

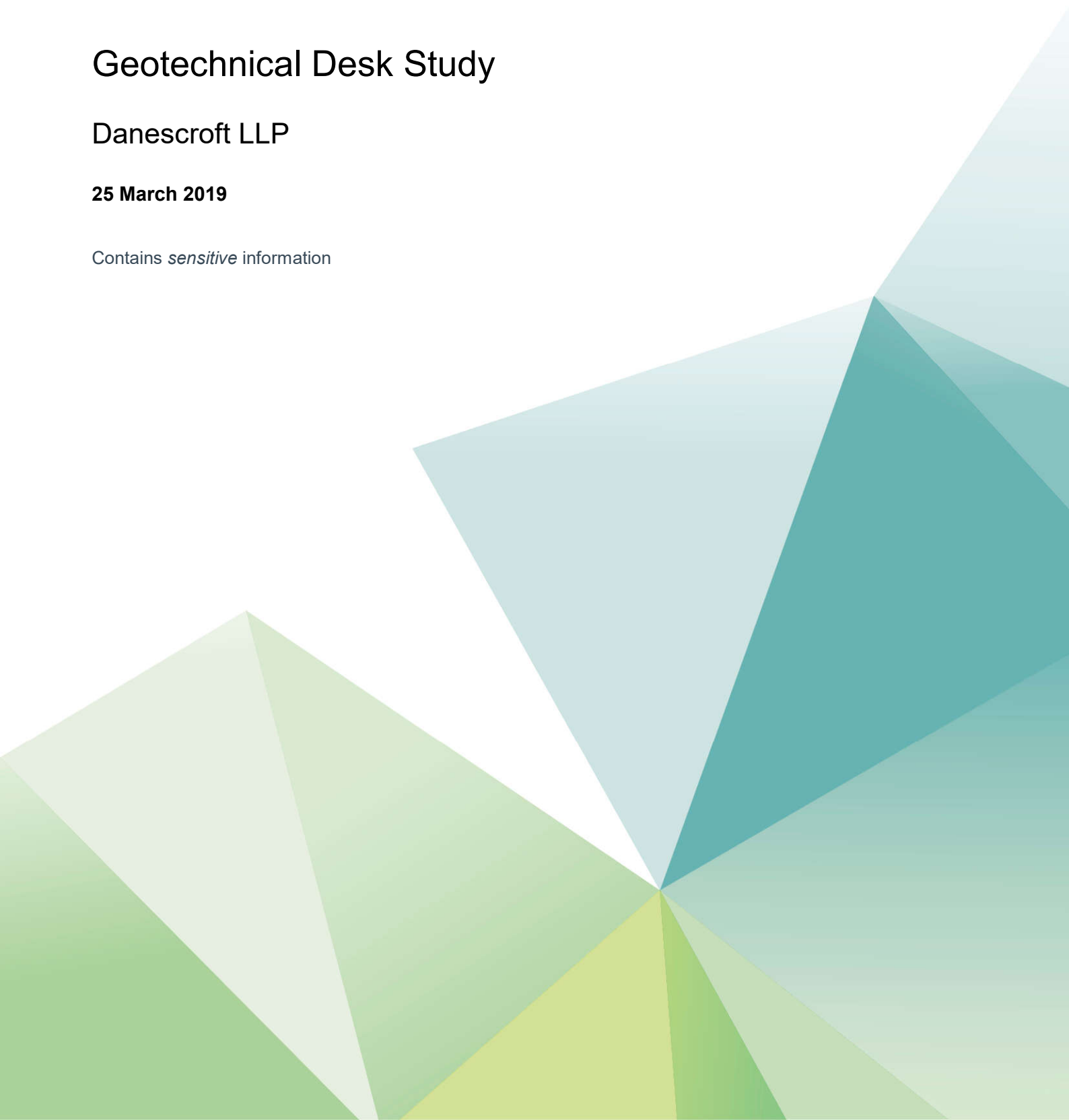
SGN Reading Gasholder

Geotechnical Desk Study

Danescroft LLP

25 March 2019

Contains *sensitive* information



Notice

This document and its contents have been prepared and are intended solely for Danescroft LLP's information and use in relation to Gas Holder Site, Reading.

Atkins assumes no responsibility to any other party in respect of or arising out of or in connection with this document and/or its contents.

This document has 34 pages including the cover.

Document history

Job number: 5182967			Document ref:			
Revision	Purpose description	Originated	Checked	Reviewed	Authorised	Date
Rev 00	Draft for comment	D. Neto	S. Mortimore	D.S. Fort		06-12-2018
Rev 01	For planning	C.Hua	S. Mortimore	D.S. Fort		25-03-2019

Client signoff

Client	Danescroft LLP
Project	Gas Holder Site, Reading
Document title	Geotechnical Desk Study
Job no.	5182967
Copy no.	-
Document reference	

Table of contents

Chapter	Pages
1. Introduction	5
1.1. Brief	5
1.2. Proposed Development	5
1.3. Scope and objective of report	5
1.4. Geotechnical category	5
1.5. Limit of study area	5
2. Existing Information	7
2.1. Sources of Information	7
2.2. Site Location and Topography	7
2.3. Site History	8
2.4. Current condition of Gasholder No. 4 Structure	9
2.5. Unexploded Ordnance	9
3. Existing Structures	10
3.1. Structures within the Site footprint	10
3.2. Existing foundations	10
4. Ground Conditions	11
4.1. Geology	11
4.2. Made Ground	11
4.3. Alluvium	12
4.4. River Terrace Deposits	12
4.5. Seaford and Newhaven Chalk Formations (undifferentiated)	12
4.6. BGS Archive Information	12
4.7. Nearby historical ground investigation reports	12
4.8. Expected Stratigraphy	14
4.9. Hydrology and Hydrogeology	15
4.10. Ground Contamination	15
4.11. Chemical Ground Conditions	15
5. Geotechnical Assessment	17
5.1. Site Overview	17
5.2. Development Proposal	17
5.3. Characteristic Geotechnical Parameters	17
5.4. Excavations and Earthworks	18
5.5. Foundation Options	18
5.6. Spread Foundations	19
5.7. Ground Improvement of Made Ground/Alluvium	20
5.8. Piled Foundations	20
5.9. Floor Slabs	21
5.10. Pavement Construction and CBR	21
5.11. Soakaways	22
6. Geotechnical Risk Register	23
7. Conclusions and Recommendations	27
7.1. Conclusions	27
7.2. Recommendations	27
8. References	28

Appendices	29
Appendix A. Drawings	30
Appendix B. Historical/Nearby Ground Investigation Exploratory Hole Logs	31
Appendix C. Zetica UXO Map	32
Appendix D. Pile Bearing Capacity with depth	33

Tables

Table 2-1	Sources of information	7
Table 4-1	Summary of ground conditions described in BGS and nearby/on-site borehole logs	14
Table 4-2	Summary of the number of historical exploratory hole records available and the number of which are located within the Site	14
Table 4-3	Concrete aggressiveness	15
Table 5-1	Summary of the Site Stratigraphy	17
Table 5-2	Summary of Characteristic Geotechnical Parameters	18
Table 5-3	Design Bearing Resistances for Strip Foundations (DA1-C2)	19
Table 5-4	Design Bearing Resistances for Pad Foundations (DA1-C2)	19
Table 5-5	Reasons For & Against Adoption of Differing Pile Types	20
Table 5-6	Indicative Pile Capacities for the SGN Reading Gasholder Site (Based on Geotechnical Parameters in Table 5-2)	21
Table 5-7	Design CBR values	22
Table 6-1	Assessment of probability	23
Table 6-2	Assessment of consequence / severity	23
Table 6-3	Perceived risk level and mitigation action	24
Table 6-4	Geotechnical risk register	24

Figures

Figure 1-1	Proposed redevelopment of the SGN Reading Gasholder site (Adapted from Collado Collins Architects, 2018) and aerial photography of Site location plan (not to scale)	6
Figure 2-1	Site location plan (Adapted from ©Google, not to scale)	8
Figure 2-2	1930s historical plan showing Gasholders and associated structures (not to scale)	8
Figure 3-1	Structures within the Site footprint (red lines and shaded areas) and proposed building structures	10
Figure 4-1	Drift and Solid Map of Reading (Reproduced with the permission of the British Geological Survey ©NERC. All rights Reserved)	11
Figure 4-2	Nearby AIGE GI 2002 & 2005 showing Development Site Area "Demise 5" (easternmost) which is closest to the Site	14

1. Introduction

1.1. Brief

Atkins Limited (Atkins hereafter) has been commissioned by Danescroft LLP (Danescroft hereafter) to provide a high level Geotechnical Desk Study report for a proposed redevelopment of the Gas Holder Site, Reading. This report determines a ground and groundwater model and presents geotechnical material parameters for the site to inform the design of the proposed works in accordance with BS EN 1997 (Eurocode 7).

