Phase 2 (geo-environmental ground investigation) report

Keston Nursery, Pinchbeck

Produced for D. Brown Building Contractors Ltd

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Ground investigation report

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EXECUTIVE SUMMARY

Proposed works	Large scale residential development (low rise dwellings with estate roads and other associated infrastructure)
Site history	Site has been used as a nursery (i.e. greenhouse nursery)
Published geology	*Drift:* Tidal Flat Deposits (locally previously known as Terrington Beds - sandy silt, loose sand and soft clay) [Glacial till anticipated at depth; probably as boulder clay] *Solid:* Oxford Clay Formation
Topography	Relatively flat, level and low-lying; c. 3m above Ordnance Datum
Hydrology	Drainage ditches along some of the site boundaries Blue Gowt Drain c. 300m east
Site works	10 windowless-sampler boreholes (WS101-WS110) to 2.0 – 4.0m bgl 3 shell and auger boreholes (BH1-BH3) 5 super heavy DCP locations (WSs 103, 105, 106, 108 & 109) to depths of 8m to 13m bgl
Revealed ground conditions	*North part of the site (e.g. BH1, BH2, WSs 101 to 105)* **Topsoil**: (across much of the area) Brown with rootlets *[max depth: 0.55m bgl]* **Made ground:** (in WSs 101, 102, 103 & 105) Natural reworked soils or sub-base *[max depth: 1.0m bgl]* **Tidal Flat Deposits (TFD)**: • Bands of cohesive silts/clays and granular silts/sands. • Typically soft/firm to about 1.5m bgl and very soft/loose below 2m bgl. • Possible slightly organic silt/clay below about 6.6m bgl • Base of deposits about 8.4m to 9.9m bgl *South part of the site (e.g. BH3, WSs 106 to 109)* **Topsoil**: (across much of the area) Brown with rootlets *[max depth: 0.45m bgl]* **Tidal Flat Deposits (TFD)**: • (Very) loose granular silts/sands. • Possible slightly organic silt/clay below about 7.0m bgl • Base of deposits about 8.7m bgl *The whole site* **Glacial till**: revealed as boulder clay *[proven to at least 16m bgl]* • Stiff becoming very stiff

Groundwater	Standing groundwater levels after drilling: 0.89m to 2.12m bgl Groundwater monitoring: 0.71m to 1.54m bgl
Allowable net bearing pressures	Strip footings in cohesive soft/firm clay (north): 40kPa (0.9m-deep, 0.6m-wide footings) Strip footings in granular sands/silts (south): 50kPa (0.5m-deep, 0.6m-wide footings), 61kPa (0.75m-deep, 0.6m-wide footings) Rafts in granular sands/silts (south): 31kPa (40mm settlement) Rafts in silts/clays (north): 20kPa (40mm – 60mm settlement) Piles: anticipated working load = 87kN (300mm-wide pile, 11m deep)
Shrinkable soil	Clay swelling/heave may occur due to adjacent trees (north) Shallow TFD soils revealed in the south part of the site are granular silts and sands (i.e. they are non-shrinkable soils)
Buried concrete	No special sulphate protection (ACEC Class AC1) required for shallow concrete (e.g. to about 3m bgl) Some protection (ACEC Class AC2) needed for piles
Gas protection	Underlying TFD found to be of low organic content (SOM <4%) No elevated gas levels/flows detected during monitoring Assessed NHBC classification of Green and CIRIA classification of CS1 No gas protection measures should be required
Road pavement design	CBR values (north): 0.5%-28% CBR values (south): 2.0%-37%
Contamination test results	This investigation (and the previous GEL investigation) have tested numerous soil samples and found: • No elevated levels of metals or metalloids • No elevated levels of PAHs • No elevated levels of petroleum hydrocarbons • No asbestos fibres (or asbestos containing materials) • No detected levels of pesticides (i.e. organochlorines or organophosphates)
Contamination assessment	No unacceptable risks identified Any unforeseen contamination (for example, hydrocarbon odours or staining) encountered during the development of the site should be monitored and reported to the local authority.

1 Introduction

Humberside Materials Laboratory Limited (HML) has undertaken a geo-environmental ground investigation for a proposed development on land at Keston Nursery, Pinchbeck, Lincolnshire, as requested by the client, D. Brown Building Contractors Ltd. The land will be hereinafter referred to as *the site*.

1.1 Aims

The aim is to investigate and assess the geo-environmental (i.e. geotechnical and environmental) ground conditions to facilitate the engineering design and contamination assessment of the proposed development.

1.2 Scope

The scope of the investigation includes:

- Review of desk study information (including previous site investigation reports)
- Intrusive investigation works
- Geotechnical and geochemical laboratory testing
- Assessment of geotechnical engineering issues
- Assessment of contamination using a source-pathway-receptor approach
- Recommendations for further works, where appropriate

1.3 Conditions and Limitations

This report is produced solely for the client and should only be copied in full. When transmitted electronically, the definitive copy of the report is held by Humberside Materials Laboratory Ltd.

