Gascoyne Holdings Ltd

Salisbury Square, Old Hatfield, Hertfordshire, AL9 5AD

Phase 2 Supplementary Geotechnical Site Investigation

1922048-R02 (02)

RSK GEOSCIENCES

OCTOBER 2023

RSK GENERAL NOTES

Project No.:	1922048
Title:	Phase 2 Supplementary Geotechnical Site Investigation: Salisbury Square, Old Hatfield, Hertfordshire, AL9 5AD
Client:	Gascoyne Holdings Ltd
Date:	6th October 2023
Office:	RSK Environment Limited, 18 Frogmore Road, Hemel Hempstead, Herts, HP3 9RT. Tel 01442 437500
Status:	Rev 00

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Revision control sheet				
Revision ref.	**Date**	**Reason for revision**	**Amended by:**	**Approved by:**
Rev 00	04.07.23	First issue	n/a	see above
Rev 01	01.08.23	Updates	Duncan Sharp	Ben Winch
Rev 02	06.10.23	Consistency review	Duncan Sharp	Ben Winch

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EXECUTIVE SUMMARY

Commissioning and purpose of assessment	RSK Environment Limited (RSK) was commissioned by Gascoyne Holdings Ltd to carry out a Phase 2 Supplementary Geotechnical Site Investigation of the land at Salisbury Square, Old Hatfield, grid reference 523321, 208687. The overall aim of the project was to provide supplementary information to inform the design of deep foundations.
DESK-BASED ASSESSMENT	
Site description and proposed development	The site currently comprises mixed commercial and residential use, occupies an area of 0.9 hectares and is being considered for development for mixed residential and commercial use following demolition of the existing shopping parade.
Previous site investigation (SI) reports	One site investigation was previously undertaken for the site in 2011. Following that report, RSK produced a supplementary report pertaining to the discharge of planning conditions in 2015. In 2021, RSK produced a report that was an update to the 2011 report. This report serves as a further update to the previous two reports.
Geology and environmental setting	The exploratory hole encountered a variable thickness of made ground over the Lowestoft Formation and Kesgrave Catchment Subgroup, with a further basal horizon of Lowestoft Formation mantling the Lewes Nodular Chalk Formation and Seaford Chalk Formation at depth. This is generally in line with the stratigraphical succession observed during the 2011 site investigation. The site is not located within an 'Affected Area' with regard to radon and is not located within previous coal mining areas. Based on the Edmund's risk assessment model for natural dissolution features referred to in CIRIA Report C574 (Lord et al. 2002), the site falls into the 'low anticipated subsidence risk' category. Notwithstanding the above, a single sink hole and a single solution pipe have been recorded some 166 m to the west and historical chalk excavations / potential mining some 91 m and 138 m to the east and south, respectively. In addition, reference to the Welwyn and Hatfield Borough Council Chalk Mine Hazards Map has identified a former chalk mine, designated as Area 25 "Hill House" located immediately east of Park Street (some 50 m east of the site).
Site reconnaissance findings	The majority of the northern portion of the site was observed to be covered by hardstanding. Two large landscaped areas are located in the central portion of Salisbury Square and a shopping parade was identified in the central portion of the site.
INTRUSIVE INVESTIGATION & ASSESSMENT	
SI scope	The supplementary site investigation involved the following: • Drilling of one cable percussion borehole to 30 m bgl; • Water level monitoring in shallow installations; and • Ground gas monitoring in monitoring well installations.

SI factual findings	The site investigation identified the presence of the following strata: • Made ground from surface to 2.5 m bgl; • Lowestoft Formation from 2.5 m bgl to 7.0 m bgl; • Kesgrave Catchment Subgroup from 7.0 m bgl to 13.0 m bgl; • Clay-with-Flint Formation from 13.0 m bgl to 17.2 m bgl; and • Lewes Nodular Chalk & Seaford Chalk Formation from 17.2 m bgl to 30.45 m bgl.
Geotechnical assessment	The ground conditions encountered in the northern area of the site are unlikely to be suitable for the design and construction of conventional shallow spread foundations due to the depth of made ground and the initially softened surface to the underlying natural stratum. Whilst deep spread foundations may be technically feasible, the practicality of construction may be prohibited by the thickness and instability of the made ground (unless ground levels are to be reduced). Prior ground improvement, utilising techniques such as vibro-displacement appear feasible to facilitate the subsequent construction of reinforced strip foundations, however, all remnant sub structures would require removal and any backfilling completed in accordance with a strict engineering specification. Continuous flight augured piled foundations are likely to provide the most appropriate foundation option for the proposed structures due to variable thickness of the made ground and the presence of very soft clays in the upper Lowestoft Formation.
Recommendations including issues for further assessment	All foundation excavations should be inspected, and any made ground, soft, organic or otherwise unsuitable materials removed and replaced with mass concrete. If piling works are to extend deeper than 16.0 m bgl, a foundation works risk assessment report may be required for the development.
The information given in this summary is necessarily incomplete and is provided for initial briefing purposes only. The summary must not be used as a substitute for the full text of the report.	

1 INTRODUCTION

1.1 Commissioning

RSK Environment Limited (RSK) was commissioned by Conisbee, on behalf of Gascoyne Holdings Ltd ('the client'), to carry out a Phase 2 Supplementary Geotechnical Site Investigation of the land at Salisbury Square, Old Hatfield. The project was carried out to an agreed brief as set out in RSK's proposal (Ref. 1922048 T01 (00), dated 2nd February 2023).

RSK's service constraints are shown in **Appendix A**.

The Site in question is being considered for development for mixed residential and commercial use.

1.2 Objectives

The objective of the work is to provide supplementary information to inform the design of deep foundations.

1.3 Scope of works

The scope of the intrusive investigation has been designed in line with the recommendations of BS5930:2015+A1:2020 Code of practice for ground investigations (BSI, 2020), 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.

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

- design and implementation of an intrusive investigation including 1No. light cable-percussion borehole to 30 m bgl.
- in situ testing, soil sampling, laboratory geotechnical testing, groundwater and ground gas monitoring of installed boreholes
- interpretation of ground conditions and geotechnical data to provide preliminary recommendations with respect to foundations and infrastructure design
- preparation of this interpretative report.