The purpose of this report is to highlight potential ground based geotechnical risks associated with the development site and to identify where further assessment may be required to inform a strategy for addressing the identified risks. The report will aim to make preliminary recommendations on potential foundation and substructure options. Geoenvironmental aspects of the Site will be addressed by WSP in a separate report.

1.2. Proposed Development

The proposals for the development of the former Gasholder site comprise the following:

- Block A: 5-storey block of 30 apartments;
- Block B: 9-storey block containing 96 apartments;
- Block C: 3-storey flats building; and
- A 2-storey car park.

No basements are proposed at present. Traditional shallow foundation options will be considered as appropriate solutions for structures which are generally lightly loaded, such as two to three storey buildings. Piled and / or raft foundation options will be considered for more heavily loaded structures, or where ground conditions are unfavourable.

Appendix A presents current redevelopment plans at the time of writing this report.

1.3. Scope and objective of report

The objective of this Desk Study report is to collate and review information related to the site in order to obtain an understanding of the existing site conditions, structures, anticipated geology and to identify significant geotechnical risks and constraints pertinent to the proposed development.

The objectives of this report are:

- To identify the existing ground-based site conditions based on desktop research;
- To review the ground conditions based on information from the British Geological Survey (BGS) and previous ground investigation works;
- To identify the significant underground assets running beneath/in close proximity to the site;
- To present preliminary substructure scheme design options; and
- To compile a Risk Register detailing identified ground-based risks for the proposed scheme and ultimately a set of recommendations for future work based on the findings within this report.

1.4. Geotechnical category

In accordance with Eurocode 7 and British Standards, the project has been assigned a Geotechnical Category 2, which is described as conventional types of structure and foundation with no exceptional risk or difficult soil or loading conditions. This will be assessed and subject to change at each stage of the project.

1.5. Limit of study area

This report specifically relates to the proposed redevelopment of the eastern end of the former SGN Reading gasworks site and proposed buildings as shown in Figure 1-1 below. This eastern end shall be referred as the 'Site' hereafter.

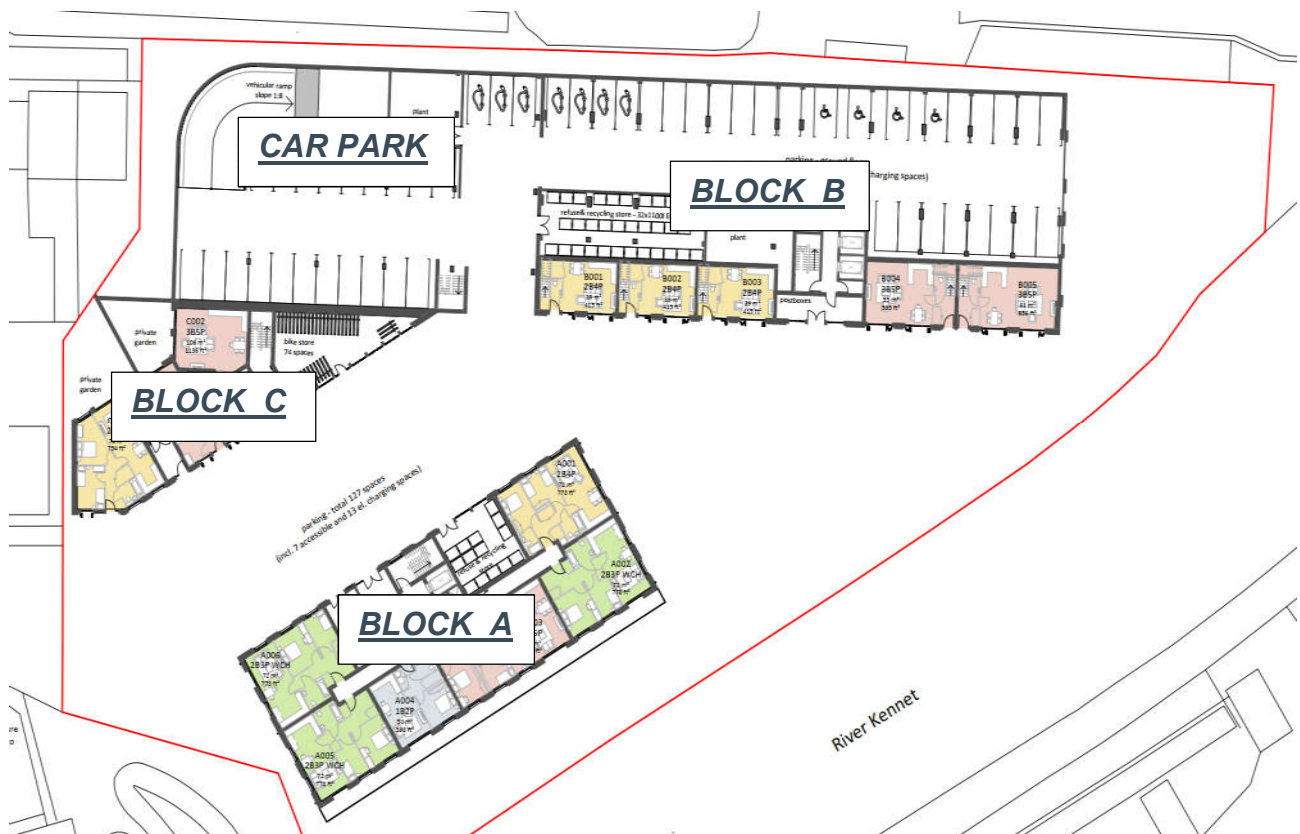


Figure 1-1 Proposed redevelopment of the SGN Reading Gasholder site (Adapted from Collado Collins Architects, 2018) and aerial photography of Site location plan (not to scale)

2. Existing Information

2.1. Sources of Information

Table 2-1 presents the sources of information used in the preparation of this report and a summary of the information received.

Table 2-1 Sources of information

Subject	Type	Reference and additional comments	Source
Topography	Maps and Aerial Photography	Google Maps	Google (2018)
	Topographical data	Google Maps	
Geology	BGS Online GeoIndex	British Geological Survey, "Geoindex onshore"	British Geological Survey (2018)
	Maps		
	Historical reports and borehole records	British Geological Survey, "Geoindex onshore"; Desk Study Report 42 Kenavon Drive, Reading, Berkshire; Ground Conditions Report, The Leasing Group Site, Kenavon Drive, Reading, Berkshire; Clarification Ground Investigation Report 42 Kenavon Drive, Reading, Berkshire Excerpt of Gas Journal	British Geological Survey (2018); AIG Engineering Group Limited (2005); AIG Engineering Group Limited (2002) AIG Engineering Group Limited (2005) Gas Journal (1924)
Environment	Historical Maps	Freely available historical Ordnance Survey mapping websites	-
	Hydrology and Hydrogeology	Human Health & Controlled Waters Quantitative Risk Assessment Report 42 Kenavon Drive, Reading; Ground contamination remediation options appraisal and remediation method statement; Ground contamination remedial statement 42 Kenavon Drive, Reading, Berkshire	AIG Engineering Group Limited (2008); AIG Engineering Group Limited (2008); AIG Engineering Group Limited (2005)
Additional	Unexploded Ordnance	Zetica Map (Zetica, 2013)	Zetica (2017)

2.2. Site Location and Topography

The Site is located at the former SGN Gasholder No. 4, Kenavon Road, Reading in Berkshire. The Gasholder No. 4 Site is located at the eastern end of Kenavon Drive, approximately 1500m north east of Reading town centre in Berkshire. The Site is currently used for temporary offices, car parking and vehicle storage. It occupies an area of approximately 0.7 hectares. The Site is centred at approximate OS grid reference 472936E 173696N. Access is gained via Alexander Turner Close road from the west of the Site.