This report is prepared on the assumption that all facts have been disclosed.

The comments given in this report and the opinions expressed assume that conditions do not vary beyond the range revealed by this study and the information provided in the production of this report is complete and reliable.

2 Desk study information

2.1 Sources of site information

A variety of sources of desk study information have been used to gather information about the site, as listed below (in Table 1).

Table 1: Sources of site information

Source description		*Details*	*Date*
Site specific	Site visit and walkover	Photos (in Appendix A) and notes of HML visit	22 Jun 2020
	Topographical survey	Existing Site Plan LDC drawing number LDC2238-02	Oct 2018
	Combined topographical and proposed plan	Proposed Block Plan LDC drawing number LDC2328	Jan 2019
	Proposed plan	Proposed Site Plan LDC drawing number LDC2902-BR001	Jan 2019
	Site Investigation Report	Ground Engineering Ltd report (ref. C13895)	Aug 2016
Local information	Local maps	www.google.co.uk, www.openstreetmap.org, www.promap.co.uk & www.streetmap.co.uk	Jul 2020
	Online British Geological Survey (BGS) data	www.bgs.ac.uk; geological maps & borehole records	Jul 2020
	Local BGS geology maps	1:50000 Series, Spalding, Sheet 144, Solid & Drift	1992
	Historical maps	www.oldmapsonline.org; www.old-maps.co.uk	Jul 2020
	Topographical data	http://en-gb.topographic-map.com	Jul 2020
	Radon gas data	Interactive map at www.ukradon.org	Jul 2020
	Environmental data	https://magic.defra.gov.uk/MagicMap.aspx	Jul 2020
	Historic landfill data	https://environment.data.gov.uk/	Jul 2020
	Coal authority data	http://mapapps2.bgs.ac.uk/coalauthority/home.html	Jul 2020

2.2 The site

2.2.1 Location and size

The site consists of an area of land currently occupied by Keston Nursery, off Mill Green Road, Pinchbeck in the South Holland district of Lincolnshire. The site is centred around grid reference 523633, 324959.

The site occupies a roughly rectangular piece of land, which measures about 370m in length and about 125m in average width. The total area is about 4.6 hectares.

Current pedestrian and vehicle access is via Mill Green Road using a gated entrance in the northwest corner of the site. There is an alternative (unused) pedestrian and vehicle access roughly midway along the western boundary via a wooden gate.

A site location plan is included in Appendix A.

2.2.2 Site features

At the time of the site walkover in June 2020 the following site features were noted. Photographic views (Photos 1 to 7) of the site are shown later (in Appendix A) as well as an aerial view (from Google Earth).

The site is currently a commercial nursery which appears to be largely disused. The only active area appeared to be around the northwest quarter of the site which includes an asphalt parking and loading area, a storage building, some offices and a large greenhouse. It is understood the site is earmarked for disuse.

The remainder of the site appears to comprise of former growing areas which are no longer in use. There is a network of single-track concrete roads that reach most areas of the site. In the northeast part of the site, the concrete tracks are bounded by former greenhouse areas which are now largely covered by sand or thin woven geotextile over clean sand. The southern area of the site is partly covered by existing greenhouses and partly by bare earth.

There is an artificial pond within the site near the western boundary. This is about 30m in diameter and is formed from a plastic membrane lapped over a 1m-high earth embankment around the periphery. The pond is not expected to reach more than a depth of about 1m below ground level (bgl).

There are two above-ground water tanks just inside the western boundary a few metres to the north of the artificial pond. These are evidently made of fibre glass or plastic and are about 2m in height.

There is a location in the north part of the site that appears to have formerly housed an above-ground fuel tank. There is no longer a tank evident but a short (about 1m high) masonry wall is still present that could have supported a tank as well as a fuel delivery hose and nozzle (such as used at a filling station).

The site boundaries are largely defined by vegetation, i.e. hedges and trees. A line of short trees lines the eastern and southern boundaries. A line of hedgerow defines the northern boundary while a line of conifers (possible lleyland cypresses) runs along the north half of the western boundary. The south half of the western boundary is generally undefined and is within a farm field. Some fencing is also present in places.

2.2.3 Adjacent land use

North: Residential dwelling and gardens
East: Railway embankment (in active use) beyond which are domestic dwellings
South: Farmer's field and driver training centre
West: Mill Green Road and a couple of domestic dwellings beyond which are fields

2.2.4 Site topography

The site is flat and level. Ground levels at the site are about 3.0m to 3.5m above Ordnance datum (AOD).

The adjacent railway embankment (adjacent to the east) reaches a height of about 1m above ground level. Otherwise, the land near the site is generally flat and level.

2.2.5 Proposed development

Development proposals comprise the construction of a new residential development comprising about 100 plots for low-rise detached and semi-detached dwellings with gardens and parking areas. The proposed dwellings are generally one or two storey houses.

Two proposed attenuation ponds are included within the development. One is located just inside and halfway along the eastern boundary. The other is in the northwest corner of the site.

