APPENDIX 9.3
GROUND INVESTIGATION REPORT

Ground Investigation Report

Beehive Retail Centre; Cambridge Retail Park; and 230 Newmarket Road, Cambridge

February 2023

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Quality Assurance – Approval Status

This document has been prepared and checked in accordance with
Waterman Group's IMS (BS EN ISO 9001: 2015, BS EN ISO 14001: 2015 and BS EN ISO 45001:2018)

Issue	Date	Prepared by	Checked by	Approved by
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Comments

01	Issue for comments.

Comments

Disclaimer

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1. Introduction

1.1 Background

Gardiner and Theobald LLP has instructed Waterman Infrastructure & Environment Limited (“Waterman”) to prepare a Ground Investigation Report (GIR) for the proposed development of Beehive Retail Park, Cambridge (hereafter termed “the Site”).

The GIR will present all available geotechnical information including geological features for the entire Site, and a geotechnical evaluation of the information, stating the assumptions made in the interpretation of the test results.

1.2 Site Location and Description

The site is located in Cambridge (Figure 1) and centred approximately at CB5 8WR. The site is irregular in shape, generally flat, has a total area of 13.86 ha and comprises of three sub-areas: the Beehive Retail Centre, 230 Newmarket Road (230 NMR) and the Cambridge Retail Park (CRP).

The site area is surrounded by the following features/land uses:

- North - Retail and industrial areas including car dealership and metal recycling plant.
- East - Commercial units, Railway lines and residential housing.
- South - Residential housing.
- West - Residential housing, retail units and the A1134.

Beehive Retail Centre

This section of the Site is centred at National Grid Reference 546677 258593. The area is 7.22ha and comprises thirteen retail units with associated car parking.

230 NMR

This section of the Site is centred at National Grid Reference 546821 259139. It spans 0.45ha and is currently occupied by car parking areas relating to the wider retail park.

CRP

This area is centred at National Grid Reference 546791 258964. It occupies 6.19ha and is currently commercial retail units, car parking areas and service yards. Soft landscaping borders are present in the north, west and south of the Site.

A plan detailing the three individual Sites which form the Cambridge Development masterplan is included in Appendix A.

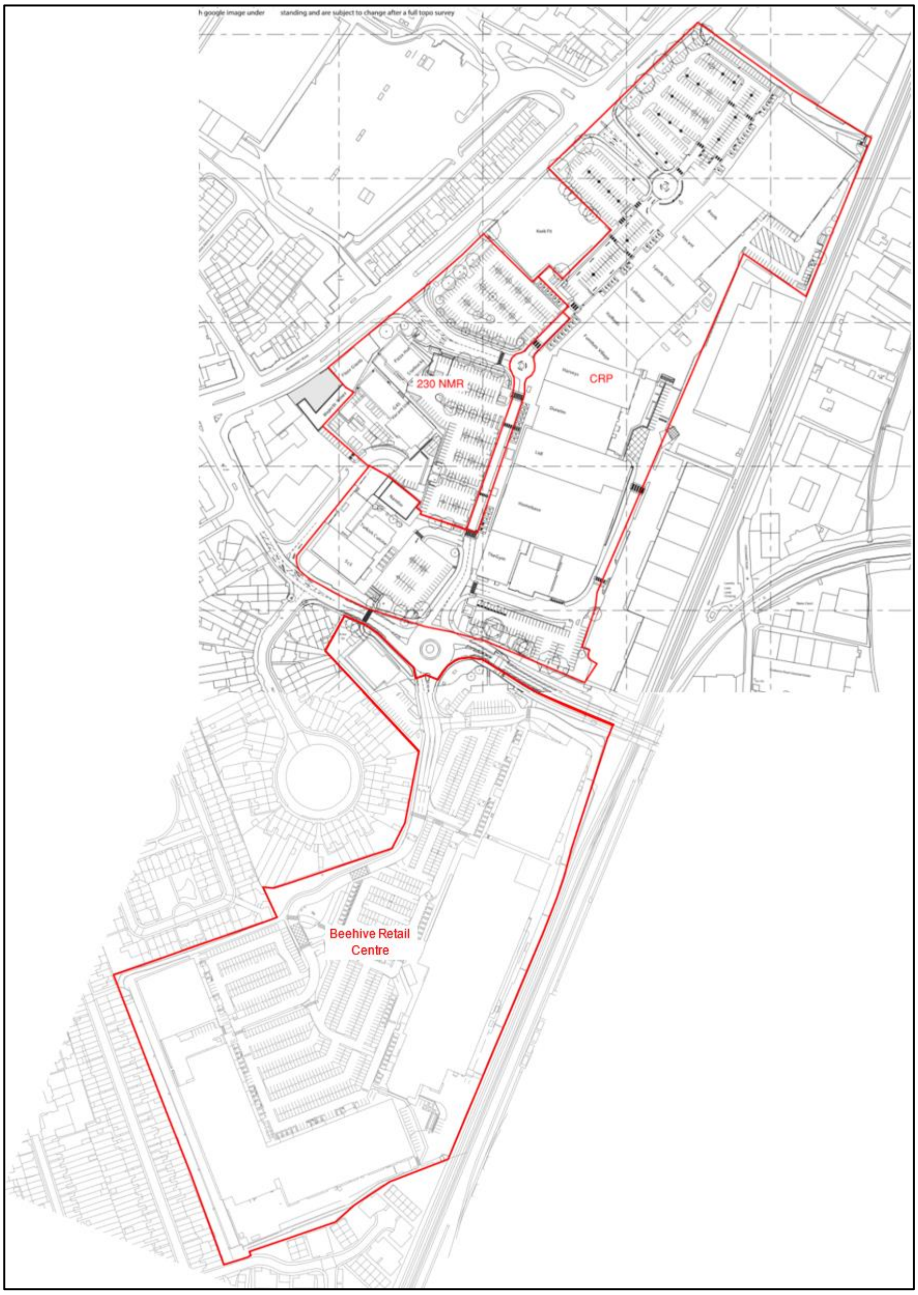


Figure 1: Combined site plan and location

1.3 Proposed Development

The proposed Development's for the three individual Sites are included below. Note the Cambridge Masterplan Development is at an early stage and changes may occur to the proposed Development as part of the design process.

Beehive Retail Centre

Fourteen building plots, with buildings 1 to 9 storeys for commercial use. Soft and hard communal landscaping is proposed with Sustainable Urban Drainage Systems (SUDs) incorporated into the development scheme. Basements and private soft landscaping are not proposed.

230 NMR

Four-storey commercial building in the southern half and a separate single-storey retail unit in the northern half. Soft communal landscaping is proposed with existing trees to be retained on the western boundary. Associated car parking areas are proposed to the northeast and eastern portion of the Site. Basements and private soft landscaping are not proposed.

CRP

The existing retail units on CRP will be retained and used for the relocation of the occupants of the Beehive Retail Centre. The existing Currys located on the northern boundary of CRP will be retained and extended westward into the existing service yard to form an additional warehouse. It is understood Currys will be occupied by Asda. The proposed Development for the small section of land adjacent Henley Road (southern boundary of CRP) is not known at this time.

1.4 Geotechnical Category

The proposed scheme comprises conventional types of geotechnical structures, earthworks and activities, with no exceptional geotechnical risks, unusual or difficult soil or loading conditions. In accordance with Clause 2.1 of BS EN 1997-1:2004, the scheme is therefore considered to be **Geotechnical Category 2**.

Partially, due to presence of thick layer of landfill material, an area of the site might fall under Geotechnical Category 3. However, this will need further assessment based on landfill material findings and detailed proposed design scheme.

The design should therefore include quantitative geotechnical data and analysis to ensure fundamental requirements are satisfied. Routine procedures for field and laboratory testing may be used for design.

1.5 Scope and Objectives

The scope and objectives of the current Ground Investigation is summarised as the following:

- Interpretation of available GI information for the entire Site;
- Production of ground models and characteristic soil parameters for the geotechnical design works of the proposed structures;
- Preliminary foundation design recommendations including bearing capacity for shallow foundations and pile resistance calculations;
- Production of geotechnical risk register.

1.6 Design Standards

The current Ground Investigation Report has been undertaken in general accordance with the following Design Standards and Specifications:

- BS EN 1997 - 1:2004 (+A1:2013) ‘Eurocode 7: Geotechnical Design – Part 1 General Rules;
- BS EN 1997 - 2:2004 ‘Eurocode 7: Geotechnical Design – Part 2 Ground investigation and testing;
- BS 8004:2015 (+A1:2020) Code of practice for foundations;
- BS 8002:2015 Code of practice for earth retaining structures;
- BS 6031 2009 Code of practice for earthworks;

- BS 8500 2006 Concrete - Complementary British Standard to BS EN 206-1. Method of specifying and guidance for the specifier (+A1:2012) (incorporating corrigendum No. 1);
- CIRIA C760, Guidance on Embedded Retaining Wall Design;
- DMRB CD 622 - Managing geotechnical risk, March 2020;
- UK National Annex to Eurocode 7: Geotechnical design – Part 1: General Rules (NA+A1:2014 to BS EN 1997 - 1:2004+A:2013); and
- The Construction (Design & Management) Regulations 2015.

1.7 Limitations and Constraints

The information contained in this report is based on existing ground investigation data comprising exploratory hole records, laboratory test results, and groundwater monitoring as well as on historical, geological, and hydrogeological sources and consultation with the regulatory authorities.

Waterman has endeavoured to assess all information provided to them but makes no guarantees or warranties as to the accuracy or completeness of this information.

The benefit of this report is made to Gardiner and Theobald LLP.

The conclusions resulting from this study are not necessarily indicative of future conditions or operating practices at or adjacent to the Site.

2. Existing Information

2.1 Site History

Beehive Retail Centre

Historical mapping records the Beehive Retail Centre site undeveloped or in use as allotments up until the 1960's whereby various warehouses, a dairy, builders' yards, and a bakery are on-site. The northern half was redeveloped initially in the 1980's into the existing Beehive Retail Centre layout. The remainder of the site followed by 1994. A petrol filling station was constructed on the western boundary as part of the initial Beehive Retail Centre before being decommissioned by 2003.

230 NMR

Historical mapping records the north/northeast of 230 NMR occupied by a clay pit that extended northwards off-site from at least the 1880's. By the late-1920's the clay pit expanded beneath the south/southeast portion of the site and joined the Brick and Tile works clay pit to the north. Between the 1950's and 1970's the pit was infilled, the infilled material was not recorded. From the mid-1970's garages and warehouses were constructed on the western boundary with large industrial units associated with Coral Park Trading Estate constructed across the remainder of the Site. No significant changes until 2010 when the warehouses were demolished and redeveloped into the existing retail centre layout.

