dead Load/Live Load split. The allowable bearing capacity is 200 kPa, based on a foundation founded at 1.5mbgl on the top of the London Clay. The corresponding predicted settlements are between 15-25mm, for square footing ranging between 1.5m to 3.0m wide.



6.9. Concrete Aggressivity

8no. samples were taken for chemical testing from the London Clay. The characteristic sulphate values and resulting design sulphate and aggressive chemical environment for concrete classes are shown below.

Table 6-5 Concrete aggressivity assessment

Stratum	No. of samples	Water Soluble Sulphate (mg/l)	Total Potential Sulphate (%)	pH value	Design Sulphate (DS) Class	Aggressive Chemical Environment for Concrete (ACEC) Class
London Clay	8	2600	3	7.7	DS-5	AC-5s

Based on the above, the Design Sulphate Class for London Clay Formation is DS-5 and the corresponding ACEC Class is AC-5s for mobile water, being based on one sample (WS07) which presents a significant quantity of oxidisable sulphides higher than 0.3%. However, if the London Clay is protected during construction such that it will not be subject to significant disturbance or exposure to oxygen over extended periods of time, this may be reduced to DS-3 and AC-3s for selected structures and construction means and methods, subject to confirmation by the geotechnical engineer. The amount of oxidisable sulphides is lower than 0.3% SO₄ in 7 out of 8 samples, therefore pyrite is not likely to be present across extensive parts of the site.

6.10. Shrinkage and swelling characteristics of the soil

Change in volume is directly related to changes in loading conditions and the fluctuation of moisture content in the soils. The resulting shrinkage or swelling cause subsidence or heave damage to foundations, the supported structure and utilities. As reported in section 5.1.2 the Plasticity Index of the London Clay is in the range of 20% to 35% (one sample with value 68%) which indicates a medium volume change potential.

6.11. Design CBR values for pavement construction.

In the absence of CBR tests, the CBR values have been obtained via the correlation between the Plasticity Index and CBR values. For the London Clay the PI varies from ranges from 20% to 35%. Based on this, the equivalent CBR value is between 3.5% to 5%.

6.12. Other Risks/Further Considerations

Further ground engineering considerations are summarised below:

- **Below ground obstructions.** Obstructions were noted in the ground investigation, and the project team should consider the presence of potential below ground obstructions across the site (natural and anthropogenic), particularly in relation to historical foundations.
- **Thickness of Made Ground.** Additional thicknesses of Made Ground and/or soft spots may be present across the site. Provisions should be made in the scheme earthworks specification for testing and inspection of substructure formations by a suitably competent groundworks engineer. This risk should be considered appropriately by the wider project team as the scheme and design proposals progress. #
- **Aggressive ground conditions.** One sample taken from the London Clay indicates a significant quantity of Total Potential Sulphate is present, leading to a high Design Sulphate Class (DS-5, AC5). However, if the London Clay is protected during construction such that it will not be subject to significant disturbance or exposure to oxygen over extended periods of time, this may be reduced to DS-3 and AC-3s for selected structures and construction means and methods, subject to confirmation by the geotechnical engineer.



- **Groundwater ingress:** Local dewatering is anticipated to be required for the proposed development works, and it is suggested that appropriate provisions for nominal dewatering via sumps and pumps are made with regards to construction means and methods, temporary works and groundwater control. Any requirements for dewatering and the subsequent ground movement should be considered as part of the design proposals.
- **Site logistics and construction means and methods.** Specialist contractor advice should be sought in relation to site access, logistics and any plant limitations and constraints. The presence of the DLR viaduct should also be considered alongside any restrictions that may apply to plant working in close proximity to live rail assets.
- **Surrounding buildings and third-party assets:** a ground movement assessment, looking at the impact of the proposed works on surrounding buildings, may be required to assist with planning approvals, Party Wall liaison and to facilitate detailed design of the proposed scheme. The risk of damage to existing buildings due to ground movements associated with construction and operation of the proposed building is generally submitted to the Local Authority for review of planning consent. The assessment of risk to buildings is usually based on Burland's Damage Scale. Ground movements may also impact buried services/utilities, substations (present on site) and further ground movements assessments will be required for these assets.