1.4 Existing reports

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

- RSK (2011), "Geotechnical and Geo-Environmental Report," ref. 241882-01 (01), dated 23rd March 2011; and
- RSK (2015), "Supplementary Geo-Environmental Information Pertaining to the Discharge of Planning Condition 20," ref. 27975-L02 (01), dated 3rd September 2015; and

- RSK (2021), "Updated Geotechnical and Geo-Environmental Report," ref. 1922048 R02 (01), dated 25th November 2021.

Pertinent information from these reports has been summarised in Section 3.

1.5 Limitations

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

2 SITE DETAILS

2.1 Site location

Site location details are presented in **Table 1** and a site location plan is provided on **Figure 1**.

Table 1 Site location details

Site name	Salisbury Square, Old Hatfield
Full site address and Post code	Salisbury Square, Old Hatfield, Hertfordshire, AL9 5AD
National Grid reference (centre of site)	523321, 208687

2.2 Site description

The Site boundary and current site layout are shown on **Figure 2.** The Site covers an area of c. 0.9 hectares and comprises ground-level parking within the northern portion of the site, with Salisbury Square and surrounding retail and commercial units in the south.

2.3 Surrounding land uses

The Site is located within a mixed residential and commercial setting. Immediate surrounding land uses are described in **Table 2.**

Table 2 Surrounding land uses

North	North of the site is predominantly residential with some amenities such as a community social club, as well as greenspace for recreational use.
East	Immediately east of the site is predominantly residential housing, and moving further east there is some greenspace. A large pond is located in Hatfield House approximately 500 m to the east of the site.
South	South of the site is mixed retail, commercial and residential, including an accountants, a barber, a gym, and a marketing agency.
West	The Great North Road (A1000) runs north-to-south immediately adjacent to the western boundary of the site. Hatfield railway station lies some 50 m further to the west with the associated railway lines running parallel to the western site boundary immediately beyond. A number of industrial units are located beyond the railway tracks to the west approximately 100 m-150 m from the site.

2.4 Development plans

The proposed layout of the site, at the time of preparing this report, is shown in **Appendix B**.

It is understood that the proposed development is in relation to the improvement of Salisbury Square. This is intended to comprise the improvement of Salisbury Square with a mixed commercial and residential development. Specifically, this will comprise the demolition of the existing shopping parade with seven maisonettes above and retaining wall structures; alterations to existing and construction of new parking areas; layout of public spaces; erection of new building containing three flats, eleven offices and one retail unit (Use Class E – Commercial, Business and Service); erection of a terrace of five houses with parking and footways; foul and surface water drainage and all ancillary works.

The commercial units are also to include a basement level. A ground level parking area is proposed across the site associated with the residential and commercial developments and public access totalling 77 car spaces.

No details of the proposed ground levels have been provided therefore for the purpose of this report it has been assumed that the current levels will remain unchanged.

3 SUMMARY OF PREVIOUS REPORTS

All previous reports can be found in **Appendix C**.

3.1 RSK (2011)

In March 2011, RSK produced a report titled "Geotechnical and Geo-Environmental Report", report reference 241882-01 (01).

3.1.1 Scope

RSK were previously commissioned by Gascoyne Cecil Estates to carry out a Geotechnical and Geo environmental site assessment. (Ref RSK Geotechnical and Geo-Environmental Report," ref. 241882-01 (01), dated 23rd March 2011).

The aim of the works was to provide a preliminary risk assessment (PRA) followed by an intrusive site investigation undertaken on 2nd to 4th February 2011.

The intrusive investigation comprised of 2No. light cable percussive boreholes to 15 m, 4No drive-in window sampler boreholes, the excavation of 3no. shallow hand pits to collect samples for contamination testing. Associated sampling and testing including 3No clegg hammer tests and the use of a photo-ionisation detector (PID) to screen for volatile organic compounds. Soil samples were collected and sent for environmental and geotechnical testing.

3.1.2 Site history

Details obtained from historical mapping, Groundsure report and the Local Authority indicate that the site and surrounding area has historically been subject to a number of light industrial, commercial and residential land-uses, some of which could have contributed to the contamination of the site. Furthermore, the information provided by Welwyn Hatfield Borough Council confirm that the historic site use as a brewery is a potential source of contamination.

3.1.3 Site conditions

Geological maps and BGS borehole log records indicate that the site is underlain by glacial deposits overlying the white chalk subgroup and groundwater was initially encountered within the chalk at approximately 27 m bgl.

The current site investigation confirmed the geological succession predicted in the preliminary conceptual model in that beneath a variable thickness of made ground, a sequence of granular Glacial Deposits and cohesive Lowestoft Formation was encountered. The greatest thickness of made ground was encountered in BH2 in the southern portion of the site. This is assumed to be related to the infilling of a former basement associated with historic land-use of the site.

3.1.4 Hydrological and hydrogeological Conditions

The hydrogeology of the site is likely be characterised by the presence of a deep aquifer within the White Chalk Subgroup. The regional direction of groundwater flow is to the northeast.

The White Chalk Subgroup beneath the site is classified by the Environment Agency (EA) as a Principal Aquifer (as indicated on the Environment Agency Groundwater Vulnerability Map of the area, Sheet No. 39 'West London'. Furthermore, the Glacial Gravels have been classified as a Secondary (A) Aquifer. The Principal Aquifer has been classified with a High (Urban) Vulnerability rating. Soil information for urban areas is less reliable and based on fewer observations that in rural areas. The worst case (i.e., high leaching potential) is therefore assumed until proved otherwise.

The potential presence of low permeability glacial deposits at relatively shallow depths beneath the site, whilst restricting downwards migration, may increase the potential for lateral migration of shallow groundwater (and therefore mobile contamination, if present).

The environmental database report indicates that there are no current licensed groundwater abstractions within a 1 km radius of the site.

There are no ponds, streams, or drainage ditches on or adjacent to the site. The nearest identified surface watercourse / feature to the site (with the exception of small ponds and drainage ditches within Hatfield Park to the east) is the river Lea located approximately 1.2 km to the north of the site.

The indicative floodplain map for the area, published by the EA, shows that the site does not lie within a floodplain.

3.1.5 Conceptual Site model summary.

The preliminary risk assessment (PRA) identified potential sources of contamination, sensitive receptors and considered the plausible pathways that may link them and have been summarised below.