The River Kennet is situated some 15m south east of Block A and some 25m east of Block B which flows into the River Thames some 250m to the north east. A multi-track railway line with a superstore beyond is located some 30m north of the Site. To the west are residential flats of recent construction. An existing water tower located in the north east corner of the Site is to be retained. Based on the available aerial photographs, the Site is generally flat at an elevation of approximately 38m AOD. A site location plan is provided as shown in Figure 2-1 below.



Figure 2-1 Site location plan (Adapted from ©Google, not to scale)

2.3. Site History

A gas works opened on the site in 1886 and underwent several phases of development and redevelopment until gas manufacture ceased in the mid-1970s. Generally, gas production processes were undertaken in the western part of the gasworks site, with gas purification, storage and by-product processing being undertaken in the eastern area (See Figure 2-2). Following demolition of most of the former gas works adjacent structures in the late 1960s/early 1970s, the site was utilised as a gas distribution depot and storage yard prior to redevelopment in the 1990s.

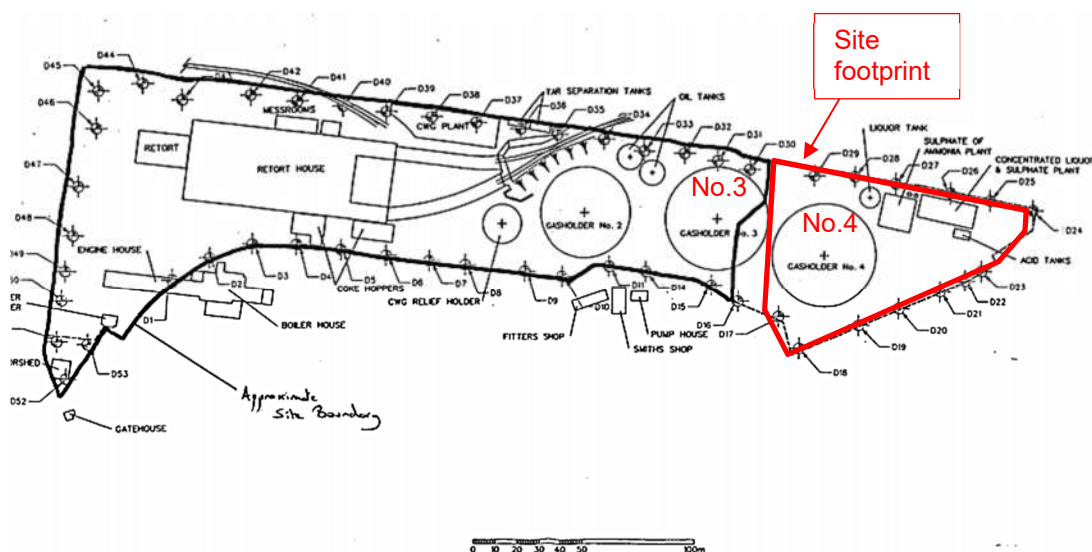


Figure 2-2 1930s historical plan showing Gasholders and associated structures (not to scale)

2.4. Current condition of Gasholder No. 4 Structure

The former gasworks Gasholder No. 4 structure as shown above in Figure 2-2 remains on-site and has not been demolished. The structure is assumed to be founded 4mbgl as per adjacent former Gasholder No. 3 (demolished) information obtained from AIGE historical records. Also, from Google maps aerial imagery, the structure appears to be about 50m in diameter with the wall elements made of steel and raising 3m above ground approximately. At this stage it is unknown whether the Gasholder has been backfilled or remained empty.

2.5. Unexploded Ordnance

The Zetica unexploded ordnance (UXO) risk map (shown in Appendix C) for the Site indicates that the site lies within a region of low risk. Low risk is defined as areas with a bomb density of 15 bombs per 1000 acres or less.

3. Existing Structures

The following section includes descriptions of the existing structures that currently lie within the gasworks footprint. Area plans taken through the Site are presented in Appendix A. The information available was taken from existing reports from previous work undertaken as part of the adjacent gasworks site redevelopment.

3.1. Structures within the Site footprint

Figure 3-1 below shows the existing Gasholder No.4 (shaded area in purple) and associated structures on the Site footprint including the River Kennet walls (shaded area in yellow). On the north east end corner of the site, a former water tower (shaded area in purple) can be found and is deemed to be retained as part of the site redevelopment plan.



Figure 3-1 Structures within the Site footprint (red lines and shaded areas) and proposed building structures

3.2. Existing foundations

Based on historical information it appears that the Gasholder No. 4 foundations may be supported by 'Simplex' concrete piles (>600No.) with a reinforced concrete raft above, if of similar construction to the Gasholder No. 3 in the adjacent site to the west. Furthermore, the existing reports on the adjacent site informed on the former Gasholder No. 2 and No. 3 being remediated and backfilled. A historical cable percussion borehole (BH901) carried out in Gasholder No. 3, located in the adjacent site showed evidence of the backfilling sequence to a maximum depth of 4.0mbgl with a concrete slab of 0.8m thickness beneath the backfill. The vertical extent (maximum depth) of the original excavation to form the Gasholder No. 3 structure is understood to have been limited to the top of the natural River Terrace Deposits. No records are currently available with respect to the construction of Gasholder No. 4 and River Kennet Walls.

Figure 2-2 presented above as an excerpt of the 1930 historical plan, which is included in the Envirocheck report for the adjacent gasworks site (presented within the AIGE 2002 Ground Conditions Report), shows the proximity of former Gasholder No. 3 (demolished) to the proposed development site.

4. Ground Conditions

4.1. Geology

The anticipated ground conditions beneath the Site are discussed in this chapter and have been assessed using the following sources:

- British Geological Survey (BGS) 1:50,000 scale Sheet 268 for Reading, Solid and Drift Edition (2000);
- Historical Borehole Logs from the British Geological Society (BGS) website (Appendix B)
- online BGS Geology of Britain viewer

The BGS maps indicate the geology beneath the Site underlain by a superficial cover of Alluvium overlying River Terrace Deposits with Upper Chalk at a depth (circa 7.0mbgl) as shown in Figure 4-1. This is confirmed by the local BGS borehole logs, which also note variable thickness of the overlying Made Ground. Based on the sources listed above, it could be anticipated that the geological sequence indicated on the BGS boreholes would be encountered underlying the Site. Two historical site-specific boreholes are provided in Appendix B.