7. Supervision and Monitoring

7.1. Introduction

A supervision and monitoring plan will be required to support the management of geotechnical risks during construction.

The supervision and monitoring plan should state (reproduced from EC7):

- The purpose of each set of observations or measurements.
- The parts of the structure, which are to be monitored and the locations at which observations are to be made.
- The frequency with which readings are to be taken.
- The ways in which the results are to be evaluated.
- The range of values within which the results are to be expected.
- The period of time for which monitoring is to continue after construction is complete.
- The parties responsible for making measurements and observations, for interpreting the results obtained and maintaining the instruments.

An Action Plan document and procedure should be developed in order to present the excavation and construction performance criteria, alongside agreed trigger levels for the primary phases of construction (in order to safeguard adjacent assets).

The development of the Action Plan will need to meet the design assurance requirements of all relevant stakeholders. The Action Plan should embody input from the design and construction teams and should include appropriate mitigation measures should any aspect of the substructure performance not be met.

7.2. Preliminary Recommendations

The supervision and monitoring plan will require further development as the scheme and detailed design of the structure/substructure are progressed. Table 7-1 provides preliminary recommendations regarding monitoring and supervision requirements for the proposed development.

Table 7-1 Preliminary monitoring and supervision requirements

Item	Monitoring/supervision description	Measurement
1	Monitoring of the adjacent buildings, assets and infrastructure during the progression of the works in order to satisfy any potential Party Wall agreements and design assurance criteria.	X, Y, Z displacements
3	Monitoring of groundwater levels during construction works.	Groundwater elevation
4	Supervision of ground works by suitably experienced contractor.	-



8. Quantitative Risk Assessment (Geo-environmental)

The following section provides a detailed assessment of the available information including data gathered from the recent A2SI ground investigation. The CSM is then developed from that presented in the Soil Consultants Phase I (summarised in the earlier sections of this Report). This section comprises GQRA. The assessments in this section have been undertaken to assess potential land contamination issues with respect to the specific proposed development.

It is considered that risks to site workers and the environment during the construction phase of the proposed redevelopment can be appropriately managed by successful implementation of construction phase risk assessments and method statements (RAMS). The associated construction phase risks from potential contamination are not considered further in this document but should be appropriately considered and mitigated by the Principal Contractor in their preparation and implementation of construction phase RAMS and Construction Phase Plan (CPP).

8.1. Human Health Risk Assessment (Dermal Contact, Ingestion and Inhalation of Soil)

The soil sample laboratory analytical results have been compared to generic assessment criteria (GAC) considered appropriate for the assessing the risks to the specific proposed development. The selected human health GAC include the LQM/CIEH 'Suitable 4 Use Levels' (S4ULs). The S4ULs are based on Health Criteria Values that represent minimal or tolerable levels of risks to health as described in the Environment Agency's SR2 guidance.

For each chemical substance, S4ULs include individual GAC for six generic land uses (residential with home grown produce, residential without home grown produce, allotments, commercial and two Public Open Space land uses) and a range of Soil Organic Matter (SOM) contents. All toxicological and physical-chemical parameters used in the derivation of the S4ULs are presented and discussed in the source publication.

In some instances, selected human health GAC used in this Report have been applied from the DEFRA Category 4 Screening Levels (C4SLs), CL:AIRE GAC and the Environment Agency (EA) Soil Guideline Values. The source reference used for the human health GAC for each chemical determined is presented in the screening tables included in Appendix E. When available for a chemical compound, C4SLs have been used preferentially.

The generic land use scenarios used for selecting GAC are 'residential without homegrown produce'. This scenario was selected as the exposure assumptions best represent the proposed site end-use for residential buildings with soft landscaped areas. The scenarios are also appropriate for the existing residential land-uses located immediately adjacent to the site.