Potentially complete contaminant linkages with a potential risk of moderate to low or higher that were identified in the previous 2011 report included:

- Direct contact, ingestion, and dust/vapour inhalation of contaminants of potential concern within shallow made ground by construction workers and future site users.
- Leakage of contaminants from shallow made ground into unsaturated zone and vertical migration to shallow groundwater.
- Root uptake of contaminants of potential concern by proposed vegetation
- Lateral migration of contaminants from shallow made ground via shallow ground water to wider groundwater body.
- Impact and degradation of plastic utilities and building structures by contaminants within made ground.
- Direct contact, ingestion, and dust/vapour inhalation of contaminants of potential concern within past industrial use by construction workers.

- Leakage of contaminants from past industrial use into unsaturated zone and vertical migration to shallow groundwater.
- Lateral migration of contaminants from past industrial use via shallow ground water to wider groundwater body.

3.1.6 Ground and ground water conditions

The ground conditions encountered during the investigation generally comprised of made ground to a maximum depth of 4.90 m bgl, underlain by an interbedded sequence of granular and cohesive glacial deposits. The White Chalk Subgroup was not encountered within the terminal depth of the investigation. Groundwater was only encountered in the form of a perched groundwater table on top of the Lowestoft Formation and at the base of the made ground soils.

3.1.7 Geo-Environmental Findings

The Preliminary desk study highlighted that the site had historically been subjected to a number of light industrial, commercial and residential land-uses, of which could potentially have contributed to a source of contamination.

Chemical analysis in the northern portion of the site did not identify any elevated concentrations of contaminants, and therefore no alleviation measure is considered necessary.

Chemical analysis in the southern portion of the site identified a single exceedance in the made ground soils with respect to Benzo(a)pyrene. This concentration was linked to bitumen and clinker-rich soils identified at this depth during the site investigation. No alleviation measures are considered necessary should this area be encapsulated beneath hardstanding post-development; however localised sampling would be required should this area be proposed for communal soft landscaping or private gardens.

A single elevated concentration with respect to the phytotoxic element zinc was encountered within the shallow made ground soils in the southern portion of the site although marginal and only recorded with in sample it was recommended that topsoil be imported to any tree pits or areas of soft landscaping to ensure an appropriate depth of a suitable growing medium is provided.

After 3no monitoring rounds were undertaken, no risk was identified in relation to ground gas and the site was categorised as Characteristic Situation 1, therefore no ground gas protection measures were deemed necessary.

Results suggested that made ground and natural soils would not be classed as hazardous waste, but WAC testing would be required to establish a non-hazardous or inert category.

Due to slightly elevated concentrations of petroleum hydrocarbons around BH2, barrier pipe is recommended in this area.

3.1.8 Geotechnical Findings

The ground conditions in the northern area of the site are not considered suitable for shallow spread foundations for the proposed terraced houses. Deep trench fill foundations would be technically feasible, and ground improvement techniques would facilitate the use of shallow spread footings.

The suitability of spread foundations to support the proposed mixed commercial development would depend upon the structural loads and the extent of the made ground. The depth to which spread foundations will need to extend and the anticipated structural loads may mean that piles will provide the most suitable foundation solution.

The development in the central and southern portion of the site is to involve the construction of a single storey basement structure with associated retaining structures. For the soil parameters pertaining to the preliminary design for retaining walls can be obtained from the previous report.

The Site is generally underlain by more than 600 mm of existing made ground. National House-Building Council (NHBC) standards require that ground floor slabs should be suspended in areas where made ground is greater than 600 mm in thickness.

The results of in situ Clegg Hammer testing indicates that the near surface soils have a CBR value that ranges from between 2 and 28. From these results the recommended sub-grade soil CBR value for road pavement design is therefore 2%. This value assumes that during construction the formation level will be carefully compacted and any soft spots removed and replaced with well-compacted granular fill.

The sub-grade soils are regarded as frost-susceptible therefore the thickness of the sub-base must be sufficient to give a total thickness of non-frost susceptible pavement construction over the soil of not less than 450 mm.

Aggressive Chemical Environment for Concrete classification is AC-1, with Design Sulphate Class DS-1. This assumes static/mobile groundwater conditions and that no significantly disturbed clay comes into contact with concrete foundations or structures.

3.2 RSK (2021)

In November 2021, RSK produced a report titled “Updated Geotechnical and Geo-Environmental Report”, report reference 1922048 R02 (01).

RSK were previously commissioned by JB Planning Associates on behalf of Gascoyne Holdings Ltd to carry out an update of the previous Phase1 and 3 geotechnical and geo-environmental investigation and report (summarised above). This report pertains to the same site footprint as the 2011 investigation.

The overall aim of the project was to assess land contamination sources and geotechnical constraints to the proposed development using information and data collected during the 2011 investigation and provide:

- An updated study of local geology, hydrogeology, and surface water setting:
- An updated review of relevant environmental data held by appropriate statutory authorities; and
- An interpretation of results against updated guidelines.

3.2.1 Refinement of the initial CSM

The investigation confirmed the presence of a moderate to significant thickness of made ground overlying mixed granular and cohesive glacial superficial deposits to depths in excess of 14.5 m bgl. A persistent groundwater table was not encountered, although perched groundwater was encountered within areas of deeper made ground.

In light of the absence of a groundwater table beneath the site, the following pollutant linkages have been removed:

- Leakage of contaminants from shallow made ground/ past industrial use into unsaturated zone and vertical migration to shallow groundwater: and
- Lateral migration of contaminants within shallow aquifer to wider groundwater body.

All other pollutant linkages identified within the initial CSM remain unchanged.

3.2.2 Geo-Environmental Findings

The Geo-environmental assessment undertaken in this report was carried out using the test data and results obtained in the 2011 report and compared to current updated guidelines.

The updated geo-environmental assessment highlighted exceedances of lead at BH1 and TP3. It is understood that the location of TP3 is intended to be encapsulated beneath hardstanding, and therefore this would sever any potential pollutant linkages. However, the location of BH1 is intended for residential housing with private gardens, therefore mitigation measures will be required in this area.

Lead was identified to exceed the adopted GAC (310 mg/kg) in TP3 0.1 at 345 mg/kg. However, TP3 is currently located within an area of proposed hardstanding, and this would sever any potential pollutant linkages (including phytotoxic effects associated with elevated concentrations of zinc).