Two pumping stations are also proposed. One is at the location of the current artificial pond (just inside and about halfway along the western boundary). The other is just inside and about halfway along the eastern boundary.

2.3 Site history

Historical maps show the site was nearly all just fields (or orchard) probably until the 1920s. A few drainage ditches criss-crossed the site. Otherwise, the site was featureless.

However, between the early part of the 20th century and the early 2000s, the site slowly grew as a nursery business. A map of 1929 shows a former small building in the centre of the site. By 1967, some glazed structures and a couple of small buildings were labelled as a Nursery in the north part of the site and there were some smaller buildings in the centre and west part of the site. By 1975 to 1980, the buildings near the centre of the site had been removed. On the other hand, by 1984, many additional nursery buildings (mostly glazed) were shown in the northwest part of the site along with a small tank feature. Further nursery buildings in the north part of the site were shown in 2000 mapping. A 2014 map shows glazed structures also in the south part of the site along

with the reservoir / pond in the west part of the site. A 2016 map shows the existing water tanks near the centre of the site.

2.4 Published geology

The local geology map (BGS 1:50000 Series, Spalding, Sheet 144) shows the town of Pinchbeck is just inland of the former Saxon coastline. That is, before land reclamation took place to create much of the existing fenland about 1000 to 1500 years ago, the site was probably only one or two kilometres from the North Sea coast.

2.4.1 Superficial drift geology

Tidal Flat Deposits

The local British Geological Survey (BGS) 1:50000 Series geology map shows the site is underlain by **Terrington Beds**. These are described as: *"younger marine alluvium; salt marsh, tidal creek and river deposits (sandy silt, sand and clay)."*

Current BGS online mapping now refers to the superficial drift deposits at the site as **Tidal Flat Deposits – Clay and Silt**. These include *"Mud flat and sand flat deposits… [which] consist of unconsolidated sediment, mainly mud and/or sand… Normally a consolidated soft silty clay, with layers of sand, gravel and peat."*

There are no available BGS historical records of previous boreholes or wells drilled within about 1km of the site. Nevertheless, historical records of boreholes drilled within the Tidal Flat Deposits in the wider area suggest they can typically reach depths of between 9.8m and 16.3m bgl.

The Tidal Flat Deposits tend to have poor engineering properties, but this can vary quite substantially. Nearby deposits usually include soft clays at shallow depth, but some (medium dense) sands have also been revealed. Traces of peat or organic matter are reasonably common.

Glacial Till

Glacial till is anticipated at depth below the site. Glacial till typically mainly consists of stiff gravelly clay (i.e. boulder clay) with occasional bands of (water-bearing) sands and gravels. BGS historical records suggest the boulder clay could be present beneath the Tidal Flat Deposits (below about 10m to 16m bgl) and could reach depths of about 23m bgl.

2.4.2 Solid geology

BGS online mapping shows the site is underlain by the Oxford Clay Formation (OCF). This is described as: *"Silicate-mudstone, grey, generally smooth to slightly silty, with sporadic beds of argillaceous limestone nodules."*

The thickness of the Oxford Clay Formation (OCF) beneath the site could be about 50m to 60m. It is underlain by the Kellaways Sand and Clay Members.

2.4.3 Mining

The site is not in an area suspected to be affected by coal mining.

2.5 Hydrogeology

The depth to the groundwater table is not known but based on the low-lying topography (c. 3m AOD) and historical BGS borehole records, it could be less than 2m bgl.

The general permeability of the anticipated underlying Tidal Flat Deposits is likely to be relatively low. Where silts and clays predominate, the permeability is probably very low. The deeper anticipated Glacial Till and OCF are also expected to be of very low permeability.

Unsurprisingly, the EA classes the superficial drift deposits and the deeper solid geology as unproductive strata. There are no known nearby groundwater abstractions.

2.6 Hydrology

The pond / reservoir on site is not a natural surface water feature. It appears to be underlain by an impermeable membrane and filled or emptied artificially.

Evidently, there are drainage ditches along many of the boundaries of the site. Local mapping shows drains along the eastern, southern and north-western boundaries.

Local drains appear to flow into the nearby Blue Gowt Drain which is about 300m east of the site. This appears to feed into Vernatt's Drain further east and eventually into the River Welland.

2.6.1 Flooding

The site is shown by UK government mapping to be located in Flood Zone 3. Land and property in Flood Zone 3 have a high probability of flooding.

2.7 Waste

There are no reported nearby historical landfills. There are none within 0.5km of the site.

2.8 Radon gas

The site is not in an area that is affected by radon. The site is within a 1km grid square in which less than 1% of homes are above the Action Level. Therefore, there should be ***no requirement*** to install radon protection measures in new buildings and extensions at the site.

2.9 Industrial land use

There is little evidence of significant industry at or near the site beyond the use of the site and nearby land for nurseries. The other features are noted as follows.

- a railway line (0m east)
- railway sidings (69m north)
- sewage works (79m southwest)
- an abattoir (112m west)
- unspecified tanks at the sewage works (129m southwest)
- saw pit (211m north)

Some tanks are noted within the site itself. This includes a tank in the northwest part of the site (e.g. in mapping dated 1984-1988). An above ground fuel tank has also been identified in the north part of the site. There are also existing above ground water tanks.