CRP

Historical mapping records the CRP as predominately being occupied by a clay pit and associated brisk and tile works (northern and central portions) in 1886, the southern portion was occupied by a coal yard. By 1927 the clay pit and associated brick and tile works infrastructure to cover the whole site displacing the coal yard historically present on the southern portion. The 1950 historical maps recorded a tyre depot, garage, and warehouse in the south western corner. Between the 1950's and 1970's the clay pit was infilled and warehouses and depot constructed known as the Coral Park Trading Estate. The Coral Park Trading Estate was redeveloped post 2010 to form the Cambridge Retail Park which has maintained its current layout up to the present day.

2.2 Geology

The Site's geology as established from, British Geological Survey (BGS) mapping and boreholes is summarised in Table 1.

Table 1: Site Geology

Stratum	Area Covered	Typical Thickness (m)	Description
Made Ground	230 NMR, CRP	0.5 – 2.5	Brown and dark grey clayey sand and gravel. Coarse fragments of limestone, brick, tile, glass, flint, steel, wood, ash, and concrete, with pockets of soft grey or brown slightly sandy clay.
	Beehive Retail Centre	1.0 – 2.0	Granular material generated during the multiple redevelopment phases. Reworked natural material becoming present with depth.
Landfill Material	230 NMR, CRP	5.0 – 15.0 (230 NMR) 5.0 – 23.7 (CRP)	Wet black loose fill comprising brick, concrete and ash with wood, nails, plastic,

			metal, pottery, electrical components and bands of stiff grey silty clay.
River Terrace Deposits	South western corner of Beehive Retail Centre	3.0 – 5.0	Brown/orange brown slightly clayey/clayey sand/gravel.
West Melbury Chalk Formation	230 NMR, CRP, Beehive Retail Centre	3.0 – 5.0	Grey marly structureless chalk.
Gault Formation	230 NMR, CRP, Beehive Retail Centre	30m	Grey silty clay.
Lower Greensand Formation	230 NMR, CRP, Beehive Retail Centre	>20m	Light brown/grey interbedded sands and sandstone

The Site's historical use identifies two primary redevelopment stages, construction of commercial/industrial uses in the early 20th century before their demolition and construction of the existing Beehive Retail Centre. Made Ground associated with the construction of these redevelopment stages is anticipated. The Made Ground is likely to be granular in nature, becoming a reworked natural deposit with depth.

BGS maps identify the Site as being underlain by a thin River Terrace Deposit in the southwest corner, consistent with the former location of a gravel pit immediately off-site to the west. The extent to which the gravel deposit extends on-site is unclear. The determining of this will form an output of a proposed ground investigation.

Beyond the southwest Site corner, BGS mapping indicates the remainder of the Site as being underlain by the West Mulbury Chalk Formation, underlain in turn by the Gault Formation. The Gault Formation outcrops in the area surrounding the Site including land immediately north which was a former clay pit, and a former brick and tile works. The Chalk Formation on-site is expected to be thin and may in some places be absent. A ground investigation objective will be to confirm the presence and thickness of Chalk Formation on-site.

2.3 Ground Stability

The Groundsure Report has identified the following geological hazards (Table 2).

Table 2: Geological Hazard Risk to the Site

Geological Hazard	Risk
Shrink Swell Clay	Moderate
Landslides	Very Low
Ground Dissolution of Soluble Rocks	Very Low
Compressible Deposits	Negligible
Running Sand	Very Low

The BGS map does not reveal any structural, geomorphological, or geochemical features on or near to the Site.

The Site is not in an area that could be affected by coal mining activity.

2.4 Hydrogeology

2.4.1 Surface Waters

Surface waters close to the Site include:

- Cherry Hinton Brook – 350m northeast;
- Coldham Brook – 530m northeast; and
- River Cam – 530m northwest.

Surface water abstractions are absent in the surrounding area.

Given the distance to these surface water receptors and intervening potentially contaminated land uses any impact on these receptors is unlikely to be attributable to contaminants originating on-site. In addition, given the distance potential for significant attenuation in the environment prior to impacting these receptors is expected. For further information please refer to WIE17469-100-R-12-1-1-GQRA report.

2.4.2 Groundwater

The EA has classified the geological deposits on-site as having the following classification (Table 3).

Table 3: Summary of Hydrogeological Properties of the Main Geological Strata

Stratum	EA Classification	Hydrogeological Significance
Made Ground	Unproductive Strata	Contains insignificant quantities of vertically or laterally extensive groundwater
River Terrace Deposits	Secondary A Aquifer	May be important in supporting local abstractions or in providing baseflow to rivers and streams
West Melbury Chalk Formation	Principal Aquifer	Regionally important aquifer, likely to be used to support potable abstractions
Gault Formation	Unproductive Strata	Contains insignificant quantities of vertically or laterally extensive groundwater
Lower Greensand Formation	Principal Aquifer	Regionally important aquifer, likely to be used to support potable abstractions

Where the River Terrace Deposits overlie the West Melbury Chalk Formation groundwater between the two deposits are expected to be in hydraulic continuity.

The Gault Clay Formation is an aquiclude and will restrict the vertical migration of contaminants to the underlying Lower Greensand Formation. Piled foundations which penetrate the Gault Clay Formation will require completion of a Foundation Works Risk Assessment (FWRA) to assess the possible risk of preferential pathway creation through completion of piles. Where the Gault Clay Formation directly underlies the Made Ground, with both River Terrace Deposits and West Melbury Chalk Formation absent, the lateral migration of contaminants onto and off-site will be restricted.

The Site is not in a groundwater Source Protection Zone (I, II, III). Active groundwater abstractions are absent in the surrounding area.

Groundwater flow within the River Terrace Deposits and West Melbury Chalk Formation which overlie the Gault Clay Formation are likely to be influenced by the surface water receptors in the surrounding area, given the absence of groundwater abstractions. Surface water receptors include the smaller Cherry Hinton Brook and Coldham Brook (northeast) and the larger River Cam (west). The larger River Cam is expected to be the dominant influence on groundwater flow direction on-site and in the surrounding area, with a lower effect from the smaller Cherry Hinton Brook and Coldham Brook. Groundwater flow in both the River Terrace Deposits and West Mulbery Chalk Formation is anticipated to be north/northwest.

This assessment is preliminary only and completion of a ground investigation which assess groundwater flow in deposits above the Gault Clay Formation is required.

For further information please refer to WIE17469-100-R-12-1-1-GQRA report.

3. Fieldwork

Information presented in this section are related to the ground investigation between 31st October and 2nd December 2022, undertaken by Groundtech as described in Section 3.1.

3.1 Scope of Fieldwork

In summary, the scope of works undertaken within the Site boundary include the following:

- 6 No. Cable Percussive Boreholes (WBH111-WBH116) to a depth of 40.0m bgl using a Dando Cable Percussive Rig.
- 2 No. Cable Percussive Boreholes (WBH101 & WBH102) to a depth of 30.0m bgl using a Dando Cable Percussive Rig.
- 8 No. Cable Percussive Boreholes (WBH103 – WBH110) to a depth of 25.0m bgl using a Dando Cable Percussive Rig.
- Rotary coring from 30.0mbgl to 32.0mbgl at WBH111 using a Commacchio 305.
- WBH103A and WBH103B were advanced to depths between 0.25m and 0.27m bgl where they were terminated on a concrete obstruction.
- Coring of the concrete hardsurfacing at 3 No. locations (WBH103, WBH111 & WBH116);
- In-situ testing and sampling in all boreholes;
- Photo Ionisation Detector (PID) screening of all environmental samples.
- Radiation Screening of all samples and arisings from within the historical landfill.
- Groundwater sampling carried out using low flow sampling techniques.
- Gas monitoring, including cannister sampling in selected locations;

3.2 Deep Boreholes

The deep boreholes were drilled to depths between 25.0m and 40.0m using cable percussive method as indicated in Table 4. The only exception was for WBH111, where rotary coring was used from 30.0mbgl to 32.0mbgl.

Table 4: Details of Deep Boreholes

Borehole ID	Depth (m)
WBH101	30.45
WBH102	30.45
WBH103	25
WBH104	25
WBH105	25
WBH106	25
WBH107	25
WBH108	25
WBH109	25
WBH110	25
WBH111	40
WBH112	40

Borehole ID	Depth (m)
WBH113	40
WBH114	40
WBH115	40
WBH116	40

3.3 Sampling

3.3.1 Geotechnical Sampling

SPT testing was undertaken between 1.0m and 2.0m intervals. SPT testing and UT100 samples were taken on alternate 1.50m intervals below 5.00m within suitable strata; and small disturbed and bulk geotechnical samples were obtained between every SPT/UT100 interval.

During the intrusive investigation, representative samples were taken at regular intervals, changes of strata and where evidence of contamination existed. Laboratory analysis was scheduled on the samples obtained.

Disturbed samples of soil for chemical analysis were placed in the correct sampling containers as required by the laboratory in accordance with their MCERTS and UKAS Accreditation. Transportation was arranged in a timely manner and the samples were at the correct temperature.

3.4 Instrumentation and Monitoring

3.4.1 Groundwater Monitoring

Dual gas and groundwater monitoring installations were constructed in the boreholes. The standpipes consisted of high-density polyethylene (HDPE) pipe - a bentonite seal was placed around the plain pipe and a clean gravel pack was placed around the slotted pipe.

All exploratory hole location plan of the existing monitoring installations is presented in Appendix E.

4. Laboratory Testing

4.1 Geotechnical Testing

A summary of the geotechnical testing carried out is shown in Table 5.

The results of the testing are included within the Factual Reports in Appendix E.

Table 5: Geotechnical Testing Summary

Test Description	Total Number of Tests
Moisture Content	14
Plasticity Index Analysis	40
Undrained Shear Strength in Triaxial Compression	43
BRE Suite D Brownfield (Pyrite present)	30
California Bearing Ratio Test	5
Consolidated Drained Shearbox Test	16
Particle Size Distribution	44
Particle Size Distribution Test Pipette Analysis	24
Unconfined Compressive Strength	4
Point Load	6
One Dimensional Consolidation Test	5
Quick Undrained Shear Box Test	2
Organic Content	2
Permeability in a Triaxial Cell	2
2.5kg Compaction Test	2

4.2 Chemical Testing

A series of chemical laboratory tests were undertaken on samples obtained from the exploratory hole locations, surface water monitoring points, and gas and groundwater samples taken from the monitoring installations.

The results of the testing are included within the Factual Reports in Appendix E.

5. Ground Summary

This section provides a summary of the results of the different testing carried out as part of the GI works as discussed in the previous sections of this report. The results will be split into two designated areas; one applicable to 230NMR and CPR areas of the site and one for Beehive Retail Centre.