Whilst no significant organic contamination has been recorded on-site, mild concentrations of petroleum hydrocarbons in the made ground soils in the vicinity of BH2 may have the potential to permeate plastic water supply pipes. It is therefore recommended that barrier pipe be adopted in this area of the site if it is proposed to install potable water supply pipes through this area.

Reassessment of the ground gas confirmed the findings in the previous report, with no specific precautions with respect to gas protection measures are considered necessary.

3.2.3 Geotechnical

The geotechnical findings of the updated report were the same as the original 2011 report.

4 PRELIMINARY GEOTECHNICAL CONSTRAINTS

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 of 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 is given in **Table 3** together with an assessment of whether the site may be affected by each of the stated hazards.

Table 3 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	☒	☐	Likely to affect ground engineering and foundation design and construction
Shrinkable clay soils	☒	☐	Design to NHBC Standards Chapter 4 or similar

<table>
<tr><th rowspan="2">Hazard category</th><th colspan="2">Hazard status based on desk study findings and proposed development</th><th rowspan="2">Engineering considerations if hazard affects site</th></tr>
<tr><th>Could be present and/or affect site</th><th>Unlikely to be present and/or affect site</th></tr>
<tr><td>Highly compressible and low bearing capacity soils, (including peat and soft clay)</td><td>☒</td><td>☐</td><td>Likely to affect ground engineering and foundation design and construction</td></tr>
<tr><td>Running sand at and below water table</td><td>☒</td><td>☐</td><td>Likely to affect ground engineering and foundation design and construction</td></tr>
<tr><td>Karstic dissolution features (including ‘swallow holes’ in Chalk terrain)</td><td>☒</td><td>☐</td><td>May affect ground engineering and foundation design and construction</td></tr>
<tr><td>Underground mining including shafts and adits (e.g. coal, mineral)</td><td>☒</td><td>☐</td><td>Likely to require further assessment including potentially special stabilisation measures</td></tr>
<tr><td>Effects of extreme temperature (e.g. cold stores or brick kilns/furnaces)</td><td>☒</td><td>☐</td><td>Likely to affect ground engineering and foundation design and construction</td></tr>
<tr><td>Existing sub-structures (e.g. tunnels, foundations, basements, and adjacent sub-structures)</td><td>☒</td><td>☐</td><td>Likely to affect ground engineering and foundation design and construction</td></tr>
<tr><td>Filled and made ground</td><td>☒</td><td>☐</td><td>Likely to affect ground engineering and foundation design and construction</td></tr>
<tr><td>Adverse ground chemistry (including expansive slags and weathering of sulphides to sulphates)</td><td>☐</td><td>☒</td><td>May affect ground engineering and foundation design and construction</td></tr>
<tr><td>Site topography</td><td>☐</td><td>☒</td><td>May affect ground engineering and foundation design and construction</td></tr>
<tr><td colspan="4">Note: Seismicity is not included in the above table as this is not normally a design consideration in the UK.</td></tr>
</table>

4.2.1 Chalk

In view of the prevailing ground conditions, with Chalk at shallow depth beneath the site, it is normal practice to consider the potential risk of ground subsidence related to the presence of swallow holes and other natural chalk solution features or man-made cavities.

Based on the Edmund’s risk assessment model for natural dissolution features referred to in CIRIA Report C574 (Lord et al. 2002), the site falls into the ‘low anticipated subsidence risk’ category. With reference to the Envirocheck database of known natural chalk solution features there is one known feature within a 500 m radius of the site. The record relates to a single sink hole and a single solution pipe located some 166 m to the west. In respect to man-made features, there are three recorded within a 500 m radius. The records relate to historical chalk excavations / potential mining some 91 m and 138 m to the east and south, respectively and an historical brick kiln / chalk mining some 457 m to the west. In addition, reference to the Welwyn and Hatfield Borough Council Chalk Mine Hazards Map has identified a former chalk mine, designated as Area 25 "Hill House" located immediately east of Park Street (some 50m east of the site).

5 SITE INVESTIGATION STRATEGY & METHODOLOGY

5.1 Introduction

RSK carried out supplementary intrusive investigation works and subsequent monitoring of boreholes between 4th May 2023 and 16th May 2023.

5.2 Objectives

The specific objective of the investigation was to provide supplementary information to inform the design of deep foundations.

5.3 Selection of investigation methods

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

Cable percussion drilling was chosen based on the targeted drill depth, requirement for in-situ geotechnical data, the opportunity to collect both disturbed and undisturbed samples and install a monitoring well.

Prior to conducting intrusive works, utility service plans were obtained and buried service clearance undertaken in line with RSK's health and safety procedures. Copies of statutory service records obtained by RSK as part of the agreed scope of works are contained in **Appendix D**.

5.4 Investigation strategy

The ground investigation was carried out using intrusive ground investigation techniques in general accordance with the recommendations of BS5930:2015+A1:2020, 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 a targeted borehole in the south-eastern corner of the current car park.

The primary constraint to the investigation was underground services.

The borehole location required breaking out prior to undertaking the hand pit and subsequent drilling works.

Details of the investigation locations, installations and rationale are presented in **Table 4**. A single cable percussive borehole was drilled to a maximum depth of 30.45 m bgl and was installed with a combined gas and groundwater monitoring well. An exploratory hole location plan is shown in **Figure 3**.

Table 4 **Exploratory hole and monitoring well location rationale**

Investigation type	Number	Designation	Monitoring well installation	Rationale examples below
Boreholes by cable percussive methods	1	BH201	Gas and groundwater	To prove the geological succession beneath the site and obtain geotechnical data.

5.4.1 Implementation of investigation works

The site investigation works were carried out in general accordance with the UK Specification for Ground Investigation (UKSGI), third edition (AGS, 2022).

The exploratory holes were logged by an engineer in general accordance with the recommendations of BS5930:2015+A1:2020 (which incorporates the requirements of BS EN ISO 14688-1, 14688-2 and 14689-1).

The monitoring well construction and associated response zones are detailed on the exploratory hole records in **Appendix E**. The response zone was installed in order to establish the presence or absence of a groundwater table. Gas monitoring was carried out as a precautionary measure, not due to any risks identified in a preliminary CSM.

The soil sampling and analysis strategy was designed to investigate the geotechnical characteristics.

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.

5.5 Monitoring programme

5.5.1 Ground gas monitoring

As a precautionary measure, a single round of ground gas monitoring was undertaken, targeting the made ground.