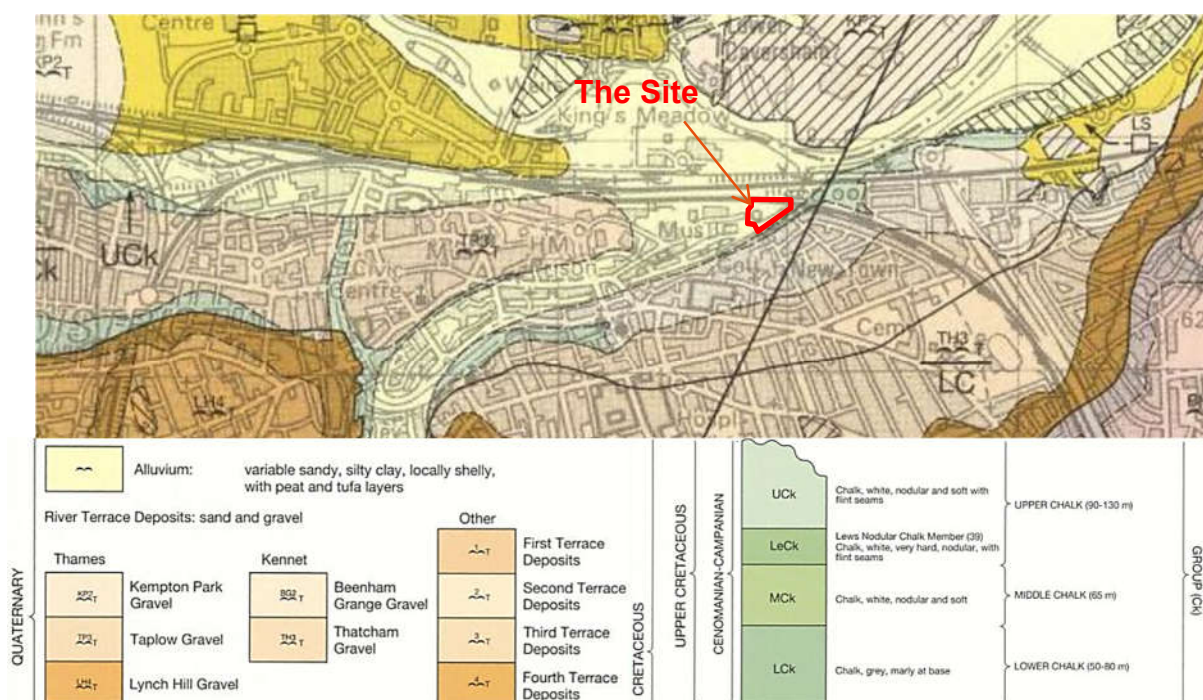


Figure 4-1 Drift and Solid Map of Reading (Reproduced with the permission of the British Geological Survey ©NERC. All rights Reserved)

4.2. Made Ground

The historical borehole logs show the Made Ground to have an approximate thickness of between 2.1m and 3.3m. The Made Ground typically comprises Top Soil/Fill (loose brick fragments, concrete rubble to boulder size, ashes, wood, slag, white fibres) and loose brown silty fine sand to coarse gravel with soft to firm greenish grey locally slightly sandy clay. Due to the variable nature of Made Ground, it is not recommended for use as a founding stratum.

Old buried foundations including concrete slabs associated with the historical development of the site should also be expected locally across the site.

4.3. Alluvium

Underlying the Made Ground, alluvium is anticipated. This comprises firm brown sandy silty clay with black organic streaks and black peaty fragments becoming loose brown very clayey fine to coarse sand with strong hydrocarbon odour. Approximate thickness of 1.0m.

4.4. River Terrace Deposits

Some of the nearby borehole logs suggested River Terrace Deposits (RTD) possibly from the Kempton Park Gravel Formation. The RTD comprises medium dense black-stained medium to coarse angular to sub-round flint gravel with little fine to coarse sand and rare cobbles.

4.5. Seaford and Newhaven Chalk Formations (undifferentiated)

The online BGS Geology of Britain viewer shows the bedrock beneath the site to comprise the Seaford and Newhaven Chalk Formations (undifferentiated) of the White Chalk Subgroup, (formerly Upper Chalk).

The Chalk encountered was recorded as structureless (Grade Dm) and comprised of white cream grey sandy slightly gravelly silt with flint and chalk cobbles. Gravel of fine occasionally medium and coarse of chalk and flint. Hydrocarbon odour and visible contamination was observed at the top.

Standard Penetration Tests (SPT) showed the competency of the Chalk to increase with depth as the degradational effects of weathering of the material decreased with the chalk becoming a weak white Chalk (Grade Dc) at depth with clasts becoming medium to high density. The boreholes were of insufficient depth to identify the presence of structured chalk.

4.6. BGS Archive Information

A search was made of the British Geological Survey (BGS) archives for borehole information pertinent to the sites. A total of nine borehole records were identified and are included in Appendix B.

The geological information obtained was based on records dating back to 1990. Thus, the logging standards will vary and should be considered when being used to derive the anticipated ground profile.

A review of the records indicates that out of the nine boreholes, two are situated adjacent to Block A (proposed development to the south westernmost part of the Site), eight records proved the top of the Chalk stratum, seven records contained groundwater information, and all provided in-situ data but no laboratory test result data.

4.7. Nearby historical ground investigation reports

4.7.1. AIGE Ground Investigation, 2002

The ground investigation was designed by AIG Engineering Limited (AIGE) to undertake an assessment of potential significant risks associated with ground contamination at the site, with specific regard to its continued use under the then current conditions (offices, workshops and vehicle storage). The findings of the site investigation and assessment were produced in a Ground Conditions Report in November 2002.

The fieldwork was carried out in October 2002 comprising:

- Four cable percussion boreholes with standpipe installations;
- 14 No. boreholes using a drive-in sampler system together with 5 combined gas/groundwater standpipe monitoring installations;
- Chemical analysis of selected soil and groundwater samples;
- In-situ and laboratory geotechnical testing;

- Monitoring of soil gas and groundwater regimes.

4.7.2. AIGE Ground Investigation, 2005

The ground investigation was designed by AIG Engineering Limited (AIGE) to undertake a reassessment of potential significant risks associated with ground contamination at the site in relation to two possible residential redevelopment schemes. The findings of the site investigation and assessment were produced in a Clarification Site Investigation Report in December 2005.

The fieldwork was carried out between 28th September and 12th October 2005 comprising:

- Five light cable percussion boreholes, designated BH901 to BH905 with standpipe installations;
- 28No. drive-in sampler boreholes, designated DS701 to DS721 twelve with standpipe installations;
- 32No. trial pits, designated TP801 to TP830, TP/BH903A and TP/BH904;
- Subsequent laboratory chemical analysis of selected soil and groundwater samples;
- Soil gas and groundwater monitoring from borehole standpipes.

The nature and characteristics of the various geological units encountered during the 2002 and 2005 AIGE investigations for the adjacent site have been summarised by segmenting the site into four proposed development areas as per the excerpt of proposed scheme and exploratory hole location plan drawing 100199/2-002 shown in Figure 4-2.

