The Trustees of the St Pancras Way Block A Unit Trust & Big Lobster

The Ugly Brown Building

Geoenvironmental and Geotechnical Site Investigation

371654-01 (01)

AUGUST 2019

RSK GENERAL NOTES

Project No.: 371654-01 (00)

Title: Geoenvironmental and Geotechnical Site Investigation : The Ugly Brown Building, 2 - 6 St Pancras Way, NW1 0TB

Client: The Trustees of the St Pancras Way Block A Unit Trust & Big Lobster Ltd. c/o Jason Russel of Reef Group, 51 Welbeck Street, London, W1G 9HL

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Office: RSK Environment Ltd, 18 Frogmore Road, Hemel Hempstead, Herts, HP3 9RT tel:44 01442 437500

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Author	Mike McCann	**Technical reviewer**	Jonathan Bailey / Alena Landers
Signature		Signature	
Date:	09/08/2019	Date:	09/08/2019

Project manager	Claire Siberry	**Quality reviewer**	Linda York
Signature		Signature	
Date:	09/08/2019	Date:	09/08/2019

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1 INTRODUCTION

1.1 Commissioning

On the instruction of GD Partnership and on behalf of Reef Group c/o the Trustees of the St Pancras Way Block A Unit Trust & Big Lobster Limited (the 'client'), RSK Environment Limited (RSK) was commissioned to carry out a Geoenvironmental and Geotechnical Site Investigation of the land at 2 - 6 St Pancras Way, London NW1 0TB. The project was carried out to an agreed brief as set out in RSK's proposal (371654-T01 (06)).

This report is subject to the RSK service constraints given in **Appendix A** and limitations that may be described through this document.

1.2 Proposed redevelopment

The proposed redevelopment will involve the demolition of the existing Ugly Brown Building (UBB) and erection of six new buildings ranging in height from two storeys to twelve storeys above ground and up to two basement levels comprising a mixed-use business floorspace, residential, hotel, gym, flexible retail and storage space development with associated limited landscaping work. The new development comprises three plots (A, B & C) in which 'Plot A' will be offices, 'Plot B' will be the Ted Baker hotel & headquarters, and 'Plot C' will comprise four major buildings for mixed-use offices, gym, residential & retail spaces. The proposed basement level at the site varies from 13.40m to 19.10m AOD while the canal water level is at approximately 23m AOD and canal bed is understood to be at an average 21.15m AOD. Plot A will have a single basement at 17.50m and 18.00m AOD. Plot B will also have a single basement but at two different levels: the deeper, double height area adjacent to St Pancras way at 16.30m AOD, and along the canal edge at 19.10m AOD. Plot C will have two basements with lower basement level at 13.40m AOD.

None of the existing foundations, which comprise piled foundations, are to be reused as part of the new scheme, with the exception of part of the piles spanning either side of the Thames Water Sewer beneath the southern part of Plot A which will be retained and included in the new scheme, and possibly the existing contiguous pile wall on the northern site boundary. However, it is understood the latter is under discussion at the time of this report.

The planned layout of the site is shown in **Appendix B**.

The redevelopment will be divided into three phases; the demolition and construction of Block A, followed by Block B and finally Block C.

1.3 Project background

Historically the site was occupied by a five-storey masonry structure called 'St Pancras Ale & Corn store', which later became known as the Granary. Based on the archive IStructE Papers provided by GD Partnership, the former granary building was founded upon a concrete raft foundation that was placed directly upon London Clay by excavating approximately 6m below the canal water level. This was confirmed within nine borehole records which indicated that the hardstanding of the former granary building was underlain by approximately 20m of London Clay that is in turn underlain by clay of the ‘Woolwich and Reading Beds’. The IStructE Papers also noted the 750mm thick hydraulic concrete lime raft supported 225mm thick massive sandstone blocks on a 4.2m grid, with 1.20m of clay infill placed around and above the sandstone blocks and upon which a brick sett was placed. Standing water was encountered during the site investigation for the Royal Mail sorting office within the brown London Clay on the underside of the concrete raft.

It is understood from information provided by GD Partnership Ltd that the middle level sewer No. 2, a part of London’s historic sewer system and now part of the Thames Water Authority’s system, was constructed by tunnelling across the northern part of the site. The Thames Water service plan indicates the sewer is approximately 2m in diameter and is brick lined throughout, with its crown approximately 4.50m below the canal bed level at an approximate level of 15.60m AOD. The southern end of the administration block of UBB has been built bridging over the sewer with contiguous piles outside the easement area.

A Preliminary Basement Impact Assessment (BIA) was issued by RSK dated 25th October 2017 (report ref. 371654 L01 (01) which assessed potential damage to various neighbouring properties and assets belonging to third parties during the three stages of demolition and construction of the proposed development. Geotechnical parameters were obtained from surrounding data previously acquired through RSK ground investigations, BGS records and IStructE drawings provided by GD Partnership.

A Phase 1 Desk Study report issued by DTS Raeburn Ltd (report ref. E12897/1 dated June 2017) has been made available to RSK which enabled the design of the Phase II Geoenvironmental and Geotechnical Investigation. The study outlined the potential risk of Unexploded Ordnance (UXO) within Block A.

1.4 Objectives

The investigation targets the developments associated with Block A and B only. The objective of the work is:

- to provide geotechnical information to aid in foundation design and other geotechnical matters pertaining to the proposed development at Block A and B.
- to identify any land contamination and/or geotechnical constraints to the proposed development and to support discharge of relevant planning conditions and relevant building control requirements.
- to provide geotechnical information to aid in stability calculations for the canal walls and capacity calculations for the existing contiguous piled walls. The report for these elements will follow under separate cover.
- to refine the ground movement assessment and accurately model displacements associated with the demolition and construction of Block A and B while assessing the feasibility of reusing the contiguous piles surrounding the Thames Water sewer and those adjacent to Canal Side Studios. The report will follow under separate cover.
- to identify the need for any additional investigation or remediation works to demonstrate that the site is suitable for its proposed use.

1.5 Scope of works

The scope of this assessment has been developed in accordance with relevant British Standards and authoritative technical guidance as referenced through the report. The assessment of the contamination status of the site is in line with the technical approach presented in CLR 11 Model Procedures for the Management of Land Contamination (Environment Agency, 2004) and in general accordance with BS 10175: 2011 + A2 2017 (BSI, 2017). It is also compliant with relevant planning policy and guidance.

The scope of the intrusive investigation has been designed in line with the recommendations of BS5930: 2015 Code of practice for ground investigations (Bsi, 2016), which maintains compliance with BS EN 1997-1 and 1997-2 and their related standards. It has also been developed in general accordance with BS 10175: 2011 + A2 2017.

A brief summary of relevant legislation and policy relating to contaminated land is given in **Appendix C**.

The scope of works for the assessment has included the following:

Intrusive Investigation

- Geophysical GPR survey to clear borehole and trial pit locations of buried services;
- Concrete coring of internal boreholes and window sampling locations;
- 13 no. 25m to 40m deep percussive boreholes with sampling and in situ testing to prove geology and obtain geotechnical parameters for design;
- 6 no shallow boreholes by way of windowless dynamic sampling of up to 5.45m to determine the soil profile and increase coverage for contamination sampling;
- Installation of 35mm and 50mm dia. gas/groundwater monitoring standpipes in selected boreholes and 80 mm diameter plain casing in boreholes positioned adjacent to the existing contiguous/sheet piled walls to facilitate downhole geophysics;
- Downhole parallel seismic and magnetometer testing in 7 no. boreholes to estimate pile toe depths: 3 no. to the existing contiguous piled walls, 4no. boreholes to estimate pile toe depths to the existing canal sheet piled wall (of which 2 no. also to find the depth of the old buried masonry canal wall);
- Hand/machine assisted trial pits to investigate existing structures: 2.no. trial pits to investigate the former masonry canal wall, 4no. trial pits to expose the existing sheet pile wall and position boreholes for downhole geophysics, 2no. trial pits to expose existing contiguous pile heads and party wall foundations to the neighbouring Canal Side Studios;
- Visual structural surveys of the exposed contiguous pile and sheet pile capping beam and horizontal/vertical cores to recover samples for testing. Also, a condition survey of the existing sheet piles including ultrasonic thickness testing to confirm thickness and condition of rear face of sheet pile;
- Laboratory analysis of selected soil, groundwater and concrete samples.
- 6no. post fieldwork visits to monitor gas/groundwater levels over a 6 month period.

- Generic quantitative risk assessment (GQRA) to evaluate potentially complete contaminant linkages identified in the refined CSM
- Identification of the need for further action, e.g. supplementary intrusive investigations/ monitoring, remediation works or other mitigation, if any.
- Interpretation of ground conditions and geotechnical data to provide preliminary recommendations with respect to foundations and infrastructure design;
- Preliminary assessment of the potential waste classification (hazardous / non-hazardous) implications of soil arisings
- Preparation of this factual and interpretative report with recommendations for further works (i.e. undertake a remedial options appraisal to identify appropriate mitigation measures/produce a remedial implementation and verification plan) and/or remediation as necessary.

This report has not been prepared in accordance with the National Quality Mark Scheme for Land Contamination (NQMS).

1.6 Existing reports

The following reports detailing previous works at the site were made available for review:

- DTS Raeburn Ltd, Preliminary Risk Assessment Report for Ugly Brown Building 2 – 6 St Pancras Way London NW1 0TB, E12897/1 dated June 2017.
- RSK Environment Limited, Site at the Ugly Brown Building, London NW1 0TB Preliminary Ground Movement Assessment, 371654-L01 (01) dated 25th October 2017.
- *Ove Arup & Partners, Granary Site, St Pancras Way, Design Calculations for Piles Subject to Heave and Lateral Loads, report ref 12727/BS/AT/KAH dated August 1983.
- *White, Myers and Dutton, Ordinary Meeting Paper: The Granary site – design and construction of a mechanised letter-sorting office, dated 18th April 1985.
- *IstructE, White, Myers and Dutton, Discussion Paper: The Granary site – design and construction of a mechanised letter-sorting office, dated January 1986.

Pertinent information from these reports has been summarised in Section 2.

In addition to the above, the following reports detailing works associated with the redevelopment of Canal Side Studios were made available to RSK for review:

- Curtins, 8-14 St Pancras Way, Appraisal of Structure, LO1923 Rev 00, dated 29th July 2015, and associated historical drawings from 2004 by Dewhurst Macfarlane and Partners.
- Curtins, 8-14 St Pancras Way, Site Investigation Report, LO1923 Rev 01, undated.

The above reports which have been asterisked (*) are provided in Appendix E.

1.7 Limitations

The comments given in this report and the opinions expressed are based on the ground conditions encountered during the site work and on the results of tests made in the field and in the laboratory. However, there may be conditions pertaining to the site that have not been disclosed by the investigation and therefore could not be taken into account. In particular, it should be noted that there may be areas of made ground not detected due to the limited nature of the investigation or the thickness and quality of made ground across the site may be variable. In addition, groundwater levels and ground gas concentrations and flows may vary from those reported due to seasonal, or other, effects and the limitations stated in the data should be recognised.

Asbestos is often present in soils in discrete areas. Whilst asbestos-containing materials may have been locally encountered during the supporting laboratory analysis, the history of the site indicates that asbestos may be present in soils and could be encountered during more extensive ground works.

At the time of the investigation access to Block C had not been granted and was being used by Verizon. It is understood that investigation across this part of the site will take place at a later date when access can be made available.

2 SITE DETAILS

2.1 Site location and description

The site is located at the Ugly Brown Building, 2-6 St Pancras Way, London, NW1 0TB in the London Borough of Camden, at National Grid Reference 529620, 183762 as shown on **Figure 1**.

The area around the site is predominantly mixed commercial as detailed in **Table 1**.

Table 1: Surrounding land uses

North	Canal Side Studios (five storey concrete framed building with part basement)
East	Regent's Canal
West	St Pancras Way with Travis Perkins and The Gestalt Centre (a seven storey building) beyond
South	Granary Street and St Pancras Hospital beyond

The site boundary and current site layout are shown on **Figure 2** and is currently occupied by a four/five storey concrete structure known as the Ugly Brown Building.

The northern building is currently vacant (former Administration Building – Block A), the central building (former Welfare block – Block B) is occupied by Ted Baker Headquarters and the southern building (former sorting office – Block C) is occupied by the Verizon Data Centre which is very sensitive to noise and vibration.

Ground floor parking is available beneath part of Block B with access to the plant room at the southern end of Block B also possible. A number of expensive old cars owned by Ted Baker are stored in the car park. External car parking is located out the front of Block A on the St Pancras Way frontage, with a number of raised plant beds present also, one with a mature deciduous tree.

Access to the rear of the building on the eastern side of the site is possible through a double gated entrance off Granary Street to the south of Block C. A gravel towpath runs between the building and Regents Canal along the eastern boundary. The Ugly Brown Building cuts into the wider, gentle slope topography such that the ground floor sits approximately 2m below the towpath. A barge belonging to Ted Baker is located on Regent's Canal outside Block B and is understood to be used for meetings.

As mentioned in Section 1.3, the Thames Water Sewer underlies the southern end of Block A at approximately 4.50mbgl (15.60 mAOD) running southwest northeast. A row of piles run adjacent to the sewer along the easement boundary.

Further, a contiguous piled wall is present between the UBB and the Canal Side Studios to the north, understood to have been installed at the time of UBB construction to minimise ground movements on the neighbouring property associated with the construction of UBB.

2.2 Summary of previous investigations

A summary of pertinent information from previous investigations is included below in **Table 2**.

Table 2: Summary of previous investigation reports

Report Details	**1. Preliminary Risk Assessment, DTS Raeburn Ltd, June 2017**
Site coverage	Full existing building footprint and the surrounding area of up to 250 m.
Summary scope of works	Desk based study on the present and historic uses of the subject site and surrounding area.
Does the client have reliance upon the report?	Yes
Key factual findings	A potential risk of ground contamination was identified on site from former demolition activities. In addition, a UXO risk was identified and further assessment recommended.
Report Details	**2. Detailed UXO Assessment, 1st Line Defence, October 2018**
Site coverage	Building footprint
Summary scope of works	Desk based study to assess the risk of encountering buried UXO across the site.
Does the client have reliance upon the report?	Yes
Key factual findings	Medium UXO Risk over Plot A, low risk elsewhere across the site. Recommended UXO mitigation measures for site works and piling across Block A only.
Report Details	**3. Preliminary Basement Impact Assessment, RSK, October 2017**
Site coverage	Building footprint and immediate surrounding area.
Summary scope of works	Preliminary numerical modelling of various stages of the proposed development
Does the client have reliance upon the report?	Yes
Key factual findings	London Clay reaches approximately 20 mbgl which in turn lies the Woolwich and Reading beds. Damage categories exhibited for each of the adjacent structures were largely confined to Category 1 (Very Slight) or below, with the exception of the southern elevation of Canal Side Studios in the long term case, which has a damage category of 2 (Slight). This will need to be reassessed to take account of the site specific ground investigation data and latest structural scheme.

Report Details	**4. Design Calculations for Piles for Heave and Lateral Loads, ARUP, August 1983**
Site coverage	Building footprint
Summary scope of works	Design calculations for foundations to the Ugly Brown Building
Does the client have reliance upon the report?	Yes
Key factual findings	Pile configurations to the existing Ugly Brown Building

3 SITE RECONNAISSANCE FINDINGS

A site reconnaissance survey was completed on 29th November 2018 by RSK. The characteristics of the site observed during the walkover and from current ordnance Survey maps are summarised in Error! Reference source not found.**Table 3**.

A site plan is provided in **Figure 2** with photographic records included in **Appendix G** detailing the main features identified below.

Whilst the walkover summary includes consideration of current operations and housekeeping on the site as potential sources of contamination, it does not constitute a comprehensive environmental audit of the site, as covered under ISO 14001.

Table 3: Site reconnaissance findings

Feature	Description
Physical characteristics	
Access constraints	Car parks outside of Block A and B are consistently filled to maximum with some overflow. Very limited space within these areas. Gravel tow path alongside canal in addition with a narrow width.
Site topography	The site is essentially level alongside St Pancras Way and within Blocks A and B and steps up approximately 2m alongside the canal, held back by a retaining wall.
Surface cover	Soft landscaped raised plant bed areas occupy the northwest corner by Block A, while the remaining area alongside St Pancras Way is laid to paving slabs. An assumed reinforced concrete slab covers the building footprint while a gravel surface tow path occupies the stretch of land alongside the canal in the east.
Site drainage	Drainage is apparent alongside the perimeters to Block A and B and gulley's are noted within the car park at Block B. It is unknown whether the sewers drain into the deep brick lined No. 2 Thames Water Sewer that cuts across the site.
Surface water	Regent's Canal runs alongside the eastern site boundary and noted to be carrying a moderate amount of refuge.
Trees and hedges	Three semi mature deciduous trees occupy the north-eastern section of the site, adjacent to Block A. A mature deciduous tree occupies one of the raised plant beds outside Block A on the St Pancras Way frontage.
Invasive species	Based upon the walkover survey obvious evidence of Japanese Knotweed or other invasive species has not been identified on-site. However, it should be noted that a detailed survey of the possible presence or absence of invasive species is outside of the scope of investigation and consideration should be given to commissioning a specialist survey, as necessary.
Existing buildings on-site	The building footprint of the four storey Ugly Brown Building occupies the centre section of the site.

Feature	Description
Retaining walls and adjacent buildings on or close to site boundary	Immediately north lies Canal Side Studios which is believed to be separated from the site by a contiguous wall. Sheet pile retaining walls are constructed alongside the canal. The former masonry canal wall is understood to be buried beneath the tow path approximately 1m from the current sheet pile canal wall.
Basements on-site	Basements are not present on site.
Made ground, earthworks and quarrying	The raised area alongside the canal is potentially Made Ground.
Potentially unstable slopes on or close to site	None observed
Buried and overhead services present	There are several manhole covers alongside the perimeter to Block A and B with additional covers noted within the car park to Block B. Potential sub stations are identified adjacent to the barge moored up by Block B with an additional structure located at the northern end of the Verizon Building (Block C).
Environmental characteristics	
Underground/ above ground storage tanks and pipework	None observed
Potentially hazardous materials storage and use	None observed
Asbestos-containing materials	No obvious asbestos construction materials were observed but a detailed survey of the buildings would be required to confirm the presence or otherwise of asbestos-containing materials.
Waste storage	Waste from the offices/retail units is stored in wheelie bins inside the roller shutter doors to the Block B ground floor car park.
Fly-tipping	None observed.
Electricity sub-stations/ transformers	There is an existing sub-station located at the southwestern corner of Block A.
Evidence of possible land contamination on-site	None observed.
Potential off-site sources of ground contamination	The immediate surroundings are not considered to contribute to potential sources of contamination.

No potentially significant land contamination or geotechnical issues were identified during the site reconnaissance survey.

4 PRELIMINARY GEOTECHNICAL HAZARDS

4.1 Design class

BS EN 1997-1 defines three different Geotechnical Categories that structures may fall into, which are summarised as follows:

- Category 1: Small and relatively simple structures for which it is possible to ensure that the fundamental requirements will be satisfied on the basis of experience and qualitative geotechnical investigations; with negligible risk
- Category 2: Conventional types of structure and foundation with no exceptional risk or difficult ground or loading conditions
- Category 3: Structures or part of structures, which fall outside limits of Geotechnical Categories 1 and 2. Examples include very large or unusual structures; structures involving abnormal risks, or unusual or exceptionally difficult ground or loading conditions; structures in highly seismic areas; structures in areas of probable site instability or persistent ground movements that require separate investigation or special measures.

Based on the information provided above on the proposed development and in view of the anticipated ground conditions, a Geotechnical Category 2 has been assumed for the purposes of designing the geotechnical investigation. This should be reviewed at all stages of the investigation and revised where necessary.

4.2 Preliminary geotechnical hazards assessment

A summary of commonly occurring geotechnical hazards associated with the anticipated geology outlined in Section 3 above is given in **Table 4** together with an assessment of whether the site may be affected by each of the stated hazards.

Table 4: Summary of preliminary geotechnical risks that may affect site

Hazard category	Hazard status based on desk study findings and proposed development		Engineering considerations if hazard affects site
	Could be present and/or affect site	Unlikely to be present and/or affect site	
Sudden lateral changes in ground conditions	☐	☒	Surrounding available borehole data suggests London Clay is present throughout.
Shrinkable clay soils	☒	☐	Design in accordance with NHBC guidance or similar.