5.1 Geological Formations Encountered

Reference should be made to the exploratory hole records in Appendix E for full details of the strata encountered during the investigations. However, a summary of the strata encountered and their depth ranges (maximum and minimum depths) are presented in Table 6 for 230NMR and CPR area of the site and in Table 7 for the Beehive Retail Centre.

This site investigation has generally confirmed ground conditions encountered in previous investigation works and published geological maps.

Table 6: Summary of Geological Formation Encountered at 230NMR and CPR site

Stratum	Minimum depth encountered (m bgl)	Maximum depth encountered (m bgl)	Description
Made Ground	0.00	4.6	Brown gravelly occasionally clayey sand. Brown sandy clayey gravel. Soft to firm sandy gravelly clay. Reworked structureless chalk composed of firm light grey gravelly cobbly clay.
Landfill encountered only in WBH104, WBH105, and WBH108	0.00	15.9	Soft to firm gravelly ashy clay with a high cobble content. Minor constituents observed included wood, brick, glass, concrete, tiles, plastic, metal including copper, fabric and leather, bones, pottery, rubber and glass bottles. A moderate organic and hydrocarbon odour. Black clayey gravel with minor constituents included wire, copper, wood, metal, cloth, plastic, brick, tiles, glass, string, fabric, rubber and carpet. A solvent and hydrocarbon odour.
West Melbury Chalk Formation	1.40	14.0	Structureless chalk composed of soft to firm cream mottled yellow to brown silty gravelly, occasionally cobbly clay with weak low-density clasts.
Gault Formation	2.5	n/A	Stiff to very stiff high strength to very high strength grey silty clay.

Table 7: Summary of Geological Formation Encountered at Beehive Retail Centre site

Stratum	Minimum depth encountered (m bgl)	Maximum depth encountered (m bgl)	Description
Made Ground	0.00	2.5	Brown gravelly occasionally clayey fine to coarse sand with minor constituents of brick, sandstone, concrete, tarmac, quartzite and chert. Brown sandy gravel of mixed lithologies including limestone.

Stratum	Minimum depth encountered (m bgl)	Maximum depth encountered (m bgl)	Description
			Firm brown sandy gravelly clay. Reworked structureless chalk composed of light grey firm gravelly cobbly clay.
River Terrace Deposits	0.85	5.5	Light brown gravelly medium to coarse sand
West Melbury Chalk Formation	0.35	7.0	Structureless chalk composed of soft to firm cream mottled yellow to brown silty gravelly, occasionally cobbly clay with weak low-density clasts. Structureless Chalk composed of light grey clayey gravelly cobbles with weak low-density clasts (Grade Dc)
Gault Formation	4.2	n/A	Stiff to very stiff high strength to very high strength grey silty clay.

5.2 Groundwater Monitoring Results

A summary of exploratory hole groundwater strikes can be found in Table 8.

Table 8: Groundwater Summary

Stratum	Minimum strike depth		Maximum strike depth	
	(m bgl)	(mOD)	(m bgl)	(mOD)
Made Ground	1.5	7.43	3.5	6.17
West Melbury Chalk Formation	3.5	8.52	4.5	4.97
Gault Formation	4.0	6.99	4.0	6.99

Two rounds of groundwater level monitoring have been completed across the site. The groundwater level monitoring results and well details are included in Table 9. Reference should be made to the exploratory hole records and monitoring data in Appendix E for details of the groundwater encountered during the ground investigations.

Table 9: Groundwater Levels

Area	Exploratory Hole	Target Strata	Groundwater Level (mbgl)		Groundwater Level (mOD)	
			Round 1	Round 2	Round 1	Round 2
230 NMR	WBH101	Made Ground and West Melbury Formation	2.49	3.2	8.9	5.7
	WBH102	West Melbury Formation	2.5	2.25	7.9	5.7
	WBH103	Made Ground	Dry	Dry	Dry	Dry
	WBH104 (shallow)	Made Ground / Infill	Dry	N/A	Dry	N/A

Area	Exploratory Hole	Target Strata	Groundwater Level (mbgl)		Groundwater Level (mOD)	
			Round 1	Round 2	Round 1	Round 2
	WBH104 (deep)	Made Ground / Infill	2.01	1.89	7.6	5.7
	WBH105 (shallow)	Made Ground / Infill	1.92	2.14	7.8	5.6
	WBH105 (deep)	Made Ground / Infill	2.2	2.17	7.5	5.3
	WBH106	Made Ground and West Melbury Formation	Dry	Dry	Dry	Dry
CRP	WBH107	West Melbury Formation	1.13	1.67	8.4	6.8
	WBH108 (shallow)	Made Ground / Infill	Dry	Dry	Dry	Dry
	WBH108 (deep)	Made Ground / Infill	1.51	1.48	7.4	5.9
	WBH109	Made Ground, West Melbury Formation and Gault Clay Formation	2.49	2.67	6.8	4.2
	WBH110	Made Ground and West Melbury Formation	Dry	Dry	Dry	Dry
Beehive Retail Centre	WBH111	West Melbury Formation	0.39	0.97	11.3	10.3
	WBH112	West Melbury Formation	3.16	2.95	8.5	5.6
	WBH113	River Terrace Deposits and Gault Clay Formation	4.09	3.99	8.7	4.7
	WBH114 (shallow)	Made Ground	Dry	Dry	Dry	Dry
	WBH114 (deep)	West Melbury Formation	3.16	3.54	8.9	5.3
	WBH115	West Melbury Formation	2.13	3.54	7.8	4.3
	WBH116	Made Ground and West Melbury Formation	1.93	1.95	7.5	5.6

5.3 Ground Conditions and Geotechnical Properties

A summary of the in-situ and laboratory testing carried out as part of the ground investigations is presented next. Characteristic geotechnical parameters for the different ground deposits encountered have been selected in accordance with BS-EN1997 - Eurocode 7.

5.3.1 230 NMR and CRP geotechnical properties

This section presents a summary of the properties encountered for each stratum as well as their ranges at the 230 NMR and CRP area of the site based on which the ground model will be selected.

Table 10: Summary of Made Ground Properties

Parameter	Range Value	Average Value	Soil Conditions	Characteristic Value
SPT N value	0 – 14*	10	Loose to Medium Density	8
Particle Size Distribution (%)	Very Coarse = 0 - 51 Coarse = 5 – 54 Sand = 5 - 12 Fines = 17 - 85	-	Silty, clayey sand	-
Moisture Content (%)	18 - 33	25	-	25
CBR (%)	0.5 – 19.7	9.4	Competent soil	9
Optimum MC (Compaction test)	16	16	-	16
BRE - SD1 SO_4 - H_2O, TPS and pH (mg/L, %SO4, -)	109 - 1244 0.12 – 0.48 6.9 – 11.9	577 0.35 9.9	DS-2, AC-2	1244 0.48 6.9

*a value of SPT N 46 was recovered due to presence of concrete obstructions.

Table 11: Summary of Landfill Properties

Parameter	Range Value	Average Value	Soil Conditions	Characteristic Value
SPT N value	0 – 13*	4.5	Very Soft to Soft	4
Particle Size Distribution (%)	Very Coarse = 0 - 59 Coarse = 7 – 65 Sand = 4 - 22 Fines = 3 - 84	-	-	-
Moisture Content (%)	16	16	-	16
CBR (%)	15.9 – 21.9	18.7	-	15.9
Peak Shear Strength c' and φ' (kPa, º)	15 – 25 21 - 35	-	-	2 26
BRE - SD1 SO_4 - H_2O, TPS and pH (mg/L, %SO4, -)	523 - 2141 0.69 – 1.59 7.5 - 8.1	1092 0.98 7.7	DS-4, AC-4	2141 1.59 7.5

*a value of SPT N 50 was recovered due to presence of concrete obstructions.

Table 12: Summary of West Melbury Chalk Formation Properties

Parameter	Range Value	Average Value	Soil Conditions	Characteristic Value
SPT N value	6 – 28	13	Weak to Medium	11
Moisture Content (%)	20 - 33	27	-	-

Parameter	Range Value	Average Value	Soil Conditions	Characteristic Value
Atterberg Limits (%)	LL = 39 – 88 PL = 21 - 32 IP = 17 - 56	52 24 28	Clay with Intermediate to Very High Plasticity	-
Particle Size Distribution (%)	Very Coarse = 0 Coarse = 0 – 7 Sand = 1 - 19 Fines = 77 - 99	-	-	-
SPT correlation Undrained Shear Strength (kPa)	27 – 126	57	-	50
UUT Undrained Shear Strength (kPa)	48 – 146	87		
SPT correlation Shear Strength φ' (º)	28 - 35	30	-	10 30
Peak Shear Strength c' and φ' (kPa, º)	11 - 15 26 - 33	13 29.5		
USC (MPa)	0.25 – 2.49	1.07	-	1.0
BRE - SD1 SO_4 - H_2O, TPS and pH (mg/L, %SO4, -)	110 0.09 9.8	110 0.09 9.8	DS-1, AC-1s	110 0.09 9.8

Table 13: Summary of Gault Formation Properties

Parameter	Range Value	Average Value	Soil Conditions	Characteristic Value
SPT N value	10 – 53	30	Medium to Stiff SPT Increasing with Depth	12+1.5z
Moisture Content (%)	23 – 39	27	-	-
Atterberg Limits (%)	LL = 55 – 84 PL = 25 - 33 IP = 30 - 53	72 30 43	Clay with High to Very High Plasticity	-
Particle Size Distribution (%)	Very Coarse = 0 Coarse = 0 – 2 Sand = 1 - 8 Fines = 90 - 99	-	-	-
SPT correlation Undrained Shear Strength	45 – 237	135	Undrained Shear Strength Increasing with Depth	50 + 7z

Parameter	Range Value	Average Value	Soil Conditions	Characteristic Value
(kPa)				
UUT Undrained Shear Strength (kPa)	80 - 261	160		
Peak Shear Strength c' and φ' (kPa, °)	1 – 28 15 - 23	12 19	-	5 22
1D Consolidation mv and cv (m2/MN, m2/yr)	0.011 – 1.302 0.095 – 32.612	0.418 8.029	-	0.40 8.00
Permeability k (ms^{-1})	3.3^{-11} – 8.9^{-11}	6.1^{-11}		
BRE - SD1 SO_4 - H_2O, TPS and pH (mg/L, %SO4, -)	230 - 510 0.48 – 1.20 8.0 – 9.0	320 0.83 8.2	DS-3, AC-2s	510 1.20 8.0

5.3.2 Beehive Retail Centre

This section presents a summary of the properties encountered for each stratum as well as their ranges at the Beehive Retail Centre area of the site based on which the ground model will be selected.