GAC have been derived for SOM values of 1%, 2.5% and 6%. Site specific SOM values for shallow soils were determined from selected samples collected during the site investigation. SOM varied from 0.5 % to 5.1 %. Therefore, GAC derived assuming 1.0 % SOM have been used in this assessment on a conservative basis.

There is no published human health GAC with respect to asbestos or ACMs in soil. Industry best practice document '*Asbestos in soil and Made Ground: a guide to understanding and managing risks*', CIRIA C733, 2014, identifies that soils containing asbestos concentrations of 0.001 % w/w may be able to liberate airborne fibre concentrations that exceed the contemporary occupational exposure limit for nuisance dust. However, as detailed in other research, including publications such as the *CAR-SOIL Industry Guidance (2016)*, in circumstances where very low concentrations of asbestos are identified in soils, the associated risks are considered low. In this study A2SI adopt an asbestos human health GAC of <0.001 % w/w i.e. mitigation or further assessment is required if asbestos in soil is detected at or above <0.001 % w/w.

Screening tables comparing the soil laboratory results to the selected human health GAC are provided in Appendix E. All soil samples which underwent laboratory analysis for geo-environmental purposes were collected from Made Ground. Where laboratory method detection limits are greater than the human health GAC (if any) this is not recorded as an exceedance.



The proposed development does not include basements. It is noted that the proposed site landscape development plans include some areas of soft landscaping. Therefore, there is potential for on-site soils to be exposed to proposed site users in these areas.

Individual laboratory results are presented within the Factual Report in Appendix C. Screening tables comparing the laboratory analytical results for soils to the selected human health GAC are provided in Appendix E.

None of the chemical concentrations detected in the soil samples collected exceed the selected 'residential without homegrown produce' human health GAC scenario. However, suspected ACM (chrysotile) has been observed in TP08 associated with the Made Ground at this location (a suspected visible fragment was identified rather than concentrations detected by laboratory analysis). Therefore, it is recommended that Made Ground in the vicinity of TP08 is further investigated to identify whether ACM fragments are actually present here, and if they are over what area and depth they extend within Made Ground. Due to the possible present of ACMs in Made Ground the risk to on-site and off-site human health is still considered 'unacceptable' (ref. *LCRM*) at this stage due to the inhalation pathway.

An on-site source of contamination which can migrate laterally at shallow depth within the ground and impact nearby shallow ground has not been identified

8.2. Human Health and Building and Structures Risk Assessment (Ground Gas / Soil Vapour)

8.2.1. Ground Gas

During the A2SI investigation, ground gas monitoring wells were installed within the Made Ground at WS01, WS05, WS07 and BH02 to monitor the potential ground gas beneath the site. A summary of the monitoring well construction details is provided in Section 4. The equipment and processes which have been implemented are summarised in Section 4.1. A Made Ground thickness plan can be observed in Figure 8.1a and Figure 8.1b.