Hazard category	Hazard status based on desk study findings and proposed development		Engineering considerations if hazard affects site
	Could be present and/or affect site	Unlikely to be present and/or affect site	
Highly compressible and low bearing capacity soils, (including peat and soft clay)	☐	☒	Likely to affect ground engineering and foundation design and construction
Silt-rich soils susceptible to rapid loss of strength in wet conditions	☐	☒	Likely to affect ground engineering and foundation design and construction
Running sand at and below water table	☐	☒	Likely to affect ground engineering and foundation design and construction
Karstic dissolution features (including ‘swallow holes’ in Chalk terrain)	☐	☒	May affect ground engineering and foundation design and construction – refer to Section 4.1.2
Evaporite dissolution features and/or subsidence	☐	☒	May affect ground engineering and foundation design and construction
Ground subject to or at risk from landslides	☐	☒	Likely to require special stabilisation measures
Ground subject to peri-glacial valley cambering with gulls possibly present	☐	☒	Likely to affect ground engineering and foundation design and construction
Ground subject to or at risk from coastal or river erosion	☐	☒	Likely to require special protection/stabilisation measures
High groundwater table (including waterlogged ground)	☒	☐	Double storey basements pose a greater risk of encountering groundwater at depth.
Rising groundwater table due to diminishing abstraction in urban area	☐	☒	May affect deep foundations, basements and tunnels
Underground mining	☐	☒	Likely to require special stabilisation measures
Effects of extreme temperature (e.g. cold stores or brick kilns/furnaces)	☐	☒	Likely to affect ground engineering and foundation design and construction

Hazard category	Hazard status based on desk study findings and proposed development		Engineering considerations if hazard affects site
	Could be present and/or affect site	Unlikely to be present and/or affect site	
Existing sub-structures (e.g. tunnels, foundations, basements, and adjacent sub-structures)	☒	☐	Historic slab construction to the Granary Building. The Thames Water sewer beneath Block A and buried masonry canal wall on eastern boundary
Filled and made ground (including embankments, infilled ponds and quarries)	☒	☐	Made Ground present from former demolition of Granary Building. Likely to affect ground engineering, foundation design and construction
Adverse ground chemistry (including expansive slags and weathering of sulphides to sulphates)	☒	☐	May affect ground engineering and foundation design and construction
Note: Seismicity is not included in the above table as this is not normally a design consideration in the UK.			

5 PRELIMINARY RISK ASSESSMENT

5.1 DTS Raeburn Ltd Preliminary Risk Assessment, June 2017

A Preliminary Risk Assessment was carried out by DTS Raeburn Ltd in June 2017 to which the reader is referred. A brief summary of the findings is provided below.

The full report is provided in **Appendix D**.

5.1.1 Geology, Hydrogeology and Hydrology

Published records held by the BGS indicate the site is underlain by the bedrock of the London Clay Formation consisting of clay, silt and sand which extended to approximately 20m, and is in turn underlain by the Lambeth Group with the Thanet Sand Formation and White Chalk at depth.

Superficial deposits are not recorded for the site, however given the site history and with reference to two borehole records held by the BGS, made ground is expected beneath the site.

Information provided by the Environment Agency indicates the underlying London Clay Formation is classified as 'unproductive strata' whilst the Lambeth Group and Thanet Sand Formation are categorised as Secondary 'A' aquifers, and the White Chalk categorised as a Principal Aquifer. Further, the site does not lie within a Source Protection Zone and is within a Flood Zone 1 (low risk).

The nearest surface water feature is the Regent's Canal located adjacent to the northeast boundary of the site. It is noted that construction of the canal was found to be suspect during the previous intrusive site investigation for the Royal Mail sorting office in 1985. Following approval from the canal authority, a new steel sheet pile wall was installed in the canal along the length of the site and sealed back into the existing wall at each end.

5.1.2 History of the site

The historical maps and planning records consulted indicate that the site was used for the storage of ale from at least 1975 with additional granary storage from the early 1950s. There was also a small unidentified building and railway sidings located in the northwest of the site up until the late 1960s. By 1985, the site had been redeveloped into a postal sorting office that covered the entire site. The building has remained largely unchanged up until the present day and is now occupied by office accommodation. There have been a number of industrial uses in the site locality since the late nineteenth century.

5.2 Detailed UXO Risk Assessment, October 2018

In light of the sites development history and the surrounding historical land use, and with reference to online information indicating high explosive bombs were dropped in the sites vicinity, DTS Raeburn concluded in their Preliminary Risk Assessment there was a risk of encountering unexploded ordnance (UXO) beneath the site and recommended that further risk assessment was carried out.

In light of this, a Detailed UXO Risk Assessment was carried out by a specialist contractor to assess the risk to site and determine what, if any, mitigation measure would be required for any intrusive work. The report (1^{st} Line Defence report ref DA7410-00, dated 24^{th} October, 2018) determined that two bombs strikes were recorded on the northern area of the site during WWII and this area sustained some level of bomb damage although the extent of the damage was unconfirmed. Goad insurance mapping indicates that this part of the site was vacant from 1942 and therefore this area would have had little access which would have hindered UXO inspection.

By contrast, the southern end of the "Ale and Porter" sustained only "blast damage, minor in nature" and aerial photographs show the building remained intact throughout the war. Further, as this building was several stories high, the chance of UXO penetrating at depth is low and because this commercial building was fully occupied during the war, the chance of UXO going unnoticed is greatly decreased.

In consideration of the above, the report concluded that there is a **low** risk of UXO across the majority of the site, with only a small area in the north (over Plot A) assessed as being a **medium** risk.

The Detailed UXO Risk Assessment is provided in **Appendix F**.

6 INITIAL CONCEPTUAL SITE MODEL

An initial conceptual model has been outlined within DTS Raeburn's report to which the reader is referred. Potential sources of soil and groundwater contamination identified from current activities and the history of the site and surrounding area are presented within the report and summarised below

6.1.1 Potential contamination sources

The following potential sources of contamination were identified;

- Onsite current use of site including car park (generally considered to be a low likelihood of ground contamination, however the possibility of hydrocarbon contamination as a result of localised oil or fuel leakages from vehicles has been considered);
- Onsite former railway sidings (heavy metals, TPH, PAH, glycols and acids/alkalis);
- Onsite made ground from demolition of former buildings (asbestos and heavy metals);
- Onsite electrical substation in southwest corner of Block A;
- Offsite contamination from industrial activities within a 250m radius of the site (inclusive of demolition of former buildings).

6.1.2 Sensitive receptors

The following potentially sensitive receptors were identified:

- Site end users;
- Occupiers of adjacent land.
- Controlled waters (canal and deep aquifers)
- Development end use: buildings, hardstandings, services, utilities and limited landscaped areas.

Please note that construction workers have not been identified in the conceptual model as receptors because risks are considered to be managed through health and safety procedures including CDM regulations.

6.1.3 Plausible pathways

The following potential pathways were identified, on the basis of the anticipated ground conditions and the proposed commercial development:

- Dermal contact with soils and/or dust;
- Ingestion of soil;
- Inhalation of gases, vapours or dusts;
- Migration of gases through the ground and accumulation of explosive gases.

The following pathways can be added to the conceptual site model on the basis of landscaped areas and controlled waters being potential receptors:

- Root uptake;

- Vertical and lateral migration including leaching and surface run-off.

Considering the presence of low permeability London Clay at a shallow depth, the complete plausible pathways associated with the deep groundwater resources are unlikely to be present. It is understood that the canal construction comprises a sheet piled wall toed into the London Clay and therefore the canal is not in hydraulic continuity with perched water encountered beneath the site. In conclusion, plausible pathways associated with controlled waters are considered incomplete.

Furthermore, the presence of the London Clay at a shallow depth will inhibit the migration of any gases and contamination from offsite sources onto the site and therefore plausible pathways associated with offsite sources are considered incomplete. In summary, potential sources of contamination are likely to be limited to ground contamination from demolition works of former structures on site and current site uses (oil/fuel spills). Further, the inhalation and migration of ground gases arising from the made ground presents a risk to site users and buildings/services.

The preliminary risk assessment findings and potentially complete contaminant linkages generally present a **moderate/low** risk on site, and an intrusive site investigation to confirm the status of the identified potential pollutant linkages will be required.

7 THE REUSE OF EXISTING FOUNDATIONS

7.1 General

The proposed basement within Block A may incorporate the existing contiguous piled wall to the north and thereby acting as a retaining wall for both the lateral earth pressures and those imposed from the adjacent building. It is also understood that new piled transfer slabs over the Thames Sewer will utilise the existing piles adjacent to the Thames Water sewer to accommodate the change in load associated with the new development.

7.2 Considerations for reuse

In order that existing piled foundations to the contiguous wall and piled transfer slab can be incorporated into the new build with sufficient confidence, careful consideration of the following is required:

- Establishing accurately the position and size of the contiguous piled wall, neighbouring Canal Side Studios foundations, as well as confirming the depth of the piles spanning the sewer beneath Block A.
- Establishing accurately the length and thickness of the existing sheet piled wall alongside the canal.
- Confidence in the geotechnical and structural capacity of the contiguous piles and capping beams to sheet piles.
- Confidence that the pile materials have not significantly deteriorated and will not do so over their future design life.
- Satisfying regulatory bodies, project funders and future building insurers.

7.3 Philosophy of Investigation

In general terms, the proposed scope of work aims to assess selected existing contiguous piles in terms of their geotechnical and structural capacity, concrete integrity / condition and durability for the life span of the new development.

8 SITE INVESTIGATION STRATEGY & METHODOLOGY

8.1 Introduction

RSK carried out intrusive investigation works and subsequent monitoring of boreholes between January 2019 and February 2019.

Prior to conducting intrusive works, utility service plans were provided to RSK by the client and buried service clearance was undertaken by RSK Safeground in line with RSK's health and safety procedures.

8.2 Objectives

The specific objectives of the investigation were as follows:

- to establish the ground conditions underlying the site including the extent and thickness of any made ground
- to investigate specific potential sources of contamination identified in initial CSM as outlined by DTS Raeburn
- to determine the ground gas regime underlying the site
- to confirm the contiguous pile configuration alongside Canal Side Studios and to assess their strength and reinforcement details
- to expose and survey the sheet pile wall alongside the canal, recording its thickness and capping beam details
- to perform downhole geophysics to record pile toe depths to the contiguous piled wall alongside Canal Side Studios and the Thames Water Sewer, and to the sheet piled wall alongside the canal and buried masonry canal wall.
- to expose and record foundation configurations to the party walls of Canal Side Studios; and
- to assess the geotechnical properties of soils.

8.3 Selection of investigation methods

The techniques adopted for the investigation were chosen with consideration of the objectives and site constraints, which are described below.

Three types of cable percussion drilling rigs were chosen based on the targeted drill depth, head room height within buildings, terrain trafficability, requirement for in-situ geotechnical data, and the opportunity to collect both disturbed and undisturbed samples and install monitoring wells. This was supplemented by shallow windowless dynamic boreholes in order to gain greater coverage across site for contamination sampling and confirmation of the shallow soil profile.

8.4 Investigation strategy

The ground investigation was carried out using intrusive ground investigation techniques in general accordance with the recommendations of BS5930: 2015 Code of practice for ground investigations, which maintains compliance with BS EN 1997-1 and 1997-2 and their related standards. Whilst every attempt was made to record full details of the strata encountered in the boreholes, techniques of hole formation and sampling will inevitably lead to disturbance, mixing or loss of material in some soils and rocks.

The investigation strategy involved targeted and non-targeted boreholes and trial pits. The investigation comprised an extensive exploratory investigation focussing within and around Block A and B only due to access restrictions imposed at the time of this investigation.

The constraints to the investigation were as follows:

- Due to Block B being occupied by Ted Baker, any noisy works outside their offices was to be conducted out of hours; either before 9:00 am or on a Saturday between 8:00 am and 1:00 pm as outlined within the London Borough of Camden noise restrictions.
- Medium UXO risk present within Block A.
- A cut down cable percussion drilling rig was used within Block A and B due to height restrictions.
- A tracked cable percussion rig had to be used for the boreholes alongside the canal due to trafficability issues on the gravel surface.
- Any works near the barge adjacent to Block B had to be moved so as to avoid any potential damage.

Concrete surfacing covered much of the site and therefore warranted coring for internal boreholes and breaking out alongside the canal.

Details of the investigation locations, installations and rationale are presented in **Table 5**. Eight machine excavated trial pits were dug to a maximum depth of 2.40m bgl before being backfilled with arisings. Thirteen cable percussive boreholes were drilled to a maximum depth of 40m bgl, seven of which were installed with 80mm diameter standpipe to their full depth to facilitate downhole geophysical testing of targeted contiguous and sheet piles and masonry walls; of the remaining boreholes a selection were installed with gas and groundwater monitoring wells. In addition, six shallow boreholes were drilled by windowless dynamic sampler techniques to a maximum depth of 5.45m bgl with 2 no. installed with gas and groundwater monitoring wells. Exploratory hole locations are presented on **Figure 3**.

Table 5: Exploratory hole and monitoring well location rationale

Investigation Type	Number	Designation	Monitoring well installation	Rationale
Boreholes by cable	2	BH01 & 02	Downhole geophysics	To prove the geological succession beneath the site

Investigation Type	Number	Designation	Monitoring well installation	Rationale
percussive methods with full geotechnical testing and sampling				and to target a contiguous pile on the northern boundary in order to record its depth via geophysics
	1	BH06	Downhole geophysics	To prove the geological succession beneath the site and to target Pile A41 adjacent to the Thames Water Sewer investigate pile toe depths via geophysics
	4	BH03, 05, 11, 12A	Gas and/or Groundwater	To prove the geological succession beneath the site and obtain sufficient geotechnical data at a depth at least 5m below the anticipated maximum pile foundation depths7.
	5	BH04, 07, 10 & 15	Downhole geophysics	To prove the geological succession beneath the site and to prove the depth of the sheet pile wall via geophysics.
	-	BH08	-	Omitted due to limited working space – waiting for building demolition
	-	BH14	-	Cancelled due to obstructions in three attempts of the borehole.
Boreholes by dynamic/ windowless sampling methods	6	WS01 to WS06	Gas/ groundwater	To determine the contamination status of the ground beneath the site and to install additional dual-purpose groundwater and gas monitoring wells.
	2	WS07 & 08	-	Cancelled
Trial-pits excavated by hand/mechanical excavator	2	TP01 & TP02.	n/a	To expose the old canal wall to record its depth, width and foundation configuration
	2	BH01 & 02	n/a	To inspect the contiguous piled wall adjacent to the Canal Side Studio and conduct a structural survey.
	2	BH01 & TP03	n/a	To expose the party wall foundations to Canal Side Studios

Investigation Type	Number	Designation	Monitoring well installation	Rationale
	4	BH04, 07, 10 & 15	n/a	To inspect the sheet pile wall and conduct a structural survey of the capping beam and pile.
		BH06 & BH08		Omitted due to limited working space around BH08 and significant concrete thickness at BH06 inside the building – waiting for building demolition.

8.5 Limitations of fieldwork

8.5.1 Buried services and obstructions

Although great care was taken to ensure underground services were not encountered, the inspection pit at BH14A encountered a pipe at 1.20m bgl, prompting the relocation to BH14B. A buried concrete obstruction was encountered at this location at around 1.60m bgl depth which prompted a further relocation to BH14C. Unfortunately, buried concrete was encountered at approximately 2.00m bgl depth which finally forced the termination of the borehole.

Buried concrete at BH12 prompted the relocation to BH12A where the borehole was able to continue to completion.

Originally, BH06 was positioned within Block A, however, initial coring of this position proved the concrete slab to be in excess of 1m thick and therefore the borehole was moved to its ultimate location, outside of the building.

The final Standard Penetration Test (SPT) within WS01 could not be carried out due to a positive magnetometer reading at 5m depth.

WS05 was terminated at 2.15m depth on a formed brick obstruction.

8.5.2 Competent ground conditions

The made ground conditions within Block B were markedly 'dense', forcing a refusal within WS06, and slow progress initially within BH13 and within the inspection pits for the BH14 series. Whilst the concrete slab at WS07 and WS08 were cored ahead of the window sample drilling, as a result of the difficult drilling conditions elsewhere across Block B, these boreholes were cancelled.

8.5.3 Space restrictions

As intrusive investigations were not allowed within the easement of the Thames Water Sewer, boreholes and inspection pits had to be positioned on the outside of the piled rows. With the limited space available, and together with the excessive thickness of concrete slab (1m), the inspection pit for BH06 and BH08 along with its corresponding borehole, were omitted from the investigation. BH06 was relocated to the outside of Block A to target pile A41 and BH08 was omitted from the investigation for the time being due to restricted working space in Block B and it was not possible to get permission from Thames Water to drill within the sewer easement in Block A.

8.6 Implementation of investigation works

The exploratory holes were logged by an RSK engineer in general accordance with the recommendations of BS 5930:2015 (which incorporates the requirements of BS EN ISO 14688-1, 14688-2 and 14689-1).

Standard penetration tests (SPTs) were carried out within the made ground and the natural strata, at regular intervals of approximately 1m in the initial 5m depths, and then at 1.5m intervals to the terminal depth of the boring. The tests were undertaken in accordance with BS EN ISO 22476-3:2005 using a hammer, which had been calibrated for efficiency. The calibration certificates are included in **Appendix I**, and the results are given on the borehole logs presented in **Appendix I**. Plots of SPT N values against depth, and SPT N_{60} values against depth, are presented as **Figures 7 and 8**, respectively.

The full results are presented in tabular format on the Summary of Standard Penetration Tests in **Appendix I**, on which the normalised N_{60} values are also reported, which are the equivalent N-value for a hammer delivering 60% of the theoretical drop energy.

In cohesive strata the SPT tests were alternated with collecting undisturbed ‘thin wall’ UT100 samples. Further small and bulk disturbed samples were taken from each strata encountered to facilitate subsequent geotechnical classification analysis.

The monitoring well construction and associated response zones are detailed on the exploratory hole records in **Appendix I**. The response zones were installed to target identified gas generation sources detailed in the initial preliminary CSM by DTS Raeburn.

The soil sampling and analysis strategy was designed to characterise each encountered soil strata, permit an assessment of the potential contaminant linkages identified and investigate the geotechnical characteristics. In addition, samples were taken to allow for geo-environmental and geotechnical testing to be undertaken.

Soils collected for laboratory analysis were placed in a variety of containers appropriate to the anticipated testing suite required. They were dispatched to the laboratory in cool boxes under chain of custody documentation. Samples were stored in accordance with the RSK quality procedures to maintain sample integrity and preservation and to minimise the chance of cross contamination.

Selected samples were placed in polythene bags for headspace screening with a photo-ionisation detector (PID) fitted with a 10.2 eV bulb. The PID screening results are presented on the exploratory hole records.

8.7 Monitoring programme

8.7.1 Ground gas monitoring

In line with the initial CSM, response zones were installed to target the sources or pathways as detailed in Section 6.

Six monitoring rounds have been undertaken to provide data to support refining of the CSM. The number of monitoring rounds undertaken is in general accordance with the decision matrix presented as Figure 6 of BS8576.

An infrared gas meter was used to measure gas flow, concentrations of carbon dioxide (CO_2), methane (CH_4) and oxygen (O_2) in percentage by volume, while hydrogen sulphide (H_2S) and carbon monoxide (CO) were recorded in parts per million. Initial and steady state concentrations were recorded.

The atmospheric pressure before and during monitoring, together with the weather conditions, were recorded. The monitoring included periods of falling atmospheric pressures and after/during rainfall.

All ground gas monitoring results together with the temporal conditions are contained within **Appendix J**. Equipment calibration certificates are available on request.

8.7.2 Groundwater monitoring and sampling

Six rounds of groundwater monitoring has been undertaken. Two rounds of groundwater sampling were scheduled to be carried out. The monitoring records, including dates, are shown in **Appendix K**.

Depths to groundwater were recorded using an electronic dip meter on each of the return monitoring visits.

Groundwater samples were retrieved using a United States Environment Protection Agency (USEPA) approved low-flow purging and sampling methodology. Details of the low-flow sampling procedure are set out in **Appendix H**. Water samples were dispatched to the laboratory in cool boxes under chain of custody documentation.

8.8 Laboratory testing

Laboratory testing was undertaken at a UKAS accredited laboratory with ISO17025 and MCERTS accredited test methods were specified where applicable for contamination testing and as shown in the laboratory test certificates appended.

8.8.1 Chemical analysis of soil samples

The soil sampling strategy was designed to characterise the made ground.

The programme of chemical tests undertaken on soil samples obtained from the intrusive investigation is presented in **Table 6** with the laboratory testing results contained in **Appendix L**.

Table 6: Summary of chemical testing of soil samples

Stratum	Tests undertaken	No. of tests
Made ground	Hazardous Waste Suite: Asbestos screening and ID, heavy metals, pH, Total Petroleum Hydrocarbon (TPH) with ID, Polyaromatic Hydrocarbon (PAH) EPH 17 and moisture content	20

Stratum	Tests undertaken	No. of tests
	Total Organic Carbon	14

8.8.2 Geotechnical analysis of soils

Where appropriate disturbed, bulk and undisturbed soil samples were taken for geotechnical classification testing with the depth and nature of samples detailed within the exploratory hole records.