Table 14: Summary of Made Ground Properties

Parameter	Range Value	Average Value	Soil Conditions	Characteristic Value
SPT N value	4	4	Loose Density	4
Particle Size Distribution (%)	Very Coarse = 0 - 0 Coarse = 4 – 62 Sand = 27 - 40 Fines = 11 - 67	-	Silty, sandy, clayey gravel	-
Moisture Content (%)	20	20	-	20
CBR (%)	139.9 – 160.1	150	Recovered as sandy gravely	150
SPT correlation Shear Strength φ' (°)	28.1	28.1	-	28
BRE - SD1* SO_4 - H_2O, TPS and pH (mg/L, %SO4, -)	109 - 1244 0.12 – 0.48 6.9 – 11.9	577 0.35 9.9	DS-2, AC-2	1244 0.48 6.9

*due to lack of data results from 230 NMR and CRP area of the site are presented

Table 15: Summary of River Terrace Deposits Properties

Parameter	Range Value	Average Value	Soil Conditions	Characteristic Value
SPT N value	3 – 12	6	Loose to Medium Density	6
Particle Size Distribution (%)	Very Coarse = 0 Coarse = 37 Sand = 52 Fines = 11	-	Gravelly sand	-
Moisture Content (%)	14	14	-	14
SPT correlation Shear Strength φ’ (º)	27.7 – 30.7	28.7	-	2 30
Peak Shear Strength c’ and φ’ (kPa, º)	13 36	13 36		

Table 16: Summary of West Melbury Chalk Formation Properties

Parameter	Range Value	Average Value	Soil Conditions	Characteristic Value
SPT N value	1 – 29	11	Weak to Medium	11
Moisture Content (%)	22 - 26	24.5	-	24
Atterberg Limits (%)	LL = 41 – 60 PL = 22- 26 IP = 19 - 34	48 23 25	Clay with Intermediate to High Plasticity	-
Particle Size Distribution (%)	Very Coarse = 0 Coarse = 0 – 2 Sand = 9 - 21 Fines = 77 - 90	-	Sandy clay chalk	-
CBR (%)	11.1 – 12.7	11.9	Competent soil	11
SPT correlation Undrained Shear Strength (kPa)	5 – 131	52	-	50
UUT Undrained Shear Strength (kPa)	23 – 107	70		
SPT correlation Shear Strength φ’ (º)	27 – 35.7	30.5	-	10 30
1D Consolidation mv and cv (m2/MN, m2/yr)	0.022 – 0.089 2.883 – 21.667	0.055 12.275	-	0.1 12.00
BRE - SD1	45 - 545	227	DS-2, AC-1s	545

Parameter	Range Value	Average Value	Soil Conditions	Characteristic Value
SO_4 - H_2O, TPS and pH (mg/L, %SO4, -)	0.06– 0.12 8.0 – 8.4	0.09 8.1		0.12 8.0

Table 17: Summary of Gault Formation Properties

Parameter	Range Value	Average Value	Soil Conditions	Characteristic Value
SPT N value	9 – 78	39	Medium to Stiff SPT Increasing with Depth	12+1.5z
Moisture Content (%)	26 – 33	29	-	-
Atterberg Limits (%)	LL = 60 – 89 PL = 26 - 32 IP = 34 - 57	77 30 46	Clay with High to Very High Plasticity	-
Particle Size Distribution (%)	Very Coarse = 0 Coarse = 0 – 1 Sand = 0 - 9 Fines = 90 - 100	-	Sandy silty clay	-
SPT correlation Undrained Shear Strength (kPa)	41 – 355	175	Undrained Shear Strength Increasing with Depth	50 + 7z
UUT Undrained Shear Strength (kPa)	77- 316	169		
USC (MPa)	1.37 – 4.40	2.89	-	2.5
1D Consolidation mv and cv (m2/MN, m2/yr)	0.062 – 0.674 0.214 – 4.264	0.368 2.239	-	0.40 2.20
BRE - SD1 SO_4 - H_2O, TPS and pH (mg/L, %SO4, -)	21 - 1231 0.87– 0.99 8.0 – 8.3	406 0.92 8.2	DS-3, AC-2s	1231 0.99 8.0

6. Ground Model and Geotechnical Parameters

The ground models and geotechnical parameters on which the geotechnical design should be based are presented in this section and have been based on ground investigations as described in the previous sections of this report.

6.1 Geology and Stratification Models

3No. ground models are proposed in Table 18 according to the location of specific areas across the Site as shown in Figure 2. Characteristic geotechnical parameters for the different ground deposits encountered have been selected in accordance with BS-EN1997 - Eurocode 7.

The GI interpretation charts for different testing can be found in Appendix B for all proposed ground models. Reference should be made to the Final Factual Reports detailed in the previous sections and the associated GI data, details of the in-situ and laboratory testing, including data from more specialist testing that are not included within the summary tables and plots.

The GI indicated that within the area of the proposed development, the ground is comprised of Made Ground overlaying the solid geological formation comprised of West Melbury Chalk Formation and Gault Formation.

In the middle of the upper part of the whole site Landfill material was encountered to a maximum depth of 15.9mbgl. Thus, a special consideration is needed for the geotechnical design of the specific area.

River Terrace Deposits layer was found to be present between the Made Ground and West Melbury Chalk Formation layers, only in the south-west corner of the Beehive Retail Centre area of the site.

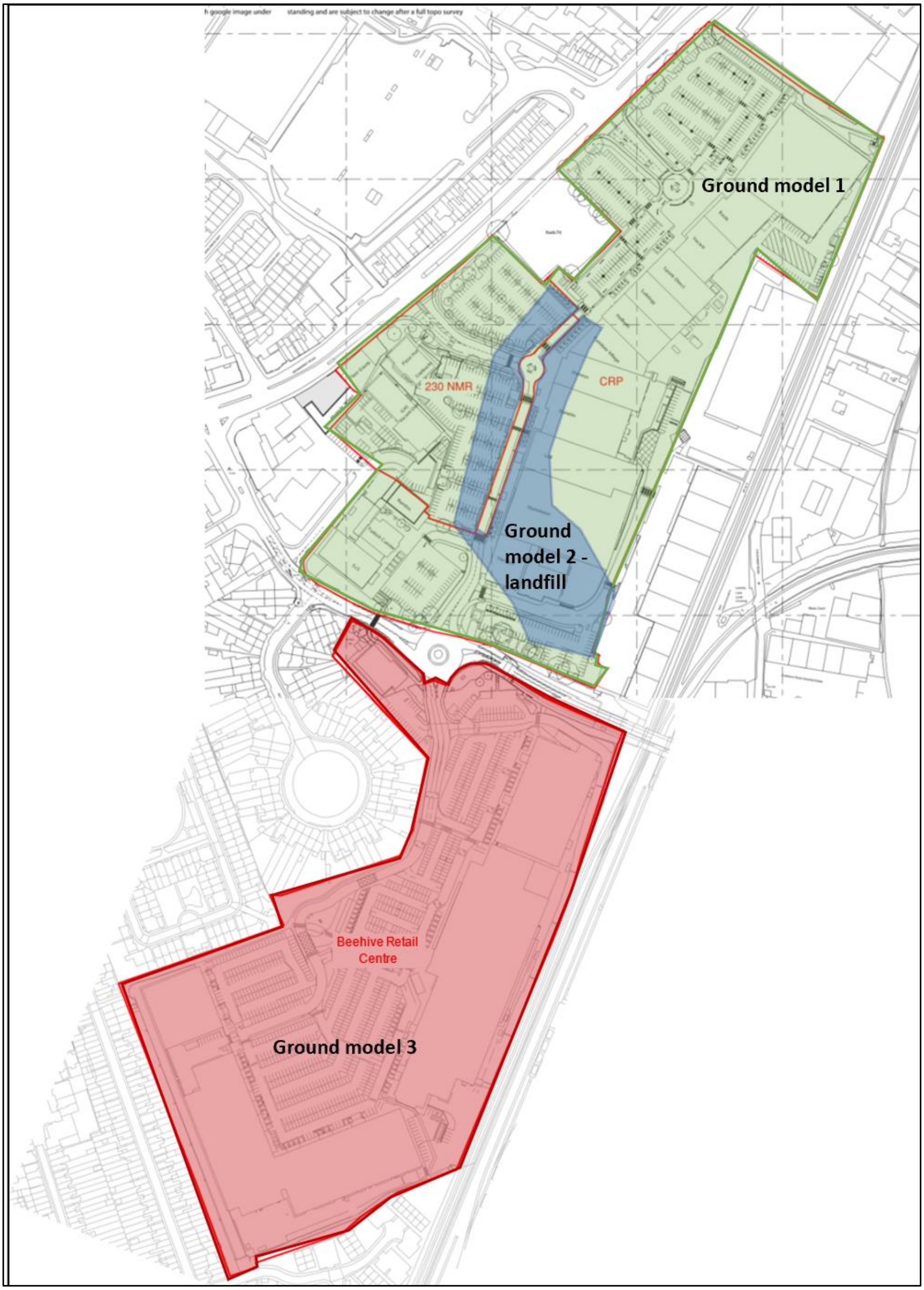


Figure 2: Proposed areas of ground models

Table 18: Proposed ground models

Stratum	Ground model 1		Ground model 2		Ground model 3	
	Top of Strata (m bgl)	Top of Strata (m AOD)	Top of Strata (m bgl)	Top of Strata (m AOD)	Top of Strata (m bgl)	Top of Strata (m AOD)
Made Ground	0.0	10.0	0.0	9.5	0.0	11.0
Landfill	-	-	1.5	8.0	-	-
River Terrace Deposits	-	-	-		0.9	10.1
West Melbury Chalk Formation	3.0	7.0	-		2.0	9.0
Gault Formation	5.0	5.0	15.9	-6.4	5.5	5.5
BH end	30	-20	25	-15.5	40.0	-29.0

6.2 Groundwater

The GI describes the shallow groundwater level been found at depths ranging from 1.5m to 4.5m below ground level.

Groundwater levels encountered may vary seasonally and in relation to rainfall, therefore, for design purposes, the groundwater level is recommended to be assumed at approximately +8.4mOD for 230NM and CPR area of the site and +8.9mOD for Beehive Retail Centre area of the site.

6.3 Geotechnical Parameters

In the absence of strength tests for superficial deposits, the derivation of soil parameters has been based on correlation between SPT N values and strength parameters as per Stroud and Butler (1975) and Peck (1967).

For solid geology deposits, the geotechnical parameters have been derived based on correlation between SPT N and strength parameters; and laboratory tests described in the previous sections.

The short-term total stress (undrained) stiffness of the cohesive strata has been obtained from Triaxial Test results and correlations with the undrained shear strength. The long-term effective stress (drained) stiffness for cohesive strata has been taken as 80% of the total stress (undrained) stiffness, following principles of elasticity theory (assuming a Poisson's Ratio of 0.2).