2.10 Anticipated ground conditions

The anticipated ground conditions are as shown below (in Table 2)

Table 2: Anticipated geology (subject to confirmation with site works)

Strata	*Description*	*Anticipated approx. depth to base (m bgl)*
Tidal Flat Deposits	Sandy silt, sand and clay	10 - 16
Glacial Till	Stiff boulder clay with occasional bands of sand and gravel	23
Oxford Clay Formation	Stiff to hard mudstone	70 - 80
Kellaways Sand Member	Sandstone	70+
Groundwater table – probably shallow (e.g. <2m bgl)		

3 Previous ground investigation

A ground investigation report of the site was completed in August 2016 by Ground Engineering Ltd (GEL), project reference number C13895. This was carried out for AMJ Construction Limited through town planners Robert Doughty Consultancy Limited.

The aims of the previous GEL investigation were twofold. One aim was to facilitate the geotechnical and drainage design of a proposed residential development and the other was to carry out a contamination assessment.

3.1 Scope of site works

Site works for the previous (GEL) investigation included the following exploratory holes.

- 20 window sampled boreholes (WS1 to WS20): 2.0m to 4.45m in depth
- 3 trial pits including soakaway tests (TP1 to TP3): 2.2m to 2.7m in depth
- Super heavy (SH) dynamic cone penetrometer (DCP) testing in the base of nine boreholes (WSs 1, 3, 5, 7, 9, 12, 14, 18 & 19): reaching 6.0m to 10.0m in depth

The locations of the previous GEL boreholes and trial pits are shown later (in Appendix A). The locations were spread around the south area of the site but confined to the eastern and northern boundaries in the north half of the site (which was apparently an active nursery at the time of the investigation).

3.2 Previously revealed sub-strata

3.2.1 Topsoil

Topsoil was revealed in all the window sampler holes (WS1 to WS20) and trial pits (TP1 to TP3). It ranged between 0.3m and 0.6m in recorded thickness.

No made ground was reported. No anthropogenic material was logged. On the other hand, the previous GEL investigation avoided hardstanding areas of the site where it is likely some sub-base or engineering fill would be found (e.g. to depths of about 0.5m bgl).

3.2.2 Tidal Flat Deposits (TFD)

The previous GEL (windowless sampler and trial pit) site investigation revealed slightly different ground conditions in the north and south parts of the site. The shallow TFD in the north and central parts of the site mainly comprised soft clays (interbedded with loose silty fine sands). In contrast, the shallow TFD in the south part of the site mainly comprised loose silty fine sands.

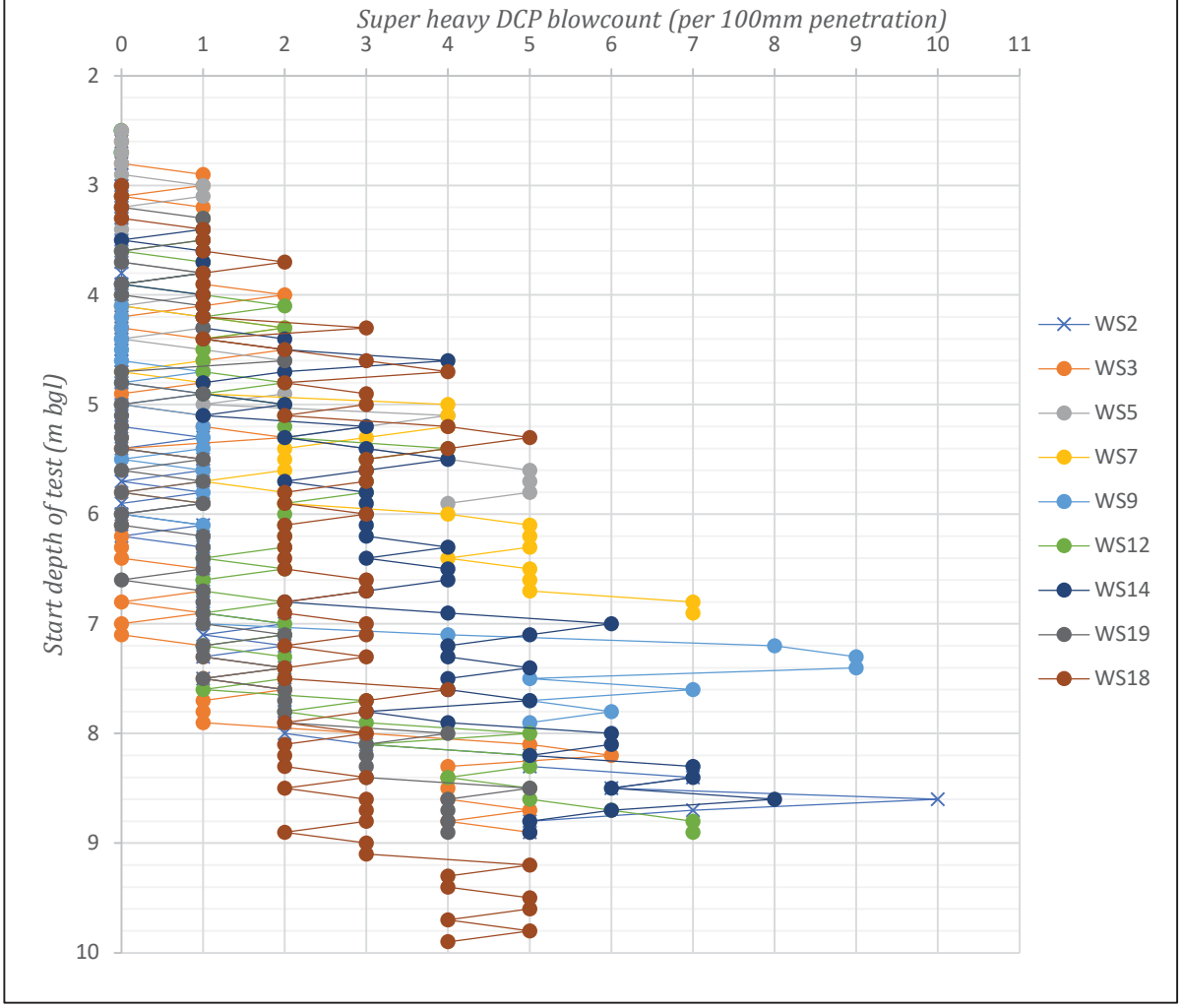
SPT N values ranged between 1 and 10 at depths of 1.2m to 4.45m bgl, as shown below (in Table 3). Such N values are typically associated with either: (1) very loose or loose sands or (2) very soft, soft or firm clays.