Where appropriate, testing was undertaken in accordance with BS 1377:1990 Method of Tests for Soils for Civil Engineering Purposes or, where superseded, by the relevant part of BS EN ISO 17892:2014 Geotechnical investigation and testing – Laboratory Testing of Soil. Tests carried out in order to classify the concrete class required on-site have been undertaken following the procedures within BRE SD1:2005.

The programme of geotechnical tests undertaken on samples obtained from the intrusive investigation is presented in **Table 7**. The results and UKAS accreditation of tests methods are shown in **Appendix M**.

Table 7: Summary of geotechnical testing undertaken

Strata	Tests undertaken	No. of tests
Made Ground	BRE brownfield pyritic soil	6
	Moisture content %	1
London Clay	Moisture content %	61
	Liquid/ plastic limits	9
	Unconsolidated undrained triaxial	66
	Consolidated undrained triaxial	2
	Laboratory vane	1
	One dimensional oedometer consolidation	3
	BRE brownfield pyritic soil	18
Lambeth Group (formerly known as Woolwich and Reading)	Moisture content %	19
	Liquid/ plastic limits	3
	Unconsolidated undrained triaxial	10
	Laboratory vane	2
	One dimensional oedometer consolidation	1
	BRE brownfield pyritic soil	7

8.8.3 Chemical analysis of groundwater

Groundwater samples were collected in containers appropriate to the anticipated testing suite required. The containers were filled to capacity and placed in a cool box to minimise volatilisation.

Chemical testing undertaken on water samples obtained during the monitoring programme is presented in **Table 8** with the laboratory testing results contained in **Appendix N**.

Table 8: Summary of chemical testing of water samples

Sample type	Tests undertaken	No. of tests
Groundwater (Made Ground and London Clay)	Speciated PAH-16MS, TPHCWG (spec.TPH), pH, As, Cd, Cr, Cu, Hg, Pb, Ni, Se, Zn, Sulphate	6
Groundwater (London Clay)	pH and sulphate	2

9 SITE INVESTIGATION FACTUAL FINDINGS

The results of the intrusive investigation and subsequent geo-environmental and geotechnical laboratory analysis undertaken are detailed below.

9.1 Ground conditions encountered

The descriptions of the strata encountered, notes regarding visual or olfactory evidence of contamination, list of samples taken, field observations of soil and groundwater, in-situ testing and details of monitoring well installations are included on the exploratory hole records presented in **Appendix I**.

The exploratory holes revealed that the site is underlain by a variable thickness of made ground over London Clay Formation with the Lower Mottled Beds of the Lambeth Group encountered at depth. This appears to confirm the stratigraphical succession described within the preliminary BIA and DTS Raeburn's Preliminary Risk Assessment.

Due to the significant increase in levels alongside the canal within the eastern section of the site, all strata depths are reported in metres above ordnance datum only (mAOD).

For the purpose of discussion, the ground conditions encountered during the fieldwork are summarised in **Table 9** with the strata discussed in subsequent subsections.

Table 9: General succession of strata encountered

Stratum	Exploratory holes encountered	Depth to top of stratum mAOD	Proven thickness (m)
Made ground	All positions	21.24 to 23.75	0.29 to 5.50
Weathered London Clay	All positions with the exception of WS05 & 06, BH12, BH14A, 14B and 14C.	16.33 to 21.55	2.50 to 6.20
London Clay	All positions with the exception of WS05 & 06, BH12, BH14A, 14B and 14C.	13.83 to 15.84	16.30 to 20.30
Lambeth Group	BH01, 02, 03, 05, 06, 12A, 13 and 15	-4.46 to -2.68	3.80 to 15.50+
Notes: + Thickness of strata extends beyond proven depth			

9.1.1 Made ground

The varied thickness and composition of the Made Ground across site has been categorised into different areas.

9.1.1.1 Beneath Block A and B

Reinforced concrete was encountered at all locations with thicknesses ranging between 0.22m and 0.33m thick. Various courses of steel rebar were present, however, generally comprised two sets of 10mm diameter bars at approximately 0.10m and 0.20m depth.

In BH05, a 170mm void with fragments of clayboard was present beneath the concrete.

Elsewhere, underlying the concrete slab at Block A, the stratum generally comprised a cohesive orangish brown and brown slightly sandy gravelly clay with fragments of brick, concrete and slag to between 21.55 and 20.40mAOD.

In WS01, a 250mm thick horizon of orangish brown slightly clayey gravelly sand with brick and clinker was present between the concrete slab and underlying cohesive made ground.

Made ground was not present in WS02 beneath the concrete slab.

The stratum underlying the concrete slab within Block B was markedly different in terms of composition and thickness and generally comprised an initial dense layer of sandy gravel with fragments of concrete and brick to between 20.26 and 19.62mAOD, locally with pockets of brown clay in WS04, WS05 and WS06. This gravel was underlain by slightly sandy gravelly clay to 19.56mAOD within WS04: the thickness of the gravel layer in BH14A/B/C, WS05 and WS06 was not proven.

The initial gravel horizon was absent within BH13. Instead, the made ground comprised a brown slightly sandy gravelly clay with fragments of brick, concrete and clinker to 19.30mAOD over a series of red brick and clay rubble, sandstone and light brown sandy clayey gravel to a maximum elevation of 16.50mAOD. The sequence is considered likely to reflect the former foundations for the Granary discussed in the IstructE Papers and mentioned in Section 1.3; a 750mm thick hydraulic concrete lime raft above which 225mm thick massive sandstone blocks sat on a 4.2m grid, with 1.20m of clay infill placed around and above the sandstone blocks and upon which a brick sett was placed.

Buried obstructions were common within Block B and included formed brick within WS05 at 19.55mAOD and buried concrete within BH14B and 14C at 19.80mAOD.

Man made detritus was fairly consistent with predominantly brick, concrete and clinker within both Block A and B.

9.1.1.2 St Pancras Way

Paving slabs covered most the area alongside St Pancras Way with the exception of BH02 and TP3 within the north of the site which were advanced through a raised plant bed. The Made Ground along St Pancras generally extended to between 17.59 and 20.66mAOD.

The Made Ground generally comprised possible reworked London Clay Formation comprising soft dark brown, brown and orange-brown silty clay, locally slightly gravelly, and occasionally with fragments of brick, concrete and flint (BH02, BH03, BH06, BH11 and TP3).

In BH06, the possibly reworked London Clay was overlain by dark brown and brown very sandy gravel of concrete, flint and brick to 20.01mAOD and dark brown grey slightly gravelly sand with charcoal and brick to 18.71mAOD.

In BH12A, the made ground comprised slightly clayey very gravelly sand which extended to 18.40mAOD, with a 500mm thick horizon of light brown sandy slightly gravelly clay at 20.20mAOD. Brick, flint, concrete and clinker detritus was scattered throughout.

Possible reworked ground was identified within BH02 from 21.46 to 20.02mAOD as a multicoloured clay. Due to the proximity of the contiguous pile, it is likely that this reworked material is a product of its installation.

Anthropogenic material was present in all layers and included brick, concrete, clinker and occasional coal and timber.

9.1.1.3 Canal Side

Apart from BH01 at the northern end of the site, a thin layer of medium flint gravel overlies reinforced concrete covering the entire area alongside the canal. The concrete had various courses of 10mm and 20mm ribbed steel rebar and generally represented a uniform thickness of 0.20m.

Generally, the Made Ground consists of a granular layer that locally overlies a cohesive layer, with a significant amount of gravel and cobbles throughout. The granular layer comprised clayey gravelly sand within BH01 in the north to 20.20mAOD, while identifying as sandy gravel with cobbles of brick and concrete within the remaining boreholes parallel to the canal. Locally, red brown cobbles of whole and fragmented brick were encountered in BH10 to 21.74mAOD.

The granular layers generally extended to between 22.37 and 20.55mAOD wherein the cohesive layer comprising sandy and gravelly clay with occasional cobbles and gravel of concrete and brick was encountered to elevations of between 19.23 and 20.74mAOD (absent in BH10).

In BH04, BH10 and BH15, at depths of 3.30m, 4.70m and 4.60m, respectively, it is inferred that the outstand at the base of the historical masonry canal wall footing was encountered.

A summary of the in-situ test and laboratory results in this stratum is presented in **Table 10** and laboratory test results can be found in **Appendix M.**

Table 10: Summary of in-situ and laboratory test results for Made Ground

Soil parameters	Min. Value	Max. Value	Reference
Moisture content (%)	24 (1 sample)	24 (1 sample)	Appendix M & Figure 4
Bulk Density (Mg/m^3)	1.92	2.12	Appendix M
SPT 'N' values	2	195**	Appendix I & Figure 7
SPT 'N60' values*	2	254**	Appendix I & Figure 8
Water soluble sulphate (WSO_4) (mg/l)	10	1800	Appendix M
pH value	7.97	12.85	Appendix M
Acid soluble sulphate (SO_4) (%)	0.02	1.19	Appendix M

Soil parameters	Min. Value	Max. Value	Reference
Total sulphur (%)	0.07	0.6	Appendix M
Notes: *derived from applying efficiency factors from the relative SPT hammer certificates **SPT refusal – extrapolated value, typically on cobble			

9.1.2 London Clay Formation

This stratum was encountered underlying the Made Ground and initially comprised firm, becoming stiff, thinly laminated light brown, orangish brown and brown, locally silty, clay with occasional fine and medium sand sized selenite crystals, locally with frequent partings of orange brown silt and pockets of fine sand/silt, indicative of the weathered portion of the geological unit. Locally the weathered London Clay Formation was fissured with generally tight, extremely to closely spaced and sub horizontal with light orange silt dusting on surfaces.

This in turn was underlain by unweathered London Clay Formation at elevations of between 13.83 and 15.84mAOD and which varied between 16.30m and 20.30m in thickness. The stratum comprised stiff becoming very stiff, fissured, dark greyish brown clay or silty clay, with pockets of grey brown silt and fine sand and occasional pyritised sand or nodules of pyrite. The clay is frequently thinly laminated with slight to prominent fissuring at depth. Fissures are generally tight, extremely to closely spaced and sub horizontal.

A rare layer of clayey silt was noted between -1.50 and -2.75mAOD within BH01.

Slightly fine sandy beds ranging in thickness of between 2.00 and 11.20m were present within all deep boreholes with the exception of BH07, often with water seepages associated with them.

Claystone bands were common within this stratum and are detailed in **Table 11** below.

Table 11: Claystones within the London Clay Formation

Location	Depth mAOD
BH01	17.25*
BH02	17.62*
BH04	17.23*
	11.13
	9.78
BH10	12.19
	9.19
BH13	13.03
	11.13
	10.48
BH15	19.14*
	15.24

Location	Depth mAOD
	9.74
	8.74
Notes: * Claystone present within weathered portion of the London Clay Formation All claystones were approximately 100mm in thickness	

A summary of the in-situ and laboratory test results recorded in the stratum are presented in **Table 12** and laboratory test results can be found in **Appendix M**.

Table 12: Summary of in-situ and laboratory test results for London Clay Formation

Soil parameters	Min. Value	Max. Value	Reference
Moisture content (%)	20	37	Appendix M & Figure 4
Modified moisture content (%) *	21	35	Appendix M
Liquid limit (%)	53	78	Appendix M
Plasticity limit (%)	23	35	Appendix M
Plasticity index (%)	30	49	Appendix M & Figures 5 and 6
Modified plasticity index (%) *	30	42	Appendix M
Plasticity term	High	Very High	Appendix M
Volume change potential	Medium	High	-
Bulk Density (Mg/m^3)	1.92	2.12	Appendix M
SPT 'N' values	5	52	Appendix I & Figure 7
SPT 'N_{60}' values**	7	62	Appendix I & Figure 8
Undrained shear strength inferred from SPT'N_{60}' values (kN/m^2) (f_1 = 4.5)	29	281	Appendix M & Figure 9
Undrained shear strength measured by hand shear vane testing (kN/m^2)	55	>150***	Appendix I & Figure 9
Undrained shear strength measured by laboratory shear vane testing (kN/m^2)	>234	>234***	Appendix M & Figure 9
Undrained shear strength measured by triaxial testing (kN/m^2)	63	327	Appendix M & Figure 9
Consistency term from field description	Firm	Very Stiff	-
Strength term (inferred from Triaxial testing)	Medium	Extremely High	-
Coefficient of volume compressibility (M_v) measured by oedometer test (m^2/MN)	0.063	0.111	Appendix M

Soil parameters	Min. Value	Max. Value	Reference
Coefficient of consolidation (C_v) measured by oedometer test (m^2/year)	2.2	7.6	Appendix M
Compressibility term	Low	Medium	-
Effective Cohesion (kPa)	6	15	Appendix M
Angle of Shear Resistance (˚)	23	25	Appendix M
Water soluble sulphate (WSO_4) (mg/l)	90	2800	Appendix M
pH value	7.75	9.34	Appendix M
Acid soluble sulphate (SO_4) (%)	0.02	3.13	Appendix M
Total sulphur (%)	0.02	1.15	Appendix M
Notes: * values provided where Atterberg test carried out ** derived from applying efficiency factors from the relative SPT hammer certificates *** Maximum reading on shear van used in the field			

9.1.3 Lambeth Group – Lower Mottled Beds

This stratum was encountered underlying the London Clay Formation and continued to the limit of investigation at elevations between -7.98 and -18.30mAOD with a recorded thickness of 3.80 to 15.50m.

Based on the site descriptions and laboratory and in-situ tests carried out this layer can be described as very stiff, locally fissured, variably mottled grey, greyish blue, yellowish green, brownish red and purple, locally silty clay.

A summary of the in-situ and laboratory test results recorded in the stratum are presented in **Table 13**.

Table 13: Summary of in-situ and laboratory test results for Lambeth Group – Lower Mottled Beds

Soil parameters	Min. Value	Max. Value	Reference
Moisture content (%)	17	35	Appendix M & Figure 4
Modified moisture content (%)*	18	27	Appendix M
Liquid limit (%)	52	98	Appendix M
Plasticity limit (%)	25	39	Appendix M
Plasticity index (%)	27	59	Appendix M & Figures 5 and 6
Modified plasticity index (%)	27	58	Appendix M
Plasticity term	High	Extremely High	Appendix M
Volume change potential	Medium	High	-
Bulk Density (Mg/m^3)	1.96	2.12	Appendix M

Soil parameters	Min. Value	Max. Value	Reference
SPT 'N' values	42	200***	Appendix I & Figure 7
SPT 'N_{60}' values**	39	240***	Appendix M & Figure 8
Undrained shear strength inferred from SPT'N_{60}' values (kN/m^2) (f_1 = 4.5)**	176	1080***	Appendix M & Figure 9
Undrained shear strength measured by hand shear vane testing (kN/m^2)	>125****	>125****	Appendix I & Figure 9
Undrained shear strength measured by laboratory shear vane testing (kN/m^2)	214	>260	Appendix M & Figure 9
Undrained shear strength measured by triaxial testing (kN/m^2)	77	294	Appendix M & Figure 9
Consistency term from field description	Very Stiff	Very stiff	-
Strength term (inferred from Triaxial testing)	High	Very High	-
Coefficient of volume compressibility (M_v) measured by oedometer test (m^2/MN)	0.052	0.052	Appendix M
Coefficient of consolidation (C_v) measured by oedometer test (m^2/year)	33	33	Appendix M
Compressibility term	Low	Low	-
Water soluble sulphate (WSO_4) (mg/l)	57	374	Appendix M
pH value	9.26	9.57	Appendix M
Acid soluble sulphate (SO_4) (%)	<0.02	0.09	Appendix M
Total sulphur (%)	0.02	0.09	Appendix M

Notes: * Values provided where Atterberg test carried out
**derived from applying efficiency factors from the relative SPT hammer certificates
***SPT refusal – extrapolated value
**** Maximum reading on shear van used in the field

9.1.4 Visual/olfactory evidence of soil contamination

No visual or olfactory evidence of contamination was noted during the investigation.

A total of 40 samples were tested in the field with a Photo Ionisation Detector (PID) and recorded a maximum value of 0.3 ppm.

9.2 Groundwater

9.2.1 Groundwater encountered during intrusive works

Groundwater was encountered during the intrusive investigation works as detailed on the logs in **Appendix I**. With the exception of the seepage in BH13 at 16.83mAOD, all groundwater encountered manifested as seepages within the London Clay Formation through claystone bands or silt/sand horizons.

The seepage in BH13 at 16.83mAOD was encountered within the made ground close to the top of the London Clay and possibly represents perched water within the made ground. This appears consistent with the information provided in the IStructE paper.

Details of all seepages are summarized within **Table 14** below.

Table 14: Summary of groundwater seepages

Location	Depth mAOD
WS02	18.33
WS03	19.54
	17.54
WS04	19.66
BH01	17.75
	1.55
BH03	19.26
	15.26
	9.76
BH04	17.33
	2.03
BH05	17.82
	9.32
	-1.18
BH06	19.01
	3.31
	-2.69
BH07	2.35
BH10	9.19
BH11	15.70
	2.70
	-2.30
BH12A	10.27
BH13	16.83
	2.33
BH15	8.74
	-1.16

9.2.2 Groundwater encountered during monitoring

Rest groundwater levels recorded during the monitoring programme are summarised in **Table 15** based on the data provided in **Appendix J**. Field data measurements are also shown in **Appendix K.**

Table 15: Summary of groundwater monitoring results

Monitoring well	Response zone stratum	TOC elevation (m AOD)	Depth to water (mb TOC)	Groundwater elevation (m AOD) – min.	Groundwater elevation (m AOD) – max.
BH03*	LC	21.76	1.53 to 3.87	17.89	20.23
BH05*	LC	21.82	1.51 to 1.60	20.22	20.31
BH11*	LC	21.70	2.21 to 3.37	18.34	19.49
BH12A	MG / LC	21.27	4.17 to 4.95	16.32	17.10
BH12A*	LAM	21.27	4.61 to 4.77	16.51	16.66
BH13*	MG	21.83	4.50 to 4.58	17.25	17.33
WS03	LC	22.04	Dry** / 1.45 to 1.56	Dry** / 20.48	20.59
WS05	MG	21.77	DRY	-	-
Notes:	All boreholes purged during first monitoring visit *groundwater samples taken during first and third monitoring visits and sent for analysis ** Installation dry during third monitoring visit only				

From the results to date, the groundwater table is considered to be within the London Clay at an elevation of between 16.32 and 20.59mAOD, within 5m below ground level. Perched water is present within the Made Ground at BH13 at c.17.30mAOD.

It should be noted that groundwater levels might fluctuate for a number of reasons including seasonal variations. On-going monitoring would be required to establish both the full range of conditions and any trends in groundwater levels.

9.2.3 Visual/olfactory evidence of groundwater contamination

No visual or olfactory evidence was noted.

9.3 Chemical laboratory results

The soil and groundwater testing results are presented in **Appendix L and N**, respectively.

Asbestos fibres were present within three samples of the Made Ground out of twenty tested.

9.4 Geotechnical laboratory results

The results of the geotechnical testing are discussed in Section 11 and presented in **Appendix M**.

9.5 Ground gas monitoring

The results of the ground gas monitoring and testing carried out are given in **Appendix J** and discussed in Section 10.

10 INVESTIGATION OF EXISTING STRUCTURES

10.1 General

During the course of the works two selected piles of the contiguous piled wall neighbouring Canal Side Studios, (BH01 & BH02), were exposed allowing for a full investigation of their diameter, structural integrity and toe depth. In addition, adjacent piles were located to assess the pile centre to centre spacing. Pile A41 of the piled transfer slab to the sewer could only be assessed via downhole geophysics due to space restrictions on site, inhibiting the inspection pit necessary for a full investigation.

Along the canal edge, four locations of the sheet pile and its capping beam were exposed (BH04, 07, 10 & 15) and investigated. Two trial pits were scheduled to record the thickness and depth of the historic canal wall at two locations (TP1 & TP2) which were coincident with BH04 and BH15 investigative pits, respectively.

In addition, the foundations of the party wall to Canal Side Studios were exposed at two locations (BH01 and TP3).The findings of various investigation techniques employed are briefly summarised in the following sections and a summary of all the information collated is given in through to below. Technical drawings of each location are presented within **Appendix I**.

10.2 Trial pit investigations

10.2.1 Shallow foundations

Two trial pits were scheduled alongside the party wall to Canal Side Studios to determine the buildings foundation configuration. It was previously understood by others that the existing foundations to the neighbouring building were piled, however, this was disproved and roughly formed strip foundations were discovered instead.

In BH01 trial pit, the strip foundation was founded on firm London Clay at a depth of 1.80m (21.95mAOD) with an outstand of 0.53m from the wall.

In TP3, the strip foundation was founded on firm London Clay at a depth of 2.00m (19.99mAOD) with an outstand of 0.40m from the wall.

Full details of which are presented within the exploratory logs in **Appendix I**.

10.2.2 Pile foundations

Two sets of piles were exposed each in BH01 and BH02. Details of the findings are presented in below.