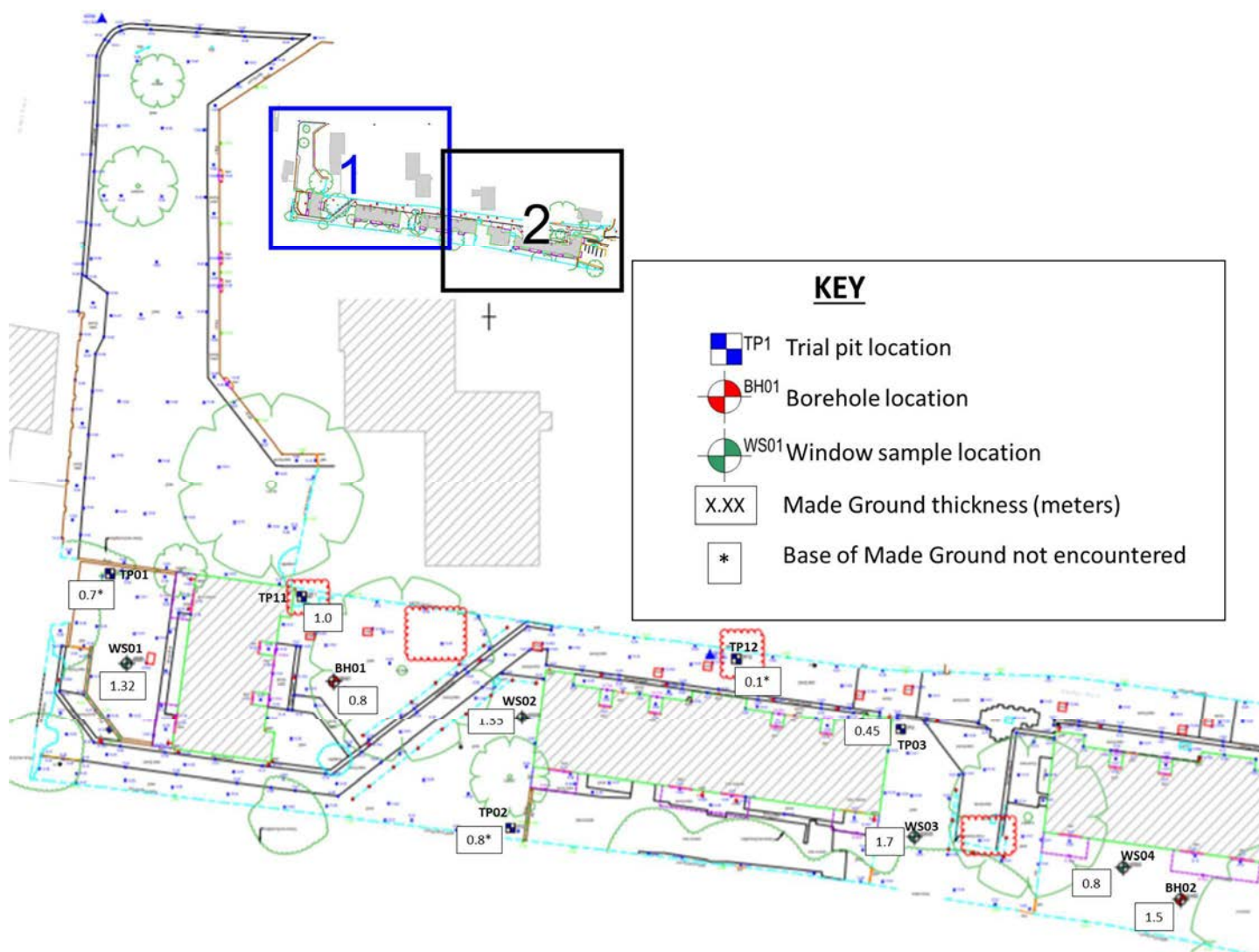


Figure 8.1a Made Ground thickness plan for the west of the site



Figure 8.1b Made Ground thickness plan for the east of the site

The ground gas assessment presented herein has been undertaken in general accordance with guidance contained within CIRIA 665, 'Assessing risks posed by hazardous ground gases to buildings' and BS8485:2015+A1:2019 Code of practice for the design of protective measures for methane and carbon dioxide ground gases for new buildings. The method requires use of both gas concentrations and ground gas well flow rates to calculate a gas screening value (GSV). The GSV is calculated as follows:

$$\text{GSV} = \frac{\text{Analyte Concentration (\%)} \times \text{Flow rate (L.hr}^{-1}\text{)}}{100}$$

The calculation is carried out for methane (CH₄) and carbon dioxide (CO₂).

Six rounds of ground gas monitoring have been undertaken. Each round of monitoring also included monitoring of all wells for VOCs. The results of the ground gas monitoring and summarised in Table 8.1.

BS8485 guidance utilises the GSV and categorises the ground gas risk into six different hazard potentials, referred to as Characteristic Situation 1 - 6 (CS1 – CS6). These are summarised in Figure 8.2.