The stiffness of granular soils has been based on the correlation E'=1*SPT N (MPa).

Table 19: Geotechnical Parameters for 230MN and CPR

Stratum	Bulk Unit Weight (kN/m^2)	Undrained Shear Strength (kPa)	φ' (°)	c' (kPa)	Eu (MPa)	E' (MPa)
Made Ground	17	-	28	2	-	8.0
Landfill (if applicable)	16	18.0	26	2	7.2	4.0
West Melbury Chalk Formation	18	50.0	30	10	19.8	15.8
Gault Formation	20	50+7z [1]	22	5	16+2.3z [1]	20+2.8z [1]

Notes:
[1] z refers to depth below top of stratum.

Table 20: Geotechnical Parameters for Beehive Retail Park

Stratum	Bulk Unit Weight (kN/m^2)	Undrained Shear Strength (kPa)	φ' (º)	c' (kPa)	Eu (MPa)	E' (MPa)
Made Ground	17	-	27	2	-	4.0
River Terrace Deposits	17	-	30	2	-	6.0
West Melbury Chalk Formation	18	50.0	30	10	19.8	15.8
Gault Formation	20	50+7z [1]	22	5	16+2.3z [1]	20+2.8z [1]

Notes:
[1] z refers to depth below top of stratum.

7. Preliminary Foundation Design

7.1 General

The proposed development comprises of three individual areas with various proposed uses; CRP, 230 NMR and Beehive Retail Centre. It shall be noted that the Cambridge Masterplan Development is at an early stage and changes may occur to the proposed Development as part of the design process.

However, the current scheme involves the construction of multi-storey residential buildings, soft and hard communal landscaping, mid-rise commercial building, single-storey retail unit, car parking areas, the extension of existing buildings and additional warehouse. The multi-storey buildings are expected to be relatively heavily loaded, therefore piled foundations are likely to be necessary for the specific structures.

The following sections will present a preliminary assessment of foundations in terms of aggressive chemical environment for concrete classification, ground bearing resistance for shallow foundations and a preliminary estimate of pile resistance values.

7.2 Road Pavements and Hardstanding Areas

Typical CBR values between <1.0% and >20.0% may be anticipated for the in situ Made Ground. For design purposes, it may be necessary to include replacement of materials to achieve a suitable CBR value for road and hard standing design.

Areas of roadway and hardstanding will be included in the development and will likely have a subgrade comprising a fill material. Within the area of the potential embankment/retaining wall backfill materials (6N/6P) are required to have a minimum design CBR value of 15% in accordance with the Design Manual for Roads and Bridges CD225.

Pavement design, subgrade design (including the need for soil improvements) and capping should be confirmed during detailed design.

Design CBR values should be confirmed during construction, by undertaking in-situ CBR testing after proof-rolling of formation to identify any soft spots and requirements for inclusion of any capping in accordance with Design Manual for Roads and Bridges document CD 225 or Local Roads Authority Guidelines as appropriate.

7.3 Aggressive Chemical Environment for Concrete Classification

To determine the outline design class for buried concrete, 21No. sets of chemical testing were undertaken in accordance with BRE SD1 2005 across all sites.

For these classifications, it is presumed that the groundwater is static in all strata except the superficial deposits, where it is assumed to be “mobile”.

According to the assumed characteristic 20% of the Sulphate (water soluble in 2:1 extract) as SO_4 results, the Design Sulphate Class results are presented in Table 21.

Table 21: Summary of Aggressive Chemical Environment for Concrete Classification

Strata	Charac. SO4-H2O sol (mg/L)	Charac. pH	Charac. TPS % SO4	DS Class from 2:1 water/soil extract	ACEC Class from 2:1 water/soil extract	DS Class from Total potential sulphate	ACEC Class from Total potential sulphate
Made Ground	1244	0.48	6.9	DS-2	AC-2	DS-2	AC-2
Landfill	2141	1.59	7.5	DS-3	AC-3	DS-4	AC-4
River Terrace Deposits	-	-	-	-	-	-	-
West Melbury Chalk Formation	545	0.12	8.0	DS-2	AC-1s	DS-1	AC-1s
Gault Formation	1231	1.20	8.0	DS-2	AC-1s	DS-3	AC-2s

7.3.1 Buried Concrete Assessment

According to BRE SD1 2005, TPS values are only relevant for disturbed ground, where disturbed ground is natural ground that is substantially disturbed e.g., by cutting and filling to terrace a site, or by excavation and backfilling, as these operations create a situation where air can enter and oxidise any pyrite contained in the soil.

It also states that cutting through the ground without opening up the ground beyond the cut face e.g. piling operations or excavation without backfill, does not generally result in disturbed ground. Therefore, the DS and ACEC classification requires the 2:1 water/soil extract values along with the corresponding pH values only.

For ***shallow foundation*** design purposes, it is anticipated that earthworks and potential backfill of excavations will result in disturbed ground, therefore concrete mixes with a design class of **DS-2, AC-2** are recommended.

For ***deep foundation*** design purposes, it is recommended using CFA piling where applicable, therefore does not resulting in disturbed ground as significant depths. However, as discussed above, the construction sequence may create disturbed ground conditions, therefore it is conservatively recommended concrete mixes with a design class of **DS-3, AC3** for superficial deposits, and design classes of **DS-2, AC-1s** for deep deposits.

The exception to the above is the area with presence of landfill material, where a design class of **DS-4, AC4** shall be used.

7.4 Shallow Foundations and Floor Slabs - Ground Bearing Resistance

From the review of borehole descriptions, the Made Ground would appear as a mix of clayey, gravelly sand and sandy gravel of mixed lithologies including limestone with minor constituents of brick, sandstone, concrete, tarmac, quartzite and chert. The depth of Made Ground is varying up to 4.6mgl. Therefore, it is unlikely the Made Ground to be suitable for the direct support of shallow foundations due to its inherently variable nature and significant thickness. However, its ground bearing resistance will be examined for further assessment.

Landfill material has been encountered in 3No boreholes and its nature appears as gravelly ashy clay with a high cobble content and minor constituents of wood, brick, glass, concrete, tiles, plastic, metal including copper, string, carpet, fabric and leather, bones, pottery, rubber and glass bottles. A solvent and hydrocarbon odour has been notified. Where encountered the depth of Landfill is varying up to 15.9mgl. Therefore, it is unlikely the Landfill to be suitable for the direct support of shallow foundations due to its inherently variable nature and inclusion of unsuitable material such as ash and rubber. It is recommended the use of piled foundations in order to surpass the layer of landfill material when encountered.

The underlying River Terrace deposits were encountered at depths ranging from 0.9m to 5.5mbgl. From SPT results, a range of N values between 3 and 12 have been measured, therefore indicating variable strength. For shallow foundation design purposes, the River Terrace deposits is considered to be a suitable stratum, however, due to its variable strength, it may be required to remove any loose/residual material before installing the foundation over a homogeneous material to avoid any unexpected differential settlements.

Shallow West Melbury Chalk Formation layers are found at depths ranging 0.35 to 14.0mbgl. For these shallow depths, the undrained shear strength is estimated to be around 50.0kPa. However, 2.5m to 3.0m deep is considered to be the limit for shallow foundations and even at this depth will require significant lost formwork within excavations. If the foundation solution is adopted in West Melbury Chalk Formation, generally piles become more economical for depths much beyond 2.5m to 3.0m.

The Ground Bearing Resistance of the Made Ground, River Terrace Deposits and West Melbury Chalk Formation have been estimated in accordance with characteristic geotechnical parameters given in Table 19. Terzaghi (1954) equation has been used in accordance with BS EN 1997-1:2004, for Design Approach DA1-C2 (M2+R2) with partial factors in accordance with Table A.4 and Table A.5 of the UK National Annex; Results presented in Table 22 are based on the dimensions of the foundation, expressed by the B/L ratio.

Table 22: Summary of Ground Bearing Resistance

B/L ratio	Ground Bearing Resistance (kN/m^2)		
	Made Ground [1]	River Terrace Deposits [1]	West Melbury Chalk Formation [1]
0.2	80	155	130
0.5	87	163	140
1	96	176	150
1.5	106	189	165
2	116	200	178
2.5	126	215	190

Notes:

[1] Ground Bearing Resistance of Made Ground have been assumed at 1.5 below ground level.

[2] Ground Bearing Resistance of River Terrace Deposits have been assumed at 2.5m below ground level.

[3] Ground Bearing Resistance of West Melbury Chalk Formation have been assumed at 3.0m below ground level.

7.4.1 Preliminary Shallow Foundation Assessment Summary and Recommendations

The bearing resistance results presented above involve assumptions in which the soil supporting the foundation is homogeneous and extends to a considerable depth. The cohesion, angle of friction, and unit weight of soil were assumed to remain constant for the bearing resistance analysis. However, in practice, layered soil profiles are encountered. In such instances, the failure surface at ultimate load may extend through two or more soil layers, and a determination of the ground bearing resistance may differ from the preliminary estimations.

Ground bearing floor slabs may be appropriate where limited depth of competent Made Ground, River Terrace Deposits and West Melbury Chalk Formation is present. This would be subject to confirmation by detailed design taking into account of loading characteristics and tolerable settlements. However, improvement of the superficial deposits by compaction and the inclusion of geotextile reinforcement in capping materials may be necessary depending on the settlement requirements; Alternatively, floor slabs may be designed as suspended.

The advice of a suitably competent earthworks and ground improvement contractor should be sought to determine the feasibility of the foundation method chosen.

7.5 Piled Foundations - Preliminary Resistance Assessment

Due to significant depths of the Made Ground, low/variable strength of the Landfill material and the multi-storey relatively heavily loaded proposed structures, piled foundations are likely to be necessary for the specific structures.

The preliminary pile resistance calculation considers a single pile case and is indicative for preliminary design purposes only; hence the final pile design is responsibility of the Piling Contractor. For calculating the pile resistances, CFA piles have been assumed with concrete strength of 32MPa, and load distribution of 40/60% for dead and live load respectively. No pile testing has been assumed; and no positive contribution to pile capacity is made from any material overlying the London Clay layer.

The calculations have been carried out in accordance with BS EN 1997-1:2004, Eurocode 7, for a ***Design Approach 1 Combination 2 (A2+M1+R4).***

The following partial factors have been applied to the resistances, as per UK National Annex to Eurocode 7: Geotechnical design – Part 1: General Rules (NA+A1:2014 to BS EN 1997 - 1:2004+A:2013):

Combination 2 - Factors for Compression:

- Base resistance (R_b): 2.0
- Shaft resistance (R_s): 1.6
- Model Factor: 1.4

Combination 2 - Factors for Tension:

- Shaft resistance (R_s): 2.0
- Model Factor: 1.4

The presented values shall be limited to the corresponding structural capacity of the pile based on the selected concrete strength as per BS EN 1992-1-1: Eurocode 2: Design of concrete structures.