Table 16: Pile construction details

Pile No.	Trial Pit	Cap thickness (m)	Measured Shaft diameter (mm)	Expected Diameter (m)
BH01*	BH01	None	600	600
BH02	BH02	None	600	600
A41 (piled transfer slab)	N/A	2.98**	N/A	600**
Notes: *Sleeved with 10mm steel **Pile measurements assumed to be as per drawing no. (A)-001				

Pile caps were non-existent in BH01 and BH02 and only the piles on the north eastern side of Block A were sleeved. Adjacent piles were exposed at both locations to confirm the pile centre spacings; a 1m pile centre spacing was measured which confirms the information provided in the Ove Arup, Design Calculations for Piles,1983 report.

In summary, the measured pile diameters are consistent with those expected from the archive drawings and therefore it is reasonable to assume that the pile dimensions for those of the piled transfer slab bridging the sewer, are accurate. However, it is understood these pile dimensions will be confirmed following demolition of Block A.

10.2.3 Sheet piles

Four trial pits were positioned alongside the canal wall at selected locations with consideration to space and the client's assets, details of which are presented in below:

Table 17: Sheet pile construction details

Location	Cap thickness (m)	Measured Sheet Thickness - top (mm)*	Measured Sheet Thickness - bottom (mm) *
BH04	0.50	9.0	9.3
BH07	0.50	7.7 – 9.3	No reading
BH10	0.50	7.7 – 9.0	7.5 – 8.9
BH15	0.50	9.1	9.2
Notes:	*Measurements taken from RSK Structural Investigation and Material Sampling report ref 1281066-01 (00), dated March 2019.		

10.2.4 Historic canal wall

Two trial pits were excavated to expose the historic canal wall; one within the north of Block A (TP1) and a second towards the south of Block B (TP2). Once the width of the wall was recorded at TP1, attempts were made to excavate down the western side of the wall to expose its depth, however, the historic backfill proved too competent to practically and economically progress with the equipment available on site and was therefore abandoned. A single tie rod was identified to be installed within the wall however its anchor point was not located/investigated.

Due to the restricted space at TP2, only the width of the historic wall could be recorded. It was also possible to record the historic wall location and width in within the inspection pits to BH07 and BH10. Details are presented in below.

Table 18: Historic canal wall dimensions

Location	Wall thickness (m)	Distance from sheet pile wall (m)
TP1	0.72	1.02
TP2	1.40	0.95
BH07	0.72	1.04
BH10	-	1.00

10.3 Investigation by percussive boring and geophysical testing

Percussive boring was undertaken adjacent to the locations of the selected piles and sheet piles outlined within Section 8.4. The general approach was to sink a single borehole as close to the selected pile or pile location and to install a sacrificial casing to facilitate downhole geophysics to estimate the pile toe depth.

The boreholes alongside the canal (BH04, 07, 10 & 15) were positioned in between the sheet pile and historic canal wall, serving a dual purpose in some locations (BH04 & 10 & 15) by chiselling through what was believed to be the historic walls foundations.

The depth to the masonry canal wall encountered in the boreholes along the canal wall are summarised in below.

Table 19: Masonry canal wall details determined during percussive boring

Borehole No	Depth to historic wall foundation base m bgl (m AOD)
BH04	3.30
BH10	4.70
BH15	4.60

The results of the down hole geophysical surveys are detailed in full in **Appendix S** and briefly summarised below.

The lengths of the three piles, four sheet pile locations (BH04, 07, 10 & 15), and the two historic canal wall locations (TP1 & 2) were investigated by geophysical parallel seismic surveying techniques. Additionally, down hole magnetic tests were undertaken to investigate the depth of reinforcing within the contiguous piles in addition to the sheet piles.

The repositioning of BH06 to outside Block A to investigate Pile A41 was interrupted further by underground services, with its final position being over 1.0m away from the selected pile. For greater accuracy, geophysical testing should be within 1.0m of the subject pile.

The results are summarised in below.

Table 20: Pile/wall details from geophysical testing

BH No	Pile description	Expected Toe Depth [1, 2] mbgl (mAOD)	Horizontal off-set of Borehole from Pile Shaft / Sheet Pile (m) [4]	Parallel Seismic Estimated Pile Depth mbgl (mAOD)	Magnetics Estimated Depth of Pile/Reinforcing mbgl (mAOD)
BH01	Contiguous	21.70 (2.05)	0.60	22.94 (0.81) +/- 1.00m	24.67 (-0.92) +/- 1.00m
BH02	Contiguous	21.70 (0.32)	0.40	22.13 (-0.11) +/- 0.50m	23.65 (-1.63) +/- 1.00m
BH04	Sheet pile	Unknown	0.80	6.79 (16.94) +/- 0.50m	7.64 (16.09) +/- 1.00m
	Historic wall	3.50 (20.23)	-	3.39 (20.34) +/- 1.00m	N/A
BH06	Contiguous (Pile A41)	24.51 (-2.70)	1.60 [5]	18.43 (3.38) +/- 1.00m	N/A [3]
BH07	Sheet pile	Unknown	0.80	7.48 (16.17) +/- 0.50m	7.94 (15.71) +/- 1.00m
BH10	Sheet pile	Unknown	0.80	7.28 (16.41) +/- 1.00m	7.25 (16.44) +/- 1.00m
BH15	Sheet pile	Unknown	0.50	8.08 (15.66) +/- 0.50m	8.08 (15.66) +/- 1.00m
	Historic wall	3.50 (20.24)	-	5.14 (18.60) +/- 1.00m	N/A

Notes:
[1] Pile depth corrected from Arup report and based on measured elevations.
[2] Canal wall depth corrected from GDP information and based on measured elevations.
[3] Low confidence in signal generated during magnetic survey.
[4] Horizontal off-set is recorded at surface and borehole inclination is not accounted for.

BH No	Pile description	Expected Toe Depth [1, 2] mbgl (mAOD)	Horizontal off-set of Borehole from Pile Shaft / Sheet Pile (m) [4]	Parallel Seismic Estimated Pile Depth mbgl (mAOD)	Magnetics Estimated Depth of Pile/Reinforcing mbgl (mAOD)
	[5] Pile shaft not exposed so assumed from archive drawings. Borehole inclination is not accounted for.				

The interpreted depth of the pile (A41) at BH06 does not have a high level of confidence and should not be relied upon for design purposes. It is recommended that further investigation of the contiguous piles to the Thames Water Sewer is carried out at a later stage in development when space restrictions are not limited, such as following demolition of the existing buildings.

As seen in the table above, the parallel seismic testing and magnetometer testing estimated different pile toe depths, varying up 1.73m in difference, largely a function of the fluxgate sensor spacing and the sensitivity settings on the magnetometer equipment. In the first instance, the parallel seismic results should be on for any capacity checks and the magnetometer for reinforcement depths within the piles.

Notwithstanding the standard potential margin of error outlined in the table, the results of the parallel seismic testing at all remaining pile locations produced a clear inflection and thus the piles toe depths are estimated with a good degree of confidence.

The historic canal wall has been recorded to depths similar to those recorded via percussion drilling and therefore can also be relied upon with a good degree of confidence.

A comparison with the archive drawings for the contiguous piles where information is provided is shown in **Table 21** below.

Table 21: Contiguous pile toe elevation comparison

Borehole No	Pile No.	Archive Toe Elevation (m AOD)	Estimated Toe Elevation (m AOD)	Toe Elevation Difference (Archive vs Calculated) [1] (m)	% Difference to Archive Toe Elevation [1]
BH01	Contiguous	2.05	0.81	1.24	5.71
BH02	Contiguous	0.32	-0.11	0.43	1.98
BH06	Contiguous (Pile A41)	-2.70	3.38	-6.08 [2]	-24.81 [2]
Notes:	[1] Negative value indicates shorter than archive drawings [2] Low degree of confidence.				

In summary, the estimated pile toe depths from the parallel seismic testing typically show the contiguous piles investigated at the northern boundary are generally between 0.43m and 1.24m longer than those in the Arup report. Although the use of parallel seismic testing allows a good indication of pile lengths to be established, it is well known that there are some possible accuracy issues that may arise concerning the way in which the data obtained is interpreted along with the various correlation and correction factors that are applied, particularly where borehole offset is greatest, such as in BH06.

Given the pile lengths derived at BH01 and BH02 are within or close to the accepted potential margins of error for the seismic technique when compared to the archive information, it is recommended that the archive pile length of 21.70m is adopted in any future assessment of the wall's geotechnical capacity.

Due to the constraints imposed on the survey completed at BH06 and uncertainty regarding the estimated length, it is recommended that further investigation is completed post demolition to assess the length of the piles adjacent to the Thames water sewer.

Canal Wall

With regards to the canal wall, provides a comparison of the archive drawings with the information obtained by drilling and geophysical parallel seismic surveying techniques.

Table 22: Canal wall toe elevation comparison

Borehole No	Archive Toe Elevation (m AOD) [1]	Toe Elevation Estimated by borehole (m AOD)	Toe Elevation Estimated by geophysics (m AOD)	Toe Elevation Difference (Archive vs Calculated) [2] (m)	
				Borehole	Geophysics
BH04	20.23	20.43	20.34	-0.20	-0.11
BH10	20.19	18.99	-	1.20	-
BH15	20.24	19.14	18.60	1.10	1.64
Notes:	[1] Depth corrected based on measured elevations [2] Negative value indicates shorter than archive drawings				

It can be seen that the depth of the canal wall at its northern end in the vicinity of BH04 shows a good correlation between the techniques adopted and the canal wall is estimated to be in the order of approximately 3.50m deep (c. 20.23mAOD). At the southern end of the site, there is less correlation between the techniques adopted to determine the wall depth, however, both techniques indicated the canal wall appears to be in the order of 1.10m to 1.50m deeper than anticipated from the archive drawings.

10.4 Concrete coring and materials testing

Concrete cores were extracted from two piles and at four locations along the sheet pile wall and tested for a range of parameters to establish concrete strength and condition. Single vertical cores, nominal 100mm diameter were extracted vertically from the piles exposed in BH01 & BH02 and from the sheet pile capping beam at BH04, 07, 10 and 15. The laboratory materials testing results are provided in full in **Appendix T** and summarised in and below.

Table 23: Summary of Results of On-Site Materials Testing

Core	Location	Element	Visual Inspection	Cover Depth (mm)	Depth of Carbonation (mm)
C1	BH02	Pile	The matrix in the upper section of the core was observed to be friable and weaker than in the remainder of the core. The depth of the weaker area varied from 2 mm to 22 mm. A crack was observed running around approximately half of the diameter of the core. The crack ran from the upper surface and reached a maximum depth of 80 mm. Fine cracks were observed associated with the main crack. Common secondary calcite deposits were observed in the crack and fine cracks. Secondary calcite was also observed lining air voids in the upper, weaker section of the core. Secondary ettringite was observed lining air voids, aggregate partings and cracks. This was commonly observed in thin section A (0-45 mm depth) and rarely observed in thin section B (235-280 mm depth). Rare alkali-silica gel was observed in cracks in a single chert particle.	71	1
C2	BH15	Capping Beam	The concrete generally appeared in sound condition with evidence of very minor leaching. Sporadic microcracks were observed running through the cement matrix. Rare secondary ettringite was observed lining air voids and microcracks.	33	0
C3	BH10	Capping Beam	The concrete generally appeared in sound condition with evidence of very minor leaching. Rare microcracks were observed running through the cement matrix. Sporadic air voids were lined with small amounts of secondary ettringite.	42	0
C4	BH04	Capping Beam	The concrete generally appeared sound with evidence of very minor leaching. Rare microcracks were observed running through the cement matrix. Common air voids were lined with small amounts of secondary ettringite.	35	0
C5	BH07	Capping Beam	(Not examined)	37	0
C6	BH01	Pile	The concrete generally appeared sound with evidence of very minor leaching. Rare microcracks were observed running through the cement matrix. Sporadic air voids were lined with small amounts of secondary ettringite.	55	0

Table 24: Summary of Results of Laboratory Materials Testing

Ref.	Location	Element	Corrected in situ compressive strength of concrete (N/mm^2)	Chloride ion content (%) by mass of cement (1)	Cement content, (kg/m^3) (2)	Water / cement Ratio	Sulfate content (% mass of cement)
C1	BH02	Pile	42.4	0.18	374	0.6	2.2
C2	BH15	Capping Beam	41.4	0.17	342	N/T	2.5
C3	BH10	Capping Beam	N/T	0.18	376	0.5	2.8
C4	BH04	Capping Beam	31.5	0.18	331	N/T	2.7
C5	BH07	Capping Beam	32.1	0.16	412	N/T	2.5
C6	BH01	Pile	40.6	0.13	392	0.6	2.1

N/T = Not tested

10.4.1 Petrographic Examination

C1 – BH02, pile

Nominal 20 mm, natural chert (flint) gravel coarse aggregate and natural quartzitic sand fine aggregate, bound by a Portland-type cement matrix. Apparently well mixed and exhibiting good compaction. Excess voidage 3 %. Apparent water/cement ratio was estimated as being in the low end of the normal range (say, 0.35 to 0.45). However, the weak upper section of the core exhibited a higher microporosity. The upper end surface of the sample was fractured, whilst the lower end surface was freshly fractured.

The depth of carbonation was difficult to determine due to the degraded nature of the upper section of the core. Areas of uncarbonated cement matrix were surrounded by secondary calcite deposits. Possible popcorn carbonation was observed in the areas of cement matrix.

A crack was observed running around approximately half of the diameter of the core. The crack ran from the upper surface and reached a maximum depth of 80 mm. Fine cracks were observed associated with the main crack. The aggregate was observed to stand slightly proud of the cement matrix in the upper ~50 mm of the core.

A 5 x 4 mm clay rich area was observed at the upper surface. An approximately 35 x 25 mm metal object was present at approximately 10 mm depth from the upper surface. A larger dark greyish brown area was present around, and likely caused by, the metal.

C2 – BH15, capping beam

Nominal 20 mm, natural chert (flint) gravel coarse aggregate and natural quartzitic sand fine aggregate, bound by a Portland-type cement matrix. Apparently well mixed and exhibiting good compaction. Excess voidage 3.0 %. Apparent water/cement ratio was estimated as being in the low end of the normal range (say, 0.35 to 0.45). The upper end surface of the sample was rough with exposed aggregate, whilst the lower end surface was freshly fractured. Sporadic microcracks were observed running through the cement matrix. Rare secondary ettringite was observed lining air voids and microcracks.

C3 – BH10, capping beam

Nominal 20 mm, natural chert (flint) gravel coarse aggregate and natural quartzitic sand fine aggregate, bound by a Portland-type cement matrix. Apparently well mixed and exhibiting good compaction. Excess voidage 3 %. Apparent water/cement ratio was estimated as being in the low end of the normal range (say, 0.35 to 0.45). The upper end surface of the sample was rough with exposed aggregate, whilst the lower end surface was freshly fractured. Rare microcracks were observed running through the cement matrix. Sporadic air voids were lined with small amounts of secondary ettringite.

C4 – BH04, capping beam

Nominal 20 mm, natural chert (flint) gravel coarse aggregate and natural quartzitic sand fine aggregate, bound by a Portland-type cement matrix. Apparently well mixed and exhibiting good compaction. Excess voidage 3 %. Apparent water/cement ratio was estimated as being in the low end of the normal range (say, 0.4 to 0.5). Rare microcracks were observed running through the cement matrix. Common air voids were lined with small amounts of secondary ettringite. The upper end surface was rough with exposed aggregate, whilst the lower end surface was freshly fractured. Rare microcracks were observed running through the cement matrix. Common air voids were lined with small amounts of secondary ettringite.

C6 – BH01, pile

Nominal 20 mm, natural chert (flint) gravel coarse aggregate and natural quartzitic sand fine aggregate, bound by a Portland-type cement matrix. Apparently well mixed and exhibiting good compaction. Excess voidage 2 %. Apparent water/cement ratio was estimated as being in the low end of the normal range (say, 0.35 to 0.45). Both the upper and lower end surfaces of the sample were freshly fractured. Rare microcracks were observed running through the cement matrix. Sporadic air voids were lined with small amounts of secondary ettringite.

11 GEO-ENVIRONMENTAL ASSESSMENT

11.1 Refinement of initial CSM

With the exception of less made ground encountered than anticipated, the ground conditions identified during the ground investigation are similar to those anticipated within the initial CSM and therefore no changes are required.

11.1.1 Linkages eliminated after refinement of the initial CSM

Based on the discussion above, no linkages have been eliminated after refinement of the initial CSM.

11.1.2 Linkages added after refinement of the initial CSM

No linkages have been added after refinement to the initial CSM.

11.2 Linkages for assessment

In line with CLR11 (Environment Agency, 2004), there are two stages of quantitative risk assessment, generic (GQRA) and detailed (DQRA). The GQRA comprises the comparison of soil, groundwater, soil gas and ground gas results with generic assessment criteria (GAC) that are appropriate to the linkage being assessed. This comparison can be undertaken directly against the laboratory results or following statistical analysis depending upon the sampling procedure that was adopted.

Following the refinement of the initial CSM, the potentially complete contaminant linkages that require further assessment and the methodology of assessment are presented in **Table 25**.

Table 25: Linkages for GQRA

Potentially relevant contaminant linkage	Assessment method
Soil	
1. Oral, dermal and inhalation exposure to impacted soil, soil vapour and dust by future residents	Human health GAC in **Appendix O** for a proposed commercial end use.
2. Inhalation exposure of future residents to asbestos fibres	Qualitative assessment based on the asbestos minerals present, their form, concentration, location and the nature of the proposed development.
3. Uptake of contaminants by vegetation potentially impacting plant growth (phytotoxicity)	Comparison of soil data to GAC in **Appendix P** for phytotoxicity.
4. Contaminants permeating potable water supply pipes	Comparison of soil data to GAC in **Appendix Q** for plastic water supply pipes using UKWIR (2010) guidance.

Potentially relevant contaminant linkage	Assessment method
Ground Gas	
5. Concentrations of methane and carbon dioxide in ground gas entering and accumulating in enclosed spaces or small rooms in new buildings, which could affect future site users. For methane this could create a potentially explosive atmosphere, while death by asphyxiation could result from carbon dioxide.	Gas screening values (GSV) have been calculated using maximum methane and carbon dioxide concentrations with maximum flow rates recorded at the site. The GSV have been compared with the revised Wilson and Card classification presented in BS8485.

11.3 Methodology and assessment of soil results

The analysis of laboratory results relating to soil samples submitted for testing, including leachate analysis, is included in the following sections.

Chemical analyses have been performed on 20 selected samples of made ground from the site. All soil samples scheduled for laboratory testing were also inspected visually on receipt at the laboratory for the presence of materials potential containing asbestos, e.g. fragments of asbestos-cement products.

The chemical results are presented within **Appendix L**.

11.3.1 Oral, dermal and inhalation exposure with impacted soil by future occupants/site users

End users of the site are defined as those who are exposed to sources of contamination on a regular and predictable basis. The development comprises a mix of commercial and residential end use with the commercial use on the ground floor. Therefore the results have been compared against commercial scenario GAC, which is considered appropriate for the site, given the scale and nature of the development. A soil organic matter (SOM) of 2.5% has been selected since laboratory results within the made ground range between 0.17 % and 1.56 %.

Results indicate that all contaminants are below the relevant GAC. Therefore, it is considered that a relevant contaminant linkage does not exist and that the site may be regarded as suitable for the proposed end use.

Furthermore, the proposed development design effectively mitigates direct exposure (oral, dermal and dust inhalation pathways). The key features associated with the development design include:

- Basement car parking. The majority of the made ground soils will be removed from site as part of the basement excavation and thus essentially removing the majority of the potential source of contamination
- The site will be covered by hardstanding at ground level
- The limited landscaping will include raised planter beds

As previously noted, the majority of the made ground will be removed off site to accommodate the proposed development. The proposed formation level of the basement garages will sit in London Clay, as indicated in the cross section in **Figure 10**.

The site investigation data included in **Appendix I** indicates the absence of visual / olfactory contamination within the London Clay. The maximum detected concentration of VOC obtained during the site works (utilising a PID detector) was 0.3 ppm. It is therefore concluded that volatile hydrocarbons are unlikely to be present in the London Clay.

Furthermore, the proposed development includes basement car parking and therefore the proposed design will include adequate venting of the basement.

11.3.2 Inhalation exposure of future occupants/site users to asbestos fibres

The laboratory screening for asbestos identified detectable asbestos fibres within three samples of made ground from BH05, BH13 and WS04. These samples were then further analysed and the presence of fibres of chrysotile and amosite were confirmed, with a maximum of 0.006 % by weight present in the sample from borehole BH05 at 0.70mbgl.

However, the made ground in these locations will be completely removed as part of the basement excavation, such that the asbestos inhalation contamination pathway will be broken.

It should also be appreciated that asbestos is often present in discrete areas and can be found in more extensive quantities during ground works. The presence of asbestos should be addressed in the contractor’s risk assessment and method statements to avoid exposure of sensitive receptors to asbestos in soils during the ground works.