Table 3: Previous SPT test results

SPT start depth (m bgl)	*SPT N Value*																					
	SPT location (WS __)																				Average N value	5th percent-ile
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20		
1.2	3	1	2	2	4	2	3	2	2	4	4	10	4	6	5	4	4	6	4	2	3.7	2.0
2	2	-	2	2	3	1	1	3	1	5	5	3	4	2	4	3	4	3	6	1	2.9	1.0
3	-	-	-	-	-	-	-	-	-	3	5	4	3	4	3	-	-	-	-	-	3.7	3.0
4	-	-	-	-	-	8	-	-	-	-	-	-	6	-	-	-	-	-	-	-	7.0	6.1
		SPT mainly within granular soil												SPT mainly within cohesive soil								

Super heavy dynamic cone penetrometer (DCP) testing (using a 67.5kg hammer dropped 750mm) within the TFD soils – down to depths of at least 5m bgl – revealed very low blowcounts indicative of very soft or very loose soils. Blowcounts (per 100mm penetration) of zero or one were often recorded to depths of about 5m or 6m, as shown below (in Figure 1). Only occasionally were higher blowcounts (e.g. of 2, 3 or 4) recorded at depths shallower than 5m bgl.

***Figure 1: DCP* results in previous investigation**



The TFD clays were noted to be initially soft or firm. Recorded shear vane values were between 31kPa and 48kPa at depths of 1.0m to 1.65m bgl, as shown below (in Table 4). These values are typical of soft to firm clays.

Table 4: Shear vane results in clays (from previous GEL investigation)

Test location	*Depth (m bgl)*	*Raw results (kPa)*
WS2	1.65	40
WS7	1.4	45
WS8	1.55	31
WS9	1.4	48
TP2	1.0	38
TP2	1.5	40

However, the TFD clays tended to become very soft with depth. Locally, some slightly organic clays were logged below about 1.45m bgl (e.g. in WS2 and WS9). The DCP probing indicated very soft (or very loose becoming loose) soils to between 5m and 9m depth.

3.2.3 Interpreted marine sand

DCP test results, as shown above (in Figure 1), reportedly indicated medium dense soils below depths of 5m to 9m bgl. This was considered to indicate the presence of marine sand although no samples were taken at these depths to confirm this inference.

3.3 Visual or olfactory evidence of contamination

A slight hydrocarbon odour was reported in WS5 at a depth of about 1.4m to 1.5m bgl. Otherwise, no visual or olfactory evidence of anthropogenic contamination was identified.

Some organic material was reported at depth within the TFD soils, typically at least 1.5m bgl. Slightly organic clays were noted in the deeper parts of WS2 and WS9. Some organics were recorded within the deeper TFD sands in WS11, WS15, WS16, WS17, WS19 and WS20.

3.4 Groundwater observations

Groundwater was reported previously as seepages within exploratory holes between depths of 1.5m and 2.0m bgl and subsequent standing water levels of between 1.2m and 1.9m bgl were measured. On the other hand, no groundwater observations were made in some holes (e.g. WS12, TP1 & TP2).

Groundwater levels can vary seasonally and annually with climactic fluctuations. Groundwater levels are often close to their highest in the later spring months (e.g. April and May) and close to their lowest in the later summer, early autumn months (e.g. September and October), although this is never certain. The previous GEL investigation took place in mid-July.