- Demise 1
- Demise 2
- Demise 3/4
- Demise 5

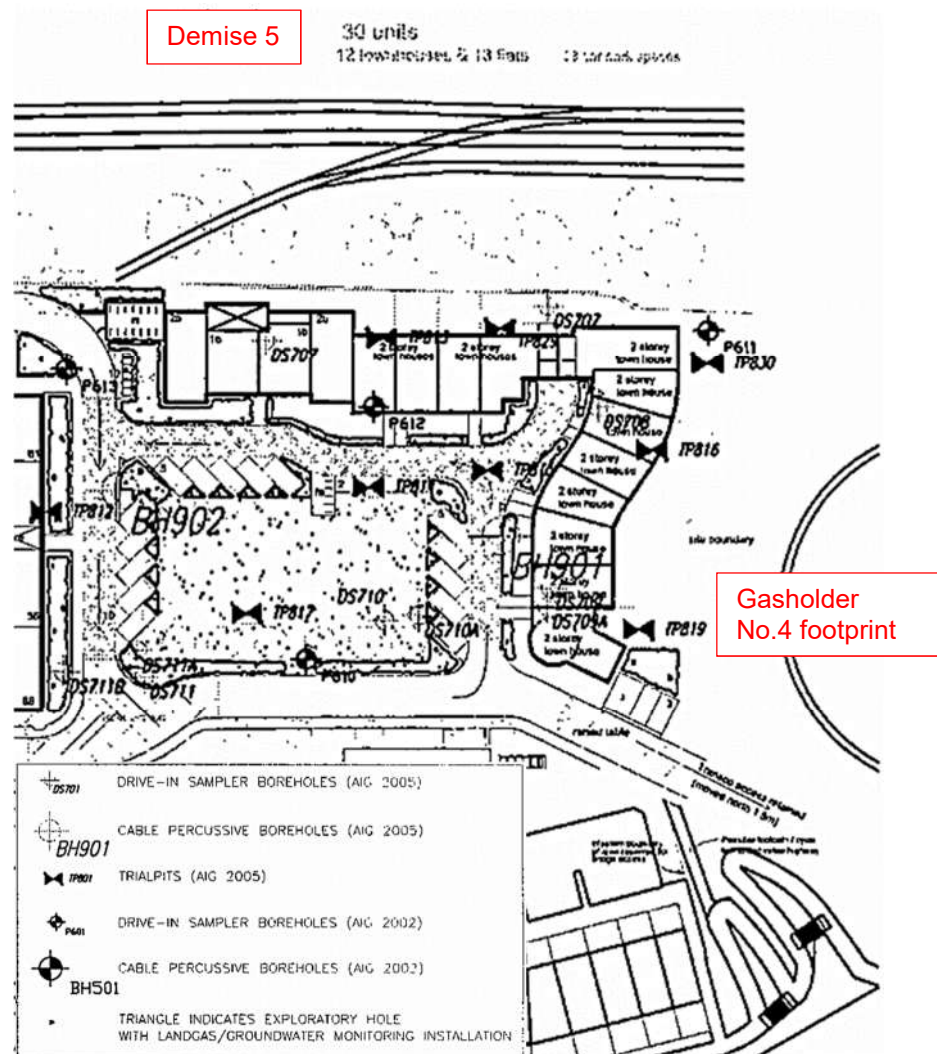


Figure 4-2 Nearby AIGE GI 2002 & 2005 showing Development Site Area “Demise 5” (easternmost) which is closest to the Site

Site Area Demise 5 is located adjacent to the Site footprint where the former Gasholder No.3 was located. BH901 is located within the former Gasholder No. 3 area footprint.

The former Gasholder No.3 was remediated and backfilled as Gasholder No.2. Exploratory locations BH901, DS708, DS709, DS709A, TP815, TP816 and TP819 were in the remediated area and show evidence of the backfilling sequence to a maximum depth of 4.0mbgl in BH901.

4.8. Expected Stratigraphy

The general stratigraphy of the Site based on BGS and nearby/on-site information is summarised in Table 4-1. Table 4-2 presents a summary of the number of historical exploratory hole records available and the number of which are located within the Site.

Table 4-1 Summary of ground conditions described in BGS and nearby/on-site borehole logs

Material	Stratigraphy	Elevation to top of stratum - Range (average) m AOD	Thickness of stratum Range (average) m	Typical Description
Made Ground	Quaternary/ Holocene	37.1 – 38.15 (37.5)	2.1 – 3.3 (3)	Brown dark grey brown silty clayey SAND and fine to medium sub angular to sub rounded GRAVEL with occasional concrete cobbles. Gravel of flint occasional concrete and brick fragments metal and decaying wood fragments. Strong hydrocarbon odour.
Alluvium (when present)		34.15 – 35.2 (34.5)	0.45 – 1.6 (1)	Soft to firm black brown blue gravelly CLAY. Gravel of fine occasionally medium sub angular flint. Hydrocarbon odour.
River Terrace Gravels		33.15 – 34 (33.5)	2 – 4 (3)	Black grey silty sandy fine to medium sub angular GRAVEL
Seaford and Newhaven Chalk Formations (undifferentiated) – Grade Dm	Upper Cretaceous	29.4 – 31.7 (30.5)	Proven to 4.5m	Structureless CHALK comprised of white cream grey sandy slightly gravelly SILT with flint and chalk cobbles. Chalk clasts of low density
Seaford and Newhaven Chalk Formations (undifferentiated) – Grade Dc		25	Proven to 2.0m	Structureless CHALK comprised of cream white very silty sandy fine to coarse sub rounded to sub angular GRAVEL of chalk and flint with flint and chalk cobbles. Chalk clasts are medium to high density.

Table 4-2 Summary of the number of historical exploratory hole records available and the number of which are located within the Site

Historical GI records	No. of boreholes / window sample holes	Trial Pits	Exploratory holes within Site	
			Boreholes / Window Samples	Trial Pits
BGS (2018)	9	0	2 (SU77SW91 & -92)	0

AIGE 2002 GI (Section 4.7.1)	28	0	0	0
AIGE 2005 GI (Section 4.7.2)	33	32	0	0

4.9. Hydrology and Hydrogeology

The River Terrace Deposits are a minor aquifer. The Chalk is a major aquifer. The canalised River Kennet lies approximately 30m offsite to the south east. The Site lies within a designated Zone III (Total Catchment) groundwater source protection zone. The River Thames lies approximately 250m offsite to the north.

Perched groundwater was encountered within the Made Ground and Alluvium between 1.6mbgl and 2.1mbgl. The highest standing perched groundwater levels recorded in three standpipes from adjacent AIGE 2005 GI are between 1.2 and 1.6mbgl. Groundwater was struck within the River Terrace Deposits at depths of between 2.5 to 3.5mbgl. The highest standing groundwater levels recorded within standpipes from adjacent AIGE 2005 GI with a response zone within the RTD or Chalk was at 1.75mbgl.

The presence of granular Made Ground on the site is discontinuous. It is considered that groundwater underlying the site is formed by smaller incremental areas of perched water rather than being representative of a larger groundwater body. Further investigation is required to characterise the groundwater regime.

4.10. Ground Contamination

The existing reports on the site show evidence that the Gasholders would have been set in large pits, filled with water to provide a seal, and were able to rise and fall with the change in volume of the gas stored. The base of the holders would have accumulated tar and contaminated water over time. Tar was often used as lubricant for the Gasholders, so it may be found spread around the perimeter of the concrete bases. Also, contaminants may have been spread around the adjacent site during site clearance, thus contaminating parts of the site where they would not normally have been found. The normal practice was to empty underground storage tanks and the bases of Gasholders before backfilling them with rubble. However, semi-solid sludges were not always removed, and some tanks may not have been cleared at all. Furthermore, during the demolition process there is the possibility that asbestos containing materials could have been spread about the adjacent site in an uncontrolled manner.

Soil contamination and geoenvironmental aspects of the Site will be covered by WSP in a separate report.

4.11. Chemical Ground Conditions

4.11.1. Visual and Olfactory Evidence

Visual and olfactory evidence of contamination was identified in the historical boreholes in the Site and in the nearby site within near surface soils. Olfactory evidence predominantly comprised hydrocarbons type odours. Visual and olfactory evidence of contamination was found within the Made ground, RTD and Chalk.

4.11.2. Ground Aggressivity

The chemical testing (undertaken by AIGE) for the adjacent site carried out in accordance with BRE Special Digest 1 classification for brownfield (Made Ground) and natural (RTD and Chalk) conditions, indicated the following results for aggressiveness of soil to buried concrete. The results are summarised in Table 4-3 for each stratum.

Table 4-3 Concrete aggressiveness

Stratum	Design Sulphate (DS) Class	Aggressive Chemical Environment for Concrete (ACEC) Class
Made Ground	DS-2	AC-2
RTD	DS-2	AC-2

Stratum	Design Sulphate (DS) Class	Aggressive Chemical Environment for Concrete (ACEC) Class
Seaford and Newhaven Chalk Formations (undifferentiated)	DS-1	AC-1s

Further site-specific testing should be undertaken to confirm the above results are representative of the conditions on the Site.

5. Geotechnical Assessment

5.1. Site Overview

The findings of this report indicate that the site is underlain by Made Ground, Alluvium (when present), River Terrace Deposits and Chalk at depth.