11.3.3 Uptake of contaminants by vegetation potentially inhibiting plant growth (phytotoxicity)

The results have been compared with the GAC presented in **Appendix P** for this linkage.

The comparison of testing results against the adopted GAC is summarised in **Table 26.** Only those determinants where exceedances have been reported are included within the table.

Table 26: Chemical testing data summary table for phytotoxic effects

Determinant	No. of samples tested	GAC (mg/kg)	No of exceedances	Maximum concentration (mg/kg)	
				Value	Location / depth
Copper	20	200	2	348 268	BH06 @ 0.70m WS04 @ 0.70m
Mercury	20	1	1	2.41	BH15 @ 1.50m

The results indicate that a relevant pollutant linkage may exist associated with phytotoxic effects within the made ground soils sampled from the site.

It should be noted that the majority of the made ground will be removed off site and proposed soft landscaping will be limited to risen planters. However, should any additional soft landscaping, in the form of tree pits, be included into the design at the later stage a clean imported topsoil should be used to provide a suitable growing medium. A landscape architect should be consulted to confirm required thickness of imported topsoil.

11.3.4 Impact of organic contaminants on potable water supply pipes

For initial assessment purposes, the results of the investigation have been compared with the GAC presented in **Appendix Q** for this linkage, which are reproduced from *UKWIR Report 10/WM/03/21. Guidance for the Selection of Water Supply Pipes to be used in Brownfield Sites* (UKWIR, 2010).

The results indicate that a relevant linkage is likely to exist associated with organic contaminants and therefore pollutant polyethylene (PE) and polyvinyl chloride (PVC) water supply pipes are expected to be unsuitable for use on the development, unless remedial measures are implemented that mitigate the risk.

It should be noted that at the time of this investigation the future routes of water supply pipes had not been established, hence the investigation and sampling strategy may not be fully compliant with UKWIR recommendations. Consequently, a targeted investigation and specific sampling/analytical strategy may be required at a later date once the route(s) of the supply pipe(s) are known. In addition, it is recommended that the relevant water supply company be contacted at an early stage to confirm its requirements for assessment, which may not necessarily be the same as those recommended by UKWIR.

11.4 Ground gas risk assessment

11.4.1 Appropriate guidance

The risks to development from ground gases have been assessed in accordance with BS8485:2015, which provides guidance on ground gas (methane and carbon dioxide) characterisation and hazard assessment, as well as a framework for the prescription of protection measures within new buildings.

The process involves characterising the gas hazard from combining the qualitative assessment of risk (using the conceptual site model) with ground investigation data so that a ‘characteristic situation’ (CS) can be derived for the site. Characteristic situations range from CS1 to CS6, the higher the CS the higher the hazard potential. Protection measures within new buildings can be prescribed using a point scoring system, taking in to consideration the CS and the proposed building type.

BS8485 indicates that the gas hazard can be characterised using the following methods:

- an empirical semi-quantitative approach using gas monitoring data to determine the ‘characteristic situation’ of the site (or zones of the site) and subsequent protective measures (Wilson and Card approach).
- an empirical semi-quantitative approach using TOC data to determine the ‘characteristic situation’ of the site (or zones of the site) and subsequent protective measures (CL:AIRE RB17 approach), or
- detailed quantitative assessment methodologies

For the purpose of this assessment, the second approach listed above, supported by the first approach, has been used to characterise the gas hazard and provide advice on the protective measures likely to be required within new buildings at the site.

11.4.2 Summary of the refined conceptual site model for ground gas

In the assessment of risks and selection of appropriate mitigation measures, BS8485 highlights the importance of the conceptual model. In summary, potential sources of ground gas within influencing distance of the site have not changed and are still relevant to the initial CSM, which includes all Made Ground associated with the demolition of former buildings on site.

However, the majority of made ground across the site will be removed during excavation of the basements and therefore the source of ground gases will be removed. Only a thin strip of made ground along the canal edge and a residual amount on the made ground at the site's frontage will be retained.

The made ground along the canal edge generally comprised a granular demolition rubble comprising gravel and cobble sized fragments of brick and concrete in a sandy matrix, locally with rare charcoal fragments and occasional pieces of timber (BH07). No traces of organic material other than the timber pieces were found and a maximum Total Organic Carbon (TOC) value of 1.56% was recorded.

As such, the Made Ground, which will be retained, is unlikely to generate significant quantities of ground gases.

This assessment has been undertaken to assess risks to building structures and proposed end users. The assessment has not taken into consideration the health and safety of construction workers. Risks may still be present to construction workers especially where works include the entry into excavations within the ground. Construction workers should undertake appropriate risk assessments and risks should be managed through health and safety procedures and safe systems of work.

The risk assessment has been undertaken based on the current understanding of the CSM.

11.4.3 Empirical semi-quantitative approach using TOC data (CL:AIRE RB17 approach)

The above empirical semi-quantitative approach is based upon the pragmatic approach to gas assessment documented in CL:AIRE RB17, wherein consideration of TOC results and made ground thickness is used to assess the risk posed by ground gas.

Made Ground as source of gas is generally considered to pose a low risk due to its low gas production potential, particularly where there is a low organic content. In accordance with Figure 6 from BS 8576:2013, '*Guidance on investigations for ground gas – Permanent gases and Volatile Organic Compounds (VOCs)*', the made ground found on site falls under the 'very low' generation potential category, as defined by the TOC results and the made ground make-up, for which the gas monitoring requirements may be considered not necessary.

As noted above, the end development will result in removal of the majority of the made ground, leaving less than 1000m^3 of made ground material, nominally less than 5m in thickness.

In line with RB17 guidance, the residual made ground is unlikely to generate significant quantities of ground gases and therefore ground gas monitoring is not required.

Gas protection measures are then defined in Table 1 of the guidance. The maximum detected TOC concentration of 1.56% was encountered in the made ground, which marginally exceeds benchmark for Characteristic Situation 1 (CS1) of 1%. However, the average concentration of TOC in the made ground is 0.67% and therefore a Characteristic Situation 1 (CS1) may be appropriate for the site.

In addition, six gas monitoring visits have been undertaken to date to confirm the above assessment and is discussed in Section 10.4.4 below.

11.4.4 Empirical semi-quantitative approach using borehole monitoring data (Wilson and Card approach)

11.4.4.1 Permanent gases – methane and carbon dioxide

The empirical semi quantitative approach using gas monitoring data is based on calculations of the gas screening value (GSV). BS8485 defines the GSV as the *'flow rate (l/hr) of a specific hazardous gas representative of a site or zone, derived from assessment of borehole concentration and flow rate measurements and taking account of all other influencing factors, in accordance with a conceptual site model'.*

Once derived for both methane and carbon dioxide the GSVs are compared to the thresholds presented in Table 2 of BS8485, so that a CS can be determined for the site, or a zone. It is important to note that the GSV thresholds are guideline values and not absolute. The GSV thresholds may be exceeded in certain circumstances, if the site conceptual model indicates it is safe to do so. Similarly, consideration of additional factors such as very high concentrations of methane, should lead to consideration of the need to adopt a higher risk classification than the GSV threshold indicates.

The results of the ground gas monitoring and testing undertaken at the site are given in **Appendix J**.

Six monitoring rounds have been completed and the results are presented in **Table 27** below.

Table 27: Summary of ground gas monitoring results

Borehole	Response zone/ stratum	Probable source(s) of ground gas	Number of monitoring visits	Methane (%)	Carbon dioxide (%)	Oxygen (%)	Flow rate (l/hr)	Water level (m b TOC)	Atmospheric pressure (mbar)
BH12A	MG	MG	6	<0.1	<0.1 to 0.2	20.6 to 21.0	<0.1 to 0.2	4.20 to 4.95	1004 to 1025
BH13	MG	MG	6	<0.1	0.4 to 1.0	17.4 to 19.9	<0.1 to 0.2	4.50 to 4.58	1003 to 1025
WS05	MG	MG	6	<0.1	0.2 to 0.6	19.7 to 20.6	<0.1 to 0.1	DRY	1004 to 1025

Note: *MG – Made Ground,*

Steady state gas concentrations and flows are presented in this table.

BS8485 suggests that the GSV should be derived by multiplying the worse credible (worst case) recorded flow value in any standpipe in that strata or zone with the maximum gas concentration in any other standpipe in that strata or zone. Further guidance is given in BS8485 section 6.3.

Considering the assessment of the gas monitoring results the following maximum GSVs have been derived for the site.

- Methane GSV (<0.0002 l/hr) = methane concentration (<0.1 % v/v)/100 x flow rate (0.2 l/hr)
- Carbon Dioxide GSV (<0.002 l/hr) = carbon dioxide concentration (1.0 % v/v)/100 x flow rate (0.2 l/hr).

Based on the GSVs derived and the method for determining the CS presented within Table 2 of BS8485, the site has been characterised as CS1.

11.4.5 Implications

Based on the current understanding of the conceptual site model and the assessment undertaken, the site has been classified as CS1. Considering the foregoing and in accordance with BS8485, ground gas protective measures are not considered necessary within proposed buildings.

12 PRELIMINARY WASTE ASSESSMENT

In accordance with the definition provided in the Waste Framework Directive (WFD), materials are only considered waste if 'they are discarded, intended to be discarded or required to be discarded, by the holder'. Naturally occurring soils are not considered waste if reused on the site of origin for the purposes of development. Soils such as made ground that are not of clean and natural origin (irrespective of whether they are contaminated or not) and other materials such as recycled aggregate, do not become waste until the criteria above are met. Further background information is provided in **Appendix H**.

Excavation arisings from the development may therefore be classified as waste if surplus to requirements or unsuitable for reuse. The following assessments assume the material tested is classified subsequently as waste.

RSK recommends that a Sampling Plan be prepared to support any waste classifications and hazardous waste assessments, prior to any material being excavated. Given the level of data obtained, scale of the development and heterogeneity of the site soils, the following assessment should be considered indicative and further assessment should be undertaken following the preparation of a waste sampling plan.

12.1 Hazardous waste assessment

Technical Guidance WM3 (EA, 2018) sets out in Appendix D requirements for waste sampling. It is a legal requirement to correctly assess and classify waste. The level of sampling should be proportionate to the volume of waste and its heterogeneity. The preliminary assessment provided below is based only upon the available sample results and may not be sufficient to adequately classify the waste.

12.1.1 Chemical contaminants

Envirolab, an RSK company, has developed a waste soils characterisation assessment tool (HASWASTE), which follows the guidance within Technical Guidance WM3. The analytical results have been assessed using this tool to assess the hazardous properties to support potential off-site disposal of materials in the future. Note that it is ultimately for landfills to confirm what wastes they are able to accept within the constraints of their permit.

With the exception of those listed in **Table 28** below, none were found to have hazardous properties on this assessment. This suggests that if applicable, the waste would require disposal at a suitably permitted inert or non-hazardous waste landfill.

The results are summarised in **Table 28** and presented in full in **Appendix R**.

Table 28: Results of waste soils characterisation assessment (HASWASTE)

Sample ref/ depth	Hazardous properties identified
BH01 @ 1.00m	HP8 Corrosive due to pH
BH10 @ 2.00m	HP8 Corrosive due to pH
BH10 @ 3.00m	HP8 Corrosive due to pH

Only three out of fifteen samples tested were assessed as potentially hazardous waste as a result of having hazardous corrosive properties associated with elevated pH values. However, it is noted from the exploratory logs that these made ground samples all contained fragments of concrete and these are considered the most likely cause of the elevated pH values. In such instances, the soil may be classified as “not hazardous” subject to confirming with the receiving landfill and they may request additional testing of the soils to confirm pH values with any concrete fragments removed.

12.1.2 Asbestos within waste soils

Technical Guidance WM3 requires that within a mixed waste the separately identifiable wastes be assessed separately.

For instance, where waste soil contains identifiable pieces of asbestos (visible to the naked eye) the asbestos should, where feasible, be separated from the soil and classified separately. This should be disposed of within a hazardous, stable non-reactive hazardous waste landfill or a special cell in a non-hazardous waste landfill.

Samples of potential asbestos containing material were collected from site and analysed for the presence of asbestos, the results of which are presented in **Appendix L**. Analysis confirmed that asbestos fibres are present within samples BH05 at 0.70m, BH13 at 0.70m and WS4 at 0.70mbgl. Visible asbestos containing material was not identified on-site.

All samples where fibres were present have been analysed for percentage asbestos fibres by weight, the results of which are presented in **Appendix L**. Analysis confirmed that percentage of asbestos fibres is less than 0.1% by weight and therefore the waste can be disposed of within a non-hazardous waste landfill, which is able to accept asbestos at non-hazardous concentrations, subject to the WAC assessment below.

12.2 WAC assessment

Samples BH01 at 1.00m, BH10 at 2.00m, BH10 at 3.00m, BH15 at 1.50m and WS06 at 0.60m were submitted for waste acceptance criteria (WAC) testing for the full WAC Inert, SNRHW and Hazardous Landfill Suite, the results of which are presented in **Appendix R**.

The results of the HASWASTE assessment indicated that the made ground may be classed as not having hazardous properties, including where elevated pH levels are the result of crushed concrete fragments within the sample. In these instances, the concrete can be removed and the made ground retested for pH to confirm the assessment. If the crushed concrete is too fine, the sample should be classified as “HP8 Corrosive Hazardous Waste due to pH”. The results of the WAC testing should be sent to the landfill operator to confirm whether they are able to accept the waste.

The results of the WAC testing indicate that the leaching limit values for inert waste landfill were not exceeded in the samples from BH01 at 1.00m and BH15 at 1.50m.

Two of the WAC results fail due to Total Dissolved Solids (TDS) exceeding the inert limits, however, the sulphate limit is not exceeded and this can be used instead of TDS. In these instances, the samples may be classified as inert waste.

Only one sample from WS06 at 0.60m exceeds the inert limits for both TDS and sulphate and the sample may therefore not be classified as inert and may be unsuitable for disposal at an inert landfill. However, this should be checked with the landfill operator receiving the waste to confirm their threshold requirements as some operators have different threshold limits. Otherwise, the waste should be disposed of at a landfill or treatment facility which is permitted to take non-hazardous waste as a minimum or further testing carried out to confirm the waste assessment.

12.3 Groundwater

When there is an intention to discard groundwater, chemical test results will indicate the appropriate disposal options. This could include disposal to treatment facility, via consent (issued by the water authority) to foul sewer or via consent (issued by the EA) to a watercourse or land.

Chemical results of samples of groundwater collected during the monitoring rounds are provided in **Appendix N**.

13 GEOTECHNICAL ASSESSMENT

13.1 Proposed development

The proposed redevelopment will involve the demolition of the existing Ugly Brown Building (UBB) and erection of 6 new buildings ranging in height from 2 storeys to 12 storeys above ground and up to 2 basement levels comprising a mixed-use business and residential development.

The proposed basement level at the site varies from 13.40m to 19.10m AOD while the canal water level is at approximately 23m AOD and canal bed is understood to be at average of circa 21m AOD. Plot A will have a single basement at 17.50m and 18.00m AOD. Plot B will also have a single basement but at two different levels; the deeper, double height area adjacent to St Pancras way at 16.30m AOD, and along the canal edge at 19.10m AOD.

None of the existing foundations, which comprise piled foundations, are to be reused as part of the new scheme, with the exception of the existing piles spanning the Thames Water Sewer beneath the southern part of Plot A which will be retained and included in the new scheme, and possibly the contiguous pile wall on the northern site boundary, subject to an assessment of the structural and geotechnical capacity of the wall.

The current load takedown information is provided within **Appendix B**.

<u>Block A</u>

The basement perimeter walls will be of contiguous pile construction with uniformly distributed loads (UDL) of 125kN/m along the eastern wall, 190kN/m along the western wall and 250kN/m along the northern wall (either new or reuse of existing). The proposed piles adjacent to the sewer will also experience a proposed UDL load of 265kN/m. The existing loads over the sewer are currently distributed by transfer slab onto pile caps on either side. The proposed load, however, will be transferred solely onto the southern wall.

Maximum column loads of 15,000kN will be transferred onto pile groups of up to six piles.

<u>Block B</u>

Similar to Block A, the basement perimeter walls be also be of contiguous pile construction with a proposed UDL of 85kN/m along the southern wall, 60kN/m along the east and between 125kN and 190kN/m to the west.

Maximum column loads of 11,900kN will be spread across similar pile caps of up to six piles.

New elevator shaft wall loads of 1,000kN/m run are proposed within both Block A and B.

13.2 Key geotechnical hazards/development constraints

The key risks identified from the available ground investigation data are discussed below:

- London Clay soils of medium to high volume change potential are present, but the potential risks posed are mitigated by the proposed piled scheme.

- Groundwater seepages have been encountered during the investigation and subsequent monitoring has recorded water levels above the proposed basement levels.
- Swelling/heave of the underlying clay soils due to removal of overburden soils to form the basements.
- Filled and made ground – ground levels raised along the eastern site boundary adjacent to the canal wall and made ground associated with demolition of former Granary Building.
- Adverse ground chemistry due to elevated sulphates in the London Clay.
- Existing sub-structures – sub-structures associated with the current and former development phases, including a contiguous piled wall along the northern site boundary
- Strip foundations to neighbouring Canal Side Studios shallower than the anticipated piled foundations originally thought.
- The buried former masonry canal wall potentially clashes with the proposed piled wall.
- The crown of the Thames Water Sewer crossing beneath Block A will be sensitive to ground movements.
- Construction adjacent to Regent's Canal – stability issues for the sheet pile retaining structure.

13.3 Assessment of existing foundations

Where it is intended to use the contiguous piles on the northern boundary and those for the piled transfer slab adjacent to the Thames Water sewer, the piles have been assessed in terms of their geotechnical vertical capacity by way of back analysis calculations. The findings are discussed in detail within the following sections.

The lateral capacity of the contiguous piles and sheet piled canal walls and associated ground movements will be covered in a separate report.

13.3.1 Back analysis of existing pile foundations

Ground model and soil parameters

The preliminary ground model summarised in has been adopted for the purpose of the preliminary foundation design recommendations.

Table 29: Ground model derived from ground investigation

Stratum	Elevation at top of stratum (mAOD)	Thickness (m)
Made ground	22.00	2.00
London Clay Formation	20.00	23.00
Lambeth Group	-3.00	Proven to -18.30mAOD [1]

Stratum	Elevation at top of stratum (mAOD)	Thickness (m)
Notes: [1] base of stratum not proven		

Groundwater has been recorded within the installations at elevations ranging between 16.32 and 20.59mAOD (within 5m below ground level).

The soil parameters used have been determined from in-situ testing undertaken in the boreholes, results of laboratory testing carried out on representative soil samples, published data and the estimated pile geometry obtained by a combination of intrusive and non-intrusive investigation techniques.

The ground model and soil parameters adopted for the back analysis are outlined in below.

Table 30: Ground model and soil parameters

Stratum	Bulk Density (kN/m^3)	Undrained Shear Strength [(1)] (kN/m^2)	Adhesion Factor (α)
London Clay Formation	20.0	80 + 4.39 z	0.5
Lambeth Group	20.3	180 + 13 z	0.5
Notes: [(1)] Figure 9			

Pile geometry and concrete strength

A summary of the information obtained from record drawings during the investigation on the pile characteristics is summarised in **Table 31** below.

Table 31: Pile geometry and characteristics

BH / Pile No.	Pile Cut off Level (mAOD)	Pile Shaft Diameter (m)	Estimated Pile Toe Level (mAOD)	Archive Toe Level (mAOD)	Corrected In Situ Cube Strength (N/mm^2)
BH01	23.400	0.60	0.81	2.05	40.6
BH02	21.670	0.60	-0.11	0.32	42.4
BH06 (A41)	18.825[1]	0.60[1]	3.38	-2.70	40.0[2]
Notes: [1] Assumed from archive drawings [2] Assumed concrete strength in the absence of testing					

For the existing piles investigated, the vertical pile capacities have been assessed using the Pile Version 19.7 computer package supplied by Oasys Ltd. The soil parameters used in the analysis are based on the ground model and pile geometries outlined above. The comparison of the parallel seismic testing and the archive drawings highlighted that the piles at BH01 and BH02 were slightly longer and in the case of BH06/Pile A41 significantly shorter, although with a low level of confidence in the geophysical test results for this location due to the constraints imposed. Therefore, as outlined in Section 10.3, the lengths presented on the archive drawings have been adopted for the purposes of the foundation back analysis.

The assessed safe working loads calculated for the selected existing piles investigated are outlined in **Table 32** below. The table presents the working load for the pile in its current configuration and for a reduced level in consideration of the proposed basements at each pile location.