CS	Hazard potential	Site characteristic GSV ^{A)} L/h	Additional factors
CS1	Very low	<0.07	Typically <1% methane concentration and <5% carbon dioxide concentration (otherwise consider an increase to CS2)
CS2	Low	0.07 to <0.7	Typical measured flow rate <70 L/h (otherwise consider an increase to CS3)
CS3	Moderate	0.7 to <3.5	–
CS4	Moderate to high	3.5 to <15	–
CS5	High	15 to <70	–
CS6	Very high	>70	–

^{A)} The figures used in this column are empirical.

NOTE The CS is equivalent to the characteristic GSV in CIRIA C665 [6].



Figure 8.2 CS vs. GSV (ref. *BS8485*)

Table 8.1 Ground Gas Monitoring Summary Data

Exploratory Hole Reference	Monitoring Round Date	Minimum O ₂ (%)	Maximum CO ₂ (%)	Maximum CH ₄ (%)	Maximum CH ₄ (%LEL)	Maximum H ₂ S (ppm)	Maximum CO (ppm)	Flow Rate (l/h)	Maximum PID (ppm)	Barometric Pressure (mb)
BH02	16/12/21	Not recorded								1033
	23/12/21	18.4	1.3	<0.1	<0.1	<1	<1	<0.1	1.6	1003
	10/01/22	17.9	1.8	<0.1	2.6	<1	<1	<0.1	1.4	1018
	19/01/22	17.5	1.6	0.1	2.6	<1	<1	<0.1	1.3	1025
	27/01/22	17.6	1.4	0.2	3.0	<1	<1	<0.1	1.2	1011
	31/01/22	18.0	1.2	<0.1	<0.1	<1	<1	<0.1	0.8	1021
WS01	16/12/21	19.3	0.7	<0.1	<0.1	<1	<1	<0.1	1.2	1033
	23/12/21	18.1	1.0	<0.1	<0.1	<1	<1	<0.1	1.7	1005
	10/01/22	19.3	0.9	<0.1	3.5	<1	<1	<0.1	1.1	1018
	19/01/22	No access								
	27/01/22	No access								
	31/01/22	19.8	0.5	<0.1	<0.1	<1	<1	<0.1	1.0	1021
WS05	16/12/21	19.4	0.8	<0.1	<0.1	<1	<1	<0.1	1.2	1033
	23/12/21	18.8	1.3	<0.1	<0.1	<1	<1	<0.1	2.3	1005
	10/01/22	19.3	3.1	<0.1	1.7	<1	<1	<0.1	2.4	1018
	19/01/22	17.7	2.6	0.1	1.7	<1	<1	<0.1	1.7	1025
	27/01/22	17.4	2.6	0.1	1.7	<1	<1	<0.1	1.9	1011
	31/01/22	18.4	1.7	<0.1	<0.1	<1	<1	<0.1	1.2	1021
WS07	16/12/21	19.8	0.8	<0.1	<0.1	<1	<1	<0.1	1.4	1034
	23/12/21	19.5	0.9	<0.1	<0.1	<1	<1	<0.1	2.4	1005
	10/01/21	14.3	2.4	0.1	3.0	<1	<1	<0.1	3.2	1018
	19/01/22	14.5	2.4	0.2	3.4	<1	<1	<0.1	2.5	1025
	27/01/22	14.8	2.1	0.1	2.6	<1	<1	<0.1	2.1	1011
	31/01/22	15.3	1.6	<0.1	<0.1	<1	<1	<0.1	2.0	1021



GSV have been calculated to define the gas regime at the site as per BS8485. The measured worst-case parameters across all wells have been adopted for the calculation on a conservative basis. The GSV for carbon dioxide and methane have been calculated using the maximum concentrations of carbon dioxide (3.1%) and methane (0.2%) detected during the return monitoring visits. The maximum detected steady gas flow rate (0.1 l/hr) has also been used in the calculation. Where parameter values are not recorded above the equipment detection limit then the limit of detection has been assumed for the calculation. The calculated GSV for CO₂ and CH₄ are:

- Carbon dioxide: 0.0031 l/hr
- Methane: 0.0002 l/hr

8.2.2. Soil Vapour

No concentrations of volatile contaminants have been identified in on-soils above the selected human health GAC. Furthermore, none have been detected above laboratory method detection limit. The risks to on- and off-site human health from a potential on-site source are considered to be low based on the absence of identified volatile contamination at the site.