Based on the historical boreholes there is no confirmation of the presence of relic foundations. However, relic foundations and both redundant and active buried utilities are likely to be present across the site, see Section 5.4).

However, one borehole (AIGE, 2005) encountered concrete at depths between 4.0mbgl to 4.8mbgl which coincide with base of former Gasholder No.3.

5.2. Development Proposal

The proposed development is understood to comprise of approximately 130 new units spread across the site. The buildings vary from 2-storey to 11-storey developments across the site-wide area red line boundary, See Figure 1-1).

The redevelopment generally involves the demolition of the existing Gasholder No.4 and associated buildings which occupy the Site, and the construction of approximately three plots (residential). There are currently no basements envisaged at this stage with an area to be utilised for car parking.

To facilitate the demolition of existing structures, excavation works and construction of the proposed buildings within the masterplan, liaison with the owners of utilities and assets within the area, residents adjacent to the site, highways specialists and tree specialists shall be required.

The proposed development will involve the construction of a high-rise building using reinforced concrete (RC) frame structure with 11 floors. The column loads (unfactored) of the proposed building have been assessed at this stage of design development and are expected to be in the range of 1500kN to 9100kN. Piled foundations will be required for this structure.

The medium-rise building using a RC frame structures or a combination of RC frame and lightweight superstructure with 4-5 floors. Here a raft option could be feasible as the maximum column loads are about 1000kN to 3000kN.

5.3. Characteristic Geotechnical Parameters

A summary of the site stratigraphy and parameters for preliminary design, based on the interpretation of the limited available data are presented in Tables 5-1 and 5-2.

Table 5-1 Summary of the Site Stratigraphy

Stratum	Top of Stratum (mAOD)	Thickness of Stratum (m)
Made Ground	+37.5	3.0
Alluvium	+34.5	1.0
RTD	+33.5	3.0
Seaford and Newhaven Chalk Formations (undifferentiated) – Grade Dm	+30.5	4.5
Seaford and Newhaven Chalk Formations (undifferentiated) – Grade Dc	+25	Proven to 2.0 (BH901)

Table 5-2 Summary of Characteristic Geotechnical Parameters

Stratum	Unit Weight [kN/m ³]	SPT 'N' Value	Estimated Effective Friction Angle based on SPT 'N' Average Value ^[1] [°]
Made Ground			N/A
Alluvium			N/A
River Terrace Deposits	19	21 to 44	35
Seaford and Newhaven Chalk Formations (undifferentiated) – Grade Dm	19	8 to 23	31 ^[2]
Seaford and Newhaven Chalk Formations (undifferentiated) – Grade Dc	19	15 to 30	33 ^[2]

^[1] CIRIA Report 143, Standard Penetration Test, 1995

^[2] CIRIA Report C574, Engineering in Chalk, 2002

5.3.1. Design Assumptions

Preliminary piled and spread foundation design was carried out using Eurocode 7 and UK's National Annex and CIRIA C574 guide for piles in Chalk. These will be based on-site borehole data available to date which indicate structureless Chalk with depth.

The preliminary design for the piles assumed zero frictional support from the Made Ground/Alluvium to the top of RTG (ca. 4mbgl) and the groundwater table was assumed to be at 1.0m bgl.

5.4. Excavations and Earthworks

Excavations will likely encounter obstructions across the entire site. The predominantly concrete and brick obstructions are likely to require breaking out prior to foundation installation. For example, the maximum depth of obstructions encountered during the works in the adjacent site to the west, recorded that at Gasholder No. 3, a 0.8m thick concrete slab was encountered at 4.0mbgl. However, deeper obstructions were also likely as it is understood that driven piles were encountered which had been driven through the Gasholder No 3's base. This evidence is corroborated by the historical gas journal information where it is documented that more than 600 No. "Simplex" concrete piles were driven in and a reinforced concrete raft then placed upon the piles.

Additionally, shallow foundation obstructions from the existing tanks and plants from the gasworks on site are likely. Shallow excavations in the generally granular made ground deposits will require temporary trench support or battering of excavation sides.

Perched groundwater is likely to be encountered locally at shallow depths (<1.5m) below ground level within the Made Ground and Alluvium deposits. Some form of groundwater control such as sump pumping will be required to control groundwater ingress at shallow depth but dewatering and cut-offs are likely to be required for excavations taken into the saturated granular River Terrace Deposits.

It is recommended that the Made Ground and Alluvium (where present) is excavated within the footprint of all proposed structures to facilitate removal of subsurface obstructions or alternatively, if appropriate, probing is carried out at proposed pile locations. Different material types from all excavations should be stock piled separately for assessment.

5.5. Foundation Options

The selection of the final foundation solution is influenced by a number of factors relating to the ground conditions and the proposed development. These are generally associated with the ability of the ground to support the loads applied by the proposed buildings and the sensitivity of the buildings to ground movement.

The available exploratory holes at the former Gasholder site have revealed a sequence of Made Ground overlying Alluvium with underlying saturated granular River Terrace Deposits and structureless Chalk at depth. As described in Section 5.2 the proposed structures foundation options will likely be comprised of, as follows:

- For **Building A (5-storey)**:
(with reinforced concrete frame)
 - *Option 1*: Piled foundations (penetrating through the existing Gasholder base);
 - *Option 2*: A raft foundation bearing on the River Terrace Deposits.
- (mix between reinforced concrete frame and light weight superstructure)
 - *Option 3*: A raft foundation founded at shallower depths;
 - *Option 4*: Pad (spread) foundations with suspended floor slabs.

For both, options 3 and 4, the Made Ground / Alluvium may be replaced or improved to achieve a minimum design bearing resistance and limit differential settlements particularly where the building footprint bridges between the existing Gasholder and the natural ground as presented in Section 5.6 below.

- For **Building B (9-storey)**:
 - Piled foundations.
- **Block C and car park**
 - Spread foundations with suspended floor slabs provided the Made Ground / Alluvium is replaced or improved to achieve a minimum design bearing resistance as presented in Section 5.6 below.

Two pile options were considered namely Rotary Bored and Continuous Flight Auger (CFA) piles. An assessment of the differing piled schemes considered is presented in Section 5.8 below.

5.6. Spread Foundations

A preliminary bearing capacity analysis has been undertaken in accordance with BS EN 1997-1:2004 and using the UK National Appendix document. Recommended design bearing resistances for strip and pad foundations are presented in Tables 5-3 and 5-4, respectively. The values recommended are to limit maximum anticipated total settlements to about 30mm (long-term) and governed by ULS DA1 Combination 2 (partial factors applied to soil parameters).

The following design bearing resistances have been calculated assuming a minimum SPT 'N' value of 25 (River Terrace Deposits). The same soil strength can be assumed at shallower depths assuming excavation and replacement of the Made Ground and Alluvium with engineered fill material and/or ground improvement is undertaken.

Table 5-3 Design Bearing Resistances for Strip Foundations (DA1-C2)

Strip Foundations	Design bearing resistance (kPa)*		
	Foundation Width		
Depth (mbgl.)	0.5m	1.0m	1.5m
1.0	250	300	400
2.0	400	500	550
3.0	550	650	750
4.0	700	800	900

*Assuming a minimum SPT 'N' value of 25 equivalent to a friction angle of 35 degrees.

Table 5-4 Design Bearing Resistances for Pad Foundations (DA1-C2)

Pad Foundations	Design bearing resistance (kPa)*		
	Foundation Size		
Depth (mbgl.)	1.0 x 1.0m	1.5 x 1.5m	2.0 x 2.0m
1.0	350	400	450
2.0	550	650	700
3.0	800	850	900
4.0	1000	1100	1150

*Assuming a minimum SPT 'N' value of 25 equivalent to a friction angle of 35 degrees.

Contains *sensitive* information

Buried structures and soil contamination may be encountered which will require consideration when assessing the most appropriate foundation solution for specific structures. As a result, ground improvement could be adopted for the Made Ground and Alluvium layers. Refer to Section 5.7 below for a discussion about possible ground improvement techniques.