Table 32: Back analysis of existing pile working loads

Pile No.	Pile Working Loads (kN) – current elevation		Pile Working Loads (kN) – proposed basement elevation *	
	Typical Design resistance for DA1 – Combinations C1 & C2 (kN)			
	C1	C2	C1	C2
BH01**	2005	1215	1433	866
BH02**	2005	1215	1433	866
BH06 (A41)	2235	1357	2157	1038
Notes:	* Proposed basement level of 18.00mAOD (BH01 & BH02) and 17.50mAOD (BH06) ** Pile working loads reduced by 20% to account of <3.D pile spacing			

It should be stressed that the above capacities do not take into consideration limiting concrete stress (to be verified by a separate load case as defined in EC2) nor pile group effects, the latter of which is more pronounced for a large number of closely spaced piles.

13.4 New foundations

Piled foundations are proposed for the developments at Block A and B.

Recommendations for the design and construction of pile foundations in relation to the ground conditions are set out in **Table 33.**

Table 33: Design and construction of piled foundations

Design/construction considerations	Design/construction recommendations
Pile type	The construction of bored/CFA piles only are considered technically feasible at this site.
Possible constraints on choice of pile type	Given the close proximity of the site to occupied office space, it is considered possible that the vibration/noise associated with pile driving will not be acceptable. This is especially true for piles close to Block C, due to sensitive equipment being situated along the boundary to Block B and C.

Design/construction considerations	Design/construction recommendations	
Temporary casing	Given the presence of groundwater strikes over the full depth of the investigation bored piles will require temporary casing throughout their depth or the use of a support fluid. Alternatively, the use of continuous-flight-auger (CFA) injected bored piles or driven piles usually overcomes this issue.	
Soft superficial deposits	For the purpose of assessing preliminary pile capacities the made ground has been presumed not to contribute to the load-carrying capacity for the piles.	
Man-made obstructions	The presence of buried sub-structures or other obstructions within made ground has been encountered within Block B and the existing piles/substructure are present beneath both blocks. The proposed basement excavation should remove most, if not all, of the made ground and any shallow obstructions. If buried obstructions are encountered it will be necessary to either relocate the pile(s) or make allowance for removing the obstruction. Further, attention is drawn to the fact that the basement retaining wall on the eastern site boundary clashes with the existing buried canal wall and this will either need to be cored through at each pile location or broken out and removed.	
Hard strata	An allowance should be made for chiselling thin 'rock' bands (claystone, limestone or cemented sandstone) within the London Clay Formation.	
Pile design parameters for London Clay	Undrained shear strength c_u (kN/m^2)	80 kN/m^2 + 4.39 z where z = depth
	Adhesion factor α	0.5
	End bearing factor N_q	9
Pile design parameters for Lambeth Group	Undrained shear strength c_u (kN/m^2)	180 kN/m^2 +13 z
	Adhesion factor α	0.5
	End bearing factor N_q	9
General parameters	Limiting shaft friction (kN/m^2)	110
	Model factor (γ_{Rd})	1.4
Special precautions relating to bored pile shafts and bases	Bored pile concrete should be cast as soon after completion of boring as possible and in any event the same day as boring. Prior to casting the base of the pile bore should be clean, otherwise a reduced safe working load will be required. Similarly, if the pile bore is left open the shaft walls may relax/soften, leading to a reduced safe working load.	

The design procedure for piles varies considerably, depending on the proposed type of pile. However, for illustrative purposes **Table 34** gives likely working pile loads for traditional bored, cast-in-situ concrete piles of various diameters and lengths, based on the design parameters given in **Table 33**. For this purpose, the average soil profile across Block A and B has been considered as shown in **Table 30**.

Table 34: Illustration of typical pile working loads for bored/CFA cast-in-situ piles

Typical Design resistance for DA1 – Combinations C1 & C2 (kN)						
Elevation of Pile toe below Block B Basement Level 19.10mOD	Pile diameter					
	600mm		750mm		900mm	
	C1	C2	C1	C2	C1	C2
-1.90	2092	1268	2715	1635	3377	2021
-3.90	2366	1435	3067	1849	3811	2284
-5.90	2689	1631	3485	2101	4331	2595
-7.90	3033	1840	3929	2369	4881	2926
-9.90	3376	2049	4373	2638	5432	3257
-11.90	3719	2258	4817	2906	5982	3588
-13.90	4063	2466	5261	3174	6533	3919
Elevation of Pile toe below Block A Basement Level 18.00mOD	Pile diameter					
	600mm		750mm		900mm	
	C1	C2	C1	C2	C1	C2
-3.00	2165	1312	2809	1692	3493	2091
-5.00	2472	1499	3208	1932	3989	2388
-7.00	2811	1704	3645	2196	4533	2715
-9.00	3154	1913	4089	2464	5083	3045
-11.00	3498	2122	4534	2733	5639	3376
-13.00	3841	2330	4978	3001	6184	3707
Elevation of Pile toe below Block A Basement Level 17.50mOD	Pile diameter					
	600mm		750mm		900mm	
	C1	C2	C1	C2	C1	C2
-3.50	2207	1337	2865	1724	3565	2133
-5.50	2523	1529	3274	1941	4074	2438
-7.50	2865	1736	3717	2238	4623	2767
-9.50	3209	1945	4161	2507	5173	3098
-11.50	3552	2154	4605	2775	5724	3429
-13.50	3895	2363	5049	3043	6275	3760
Elevation of Pile toe below	Pile diameter					
	600mm		750mm		900mm	

Typical Design resistance for DA1 – Combinations C1 & C2 (kN)						
Block B Basement Level 16.30mOD	C1	C2	C1	C2	C1	C2
-3.70	2158	1306	2805	1686	3495	2088
-5.70	2478	1450	3219	1936	4009	2396
-7.70	2820	1708	3662	223	4559	2726
-9.70	3164	1617	4106	2472	5110	3057
-11.70	3507	2125	4551	2740	5660	3388
-13.70	3851	2334	4995	3008	6211	3719

When dimensioning a pile the design load must be multiplied by the appropriate partial factor, $\gamma_{G.}$

It should be stressed that the above capacities do not take into consideration limiting concrete stress (to be verified by a separate load case as defined in EC2) nor pile group effects, the latter of which is more pronounced for a large number of closely spaced piles.

The piles should be appropriately reinforced to mitigate the risk of heave induced by the proposed basement excavation.

If serviceability is verified by preliminary/working load tests on more than 1% of the constructed piles to loads not less than 1.5 times the representative load for which they are designed then the DA1-C2 geotechnical pile capacities given in the table above could be increased by 15%.

Notwithstanding all of the above, it is recommended that the detailed advice of a specialist-piling contractor be sought as to the most suitable type of pile for the prevailing ground conditions and as to their lengths and diameters to support the required design loads.

13.4.1 Foundation works risk assessment

It is not anticipated that a foundation works risk assessment report will be required for the development because:

- there are no identified ground gas sources present at depth that could be affected by the type of foundation proposed
- shallow groundwater quality has been shown not to be impacted above GACs and no free-phase product was identified at the site

13.5 Basement floor slabs

The formation levels of the new basements are anticipated to lie within the firm/stiff London Clay Formation with groundwater expected between 16.32m and 20.59m AOD.

It is estimated the excavations for the new basements will require the removal of between 2.90m and 5.70m of overburden soil, which will lead to an unloading of approximately 60 to 115 kN/m^2 resulting in short term elastic and long term swelling of the London Clay. These movements will be mitigated to some extent by the imposed loads applied by the proposed multi-storey storey buildings.

Consideration will therefore need to be given to designing the basement slab to withstand both heave of the underlying clay soils resulting from unloading due to excavation and potential hydrostatic pressures. With regard to clay heave, it is noted that incorporating a suitable compressible layer beneath the slab will mitigate the associated uplift pressures.

The results of the preliminary ground movement assessment completed previously by RSK (report 371654-L01 (01) dated 25th October 2017) estimated the maximum unrestrained heave in the long-term condition beneath Blocks A and B ranged between approximately 40 to 50mm. As mentioned above, this should be mitigated to some extent by the imposed loads but does provide a conservative estimate for the thickness of compressible layer required.

The above will be subject to the results of the updated ground movement assessment that will take account of the site-specific data obtained during the current site investigation and to reflect the revised foundation scheme and loadings.

13.6 Excavations

It is understood that a contiguous piled retaining wall will be formed around the site perimeter and between the two basements levels beneath Block B, with the basements excavated within the wall confines. Consideration is also being given to incorporating the existing contiguous piles within the northern perimeter wall and reuse of the existing bearing piles either side of the existing sewer. The lateral capacity of the existing piles will be subject to further detailed studies and reported separately.

Groundwater seepages were encountered at various levels over the proposed basement depths and therefore allowance will need to be made for dealing with groundwater seepages within the excavations if a contiguous piled wall is adopted. It should also be noted that the relatively wide spacing of the existing piles does present a potential risk of materials collapsing/running between the piles, particularly in the presence of any groundwater entries. Consideration will need to be given to the requirement for providing additional support between the piles in these areas.

It is envisaged that temporary props will be required to provide adequate restraint to limit lateral ground movements and deflections in the contiguous piled walls resulting from the excavations. The basement slab should be cast as early as possible and tight to the retaining walls to provide additional support.

It is anticipated that the lowest basement level beneath Block A will be excavated without a piled retaining wall support between it and the highest basement level. In this case, excavation sides should be battered to a suitable safe and stable angle.

Excavations with vertical or relatively steep sides in the made ground strata are likely to be unstable and will therefore require battering back to a safe angle or appropriate support installed.

The recommended maximum safe slope angles for the strata encountered are provided in and parameters for retaining wall design are presented in the following section.

Table 35: Recommended maximum safe slope angles for temporary excavations

Strata	Temporary (Short Term)
Made ground	1v : 2h - 1v : 1.5h[1]
London Clay Formation	1v : 1h
Notes: [1]The steeper slope angle may be achievable subject to observations on site	

Suitably trained and experienced personnel should be present on site during the formation of temporary excavations to confirm suitability of the safe slope angles for the conditions encountered. It should also be noted that the safe slope angles given above do not take account of any applied loadings at or near the crest of the slope or presence of groundwater. All excavations that extend below/close to the groundwater table or encounter water ingress should be fully supported.

A detailed assessment should be undertaken by the temporary works designer for all proposed excavation slopes to account for factors such as any imposed live loadings.

13.7 Retaining Wall Design Parameters

In order to facilitate basement construction it is proposed to construct a contiguous bored pile wall along the perimeters. The advice of a specialist contractor should be sought on the design of the proposed contiguous piles walls.

Groundwater was encountered during subsequent monitoring visits within 5m below current ground level. Further groundwater monitoring visits will be carried out to confirm groundwater conditions and any impact this may have on design.

On the basis of the ground investigation information the following soil parameters in should be adopted for retaining wall design purposes.

Table 36: Retaining wall design parameters

Soil type	Unit weight γ_k (kN/m^3)	Short Term Parameters		Long Term Parameters	
		$c_{u,k}$ (kN/m^2)	$\phi'_{cv,k}$ (°)	$c'_{,k}$ (kN/m^2)	$\phi'_{cv,k}$ (°)
Made ground	18.5*	30	25	0	25*
London Clay Formation	20.0	80 + 439 z	-	5	23
Notes: *Based on published information in the absence of site-specific data.					

In order to prevent damage to adjacent structures, the design of the retaining wall and basement excavation must address the risk of excessive deformation of the wall and bracing, both in the temporary and permanent condition, to ensure that the horizontal and vertical soil movement around and below the excavation remain within acceptable levels.

The investigation and monitoring to date has indicated that ground water is likely to be present within the basement excavations to Block A and B within claystone bands within the London Clay. However, reference to Clause 6.1 of BS BS8102:2009 "Protection of Structures Against Water from the Ground" indicates that waterproofing protection measures should be designed on the basis of water to the full height of the retained ground, unless effective drainage measures can be ensured.

13.8 Roads and hardstanding

In the 1 m to 1.5 m below existing ground levels the exploratory holes have revealed a soil profile comprising a nominal thickness of variable made ground over firm London Clay Formation, with made ground thickness increasing to up to 5.30m on the eastern edge along the canal.

In pavement design terms, the groundwater conditions are anticipated to comprise a low water table, i.e. at least 1 m below the pavement formation level.

The estimated minimum, equilibrium soil-suction, California bearing ratio (CBR) value for the soils and groundwater conditions described above under a completed pavement is 3%, based upon Table C1 in TRRL (1984) Report LR1132.

The results of in-situ testing are summarised in Error! Reference source not found..

Table 37: Summary of CBR values derived from in-situ Clegg Hammer tests

Test location	Test Depth range in mbgl	Material type	Minimum CBR value determined at or just below anticipated formation level
WS01	1.20	London Clay	17%
WS02	1.20	London Clay	12%
WS03	1.20	London Clay	15%
WS04	1.20	Made ground	13%
WS05	1.20	Made ground	4.8%
WS06	1.20	Made ground	8.5%

It can be seen from the results above the that the CBR values obtained for the made ground materials are highly variable.

The recommended sub-grade soil CBR value for preliminary pavement design is therefore 3%. It is recommended that in situ plate bearing tests are conducted at final formation level to confirm the final design CBR values.

During construction the formation level should be carefully compacted and any soft spots removed and replaced with well-compacted granular fill.

The sub-grade soils can be regarded as non-frost-susceptible, based upon the criteria given in Appendix 1 of TRRL (1970) Report Road Note 29.

13.9 Chemical attack on buried concrete

This assessment of the potential for chemical attack on buried concrete at the site is based on BRE Special Digest 1: Concrete in aggressive ground, which represents the most up-to-date guidance on this topic currently available in the UK.

The desk study and site reconnaissance indicate that, for the purposes of assessing the aggressive chemical environment of the site, the site should be considered as comprising natural ground likely to contain pyrite and brownfield ground likely to contain pyrite.

Based on testing results, gives the characteristic pH, water-soluble and total sulphate content values for soils from each of the geological units and groundwater encountered on-site.

Table 38: Characteristic pH, water soluble sulphate and total sulphate values

Stratum	pH	Water Soluble Sulphate (mg/l)	Total Potential Sulphate (mg/l)
Made ground	7.97	1394	0.74
London Clay	7.84	2500	2.62
Lambeth Group	9.28	271	0.09
Groundwater	7.94	2755	-

Based on the results above and following the steps outlined in the BRE guidance, the Design Sulphate Classes and Aggressive Chemical Environment for Concrete classifications are summarised in **Table 39**, on the basis of water soluble sulphate and total potential sulphate, respectively.

Table 39: Concrete design class

Stratum	Ground water	Water Soluble Sulphate		Total Potential Sulphate	
		DS Class	AC Class	DS Class	AC Class
Made ground	Mobile	DS-2	AC-2	DS-3	AC-3
London Clay	Mobile	DS-3	AC-3	DS-5	AC-5
Lambeth Group	Mobile	DS-1	AC-1	DS-1	AC-1
Groundwater	-	DS-3	AC-3	-	-

Assuming that disturbed ground will be minimised by the use of piled foundations, the recommended ACEC Classification is therefore AC-3 with a Design Sulphate Class of DS-3. If London Clay arisings are proposed for reuse behind any concrete structures then the classification will need to be increased to AC-5 and DS-5.

17.1 Soakaways

The falling head test carried out in WS03 standpipe installation with a response zone within the London Clay soils failed to return measurable infiltration rate. The ground conditions are therefore not deemed suitable for the use of soakaways.

18 CONCLUSIONS AND RECOMMENDATIONS

18.1 Geo-environmental assessment

The results of the site investigation and GQRA indicate that relevant contaminant linkages with respect to end users are generally absent by virtue of the majority of made ground being removed from site during basement construction and the hardstanding cover across the remainder of the site.

Whilst asbestos fibres were identified in three locations around the site, the made ground in these locations will be completely removed as part of the basement excavation, such that the contamination pathway considering the future users will be broken. However, the presence of asbestos should be addressed in the contractor's risk assessment and method statements to avoid exposure of sensitive receptors to asbestos in soils during the ground works.

The results indicate that a relevant pollutant linkage may exist associated with phytotoxic effects within the made ground soils sampled from the site. It should be noted that the majority of the made ground will be removed of site and proposed soft landscaping will be limited to risen planters. However, should any additional soft landscaping, in a form of tree pits, is included into the design at the later stage a clean imported topsoil should be used to provide a suitable growing medium. A landscape architect should be consulted to confirm required thickness of imported topsoil.

A relevant linkage may exist associated with organic contaminants permeation into water supply pipes and therefore, polyethylene (PE) and polyvinyl chloride (PVC) water supply pipes are expected to be unsuitable for use on the development, unless remedial measures in form of a barrier pipe are implemented that mitigate the risk.

The conceptual model, chemical test results and ground gas monitoring conducted on site indicate that the site has been characterised as Characteristic Situation 1 (CS1), for which no gas protection measures are considered necessary.

A number of samples of made ground tested have been assessed as having hazardous properties. WAC testing carried out on five samples indicates that the samples have exceeded the pH limit values for stable non-reactive hazardous waste, likely a result of crushed concrete causing the elevated pH. It is recommended the results of the WAC testing are forwarded to the landfill operator to confirm whether they are able to accept the waste or not. Where asbestos has been identified, the asbestos fibres are less than 0.1% by weight and can therefore be disposed of within a non-hazardous landfill, which is able to accept asbestos at non-hazardous concentrations, subject to the WAC assessment.

18.2 Geotechnical assessment

The key findings of the geotechnical assessment are as follows:

- The exploratory holes revealed beneath a variable thickness of made ground, the site is underlain by London Clay Formation to approximately -3.00mAOD which is in turn underlain by the Lower Mottled Beds of the Lambeth Group which was proved to -18.60mAOD.

- Groundwater was encountered as a number of seepages within the London Clay Formation.
- Post fieldwork monitoring has recorded the general groundwater table in the London Clay Formation at an elevation of between 16.32 and 20.59mAOD, within 5m below ground level. Perched water is present within the Made Ground at BH13 at 20.31mAOD.
- Three piles were selected for investigation; access to one of these piles adjacent to the Thames Water Sewer could not be accessed to confirm its diameter and geophysical testing did not provide high confidence level results. However, the two other piles proved a diameter of 600mm with 1m centre spacing and a toe elevation of between approximately -0.11m and 0.81mAOD, which appears to confirm construction details provided in the Arup report.
- Four sheet pile locations were selected for investigation and proved their depth as being between 15.66m and 16.94mAOD and a wall thickness of between 9.3mm to 7.7mm.
- The historic buried masonry canal wall was located 1m away from the sheet pile canal wall and was found to be 0.72m to 1.40m thick and between 3.38m and 5.14m deep.
- The vertical capacity of the contiguous piles has been assessed for their re-use within the new development and is presented in Section 13.3. The lateral capacity of the contiguous pile walls and sheet piled walls will be covered in a separate report.
- The pile concrete examined from the selected piles comprised, nominal 20mm partially crushed chert (flint) gravel coarse aggregate and natural quartzitic sand fine aggregate bound by a Portland type cement matrix. Rare microcracks were observed running through the cement matrix. Sporadic and common air voids were lined with small amounts of secondary ettringite. The concrete generally appeared in sound condition with evidence of very minor leaching.
- Elsewhere across the site, new piled foundations will be installed and preliminary working pile loads have been provided. The loads do not take into consideration the effects of pile groups.
- The piles will need to be appropriately reinforced to mitigate the risk of ground heave induced by the proposed basement construction, which is expected to be between 60 and 115 kN/m^2.
- Basement floor slabs will need to be designed to withstand both heave from the unloading due to basement construction and potential hydrostatic pressures of groundwater seepages at basement formation level.
- The recommended sub-grade soil CBR value for preliminary pavement design is 3% It is recommended that in situ plate bearing tests are conducted at final formation level on completion of the proposed earthworks to confirm the final design CBR values.
- The recommended ACEC Classification is therefore AC-3 with a Design Sulphate Class of DS-3. If London Clay arisings are proposed for reuse behind any concrete structures then the classification will need to be increased to AC-5 and DS-5.

- The ground conditions are not deemed suitable for the use of soakaways.

18.3 Recommendations

The following recommendations are made for further assessment of the site to address the risks identified above and or to address remaining uncertainties:

- Carry out stability calculations for the canal walls and capacity calculations for the existing contiguous piled walls.
- Following demolition of the existing buildings, carry out an investigation on selected piles adjacent to the Thames Water sewer which was not previously accessible, to confirm their geometries and viability for use as part of the new transfer slabs,
- Refine the preliminary Ground Movement Assessment carried out by RSK in consideration of the updated ground model derived from the current site investigation and revised foundation layout and loads.
- Carry out a Thames Water Asset Impact Assessment on their assets within the vicinity of the development which may be affected.
- It is recommended that the detailed advice of a specialist-piling contractor be sought as to the most suitable type of pile for the prevailing ground conditions and as to their lengths and diameters to support the required design loads.
- Further investigation will be required beneath Block C when this becomes vacant to confirm the ground conditions and chemical status of the soils beneath that area of the development.