The previous site investigation undertaken by RSK in 2011 included three rounds of ground gas monitoring. The site was subsequently categorised as Characteristic Situation 1, and no further ground gas monitoring was required.

The purpose of this investigation was purely geotechnical in nature, and therefore the single round of gas monitoring that was undertaken was precautionary, and not a risk that needed further investigation.

A calibrated 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. In addition, during the first monitoring round, all wells were screened with a PID to establish if there are any interferences and cross-sensitivity of other hydrocarbons with the infrared gas meter.

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 G** Equipment calibration certificates are available on request.

5.5.2 Groundwater monitoring

One round of groundwater monitoring was undertaken, and no groundwater sampling was carried out. The monitoring record, including dates, is shown in **Appendix G**.

Depth to groundwater was recorded using an electronic dip meter on the return monitoring visit.

5.6 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.

5.6.1 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 5**. The results and UKAS accreditation of tests methods are shown in **Appendix F**.

Table 5 Summary of geotechnical testing undertaken

Strata	Tests undertaken	No. of tests
Made ground	Unconsolidated undrained triaxial compression	1
Lowestoft Formation (upper)	Water content %	3
	Liquid/ plastic limits	3
	Unconsolidated undrained triaxial compression	1
	BRE brownfield non-pyritic	2
	Water content %	1

Strata	Tests undertaken	No. of tests
Kesgrave Catchment Subgroup	Liquid/plastic limits	1
	Sieve analysis	2
	Sedimentation analysis	2
Lowestoft Formation (basal)	Water content %	2
	Liquid/plastic limits	2
	BRE brownfield non-pyritic	1
Lewes Nodular Chalk Formation and Seaford Chalk Formation	Saturated moisture content	4
	BRE brownfield non-pyritic	3

6 SITE INVESTIGATION FACTUAL FINDINGS

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

6.1 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 E**.

The recent exploratory hole BH201 encountered a variable thickness of made ground over the Lowestoft Formation and Kesgrave Catchment Subgroup, with a further basal horizon of Lowestoft Formation mantling the Lewes Nodular Chalk Formation and Seaford Chalk Formation at depth. This is generally in line with the stratigraphical succession observed during the 2011 site investigation, although the glacial deposits were more heterogeneous than anticipated.

For the purpose of discussion, the ground conditions encountered during the fieldworks are summarised in **Table 6** with the strata discussed in subsequent subsections.

Table 6 General succession of strata encountered

Stratum	Exploratory holes encountered	Depth to top of stratum m bgl	Proven thickness (m)
Made ground	BH201	0.00	2.80
Lowestoft Formation (Upper)		2.80	4.20
Kesgrave Catchment Subgroup		7.00	6.00
Lowestoft Formation (Basal)		13.00	4.20
Lewes Nodular Chalk Formation and Seaford Chalk Formation.		17.00	13.25

6.1.1 Made ground

Made ground was encountered beneath reinforced concrete hardstanding, and initially comprised a sandy gravel of flint, sandstone and concrete. At 0.5 m bgl, the made ground comprised a cohesive soil recovered as sandy gravelly clay, with anthropogenic materials recorded including clinker, metal wire, nails, glass and ceramic, brick, concrete and mortar.

A summary of the in-situ and laboratory test results recorded in the stratum are presented in **Table 7**.

Table 7 Summary of in-situ and laboratory test results for the Made Ground.

Soil parameters	Value	Reference
SPT 'N' values	6	**Appendix E**
SPT 'N60' values (Er= 72)	7	**-**
Undrained shear strength inferred from SPT 'N' values (kN/m^2)*	25.2	**-**
Undrained shear strength inferred from SPT 'N60' values (kN/m^2)*	30.2	**-**
Consistency term from field description	Soft	**Appendix E**
Notes: *derived using a Stroud Factor of 4.2		

6.1.2 Upper Lowestoft Formation

This upper stratum was encountered from beneath the made ground and generally comprised a layer of very soft to stiff sandy gravelly clay approximately 4.20 m in thickness. Initial SPT undertaken at 3.00 m depth within this formation penetrated under own weight.

A summary of the in-situ and laboratory test results recorded in the stratum are presented in **Table 8**.

Table 8 Summary of in-situ and laboratory test results for the upper Lowestoft Formation.

Soil parameters	Min. Value	Max. Value	Reference
Moisture content (%)	14.7	25.8	**Appendix F**
Modified moisture content (%)	16	49	**-**
Liquid limit (%)	28	78	**Appendix F**
Plasticity limit (%)	16	24	**Appendix F**
Plasticity index (%)	12	54	**Appendix F**
Modified plasticity index (%)	10.8	28.62	**-**
Plasticity term	Low	Very High	**Appendix E**
Volume change potential	Low	Medium	**-**
SPT 'N' values	0	22	**Appendix E**
SPT 'N60' values (Er= 72)	0	26	**-**
Undrained shear strength inferred from SPT 'N' values (kN/m^2)*	0	103.4	**Appendix E**

Soil parameters	Min. Value	Max. Value	Reference
Undrained shear strength inferred from SPT 'N60' values (kN/m^2)*	0	124.1	**Appendix E**
Undrained shear strength measured by triaxial testing (kN/m^2)	129		**Appendix F**
Consistency term from field description	Very soft	Very stiff	**Appendix F**
Strength term (inferred from Triaxial testing)	High		**Appendix F**
Notes: *derived using a Stroud Factor of 4.7			

6.1.3 Kesgrave Catchment Subgroup

This stratum was encountered at a depth of 7.00m below ground level and was 6m in thickness. Based on the site descriptions and laboratory and in-situ tests carried out this layer can be generally described as a medium dense to dense predominately granular soil. However, a cohesive horizon was encountered between 10.10m and 10.50m depth.

A summary of the in-situ and laboratory test results recorded in the stratum are presented in **Table 9**.

Table 9 Summary of in-situ and laboratory test results for the Kesgrave Catchment Subgroup.