5.7. Ground Improvement of Made Ground/Alluvium

Where the building loads are light, vibro stabilisation could possibly be undertaken beneath the foundations and floor slabs in the areas where any subsurface structures have been removed or confirmed as absent or where poor ground conditions and contamination is present to create a suitable load bearing resistance. However, consideration for the alluvium stratum known to be present underlying the Made ground will require careful consideration of ground improvement techniques such as vibro stone columns. These can be perceived as offering a pathway for contamination. Therefore, either a concrete plug at the base could be formed or variations on this option would be to use vibro concrete columns (VCCs). These may be preferred by regulators as they would create less of a potential migratory pathway for contamination and work well in cohesive materials if present.

5.8. Piled Foundations

Two piled options were considered. A detailed discussion of the reasons for and against the adoption of the differing pile types is shown in Table 5-5, below.

Table 5-5 Reasons For & Against Adoption of Differing Pile Types

Pile Type	Reasons for Adoption	Reasons against Adoption
Rotary Bored Piles	• Large range of diameters and depths may be achieved. • Largest load carrying potential. • Will be able to penetrate obstructions, possibly at reduced rate of penetration. • Temporary casing through the Made Ground/Alluvium/RTD acting as a barrier. • Relatively low-level vibrations	• Will generate arisings (potentially contaminated) which will require off-site disposal. • Higher mobilisation cost and longer installation times due to obstructions. • Larger corer may be required to penetrate obstructions resulting in an uneconomical pile design.
Continuous Flight Auger (CFA) Piles	• Does not require temporary support during construction. • Quicker and generally lower cost than rotary bored piles.	• Will generate arisings (potentially contaminated) which will require off-site disposal. • Limited ability to penetrate obstructions. Resulting in: • Pile location clearance. • Removal of obstructions prior to pile installation by excavation or pre-coring with a separate rig. • Relocation of piles resulting in additional piles/alterations to column locations/increased pile caps. • Cost and programme implications. • Pile depth/diameter limited by rig capacity. • Dependent on continuous core delivery throughout piling.

The use of bored piles may result in an excessively long piling program and lead to increases in cost. The piles ability to bore through hard stratum is a major advantage but they will require temporary support during construction. It should be noted that both forms of piling will generate potentially contaminated arisings which will require management off-site disposal and will require large volumes of water for washing down which may result in contaminant being washed into the aquifers below.

CFA piling presents an option whereby economy and flexibility can be maintained. CFA piling can extend to depths of up to 25m below ground and to diameters of up to 1200mm, the use of temporary supports will not be required as the pile bore is concreted simultaneously as the auger is withdrawn.

Noise and vibrations caused by CFA rigs are usually found to be acceptable on urban sites. A designed piling mat shall be required to be used as a platform for the rig, the Federation of Piling Specialists (FPS) provide guidance on its design and procurement.

Continuous Flight Auger (CFA) piles are likely to be the most appropriate type of piles for this development.

The charts showing the available pile capacity with depth are presented in Appendix D for the two different cases mentioned above. Indicative pile capacities based on CFA piles and using the geotechnical parameters determined in Section 5.3 and axial capacities for various pile diameters and lengths are shown in Table 5-6. The Pile capacities shown are for single isolated piles based on an overall factor of safety of 3.5 on skin friction and end bearing.

Table 5-6 Indicative Pile Capacities for the SGN Reading Gasholder Site (Based on Geotechnical Parameters in Table 5-2)

Pile Length (m bgl)	Rotary Bored Pile Design Resistance (kN)			CFA Pile Design Resistance (kN)		
	600mm	900mm	1200mm	600mm	900mm	1200mm
15	800	1400	2250	750	1400	2200
16	900	1600	2500	850	1550	2450
17	1000	1800	2800	950	1700	2650
18	1100	1950	3050	1000	1800	2800
19	1200	2150	3250	1050	1900	3000
20	1350	2300	3450	1150	2000	3100
21	1450	2450	3700	1200	2150	3250
22	1550	2650	4000	1300	2250	3400
23	1700	2900	4200	1400	2400	3600
24	1850	3050	4500	1500	2500	3750
25	2000	3300	4800	1550	2650	3950
26	2150	3500	5050	1700	2800	4100
27	2300	3750	5400	1750	2950	4300
28	2450	4000	5700	1850	3100	4500
29	2500	4200	6050	2000	3250	4750
30	2500	4500	6400	2050	3400	4950

A detailed pile design is outside the scope of this report. The final pile layout will depend on many factors including the existing and final layout of the structural columns, transfer structures, extent and location of the cores, location, impact of any buried structures and ground conditions. The exact type and dimensions of the piles to be used will be determined at detailed design stage by the specialist piling contractor. It is proposed that all pile locations be probed to the top of the Chalk by the Main Contractor prior to the commencement of piling works. Any obstructions encountered during probing should be removed and the excavated areas backfilled with arisings.

5.9. Floor Slabs

NHBC recommend adoption of suspended slabs where made ground thickness exceeds 600mm depth and these criteria are exceeded over the whole of the site area.

5.10. Pavement Construction and CBR

It is assumed that capping material will not be required for pavement construction onto granular Made Ground at the site. Based on information retrieved from the nearby site where design CBR values have been derived using IAN73/06 (Highways England, 2009) and TRRL guidance (TRRL, 1984) for the Made Ground assuming typical average construction conditions.

The recommended design values are summarised in Table 5-7. Soft cohesive spots encountered during construction should be dug out and replaced and loose granular Made Ground material should be adequately compacted in order to achieve the minimum CBR values summarised in Table 5-7.

Table 5-7 Design CBR values

Stratum	Design CBR (%)	
Made Ground	Granular	40 (assuming average construction conditions)
	Cohesive	2*
*Assuming thin construction		

It should be noted that the subgrade CBR value must be checked on site before foundation construction starts and must be equal or greater than the Design CBR.

5.11. Soakaways

Soakaway testing has not been undertaken at the site. The granular Made Ground and Alluvium, which overlies the RTD across the majority of the site, may be geotechnically suitable as a soakaway, although given the potential for remaining contamination, and to support the re-use of site won materials on site, this is not recommended without further review of the cut and fill requirements and subsequent discussions/approval by the Environment Agency.

6. Geotechnical Risk Register

This section presents the Geotechnical Risk Register for the proposed works to be undertaken for the scheme. The Geotechnical Risk Register identifies the risks and the consequences, recommends preventative or mitigation measures and re-assesses the risk in light of those measures. The geotechnical risks are discussed in terms of probability, severity and risk, as defined below:

Probability (P): The perceived likelihood of the identified geotechnical hazard actually occurring (defined as a rating in Table 6-1).

Severity (S): The perceived impact, in terms of safety, financial, temporal, legal, or operational consequence, of the occurrence of the identified geotechnical hazard on the identified receptor(s) (defined as a rating in Table 6-2).

Risk(R): The perceived level of concern which should be assigned to the identified hazard, based on the probability of occurrence, and taking into due account the perceived severity of impact and consequence on the receptor(s) (defined as a risk number in Table 6-3).

The Geotechnical Risk Register and terminology adopted for this project is based on the guidance given in HD22/08. It is a semi-quantitative assessment based on engineering judgment. The classification of probability and severity used in this Geotechnical Risk Register is summarised in Table 6-1 and Table 6-2 respectively.

The risk classification is based on:

Risk = Probability Rating x Severity Rating

The explanation of risk number is given in Table 6-3, which gives the assessed risk level and appropriate actions.