Results for preliminary pile resistance for all three ground models for **compression** are presented in Table 23, and results for allowable resistance for **tension** are presented in Table 24. According to the preliminary results presented in the tables above, it is recommended that piles should be installed between -15.0m

AOD and -25.0m AOD, therefore returning an allowable resistance greater than 600kN per pile.

The preliminary pile resistance charts for piles in compression and tension can be found in Appendix C.

Table 23: Summary table – preliminary pile design – combination 2 **compression** design capacities

Area of the site	Diameter/ Toe level	450mm	600mm	750mm	900mm
		Allowable Resistance (kN)			
Ground model 1	0.0m AOD	180	260	350	450
	-10.0m AOD	600	830	1080	1350
	-15.0m AOD	880	1220	1580	1970
	-20m AOD	1230	1690	2180	2690
Ground model 2	0.0m AOD	0	0	0	0
	-10.0m AOD	135	200	280	365
	-15.0m AOD	390	560	740	950
	-20m AOD	715	1000	1200	1660
Ground model 3	0.0m AOD	210	300	400	510
	-10.0m AOD	650	900	1170	1460
	-15.0m AOD	950	1320	1700	2100
	-20m AOD	1320	1810	2330	2880

Table 24: Summary table – preliminary pile design – combination 2 **tension** design capacities

Area of the site	Diameter/ Toe level	450mm	600mm	750mm	900mm
		Allowable Resistance (kN)			
Ground model 1	0.0m AOD	140	185	230	280
	-10.0m AOD	515	690	860	1030
	-15.0m AOD	790	1050	1315	1580
	-20m AOD	1120	1490	1864	2230

Area of the site	Diameter/ Toe level	450mm	600mm	750mm	900mm
		Allowable Resistance (kN)			
Ground model 2	0.0m AOD	0	0	0	0
	-10.0m AOD	90	120	180	220
	-15.0m AOD	300	405	510	615
	-20m AOD	600	800	1000	1200
Ground model 3	0.0m AOD	170	225	280	340
	-10.0m AOD	570	760	950	1140
	-15.0m AOD	850	1130	1420	1710
	-20m AOD	1200	1600	2000	2400

7.5.1 Preliminary Pile Assessment Summary and Recommendations

Due to significant depths of the Made Ground, low/variable strength of the Landfill material and the multi-storey relatively heavily loaded proposed structures, piled foundations are likely to be necessary for the specific structures.

The piled foundation solutions will need to be designed with due consideration of both shallow and deep obstructions present in the Made Ground; The advice of a suitable competent pilling contractor should be sought to determine the feasibility of a piled foundation solution and the most appropriate pile type.

Pile toe levels are recommended to be embedded at least 3m within the founding stratum, and the toe shall stay within 3x the pile diameter above the bottom of the layer.

The final piling design should be responsibility of the Piling Contractor, and it is recommended that a detailed serviceability/settlement assessment is undertaken as part of the detailed design to assess the pile behaviour under working load conditions.

The effects of noise and vibration (e.g., from piling plant) should be addressed as part of the Contractor's method statements.

Consideration should also be given to the potential of the piled foundation solution adopted to create migration pathways for groundwater contamination. Requirements for Foundation Risk Assessment may be required as part of the final design.

7.6 Groundwater and Stability of Excavations

Given the highly variable and frequently granular nature of Made Ground / natural strata, and the presence of shallow water table (1.5mbgl to 4.5mbgl) excavations may locally be unstable in Made Ground, natural strata or landfill material. Unstable excavations may require sidewall support.

The groundwater is likely to be perched in the Made Ground and natural strata. Based on the foregoing, hence, excavations may require a sump type dewatering due to significant water ingress. The inflow of water into excavations is likely to vary across the site and may increase with depth. The water may require treatment before it is discharged from the site.

Appropriate dewatering measures employed should be in accordance with relevant guidance such as CIRIA Report C750, Groundwater control: design and practice, second edition (2016).

An assessment of excavation stability should be undertaken (where necessary) to confirm the potential requirement for side wall support prior to finalising construction proposals.

The detailed assessment of the stability of excavations during construction is outside the remit of this report and should be discussed with the Groundwork's Contractor prior to the commencement of site works.

Given the fact that groundwater level is assumed to be at 2.1mbgl, any proposed excavation is anticipated at deeper level than the groundwater level; as per BS:8102 (2009), the groundwater is considered to be 'High'.

8. Conclusions

8.1 General

Gardiner and Theobald LLP has instructed Waterman Infrastructure & Environment Limited ("Waterman") to undertake a Ground Investigation Report (GIR) for the proposed development of Beehive Retail Park, Cambridge.

The following sections presents the overall findings of the report.

8.2 Ground Conditions

The borehole logs indicate the presence of Made Ground and superficial deposits of River Terrace Deposits overlaying a solid geology comprised of West Melbury Chalk Formation and Gault Formation.

In the middle of the upper part of the whole site Landfill material was encountered to a maximum depth of 15.9mbgl. Thus, a special consideration is needed for the geotechnical design of the specific area.

In the south-west corner of the Beehive Retail Centre area of the site River Terrace Deposits is present between the Made Ground and West Melbury Chalk Formation.

- **Made Ground** is described as a mix of clayey, gravelly sand and sandy gravel of mixed lithologies including limestone with minor constituents of brick, sandstone, concrete, tarmac, quartzite and chert. Generally has a consistent composition across the site.
- **Landfill** is described as gravelly ashy clay with a high cobble content and minor constituents of wood, brick, glass, concrete, tiles, plastic, metal including copper, string, carpet, fabric and leather, bones, pottery, rubber, and glass bottles. A solvent and hydrocarbon odour has been noted. Only encountered in locations WBH104, WBH105 and WBH109, all of which are located in the middle of the upper side of the site.
- **River Terrace Deposits** are described as gravelly medium to coarse sand. Only encountered in location WBH113, which is located in the south-west of the down side of the site.
- **West Melbury Chalk Formation** is described as structureless chalk composed of soft to firm cream mottled yellow to brown silty gravelly, occasionally cobbly clay with weak low-density clasts or light grey clayey gravelly cobbles with weak low-density clasts (Grade Dc)
- **Gault Formation** is described as stiff to very stiff high strength to very high strength grey silty clay.

The GI describes the shallow groundwater level been found at depths ranging from 1.5m to 4.5m below ground level.

Groundwater levels encountered may vary seasonally and in relation to rainfall, therefore, for design purposes, the groundwater level is recommended to be assumed at approximately +7.9mOD for 230NM and CPR area of the site and +8.5mOD for Beehive Retail Centre area of the site.

Laboratory testing has revealed that the cohesive materials encountered across the site have an intermediate to very high plasticity index and medium volume change potential, and this classification should be assumed for design purposes.

8.3 Road Pavements and Hardstanding Areas

Typical CBR values between <1.0% and >20.0% may be anticipated for the in situ Made Ground. For design purposes, it may be necessary to include replacement of materials to achieve a suitable CBR value for road and hard standing design.

Areas of roadway and hardstanding will be included in the development and will likely have a subgrade comprising a fill material. Within the area of the potential embankment/retaining wall backfill materials (6N/6P) are required to have a minimum design CBR value of 15% in accordance with the Design Manual for Roads and Bridges CD225.

Pavement design, subgrade design (including the need for soil improvements) and capping should be confirmed during detailed design.

Design CBR values should be confirmed during construction, by undertaking in-situ CBR testing after proof-rolling of formation to identify any soft spots and requirements for inclusion of any capping in accordance with Design Manual for Roads and Bridges document CD 225 or Local Roads Authority Guidelines as appropriate.

8.4 Shallow Foundations and Floor Slabs

It is unlikely the Landfill will be suitable for the direct support of shallow foundations due to its inherently variable nature and inclusion of unsuitable material such as ash and rubber. It is recommended the use of piled foundations in order to surpass the layer of landfill material when encountered.

The underlying River Terrace deposits were encountered at depths ranging from 0.9m to 5.5mbgl. From SPT results, a range of N values between 3 and 12 have been measured, therefore indicating variable strength. For shallow foundation design purposes, the River Terrace deposits is considered to be a suitable stratum, however, due to its variable strength, it may be required to remove any loose/residual material before installing the foundation over a homogeneous material to avoid any unexpected differential settlements.

Shallow West Melbury Chalk Formation layers are found at depths ranging 0.35 to 14.0mbgl. For these shallow depths, the undrained shear strength is estimated to be around 50.0kPa. However, 2.5m to 3.0m deep is considered to be the limit for shallow foundations and even at this depth will require significant lost formwork within excavations. If the foundation solution is adopted in West Melbury Chalk Formation, generally piles become more economical for depths much beyond 2.5m to 3.0m.

The Ground Bearing Resistance of Made Ground, River Terrace Deposits and West Melbury Chalk Formation have been estimated as per Terzaghi (1954) equation in accordance with BS EN 1997-1:2004, for Design Approach DA1-C2 (M2+R2) with partial factors in accordance with Table A.4 and Table A.5 of the UK National Annex; Results indicated a Ground Bearing Resistance for the different strata ranging from 130kN/m^2 to 230kN/m^2.

The bearing resistance results presented above involve assumptions in which the soil supporting the foundation is homogeneous and extends to a considerable depth. The cohesion, angle of friction, and unit weight of soil were assumed to remain constant for the bearing resistance analysis. However, in practice, layered soil profiles are encountered. In such instances, the failure surface at ultimate load may extend through two or more soil layers, and a determination of the ground bearing resistance may differ from the preliminary estimations.

Ground bearing floor slabs may be appropriate where limited depth of competent Made Ground, River Terrace Deposits and West Melbury Chalk Formation is present. This would be subject to confirmation by detailed design taking into account of loading characteristics and tolerable settlements. However, improvement of the superficial deposits by compaction and the inclusion of geotextile reinforcement in capping materials may be necessary depending on the settlement requirements; Alternatively, floor slabs may be designed as suspended.

The advice of a suitably competent earthworks and ground improvement contractor should be sought to determine the feasibility of the foundation method chosen.

8.5 Piled Foundations

Due to significant depths of the Made Ground, low/variable strength of the Landfill material and the multi-storey relatively heavily loaded proposed structures, piled foundations are likely to be necessary for the specific structures.

The preliminary pile assessment has been carried out in accordance with BS EN 1997-1:2004, Eurocode 7, for a ***Design Approach 1 Combination 2 (A2+M1+R4)***.

According to the preliminary results presented in the tables above, it is recommended that piles should be installed between -15.0m AOD and -25.0m AOD, therefore returning an allowable resistance greater than 600kN per pile.