Soil parameters	Min. Value	Max. Value	Reference
Moisture content (%)	7.1		**Appendix F**
Modified moisture content (%)	12		**-**
Liquid limit (%)	28		**Appendix F**
Plasticity limit (%)	15		**Appendix F**
Plasticity index (%)	13		**Appendix F**
Modified plasticity index (%)	7.54		**-**
Plasticity term	Low		**Appendix F**
Volume change potential	Low		**-**
SPT 'N' values	23	45	**Appendix E**
SPT 'N60' values (Er= 72)	28	54	**-**
Undrained shear strength inferred from SPT 'N' values (kN/m^2)*	96.6	189	**Appendix E**
Undrained shear strength inferred from SPT 'N60' values (kN/m^2)*	115.9	226.8	**Appendix E**
Density term	Medium dense	Dense	**Appendix E**
Notes: *derived using a Stroud Factor of 4.2			

6.1.4 Basal Lowestoft Formation

This stratum was encountered at a depth of 13.0 m and was 4.20 m in thickness. Based on the site descriptions and laboratory and in-situ tests carried out this layer can be described as very stiff orangish brown slightly gravelly slightly sandy clay.

A summary of the in-situ and laboratory test results recorded in the stratum are presented in **Table 10**.

Table 10 Summary of in-situ and laboratory test results for the basal Lowestoft Formation

Soil parameters	Min. Value	Max. Value	Reference
Moisture content (%)	13.4		**Appendix F**
Modified moisture content (%)	20		-
Liquid limit (%)	47		**Appendix F**
Plasticity limit (%)	20		**Appendix F**
Plasticity index (%)	27		**Appendix F**
Modified plasticity index (%)	17.82		-
Plasticity term	Intermediate		-
Volume change potential	Low		-
SPT 'N' values	31	35	**Appendix E**
SPT 'N60' values (Er= 72)	37	42	-
Undrained shear strength inferred from SPT 'N' values (kN/m^2)*	145.7	164.5	**Appendix E**
Undrained shear strength inferred from SPT 'N60' values (kN/m^2)*	174.8	194.4	**Appendix E**
Consistency term from field description	Very Stiff		**Appendix E**
Notes: *derived using a Stroud Factor of 4.7			

6.1.5 Lewes Nodular Chalk Formation and Seaford Chalk Formation

The typically Lewes Nodular Chalk Formation and Seaford Chalk Formation comprised structureless chalk composed of off-white slightly sandy slightly gravel or gravel silt (Grade Dm to Dc) with some flint.

This however is not considered likely to represent the in-situ condition of the chalk due to the inherently destructive nature of the drilling technique. The sand fraction comprised fine to coarse chalk fragments and the gravel fraction comprised fine to coarse subangular to angular chalk fragments.

Chalk was encountered at a depth of 17.00 m below ground level and extended beyond the full depth of the investigation at 30.45 m below ground level and

A summary of the in-situ and laboratory test results recorded in the stratum are presented in **Table 11** and **Table 12**.

Table 11 Summary of laboratory test results for Lewes Nodular Chalk Formation & Seaford Chalk Formation

Location	Depth (mbgl)	Moisture Content (%)	Bulk density (mg/m^3)	Dry density (mg/m^3)	Saturated Moisture Content (%)
BH201	20.00	21	2.07	1.71	**21**
BH201	21.50	16	2.16	1.86	**17**
BH201	24.00	24	2.04	1.65	**24**
BH201	29.00	30	1.92	1.48	**31**
Reference: Appendix F					

Table 12 Summary of in-situ results for Lewes Nodular Chalk Formation & Seaford Chalk Formation

Soil parameters	Min. Value	Max. Value	Reference
SPT ‘N’ values	13	46	**Appendix E**
SPT ’N60’ values (Er= 72)	16	55	**-**

6.1.6 Visual/olfactory evidence of soil contamination

There was not visual or olfactory evidence of contamination within made ground deposits and underlying natural strata.

No visual evidence of asbestos was encountered during the site investigation.

6.2 Groundwater and surface water

6.2.1 Groundwater encountered during intrusive works

Groundwater was encountered during the intrusive investigation works as detailed in the logs in **Appendix E**. Water seepage occurred at 20.50 m bgl within the Lewes Nodular Chalk Formation and Seaford Chalk Formation.

According to previous reports written by RSK, ground water seepage was encountered at 4.9 m bgl in BH02 (2011).

6.2.2 Groundwater encountered during monitoring

Resting groundwater levels recorded during the monitoring programme are summarised in **Table 13** based on the data provided in **Appendix G**.

Table 13 Summary of groundwater monitoring results

Monitoring well	Response zone stratum	TOC elevation (m AOD)	Depth to water (mb TOC)	Groundwater elevation (m AOD)
RSK 2023				

Monitoring well	Response zone stratum	TOC elevation (m AOD)	Depth to water (mb TOC)		Groundwater elevation (m AOD)
BH201	18.00 to 30.00	72.86	20.62		52.24
RSK 2011					
BH2	1.00 to 6.00	-	4.18	-	-
WS2	1.00 to 3.00	-	Dry	-	-
WS4	1.00 to 3.00	-	2.41	-	-

The findings reflect the groundwater table in the Lewes Nodular Chalk Formation and Seaford Chalk Formation, which is at an elevation of 52.24 m AOD.

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.

6.3 Ground gas monitoring

The results of the ground gas monitoring carried out are given in **Appendix G** and summarised in **Table 14** below.

Table 14 Ground gas monitoring results

Exploratory position ID	No. of monitoring rounds	Peak CH_4 max. (%/vol)	Peak CO_2 max. (%/vol)	Oxygen min. (%/vol)	Peak gas flow max. (l/hr)	Steady state gas flow range (l/hr)	Min Atm. pressure (mb)
RSK 2023							
BH201	1	0.0	2.4	17.5	0.1	0.1	1018
RSK 2011							
BH2	2	0.0	0.1	20.1	-	0.4	999
WS2	2	0.0	3.4	17.1	-	0.1	999
WS3	2	0.0	1.4	18.5	-	0.5	999

6.4 Geotechnical laboratory results

The results of the geotechnical testing are discussed in Section 7 and presented in **Appendix F**.

7 GEOTECHNICAL ASSESSMENT

7.1 Proposed development

Following demolition of the existing structures, the proposed development will principally comprise two structures, which require foundations. A row of 5 No. terraced townhouses is proposed along the northern flank of the site (fronting onto Arm and Sword Lane) and a multi-use block proposed within the centre of the site. A small basement level is proposed beneath the multi-use block.

Ground level parking area is proposed across the site associated with the residential / commercial developments and public access totalling 77 car spaces.