3.5 Soakaway test results

Water infiltration tests were previously carried out in TP1, TP2 and TP3 to investigate the potential for shallow surface water soakaways. The infiltration was generally slow so the testing did not meet BRE 365 requirements and was limited to one fill which only managed to achieve partial emptying. Nevertheless, water infiltration rates were estimated based on extrapolated curves of water level against time.

The calculated water infiltration rates ranged between 1.66×10^{-6} m/s and 2.51×10^{-6} m/s, as shown below (in Table 5). These indicate relatively poor drainage conditions. These values should be used very cautiously as they are based on only one fill and possibly unreliable extrapolations of drainage curves.

Table 5: Results of soakaway infiltration tests in previous GEL investigation						
Test location	*Base of pit (m bgl)*	*Test soil stratum*	*Initial depth to water (m bgl)*	*Final depth to water (m bgl)*	*Time taken (mins)*	*Calculated water infiltration rate* (m/s)*
TP1	1.60	Brown silty sand	0.35	0.68	240	$2.51x10^{-6}$
TP2	2.30	Firm brown sandy silty CLAY	0.37	0.62	240	$1.66x10^{-6}$
TP3	2.20	Brown sandy silt	0.64	0.74	240	$1.77x10^{-6}$
*based on extrapolated curve of water level against time for one fill only						

3.6 Previous geotechnical laboratory testing

Four plasticity (and moisture content) tests were carried out as part of the previous investigation. The results show the plasticity index of some cohesive samples ranged between 11% and 35%, as shown below (in Table 6).

Table 6: Results of previous plasticity and moisture tests						
Hole	*Sample depth (m bgl)*	*LL (%)*	*PL (%)*	*PI (%)*	*NMC (%)*	*% passing 425um sieve*
WS2	2.00	48	19	29	36	100
WS3	0.6	32	21	11	22	100
WS7	0.9	62	27	35	32	100
WS9	2.00	35	21	14	33	100

Eight particle size distribution (PSD) tests (sedimentation method) were carried out as part of the previous investigation. The results show two of the samples (from WS1 and WS6) contained more than 35% fine soil (clay or silt), as shown below (in Table 7). However, the other samples (from WSs 11, 12, 13, 16, 18 and 20) were all determined to be silty SAND with at least 65% sand.

Table 7: Results of previous particle size distribution tests						
Hole	*Sample depth (m bgl)*	*% clay*	*% silt*	*% sand*	*% gravel*	*Particle size description*
WS1	1.2-2.0	4	58	38	0	Slightly clayey sandy SILT
WS6	1.2-2.0	6	38	56	0	Slightly clayey SILT/SAND
WS11	2.0-3.0	0	22	78	0	Silty SAND
WS12	1.2-2.0	0	22	78	0	Silty SAND
WS13	1.2-2.0	1	20	79	0	Silty SAND
WS16	1.2-2.0	0	33	67	0	Silty SAND
WS18	2.0-3.0	1	20	79	0	Silty SAND
WS20	1.2-2.0	1	22	77	0	Silty SAND

Three CBR tests were carried out as part of the previous investigation. The results show the samples from TP1 and TP2 were found to have CBR values of at least 2.3%, as seen below (in Table 8). However, the sample from TP3 was determined to have a CBR value of only about 0.5%.

Table 8: Results of previous CBR lab tests							
Hole	*Sample depth (m bgl)*	*NMC (%)*	*Bulk density (Mg/m^3)*	*Dry density (Mg/m^3)*	*CBR (%)*		*% retained on 20mm sieve*
					Top	*Bottom*	
TP1	0.5-0.7	30	1.83	1.45	4.4	6.6	0
TP2	0.5-0.7	30	1.90	1.46	2.4	2.3	0
TP3	0.5-0.7	27	1.95	1.53	0.6	0.5	0

Sulphate and pH tests were carried out on the aqueous solution from eight soil samples taken from depths of between 0.9m and 2.0m bgl. The pH was found to be at least 7.5 and the sulphate concentration ranged between 25mg/l and 272mg/l, as shown below (in Table 9).

Table 9: Results of previous sulphate tests			
Hole	*Sample depth (m bgl)*	*Water soluble sulphate (mg/l)*	*pH*
WS1	1.2	25	7.8
WS4	2.0	272	7.6
WS7	1.2	33	7.5
WS13	0.9	66	7.9
WS20	1.2	82	8.0
TP1	1.00	66	7.8
TP2	1.00	49	7.6
TP3	1.00	58	7.6

3.7 Scope of previous geochemical laboratory testing

Geochemical testing was carried out on 14 soil samples and 2 water samples, as shown below (in Table 10). Many tests were carried out on a variety of contaminants including metals, PAHs and cyanides. Some tests for TPH and pesticides were also carried out on selected samples. No asbestos testing was undertaken.

By and large, the testing was carried out on near surface (topsoil) samples. Such soils tend to be the most likely to create a pathway for contaminants to reach future site users, especially with respect to ingestion, inhalation (of dust) and direct contact.