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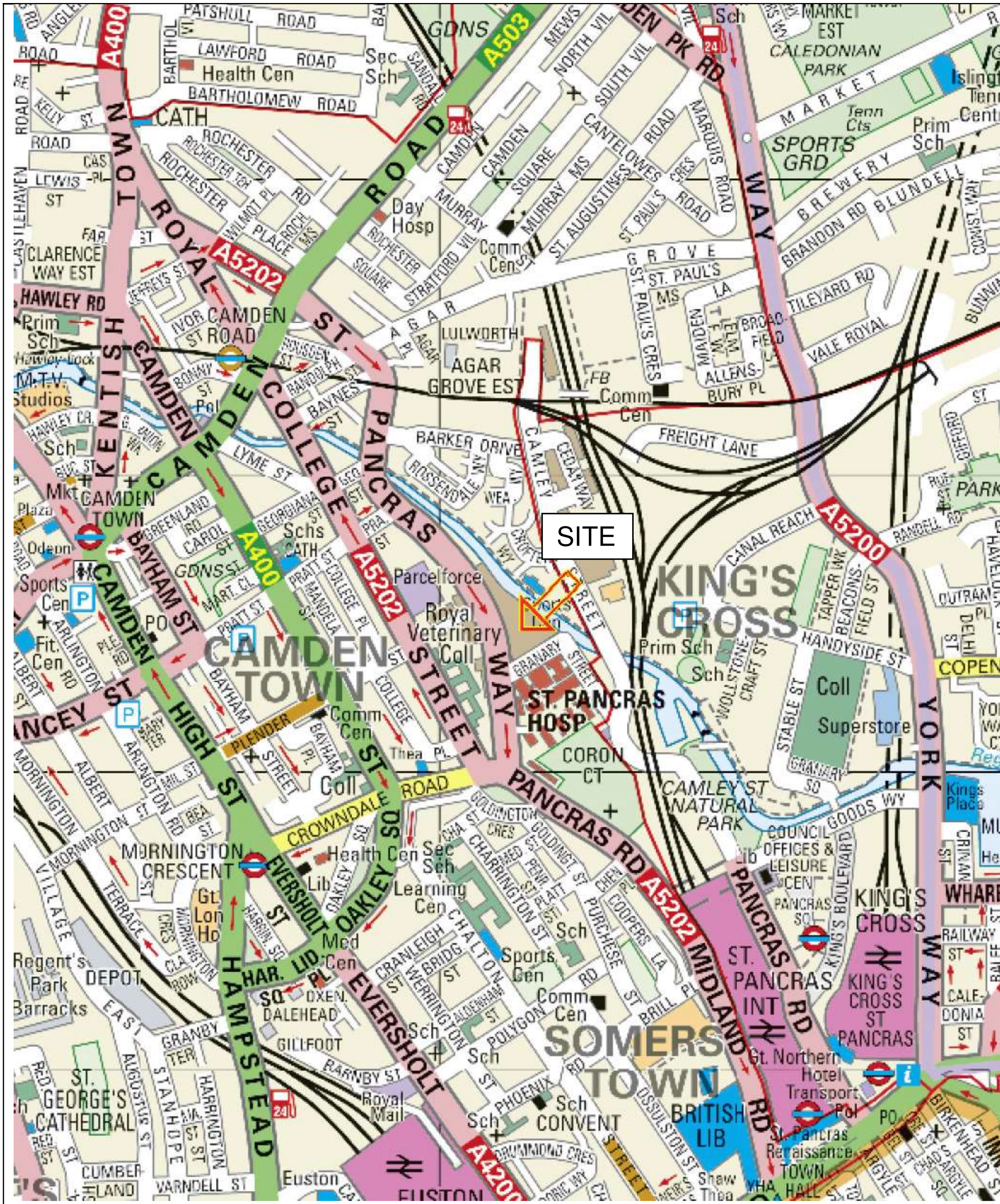
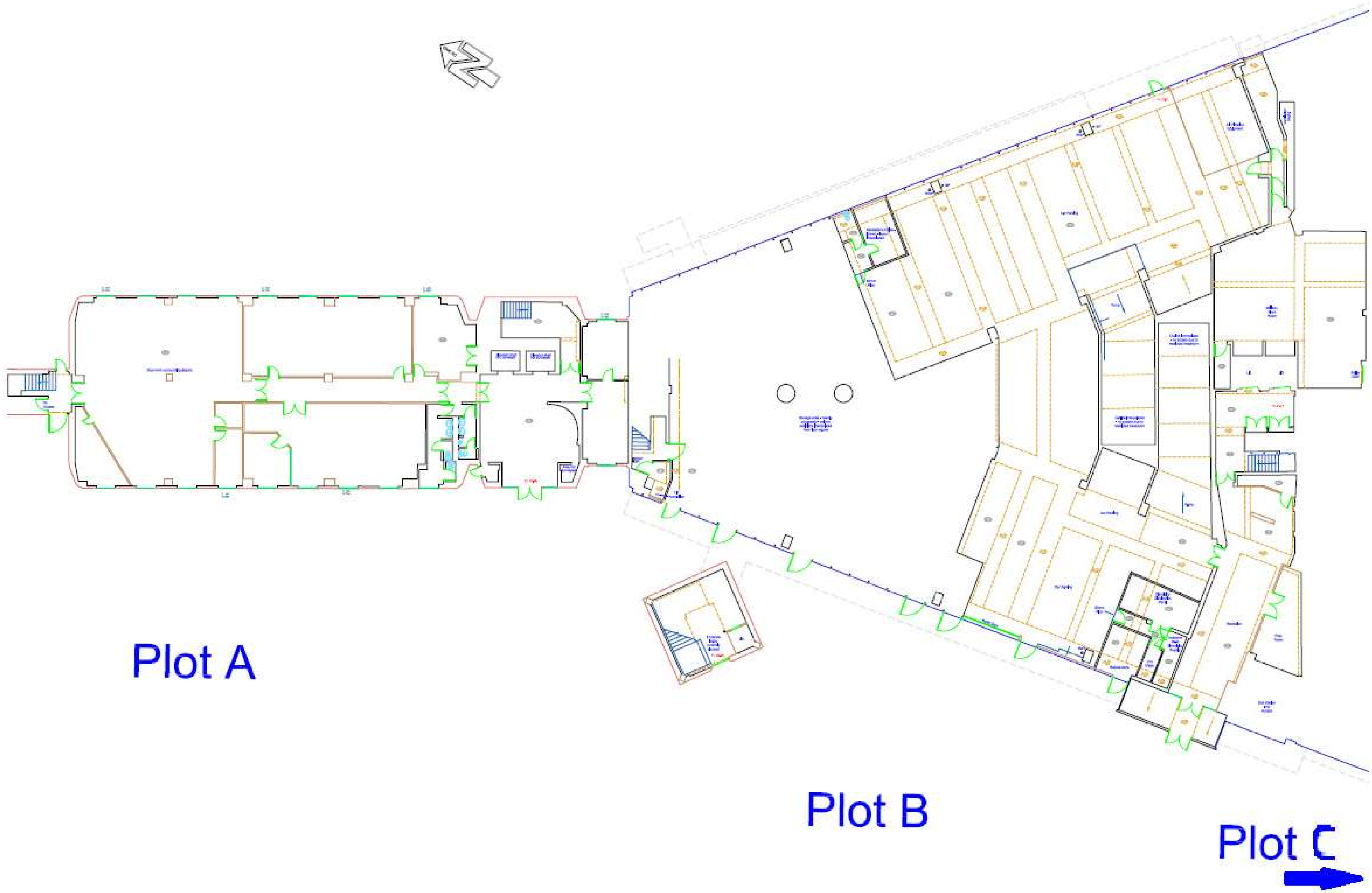
FIGURES

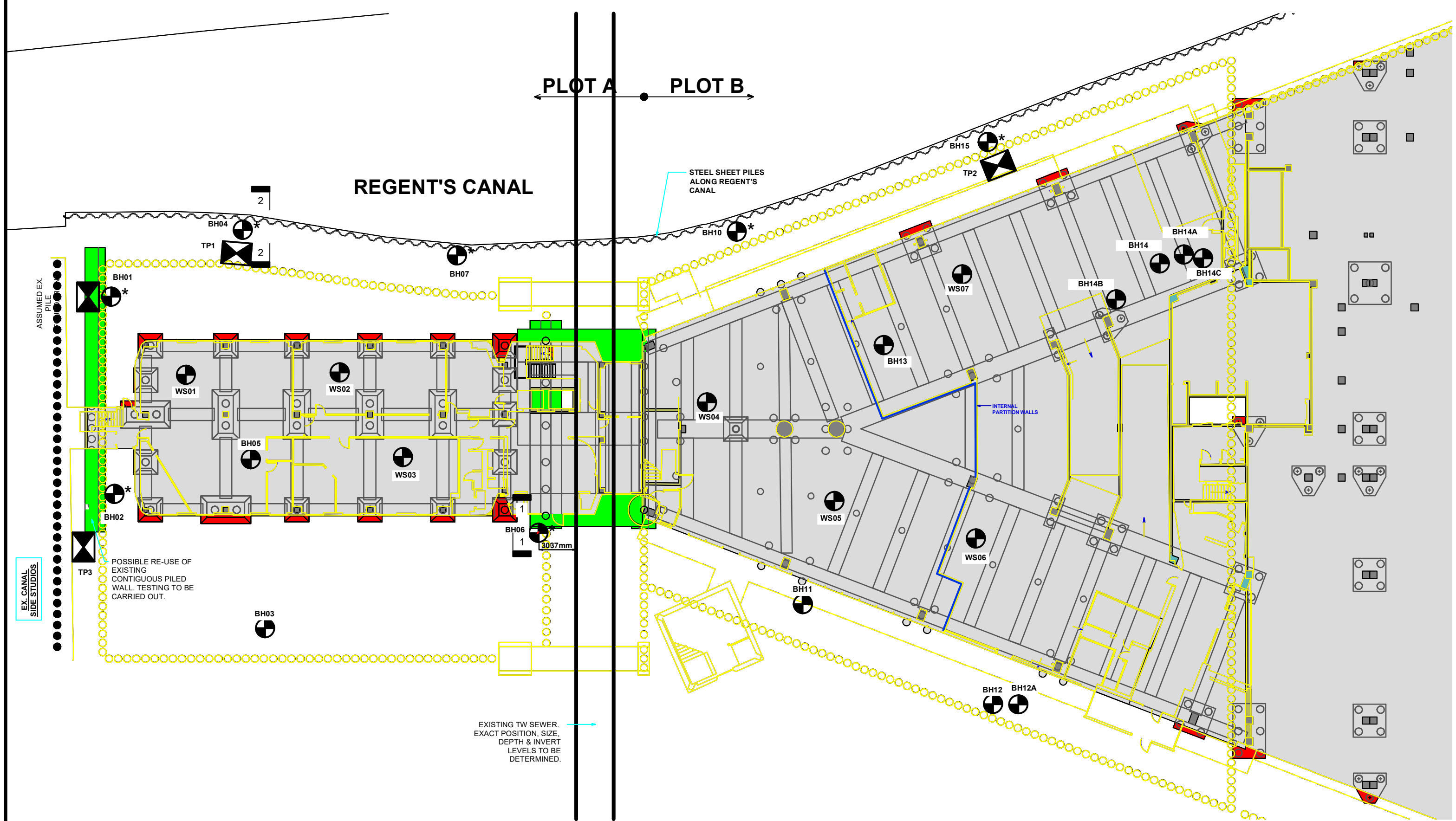


RSK

SITE LOCATION PLAN	**Client:** The Trustees of The St Pancras Way Block A Unit Trust & Big Lobster	**Figure No:** 1
	Site: Ugly Brown Building	**Job No:** 371654
	Scale: NTS	**Source:** Streetmap.co.uk



RSK	SITE LAYOUT PLAN	Client:	The Trustees of The St Pancras Way Block A Unit Trust & Big Lobster	Figure No:	2
		Site:	Ugly Brown Building	Job No:	371654
		Scale:	NTS	Source:	Clugston DWG No. 3948/30/001 A



Rev.	Date	Amendment	Drawn	Chkd.	Appd.

18 Frogmore Road
Hemel Hempstead
Hertfordshire
HP3 9RT
United Kingdom

Tel: +44 (0) 1442 437500
Fax: +44 (0) 1442 437550
Email: info@rsk.co.uk
Web: www.rsk.co.uk

Client

The Trustees of The St Pancras Way Block A Unit Trust & Big Lobster

Project Title

UGLY BROWN BUILDING, LONDON

Drawing Title

EXPLORATORY HOLE LOCATION PLAN

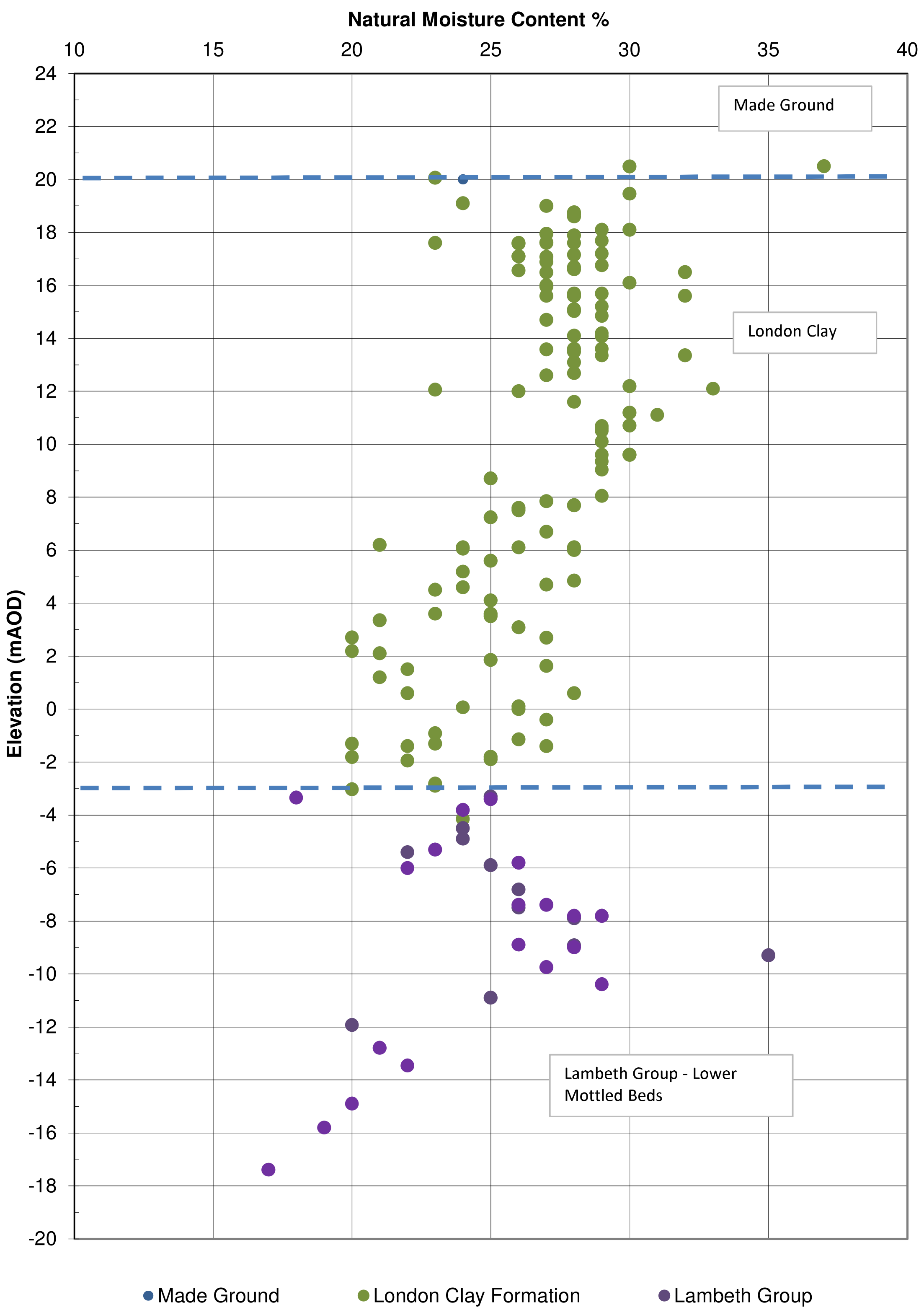
Drawn	Date	Checked	Date	Approved	Date
SA	13.03.19				

Scale	Orig Size	Dimensions
NTS	A3	N/A

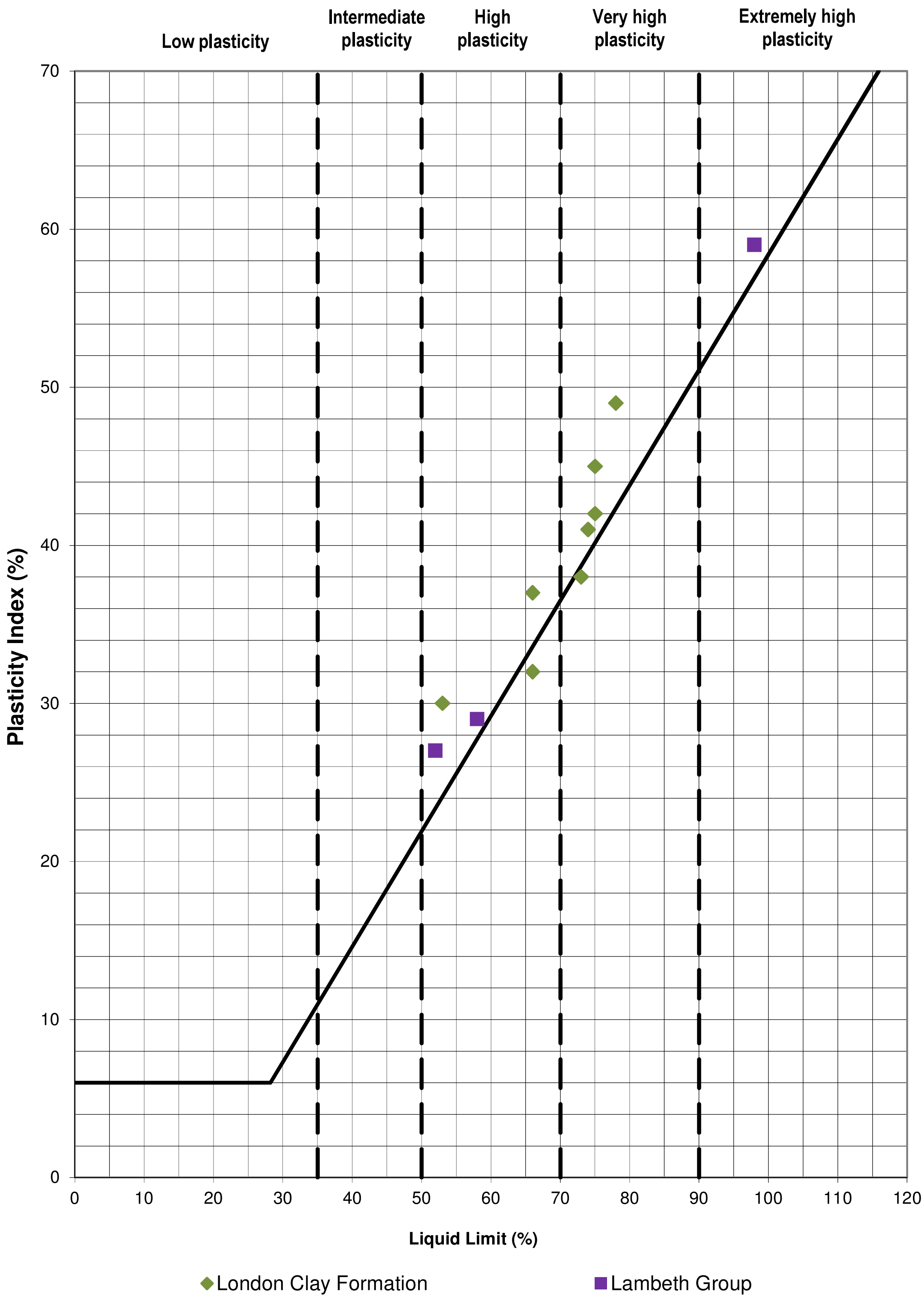
Project No.	Drawing File
371654	Figure 3 - EHL Plan.dwg

Drawing No.	Rev.
Figure 3	P1

RSK	NATURAL MOISTURE CONTENT vs ELEVATION		
Site: Ugly Brown Building	**Client:** The Trustees of The St Pancras Way Block A Unit Trust & Big Lobster	**Job Number:**	371654
		Figure:	4