Due to significant depths of the Made Ground, low/variable strength of the Landfill material and the multi-storey relatively heavily loaded proposed structures, piled foundations are likely to be necessary for the specific structures.

The piled foundation solutions will need to be designed with due consideration of both shallow and deep obstructions present in the Made Ground; The advice of a suitable competent pilling contractor should be sought to determine the feasibility of a piled foundation solution and the most appropriate pile type.

Pile toe levels are recommended to be embedded at least 3m within the founding stratum, and the toe shall stay within 3x the pile diameter above the bottom of the layer.

The final piling design should be responsibility of the Piling Contractor, and it is recommended that a detailed serviceability/settlement assessment is undertaken as part of the detailed design to assess the pile behaviour under working load conditions.

The effects of noise and vibration (e.g., from piling plant) should be addressed as part of the Contractor's method statements.

Consideration should also be given to the potential of the piled foundation solution adopted to create migration pathways for groundwater contamination. Requirements for Foundation Risk Assessment may be required as part of the final design.

8.6 Chemical Attack and Buried Concrete

According to BRE SD1 2005, TPS values are only relevant for disturbed ground, where disturbed ground is natural ground that is substantially disturbed e.g., by cutting and filling to terrace a site, or by excavation and backfilling, as these operations create a situation where air can enter and oxidise any pyrite contained in the soil.

It also states that cutting through the ground without opening up the ground beyond the cut face e.g. piling operations or excavation without backfill, does not generally result in disturbed ground. Therefore, the DS and ACEC classification requires the 2:1 water/soil extract values along with the corresponding pH values only.

For ***shallow foundation*** design purposes, it is anticipated that earthworks and potential backfill of excavations will result in disturbed ground, therefore concrete mixes with a design class of **DS-2, AC-2** are recommended.

For ***deep foundation*** design purposes, it is recommended using CFA piling where applicable, therefore does not resulting in disturbed ground as significant depths. However, as discussed above, the construction sequence may create disturbed ground conditions, therefore it is conservatively recommended concrete mixes with a design class of **DS-3, AC3** for superficial deposits, and design classes of **DS-2, AC-1s** for deep deposits.

The exception to the above is the area with presence of landfill material, where a design class of **DS-4, AC4** shall be used.

8.7 Groundwater and Excavations

Given the highly variable and frequently granular nature of Made Ground / natural strata, and the presence of shallow water table (1.5mbgl to 4.5mbgl) excavations may locally be unstable in Made Ground, natural strata or landfill material. Unstable excavations may require sidewall support.

The groundwater is likely to be perched in the Made Ground and natural strata. Based on the foregoing, hence, excavations may require a sump type dewatering due to significant water ingress. The inflow of water into excavations is likely to vary across the site and may increase with depth. The water may require treatment before it is discharged from the site.

Appropriate dewatering measures employed should be in accordance with relevant guidance such as CIRIA Report C750, Groundwater control: design and practice, second edition (2016).

An assessment of excavation stability should be undertaken (where necessary) to confirm the potential requirement for side wall support prior to finalising construction proposals.

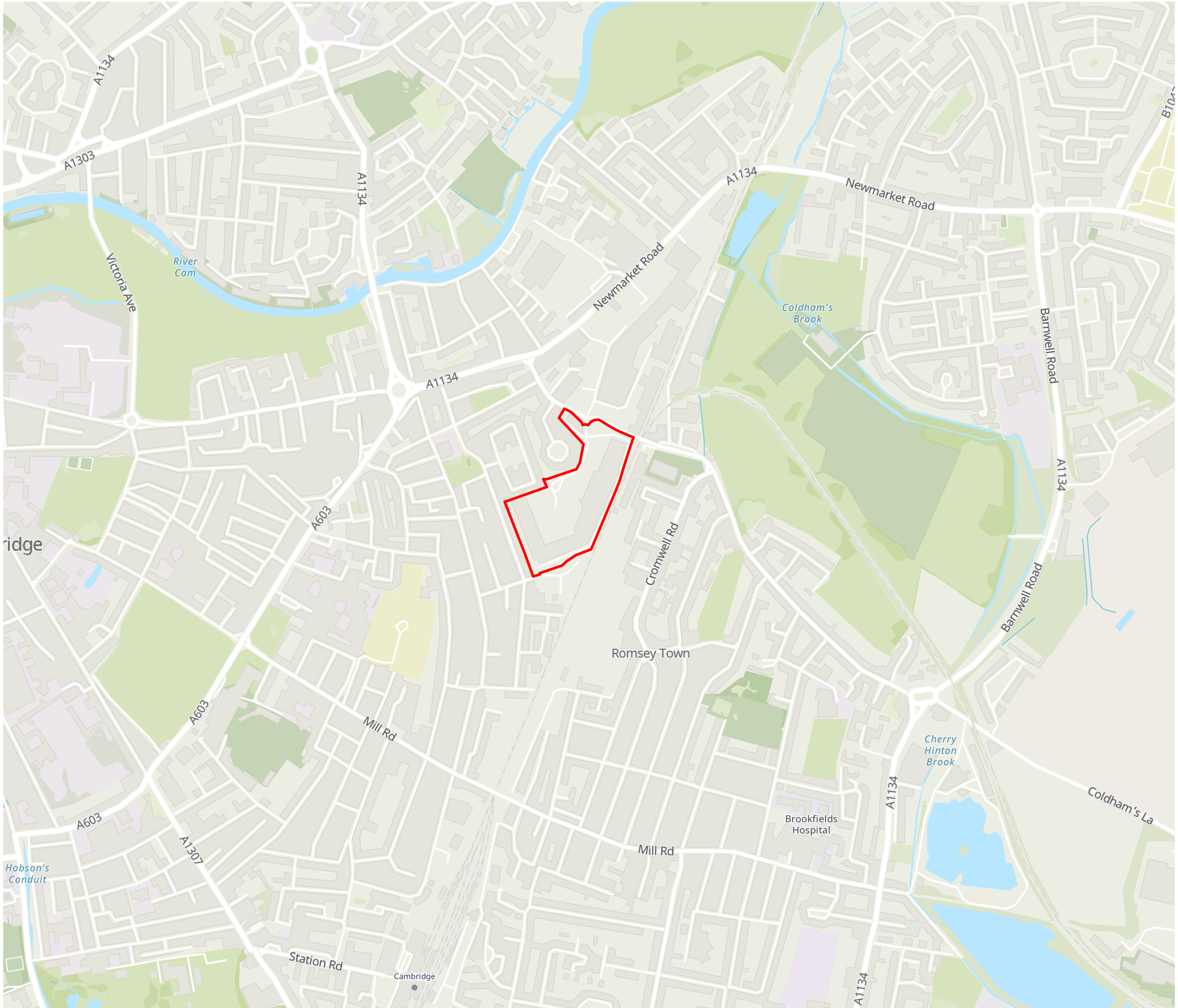
The detailed assessment of the stability of excavations during construction is outside the remit of this report and should be discussed with the Groundwork's Contractor prior to the commencement of site works.

References

1. BRE (1993), Digest 240 Low - rise buildings on shrinkable clay soils: Part 1.
2. British Geological Survey, British Regional Geology Publication for London and the Thames Valley (2013) (www.bgs.ac.uk).
3. Carter, M. and Bentley S.P. (1991) *Correlations of Soil Properties*, Pentech Press, London.
4. CIRIA R143 (1995) *The Standard Penetration Test (SPT): Methods and Use.*
5. CIRIA C760, Guidance on Embedded Retaining Wall Design.
6. CIRIA R181 'Piled Foundation in Weak Rock'
7. Kulhawy, F.H. and Mayne, P.W. (1990) "Manual on Estimating Soil Properties for Foundation Design". Report EPRI EL - 6800 Research Project 1493 - 6, Electric Power Research Institute, Palo Alto, CA, pp. 306.
8. Peck, R.B., Hanson, W.E. and Thornburn, T. H., *Foundation Engineering,* 2nd Edition 1967.
9. R.F. Craig, Craig's Soil Mechanics, 7th Edition.
10. Stroud and Butler, 1975, Standard Penetration test and the Engineering Properties of Glacial Materials, The Engineering Behaviour of Glacial Materials, The Midland Soil Mechanics and Foundation Engineering Society.
11. The Construction (Design & Management) Regulations 2015.
12. UK National Annex to Eurocode 7: Geotechnical design – Part 1: General Rules (NA+A1:2014 to BS EN 1997 - 1:2004+A:2013).

APPENDICES

A. Drawings



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Contains data from OS Zoomstack

Site Location

N

0 50 100 200 300 400 500 m

Project Details	WIE17469-100: Project Otter
Figure Title	Figure A1: Site Location Plan
Figure Ref	WIE17469-100_GIS_PRA_A1A
Date	February 2022
File Location	\\H-lncs\wiel\Projects\WIE17469\100\9_GIS\WIE17469-100_GIS_LQD\

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Waterman

Site Boundary

N

0 10 20 40 60 80 100 m

Project Details	WIE17469-100: Project Otter
Figure Title	Figure A2: Site Plan
Figure Ref	WIE17469-100_GIS_PRA_A2A
Date	February 2022
File Location	\\H-lncs\wiel\Projects\WIE17469\100\9_GIS\WIE17469-100_GIS_LQD\

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B. GI Data and Interpretation Charts

B1. 230 NM and CPR area of the site

waterman	SPT N Value
Project ID: **WIE17469**	Project Title: **Ground Investigation Report**
Client: **Gardiner and Theobald LLP**	Location: **Cambridge**