Table 10: Scope of previous geochemical tests													
Hole	*Depth (m bgl)*	*Soil / water*	*pH*	*Sulphate (water-soluble)*	*Cyanide (free and total)*	*Sulphides*	*Metals**	*SOM*	*PAHs*	*Total phenols*	*TPHs*	*VOCs*	*SVOCs TIC (pesticides)*	*WAC*
TP1	0.2	Soil	✓	✓	✓	✓	✓	✓	✓	✓		✓	✓	
TP2	0.2	Soil	✓	✓	✓	✓	✓	✓	✓	✓		✓		
TP3	0.2	Soil	✓	✓	✓	✓	✓	✓	✓	✓		✓		
WS1	0.3	Soil	✓	✓	✓	✓	✓	✓	✓	✓				
WS3	0.3	Soil	✓	✓	✓	✓	✓	✓	✓	✓				
WS5	0.3	Soil									✓			
WS5	2.0	Soil	✓	✓	✓	✓	✓	✓	✓	✓				
WS8	0.3	Soil					✓							✓
WS12	0.3	Soil	✓	✓	✓	✓	✓	✓	✓	✓				
WS13	0.3	Soil	✓	✓	✓	✓	✓	✓	✓	✓				
WS16	0.3	Soil	✓	✓	✓	✓	✓	✓	✓	✓				
WS19	0.3	Soil	✓	✓	✓	✓	✓	✓	✓	✓		✓	✓	
WS2	>1.2	Water	✓	✓	✓	✓	✓*		✓	✓		✓	✓	
WS7	>1.4	Water	✓	✓	✓	✓	✓*		✓	✓		✓	✓	
* includes Chromium VI and water-soluble Boron (except for WAC and water samples)														

3.8 Results of previous contamination testing

The detected concentrations of contaminants for the GEL investigation were found to be within acceptable levels, as shown below (in Tables 11 and 12). No measured amounts were above the adopted generic assessment criteria (GAC), i.e. screening values.

Some petroleum hydrocarbons were detected in the WS5 sample from 2.0m bgl, but all the concentrations were well below the screening values. The highest detected concentrations were in the C12 to C21 fractions, which could indicate acceptably low trace amounts of diesel range organics.

Table 11: Results of previous GEL geochemical tests (SOIL)								
Contaminant			*No. of samples tested*	*Lowest value (mg/kg)*	*Highest value (mg/kg)*	*95th percentile (mg/kg)*	*GAC* (mg/kg)*	*No of GAC exceed-ances*
SOM			10	1.5%	2.9%	-	-	-
Metals & metalloids	Arsenic		10	8.2	1.5	11.5	32	0
	Cadmium		10	0.12	0.34	0.18	10	0
	Chromium III		10	12	29	19.61	12800	0
	Chromium VI		10	<0.50	<0.50	<0.50	21	0
	Lead		10	16	57	27.57	200	0
	Mercury		10	<0.10	0.79	0.30	6	0
	Selenium		10	<0.20	0.44	0.33	350	0
	Nickel		10	9.7	18	15.66	130	0
	Copper		10	8.1	26	14.89	3970	0
	Zinc		10	32	73	53.34	16900	0
	Free cyanide		10	<0.50	<0.50	<0.50	34	0
Total phenols			10	<0.30	<0.30	<0.30	162	0
PAHs	Benzo(a)pyrene		10	<0.10	<0.10	<0.10	0.8	0
Total petroleum hydrocarbons (TPHs)	aliphatic	C5-C6	1	<1.0	<1.0	<1.0	-	-
		C6-C8	1	<1.0	<1.0	<1.0	69.8	0
		C8-C10	1	8.7	8.7	8.7	9.79	0
		C10-C12	1	25	25	25	1390	0
		C12-C16	1	100	100	100	5100	0
		C16-C21	1	120	120	120	145000	0
		C21-C35	1	30	30	30	145000	0
		C35-C44	1	<1.0	<1.0	<1.0	-	-
	aromatic	C5-C7	1	<1.0	<1.0	<1.0	0.0493	0
		C7-C8	1	<1.0	<1.0	<1.0	86.9	0
		C8-C10	1	7.6	7.6	7.6	14.8	0
		C10-C12	1	11	11	11	57.3	0
		C12-C16	1	26	26	26	142	0
		C16-C21	1	17	17	17	272	0
		C21-C35	1	4.4	4.4	4.4	888	0
		C35-C44	1	<1.0	<1.0	<1.0	-	-
* residential with home grown produce								

As discussed, no asbestos testing was reported. Nevertheless, no asbestos containing materials were recorded in the exploratory hole logs or the test samples.

The results of groundwater testing from two water samples did not identify any significant pollution. A concentration of arsenic was found to be above the standards for drinking water quality but below the environmental standards for freshwater rivers, as shown below. No pesticides were detected.