During the return monitoring rounds, VOCs were recorded in the monitoring wells at a maximum concentration of 3.2ppm (WS07 during round three). VOC monitoring enables investigation of potential volatile vapours migrating onto the site from off-site. The detected concentrations are possibly a result of soil moisture affects rather than indicative of anthropogenic contamination. However, this is only a provisional characterisation as the monitoring rounds do not capture a low atmospheric pressure event (so a 'worst case' scenario has not been captured in the data). For this reason, it is recommended that further monitoring is undertaken during low atmospheric pressure conditions to provide a more robust dataset before the soil vapour risk assessment for potential off-site sources is finalised.

8.3. Buildings and Structures Risk Assessment

No gross contamination which might represent an unacceptable risk to the structural integrity of the proposed development and off-site buildings via direct contact has been encountered. Section 6.9 of this report states the design sulphate class that should be used on site.

8.4. Human Health Risk Assessment (Drinking Water Supply Pipes)

The soil sample geo-environmental laboratory results have been compared to assessment criteria presented in UKWIR "Guidance for the selection of Water Supply Pipes to be used in Brownfield Sites (Ref 10/WM/03/21)". Only relevant assessment criteria for the available chemical results and potential contaminants of concern identified in the Phase I Desk Study have been used. The results are summarised in Table 8.3.

Table 8.3 Water Supply Pipes Assessment Summary

Determinans	Polyethylene (PE) Pipe Assessment Criteria (mg / kg)	Polyvinylchloride (PVC) Pipe Assessment Criteria (mg / kg)	Maximum Concentration Detected in Soil (mg / kg)
VOC	0.5	0.125	<0.1
Total BTEX and MTBE	0.1	0.03	<0.01 (all samples)
SVOCs and C5-10 aliphatic / aromatic hydrocarbons	2	1.4	<0.1(all samples)



Determinans	Polyethylene (PE) Pipe Assessment Criteria (mg / kg)	Polyvinylchloride (PVC) Pipe Assessment Criteria (mg / kg)	Maximum Concentration Detected in Soil (mg / kg)
C11-20 aliphatic / aromatic hydrocarbons	10	Pass	16.4 (Aliphatic and aromatic compound range in WS06 at 1.2m bgl)
C21-40 aliphatic / aromatic hydrocarbons	500	Pass	282.9 (Aliphatic and aromatic compound range in WS06 at 1.2m bgl)

A single sample taken from WS06 exceeded the criteria for PE pipe from C11-C20 aliphatic and aromatic compounds. No contaminants exceed the respective criteria for PVC pipe. Therefore, it is considered that the results indicate PVC specification pipe would be suitable where new water supply pipes are laid in Made Ground.

It should be noted that the final pipe specification to be installed at the site should be agreed with the utility provider.

An on-site source of contamination which can migrate laterally at shallow depth and impact nearby off-site water supply pipes has not been identified.

8.5. Proposed Flora and Fauna

The soil sample laboratory analytical results have been screened against generic assessment criteria considered protective of adverse phytotoxic effects. The source reference for the assessment criteria is *BS3882:2015 Specification for Topsoil*. A summary screening table is provided as Table 8.4. As the detected soil pH ranges from 7.6-8.5, the assessment criteria have been selected from 'criteria for soil with pH >7'.

Table 8.4 – Phytotoxic Metals Screening Criteria Table

Determinand	Minimum Concentration (mg/kg)	Maximum Concentration (mg/kg)	Criteria for soil with pH >7 (mg/kg)
Zinc	68.5	162	300
Nickle	22.2	52.6	110
Copper	29.6	59.0	200

The results indicate that none of the phytotoxic metal exceed their respective criteria therefore, the results are not considered to represent an unacceptable risk if Made Ground is incorporated into soft-landscaping areas at the proposed development as soft-landscaping will not include any sensitive ecological receptors relevant in the context of the Environmental Protection Act 1990, Part 2a (as amended).