7.2 Key geotechnical hazards / development constraints

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

- Variable thickness of made ground.
- Sudden lateral changes in ground conditions.
- Shrinkable clay soils.
- Existing sub-structures.
- Soils of low to medium volume change potential.
- Low subsidence risk due to the known presence of both natural and historical man-made mining cavities within the vicinity of the site.

7.3 Foundations

Piled foundations are proposed to support the structures. Working loads ranging between 500 kN and 800 kN are anticipated. The basement beneath the multi-use block is anticipated to be formed by contiguous CFA piles. The load bearing piles to both structures are understood to comprise CFA piles. The foundation solution for the terrace of townhouses will need to be designed to span the existing 1250 mm culvert, which currently has a 4700 mm exclusion zone.

The recent investigation has confirmed a similar thickness of made ground to that encountered during previous phases of work. Glacial deposits, varying between a cohesive diamicton and a granular deposit sub cropped beneath the made ground, with the upper surface of the chalk bedrock encountered at 17.0 m depth.

Water seepage occurred at 20.50 m bgl within the Lewes Nodular Chalk Formation and Seaford Chalk Formation during the investigation and at 20.62 m during the subsequent monitoring visit.

7.3.1 Foundation options

Terrace Housing

The ground conditions encountered in the northern area of the site do not appear suitable for the design and construction of conventional shallow spread foundations for the proposed terraced houses due to depth of made ground. Whilst, relatively deep trench fill foundations appear technically feasible, the depth to which such foundations will need to extend, combined with potentially poor stability of open excavations through the extensive made ground may mean that piles will provide a more economic foundation solution. The need to span the existing culvert, is also likely to increase the loads, and deepen the required founding depth to ensure the asset is not overstressed.

Mixed Commercial / Residential Block with Basement Car Park

The suitability of spread foundations to support the proposed mixed use building in the central portion of the site will depend upon the structural loads and the extent of made ground remaining below the proposed basement formation level. The depth to which spread foundations will need to extend and the anticipated structural loads may mean that piles will provide the most suitable foundation solution for this aspect of the proposed development.

In view of the low subsidence risk category determined by the Edmund's risk assessment model for natural dissolution features and the absence of evidence for such features in the site investigation, the need for any special foundation design measures does not appear justified. However, as with any site underlain by chalk, there will still be a potential for chalk dissolution related features on site and hence excavations should be carefully inspected to confirm the absence of such features beneath structures.

7.3.2 Spread foundations

The recommendations for the design and construction of spread foundations in relation to the ground conditions are set out in **Table 15**.

Table 15 Design and construction of spread foundations

Design/construction considerations	Design/construction recommendations
Founding stratum	Firm Lowestoft Formation (It is reiterated that foundations must extend below made ground soils and any soft weathered surface of the underlying natural soil. As discussed above, the depth to a competent undisturbed founding stratum is likely to be prohibitive)
Depth	Foundations should be taken to a minimum depth of 1.0 m below the final or existing ground level, whichever is lower, and at least 0.2 m into the founding stratum below any overlying made ground or to any greater depth required in respect of the special design considerations given below. The planned basement structures will result in most of the made ground being removed.
Special design considerations	

Design/construction considerations	Design/construction recommendations
Shrinkable soils	Owing to the presence of shrinkable clay soils, foundations should be designed taking into account all the normal precautions, including minimum founding depths, to minimise the risk of future foundation movements in accordance with NHBC standards or similar. The findings of the ground investigation indicate that foundations should be designed for shrinkable soils of medium volume change potential.
Variable founding soils	Owing to the significant lateral and vertical variability of the founding strata, consideration should be given to incorporating appropriate reinforcement into the strip foundations to minimise the risk of future differential foundation movements.
Presumed bearing capacity	Strip footing foundations with a width of 1.0 m and constructed on the firm Lowestoft Formation at a minimum depth of 1.0 m, or at least 0.2 m into the firm stratum, may be designed using a presumed bearing capacity of 125 kN/m^2. This value may be increased to 150 kN/m^2 for pad foundations up to 1.5 m^2. The presumed bearing capacity includes a partial factor on bearing resistance of 3 against bearing capacity failure. Total settlements associated with the presumed bearing pressure are anticipated to be less than 25 mm.
Construction considerations	All foundation excavations should be inspected, and any made ground, soft, organic or otherwise unsuitable materials removed and replaced with mass concrete. It should be noted that, due to the variability of the thickness of the made ground, foundations will need to be taken into the firm Lowestoft Formation, which can occur at depths of 4.9 m bgl. This may result in the strip or pad footing foundations being impractical in areas where basement structures are not proposed.

7.3.3 Piled foundations (bearing piles)

It is understood that working loads ranging between 500 kN and 800 kN are required to support the proposed structures. Recommendations for the design and construction of pile foundations in relation to the ground conditions are set out in **Table 16**.

Table 16 Design and construction of piled foundations

Design/construction considerations	Design/construction recommendations
Pile type	The construction of both bored and driven piles is considered technically feasible at this site.
Possible constraints on choice of pile type	Given the close proximity of the site to a residential area it is considered possible that the vibration/noise associated with pile driving may not be acceptable.
Temporary casing	Given the presence of non-self-supporting material within the soil profile, bored piles will require temporary casing throughout their depth. Alternatively, the use of continuous-flight-auger (CFA) injected bored piles or driven piles usually overcomes this issue.