Table 6-1 Assessment of probability

Rating	Likelihood (Class Descriptor)	Class Description
4	>85%	Highly likely
3	25 – 84%	Likely (regular occurrence)
2	2 – 25%	Fairly unlikely
1	<1%	Very unlikely to occur

Table 6-2 Assessment of consequence / severity

Rating	Class Descriptor	Construction Impact	Operation / Maintenance Impact	Environmental Impact
4	Disastrous	Construction unsustainable	Scheme operation unsustainable/deaths and serious injury	Major environmental incident – Threat to public health or safety
3	Substantial	Significantly increased construction costs and operational difficulty	Increased scheme costs Injury/Illness	Environmental incident triggers damage and/or nuisance prosecution and/or compensation

Contains *sensitive* information

Rating	Class Descriptor	Construction Impact	Operation / Maintenance Impact	Environmental Impact
2	Significant	Impact to costs/programme	Effect on scheme costs	Third party environmental impact requiring management response to recover
1	Marginal	Impact to programme	Small effect on scheme costs	Any environmental impact regarded as significant.

Table 6-3 Perceived risk level and mitigation action

Risk Number	Perceived Risk Level	Action Required
13 to 16	Intolerable	Eliminate Risk
9 to 12	Unacceptable (avoid)	Avoid or transfer risk
5 to 8	Unacceptable (manage)	Risk managed by prevention or mitigation
1 to 4	Negligible	Could be ignored

A summary of the potential geotechnical risks identified at this stage are detailed in Table 6-4. The Geotechnical Risk Register will be updated as the design of the scheme progresses to take into consideration any additional information obtained.

Table 6-4 Geotechnical risk register

Hazard	Consequence	Risk Rating			Recommended Mitigation Measures	Revised Risk Rating		
		P	I	R		P	I	R
Soft Ground	Excessive differential settlement between the existing and repaired pavement resulting in an uneven road surface. Additional cost and time required for additional works.	3	3	9	Removal of unsuitable material during construction phase where appropriate. Designer to specify appropriate construction materials to limit differential movements.	1	3	3

Hazard	Consequence	Risk Rating			Recommended Mitigation Measures	Revised Risk Rating		
		P	I	R		P	I	R
Unforeseen Ground Conditions	Re-design with delays to programme and increase in cost	3	3	9	Undertake site specific geotechnical and geoenvironmental ground investigation (GI) at the Site. Designer to review and confirm ground conditions assumed for preliminary design during the detailed design and construction phases. Contractor to advise on any temporary works GI required.	2	3	6
Migration of contamination within Made Ground into underlying strata	Creation of contamination pathways Contamination of confined groundwater in Chalk	3	3	9	Designer to consider piling methodology to avoid creation of contamination pathways	1	3	3
Existing Services (Relic buried services also likely to be present)	Damage to existing services during construction phase	4	3	12	Ensure privately owned utilities are traced and located prior to construction works Identify and relocate services crossing areas of proposed construction	2	3	6
Existing River Kenneth walls	Encountering anchors causing delay to construction	2	3	6	Consideration to be given to undertaking non-intrusive geophysical surveys to identify anchors	1	3	3
Buried obstructions/ foundations	Encountering buried obstructions including old foundations/piles below Gasholder No 4, causing design issues and delay to construction	2	3	6	Consideration to be given to undertaking non-intrusive geophysical surveys to identify shallow obstructions	1	3	3
Interface between Gasholder foundations and new structures	Crossing of the new structures over the existing Gasholder footprint foundations, ring wall and the adjacent natural ground. Bearing capacity issues and differential settlements	2	3	6	Consideration to be given to ground improvement and the use of engineered fill and modification or part demolition of the ring wall and Gasholder associated elements.	1	3	3

Hazard	Consequence	Risk Rating			Recommended Mitigation Measures	Revised Risk Rating		
		P	I	R		P	I	R
Perched groundwater	Presence of perched groundwater on top of the Alluvium/RTD within the granular Made Ground layers at the site causing flooding in temporary excavations	2	3	6	Monitoring of groundwater installations should be ongoing to inform long term groundwater model for the site	1	3	3
Unexploded Ordnance	Damage to construction, plant and injury/ death to site personnel or the public	2	2	12	Undertake an UXO risk assessment of the site prior to starting construction	1	3	3
Stability of Excavations	Collapsing excavations causing damage to plant or injury to site personnel	4	3	12	Design of temporary works to be undertaken by competent person	1	3	3
Asbestos	Risk to human health	3	3	9	Review of building management records for existing asbestos registers.	1	3	3

7. Conclusions and Recommendations

7.1. Conclusions

This Desk Study Report was prepared for the proposed development in order to highlight potential geotechnical and ground based risks associated with the development and to provide a strategy for addressing those identified. The potential geoenvironmental based risks are covered in a separate report by WSP.

Potential sources of contamination are largely linked to the site's historical redevelopment which may have impacted the Made Ground and the unknown ground gas regime. The previous ground investigations conducted at the adjacent gasworks site particularly Site Area "Demise 5" identified contaminants of concern which require further assessment. There is also the potential for further sources of contamination to be present within the inaccessible areas or additional unidentified underground structures.

Furthermore, in order to identify the risks posed by these potential contaminative sources to the site sensitive receptors, it is recommended that site-specific exploratory data is sought for interpretation and quantitative assessment, and to refine the potential risks identified within this report. In order to quantify the on-site ground conditions, it is recommended that a ground investigation be carried out to quantify the geotechnical properties of the key strata at the site in order to inform optimal and safe detailed design of the proposed foundations and contamination assessment. It could also delineate and explore potential historical substructures underlying the site.

A range of foundation and ground improvement solutions have been identified as possibly being viable at the site. The choice of final solution will be dependent on the ground conditions encountered during the ground investigation, the final proposed development design, and constraints related to construction such as protected tree zones, sensitive structures such as the Network Rail assets, River Kennet Canal Trust, site access and limits on vibrations, wall movements and ground movements set by third party asset owners.

7.2. Recommendations

It is recommended that a ground investigation be carried out at the Site, in order to:

- Establish the underlying ground conditions across the Site and geotechnical engineering design parameters for substructure (foundation) design, in particular nature of the Chalk grade with depth;
- Determine the presence and chemical composition of the Made Ground and underlying layers through targeted and representative soil sampling;
- Characterise the underlying groundwater and ground gas regimes through sampling and the development of a monitoring program;
- Quantify the risk of encountering subsurface obstructions and existing foundations;
- Quantify the risk of encountering subsurface utilities/services – live and abandoned;
- Assess the need for stability assessments of adjacent third-party structures;
- To provide a preliminary waste management assessment;
- Provide recommendations for any potential remedial work and verification reporting; and
- Prior to any ground intrusive works a UXO assessment of the site should be instructed to be undertaken and its recommendations adhered through to future works.

8. References

British Geological Survey (2017) *GeoIndex | Map Index to British Geological Survey (BGS) data* [online] available: <http://www.bgs.ac.uk/geoindex> [accessed November 2018].

BS 1377-1/9 *Methods of Test of Soils for Civil Engineering Purposes – Parts 1-9* (1990) BSI.

BS 8002:2015 *Code of Practice for Earth Retaining Structures* (2015) London: BSI Standards Limited.

BS EN 1997-1:2004 *Eurocode 7: Geotechnical Design – General Rules* (2004) BSI.

Building Research Establishment (2001) *BRE Special Digest 1, Concrete in Aggressive Ground*, BRE.

CIRIA C574 (2002) *Engineering in Chalk*, DTI

ICE. (2012) *ICE Manual of Geotechnical Engineering*, 1st ed., ICE.

Tomlinson, M.J. (2001) *Foundation Design and Construction*, 7th ed., Pearson.

Appendices

Appendix A. Drawings

Appendix B. Historical/Nearby Ground Investigation Exploratory Hole Logs

Appendix C. Zetica UXO Map

Appendix D. Pile Bearing Capacity with depth

Daniel Neto

Atkins

Euston Tower

286 Euston Road

London

NW1 3AT

Email

Daniel.Neto@atkinsglobal.com

Telephone

+44 2071 212000

© Atkins Ltd except where stated otherwise.

The Atkins logo, 'Carbon Critical Design' and the strapline
'Plan Design Enable' are trademarks of Atkins Ltd.