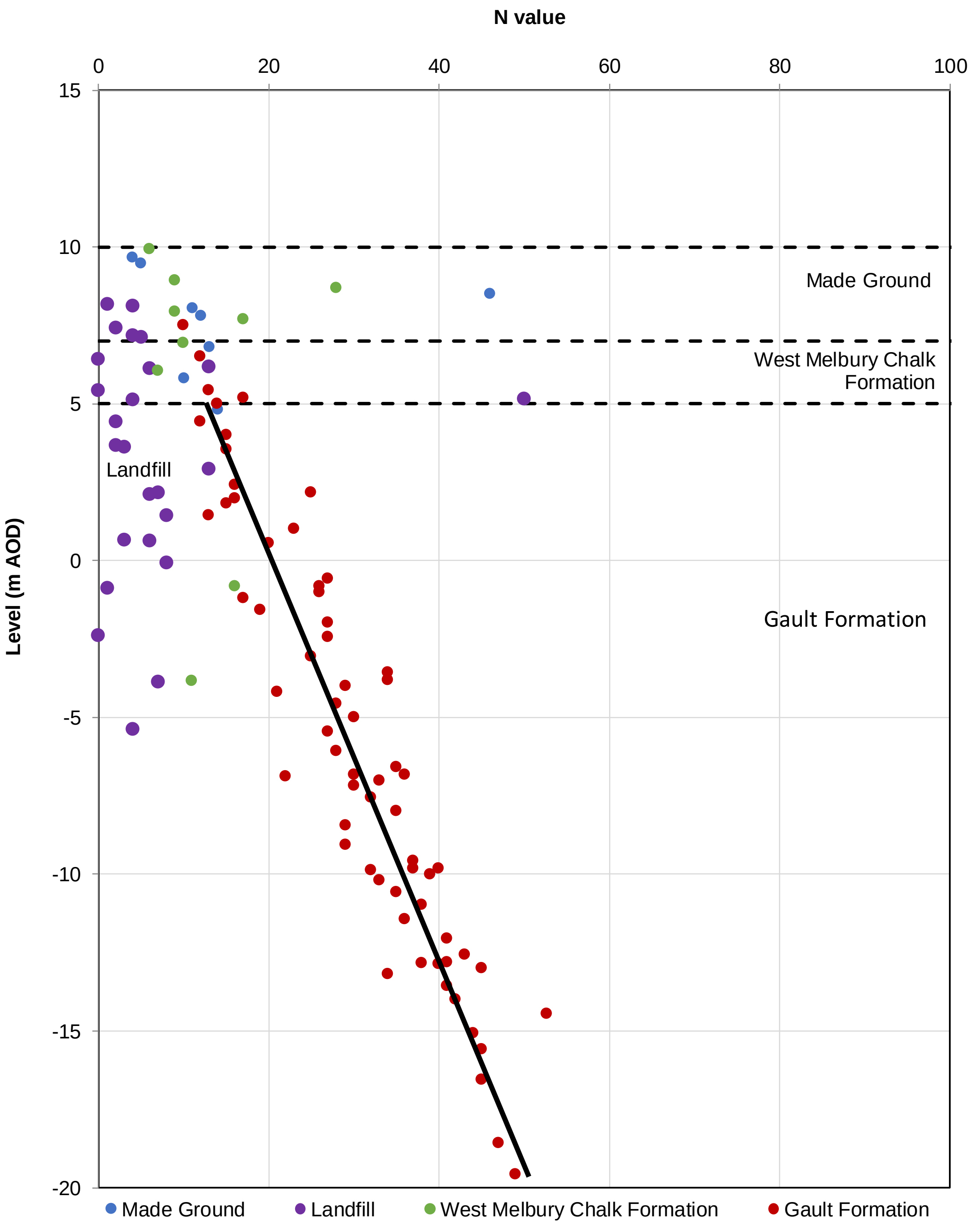

Figure B1-1: SPT value versus level (all strata) for 230 NMR and CRP

waterman	Unit Weight		
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Client:	**Gardiner and Theobald LLP**	Location:	**Cambridge**

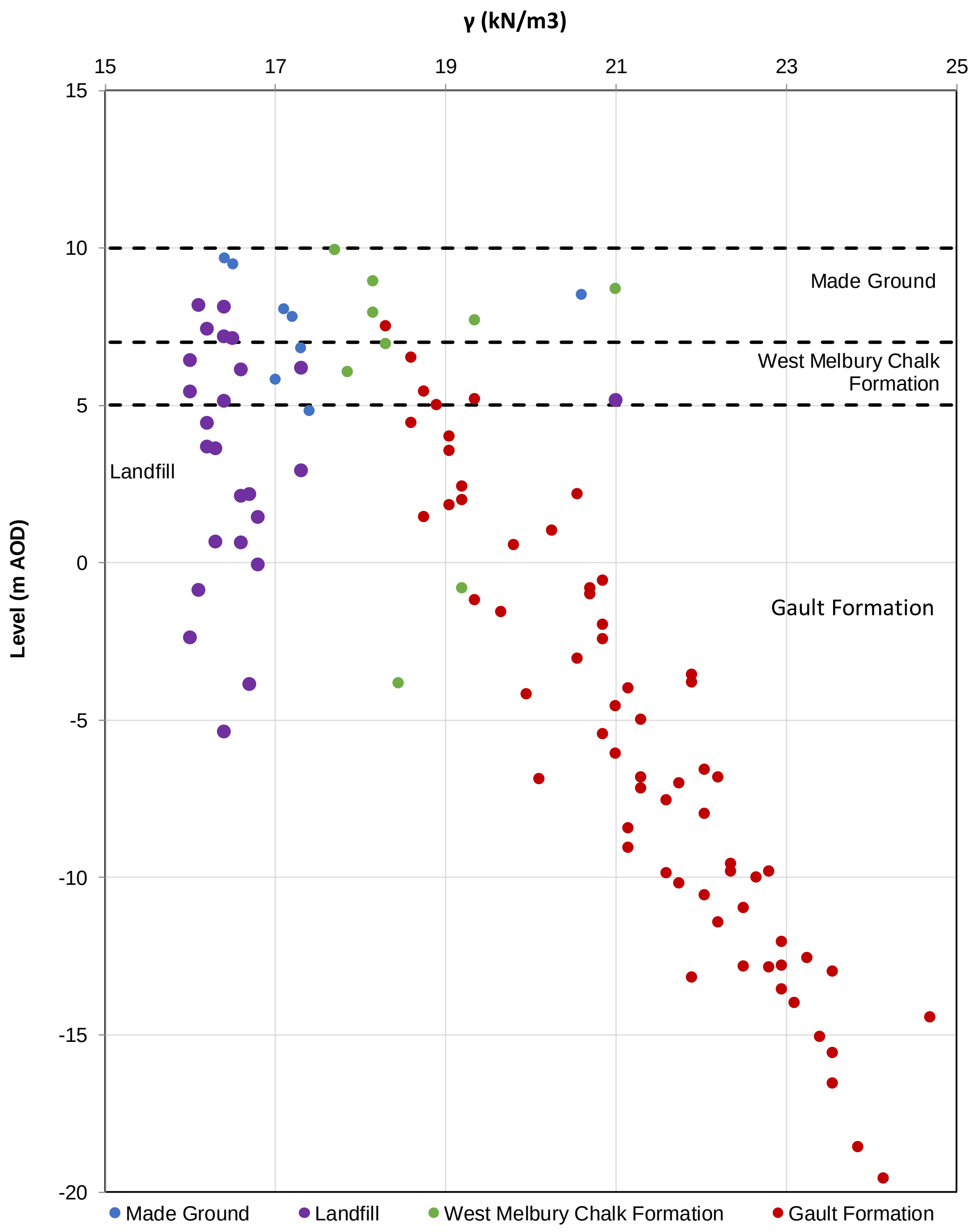

Figure B1-2: Unit weight versus level (all strata) for 230 NMR and CRP

Undrained Shear Strength

Project ID:	**WIE17469**	Project Title:	**Ground Investigation Report**
Client:	**Gardiner and Theobald LLP**	Location:	**Cambridge**

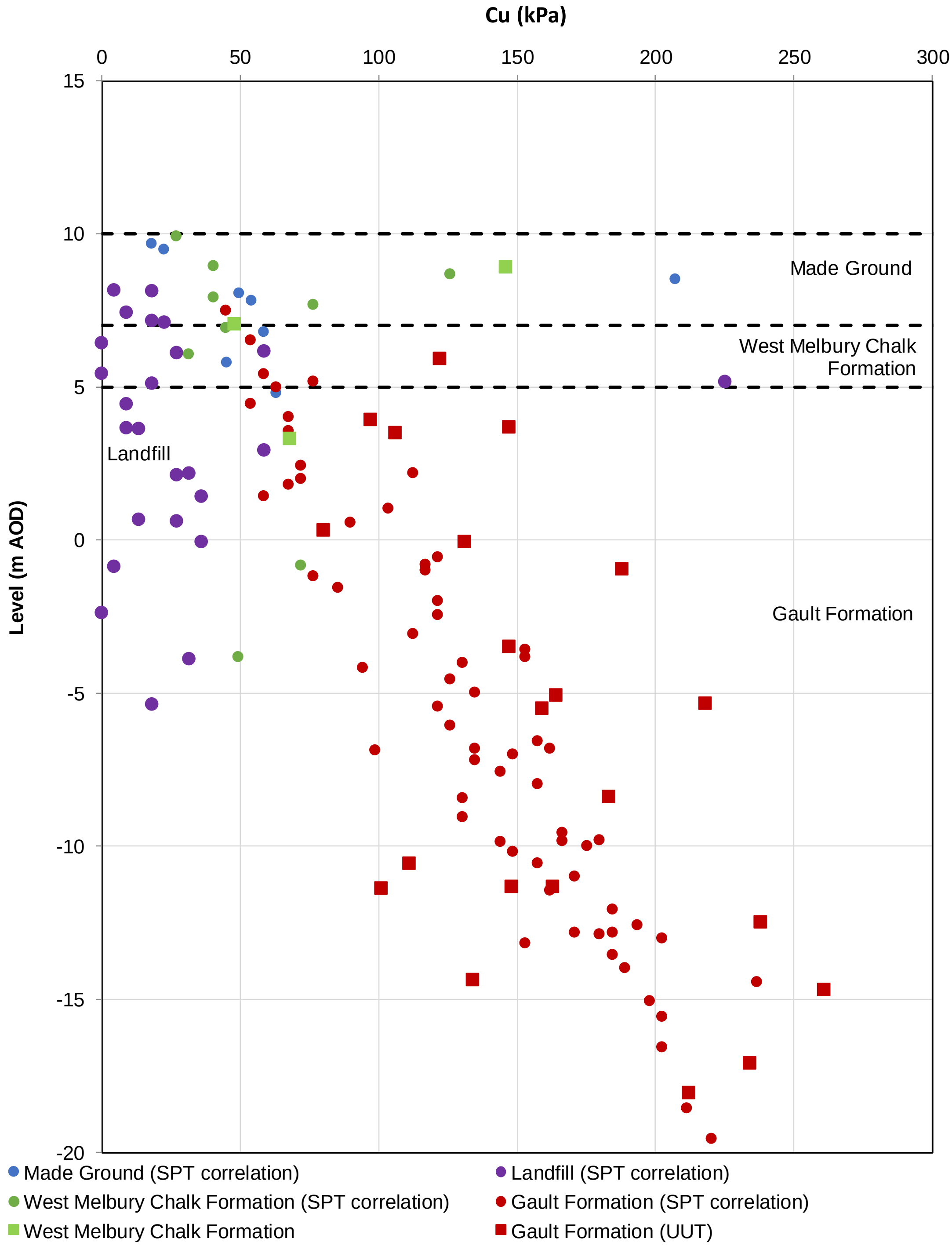

Figure B1-3: Undrained shear strength versus level (cohesive strata) for 230 NMR and CRP