Design/construction considerations	Design/construction recommendations	
Soft superficial deposits	For the purpose of assessing preliminary pile capacities the made ground and soft clays have been presumed not to contribute to the load-carrying capacity for the piles. This will be particularly important for driven piles as the development of pore pressures during driving is likely to exacerbate the problem.	
Man-made obstructions	The presence of buried sub-structures or other obstructions within made ground may lead to some difficulty during piling. It is therefore recommended that once the proposed pile layout has been determined, pre-pile probing be carried out at each of the pile positions. Where buried obstructions are encountered, it will be necessary to either relocate the pile(s) or make allowance for removing the obstruction.	
Pile design parameters for cohesive deposits – Upper Lowestoft Formation	Undrained shear strength cu (kN/m2)	Average 100
	Adhesion factor α	0.6
Pile design parameters for Kesgrave Catchment Subgroup (granular)	Internal Friction Angle (ϕ')	37
	Shaft friction factor (k_s.tan δ)	0.36
Pile design parameters for cohesive deposits – Basal Lowestoft Formation	Undrained shear strength c_u (kN/m^2)	Average 150
	Adhesion factor α	0.4
Lewes Nodular Chalk Formation & Seaford Chalk Formation	Shaft Resistance (τ_{sf})	0.80σ_v' for N>10 (Bored)
	Base resistance	Qu = 200N (Bored) Q_{all} = 600-800 kN/m2 for N>25 (17.2m-20.0m) Q_{all} = 1000-1800 kN/m2 for N<25 (20.0m-30.45m)
General parameters	Limiting concrete stress (kN/m^2)	7.5 N/mm^2
	Global margin of safety	2.5
	Limiting shaft friction (kN/m^2)	110
	Limiting end bearing pressure (kN/m2)	11000
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. The more recent investigation (BH201) has inferred both lateral and vertical variability within the interstratified Glacial Deposits. Should the pile design rely on high bearing resistance from dense granular soils within the Glacial Deposits, it is essential that the designer can satisfy themselves that the piles do not overstress any potentially weaker underlying materials. In order to achieve this, it will either be necessary for additional investigations extending to the surface of the chalk to be conducted to accurately prove the thickness of the granular layer, which is providing the bearing resistance for each structure; or for the pile designer to employ an alternative approach prior to construction. Such as adoption of driven piles installed to strict rules to ensure they are properly seated within the gravels. It is noted that driven piles would also provide the additional advantage for higher working loads when installed within the Glacial Deposits.

For illustrative purposes **Table 17** gives likely working pile loads for traditional bored piles, designed based on current site investigation data, which does not reply on bearing resistance from the granular Glacial Deposits, due to the current levels of uncertainty and based on the design parameters given in **Table 16**. For this purpose, the soil profile in BH201 has been considered.

Table 17 Illustration of conservative pile working loads for bored cast-in-situ piles

Typical pile working loads (kN)				
Depth of pile below existing ground level (m)	Pile diameter			
	300 mm	350 mm	400 mm	450 mm
18	427	507	593	680
20	430	606	721	826
22	511	705	857	983
24	530*	721*	967	1107
*Pile material limiting stress				

Notwithstanding the above an alternative working loads have been provided in **Table 18** below, which assume that an appropriate level of supplementary ground investigation has been completed to provide the designer sufficient confidence to design the pile benefitting from high end bearing resistance provided by the dense granular soils without over stressing the underlying weaker clay layer. The ground profile recorded in borehole BH201 has been adopted for this design.

Table 18 Illustration of conservative pile working loads for bored cast-in-situ piles

Typical pile working loads (kN)				
Depth of pile below existing ground level (m)	Pile diameter			
	300 mm	350 mm	400 mm	450 mm
10	367**	428**	489**	550**
11	429**	500**	572**	643**
12	497**	580**	662**	745**
** Limited by serviceability limit check, applying Factor of safety = 1.2 on shaft alone				

It should be stressed that the above capacities do not take into consideration pile group effects which is more pronounced for a large number of closely spaced piles.

Notwithstanding 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.

7.3.4 Foundation works risk assessment

Given the site's sensitive location above a Principal Aquifer, located in relatively close proximity to a public water abstraction, it is considered prudent to complete a foundation works risk assessment, particularly due to:

- To allow assessment of the bromate contamination plume within the primary chalk aquifer
- To assess risks from potential contamination within the shallow soils and secondary aquifer - a considerable thickness of cohesive Lowestoft Formation has been encountered beneath the site between the secondary aquifer and the primary chalk aquifer (average thickness of 4.2 m) and is likely to significantly retard migration pathways and the piled foundations may affect this

This should be undertaken in accordance with relevant EA guidance.

7.4 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 unlikely to contain pyrite and brownfield ground unlikely to contain pyrite.

Based on testing results, **Table 19** 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 19 Characteristic pH, water soluble sulphate and total sulphate values

Stratum	pH	Water Soluble Sulphate (mg/l)
Lowestoft Formation	8.49	194.5
Chalk	8.95	34.5
Note: * Based on the mean of the two highest results.		

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 20**, on the basis of water-soluble sulphate and total potential sulphate, respectively.

Table 20 Concrete design class

Stratum	Ground water	Water Soluble Sulphate	
		DS Class	AC Class
Lowestoft Formation	Mobile	DS-1	AC-1
Chalk	Mobile	DS-1	AC-1

Assuming that disturbed ground will be minimised by the use of piled foundations, the recommended ACEC Classification is therefore AC-1 with a Design Sulphate Class of DS-1.

8 CONCLUSIONS AND RECOMMENDATIONS

8.1 Geotechnical assessment

The key findings of the geotechnical assessment are as follows:

8.1.1 Ground Conditions

The ground conditions encountered in the northern area of the site are unlikely to be suitable for the design and construction of conventional shallow spread foundations due to the depth of made ground and the initially softened surface to the underlying natural stratum. Whilst deep spread foundations may be technically feasible, the practicality of construction may be prohibitive due to thickness and instability of the made ground, unless ground levels are to be reduced.

Prior ground improvement, utilising techniques such as vibro-displacement appear feasible to facilitate the construction of reinforced strip foundations, however, all remnant sub structures would require removal and any backfilling completed in accordance with a strict engineering specification.

Continuous flight augured piled foundations are likely to provide the most appropriate foundation option for the proposed structures due to variable thickness of the made ground and the presence of initially soft clays in the upper Lowestoft Formation. Alternatively, however, higher working loads may be attainable from a driven pile technique.

8.1.2 Foundation Works Risk Assessment

Given the sensitivity of the site’s location above a Principal Aquifer it is considered prudent for a foundation works risk assessment report to be completed.

8.1.3 Chemical Attack on Buried Concrete

The results indicate that, in accordance with BRE Special Digest 1: 2005 Concrete in aggressive ground, the Aggressive Chemical Environment for Concrete (ACEC) Classification is AC-1 with a Design Sulphate Class for the site of DS-1.

8.2 Recommendations

All foundation excavations should be inspected, and any made ground, soft, organic or otherwise unsuitable materials removed and replaced with mass concrete.

Additional deep ground investigations may enable a more economical (shallower) pile design to be adopted.

Given the site’s sensitivity in respect to controlled water resources it is recommended that a foundation works risk assessment report be prepared for the development.

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FIGURES

Figure 1 SITE LOCATION PLAN