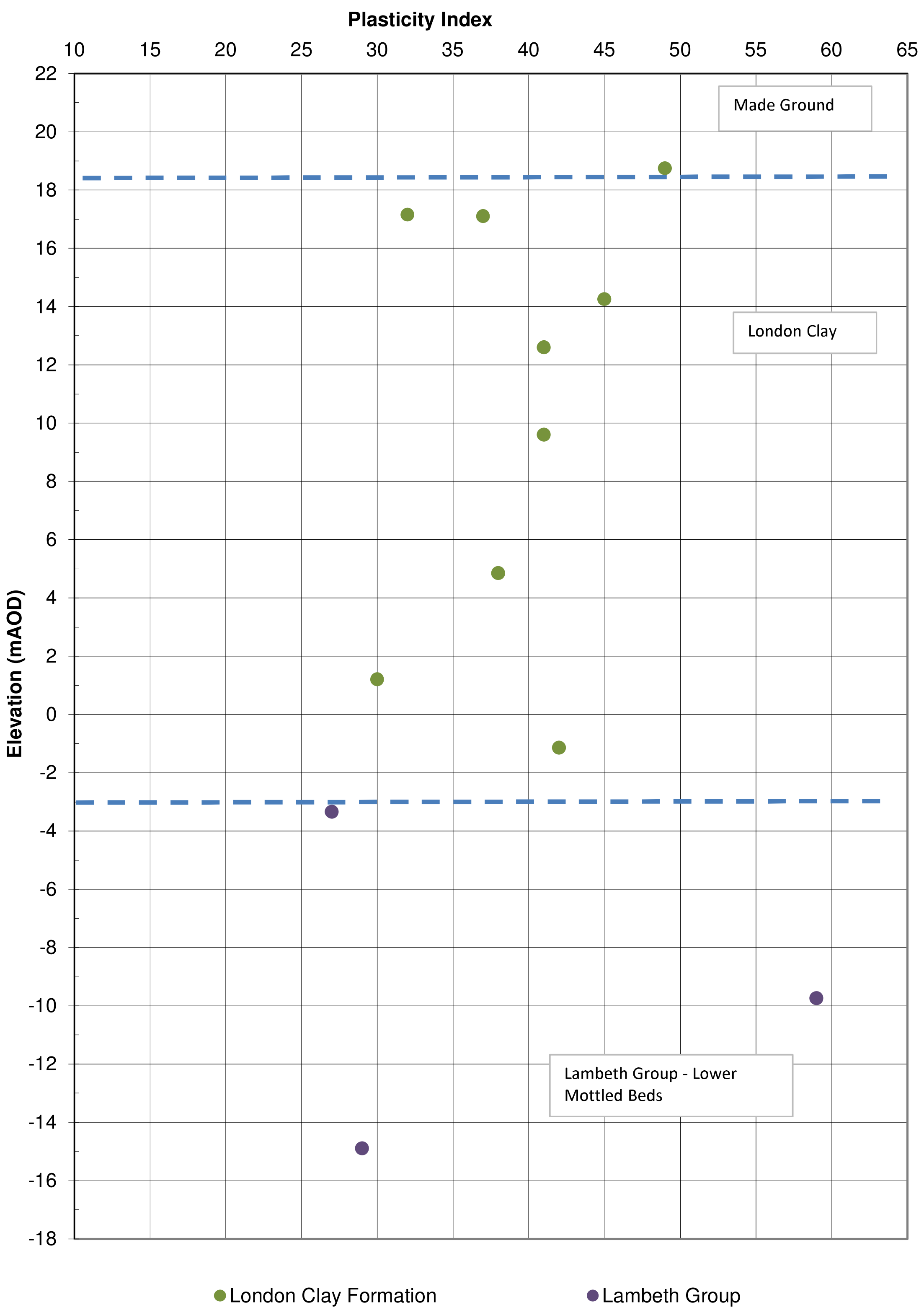


	PLASTICITY CLASSIFICATION CHART		
Site: Ugly Brown Building	**Client:** The Trustees of The St Pancras Way Block A Unit Trust & Big Lobster	**Job Number:**	371654
		Figure:	5



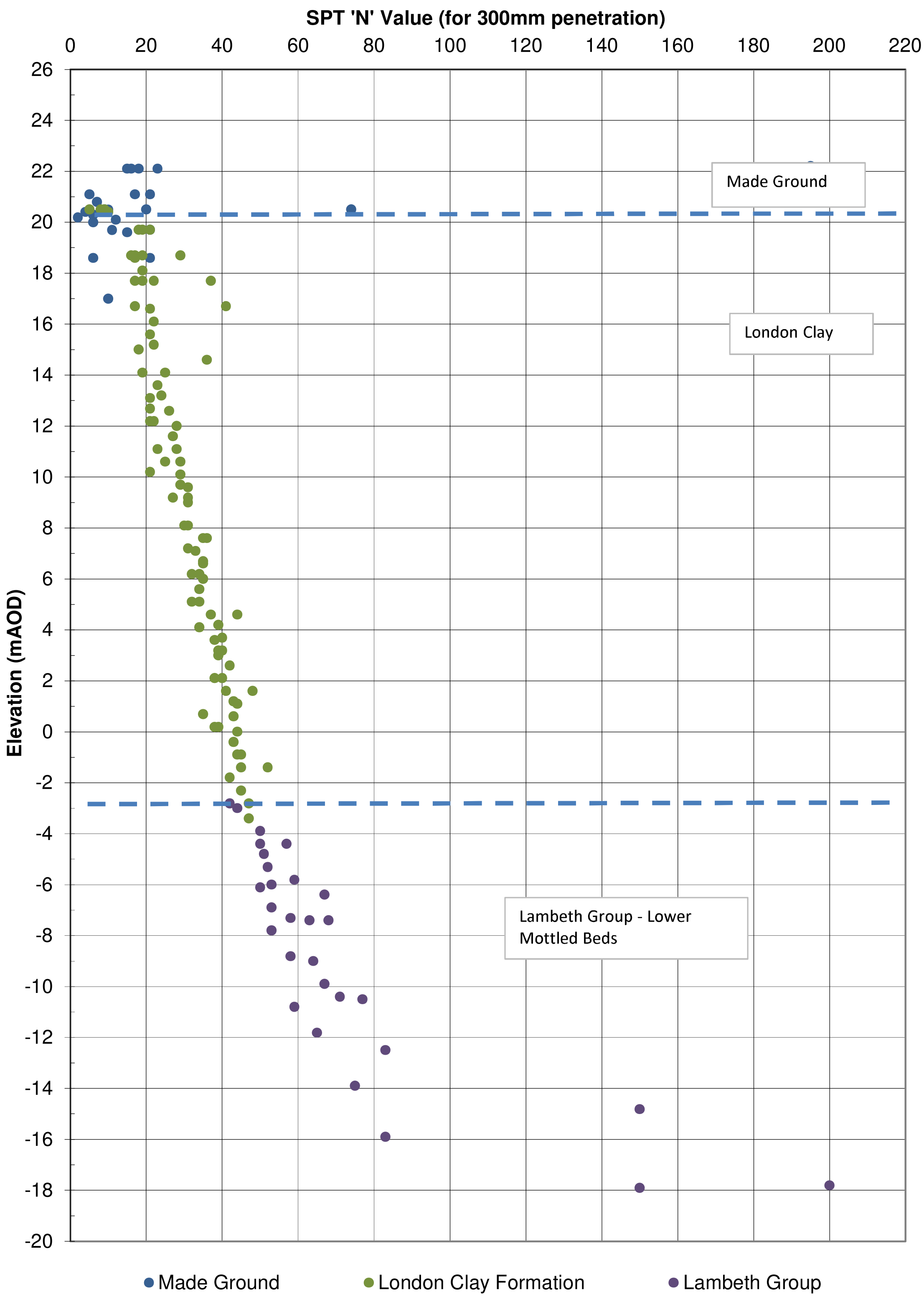
PLASTICITY INDEX vs ELEVATION (m AOD)

Site:	Client:	Job Number:	371654
Ugly Brown Building	The Trustees of The St Pancras Way Block A Unit Trust & Big Lobster	Figure:	6



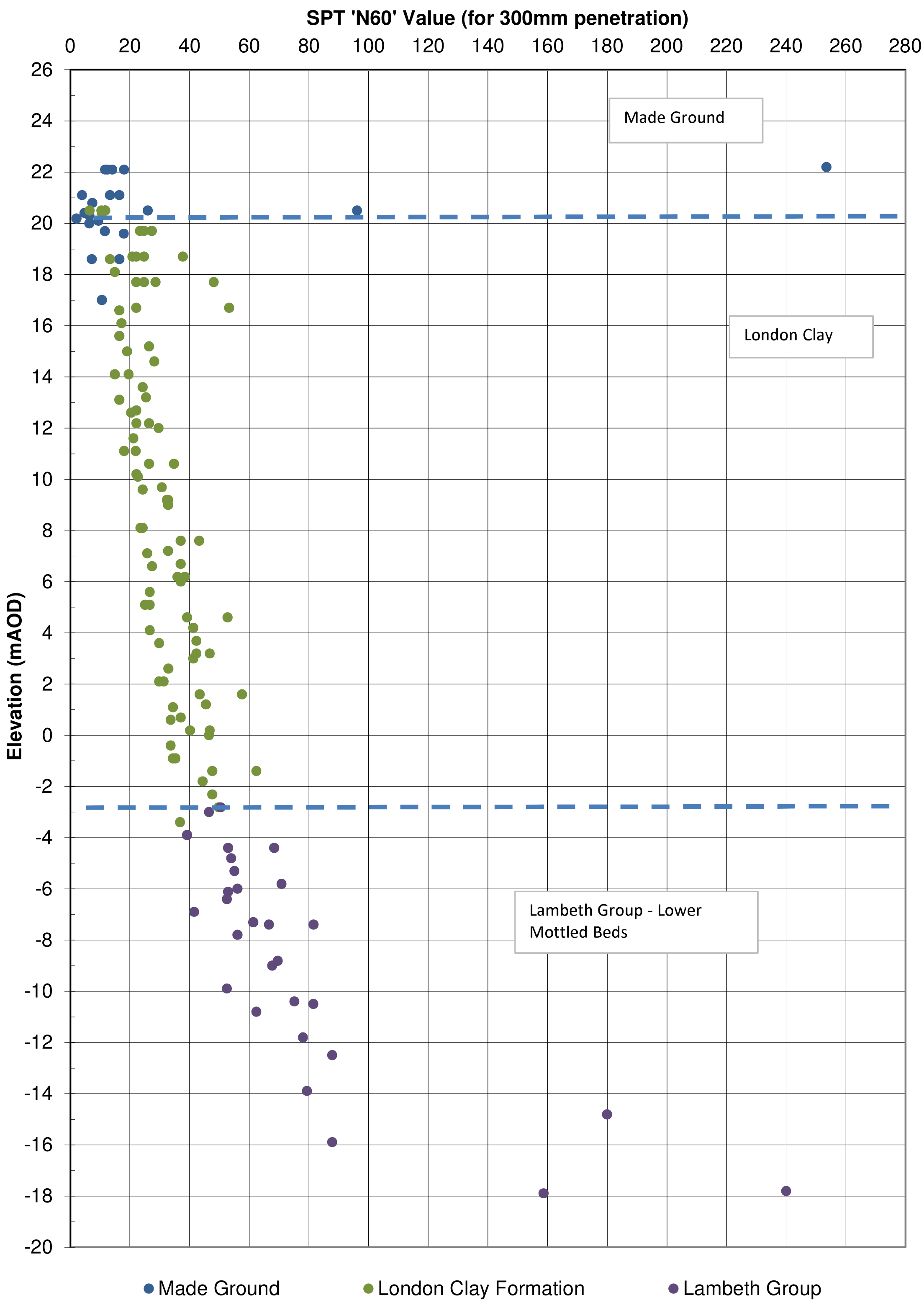
SPT 'N' Value vs Elevation (m bgl)

Site:	Client:	Job Number:	371654
Ugly Brown Building	The Trustees of The St Pancras Way Block A Unit Trust & Big Lobster	Figure:	7



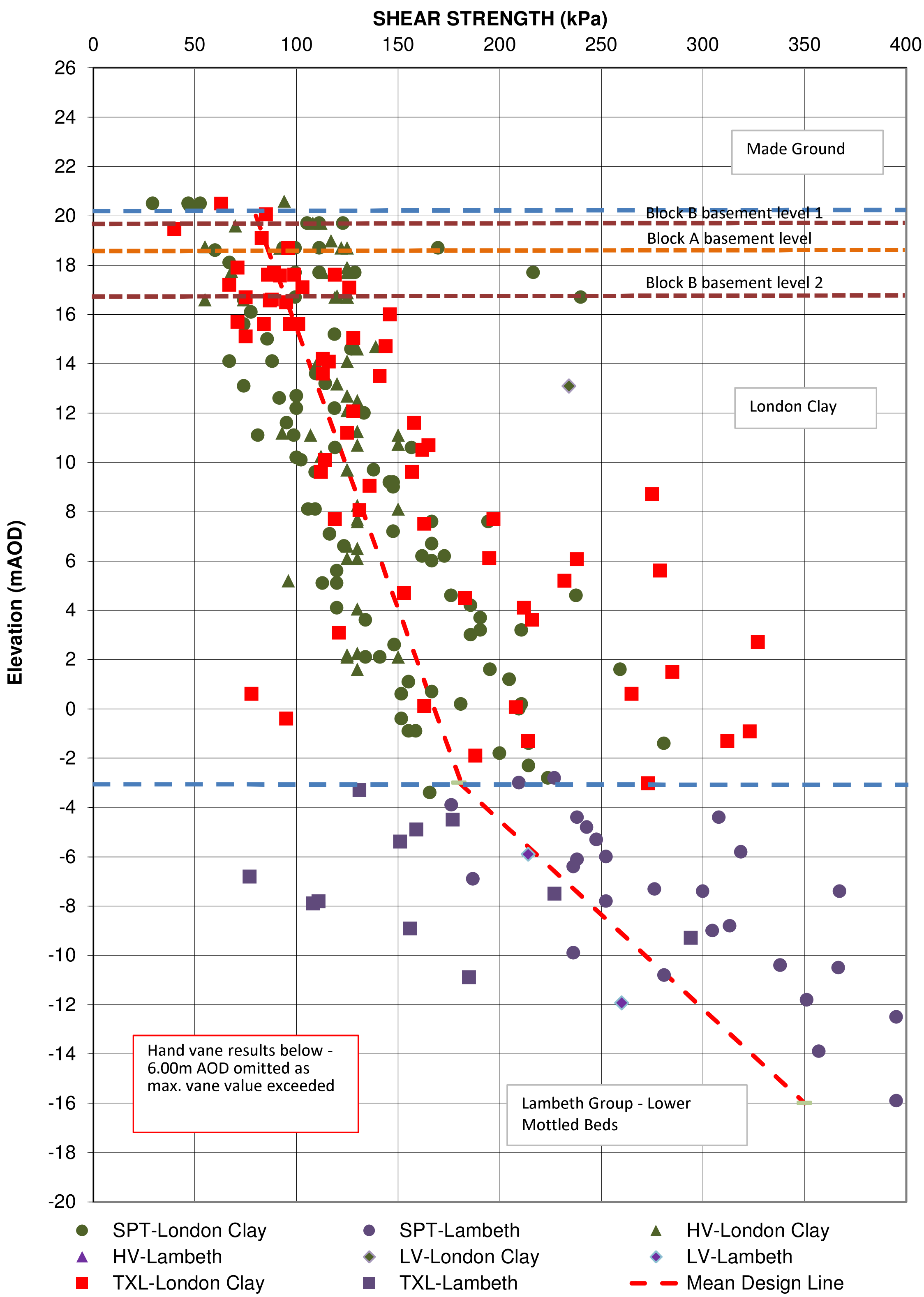
SPT 'N60' Value vs Elevation (m bgl)

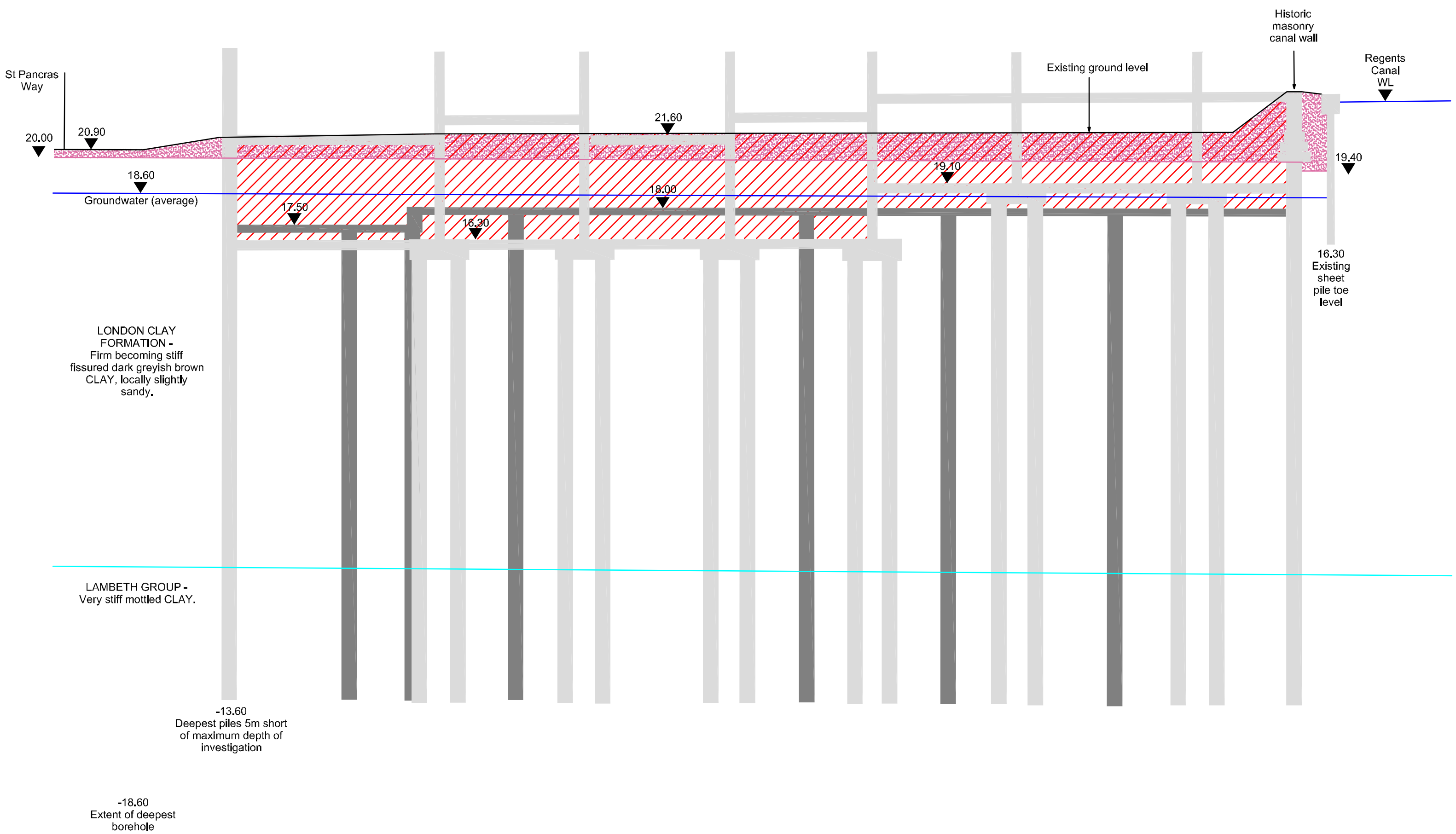
Site:	Client:	Job Number:	371654
Ugly Brown Building	The Trustees of The St Pancras Way Block A Unit Trust & Big Lobster	Figure:	8



SHEAR STRENGTH vs ELEVATION

Site:	Client:	Job Number:	371654
Ugly Brown Building	The Trustees of The St Pancras Way Block A Unit Trust & Big Lobster	Figure:	9





LEGEND

Material excavated during basement construction

MADE GROUND - variably clayey and gravelly with fragments of brick, concrete and flint. Occasional brick cobbles.

Block B - Proposed basement levels

Block A - Proposed basement levels

GEOLOGY BASED ON GROUND MODEL IN MAIN REPORT AS DETERMINED BY SITE INVESTIGATION

Rev.	Date	Amendment	Drawn	Chkd.	Appd.

RSK
18 Frogmore Road
Hemel Hempstead
Hertfordshire
HP3 9RT
United Kingdom
Tel: +44 (0) 1442 437500
Fax: +44 (0) 1442 437550
Email: info@rsk.co.uk
Web: www.rsk.co.uk

Client
THE TRUSTEES OF THE ST PANCRAS WAY BLOCK A UNIT TRUST & BIG LOBSTER

Project Title
UGLY BROWN BUILDING

Drawing Title
CONCEPTUAL SITE MODEL

Drawn	Date	Checked	Date	Approved	Date
AT	17-04-19	CS	18-04-19	JB	18-04-19

Scale	Orig Size	Dimensions
NTS	A3	MM

Project No.	Drawing File
371654	371654_R02_FIG10

Drawing No.	Rev.
371654_R02_FIG10	R1