Table 12: Results of previous GEL geochemical tests (GROUNDWATER)								
Contaminant		*No. of samples tested*	*Lowest value (ug/kg)*	*Highest value (ug/kg)*	*Water Supply GAC (ug/l)*	*No. of WQS GAC exceed-ances*	*EQS (fresh-water) GAC (ug/l)*	*No. of EQS GAC exceed-ances*
Metals & metalloids	Arsenic	2	3.4	21	10	1	25	0
	Boron	2	280	370	1000	0	2000	0
	Cadmium	2	<0.080	<0.080	5.0	0	5.0	0
	Chromium (total)	2	6.9	14	50	0	-	-
	Copper (total)	2	3.2	11	2000	0	28	0
	Cyanide (total)	2	<0.050	<0.050	0.05	0	-	-
	Lead	2	<1.0	<1.0	1.0	0	1.0	0
	Mercury	2	<0.50	<0.50	1.0	0	1.0	0
	Nickel	2	8.7	8.9	20	0	50	0
	Selenium	2	1.3	1.4	10	0	-	-
	Zinc	2	1.0	3.9	5000	0	125	0
Phenol		2	<0.030	<0.030	500	0	30	0
pH		2	7.8	7.8	6.5 – 10.0	0	-	-
PAHs	Naphthalene	2	<0.010	<0.010	10	0	10	0
	Total*	2	<0.20	<0.20	0.1	n/k	-	-
* includes only three PAHs; WQS – Water Quality Standards, EQS – Environmental Quality Standards								

4 Preliminary contamination assessment

4.1 Assessment principles

Current legislation and best practice, such as Part IIA of the Environmental Protection Act and CLR 11 (EA/Defra, 2004), recommend that potentially contaminated land is managed using a risk-based approach. Model procedures to assess the risks from contaminated land involve the development of a conceptual site model (CSM) and the identification and assessment of relevant contamination linkages.

A contamination linkage comprises a *source* (i.e. a contaminant capable of causing pollution or harm), a *receptor* and a viable *pathway* by which one can be transmitted to the other. If one of these three elements are missing, there is no significant risk.

The risk from a potential contamination can be assessed as a product of the severity of the consequences that might arise from of the contamination hazard and the probability of the hazard occurring, as outlined in CIRIA C552 (Rudland et al., 2001). The definitions used to perform the risk assessment for the subject site are shown below (in Tables 13 to 16).

Table 13: Severity of hazard being realised

Classification	*Category/Definition*			
	Human	*Controlled waters*	*Property*	*Eco system*
Severe	Short term acute risk	Short term risk	Catastrophic damage	Short term risk
Medium	Chronic damage	Pollution to sensitive water resources		Significant change
Mild		Pollution to non-sensitive water resources	Significant damage	Significant damage
Minor	Non-permanent health effects		Repairable damage	

Table 14: Probability of hazard being realised

Classification	*Definition*
Highly likely	There is a pollution linkage and an event that appears very likely in the short term and or almost inevitable over a long term, or evidence at the receptor of harm or pollution
(Medium) Likely	There is a pollution linkage and all elements are present which means that it is probable that an event will occur.
Low likelihood	There is a pollution linkage and circumstances are possible under which an event will occur. However, it is not certain that even over a long period such an event would occur and is less likely in the short term.
Unlikely	There is a pollution linkage, but circumstances are such that it is improbable that an event would occur even in the long term.

A matrix can be used to determine a suitable risk classification, as shown below (in Table 15). The largest risks are where the severity of the contaminant is great and the probability of the contaminant linkage occurring is highly likely. The definitions of the different classifications used in this assessment, based on CIRIA C552 (Rudland *et al.*, 2001), are also shown below (in Table 16).

Table 15: Risk classification matrix

		Consequence			
		Severe	*Medium*	*Mild*	*Minor*
Probability	*Highly likely*	Very high	High	Moderate	Moderate/low
	Likely	High	Moderate	Moderate/low	Low
	Low likelihood	Moderate	Moderate/low	Low	Very low
	Unlikely	Moderate/low	Low	Very low	Very low

Table 16: Risk classification definition

Classification	*Definition*
Very High	There is a high probability that severe harm could arise to a designated receptor from an identified hazard or there is evidence that severe harm to a designated receptor is currently occurring. This risk if realised is likely to result in a substantial liability. Investigation and or remediation is required.
High	Harm is likely to a designated receptor from an identified hazard or there is evidence that severe harm to a designated receptor is currently occurring. This risk if realised is likely to result in a substantial liability. Investigation and or remediation may be necessary in the short term and likely over the long term.
Moderate	It is possible that harm could arise to a designated receptor from the identified hazard. However, it is relatively unlikely that such a hazard would be severe, or if any harm were to occur it is more likely that the harm would be relatively mild. Investigation is normally required to clarify and determine the potential risk. Some remedial works may be required in the long term.
Low	It is possible that harm could arise to a designated receptor from the hazard identified but is likely that this hazard if realised would at worst normally be mild.
Very low	There is a low possibility that harm could arise to a receptor. In the event of such harm being realised it is not likely to be severe

4.2 Conceptual site model

The previous GEL investigation included conceptual site models and risk assessments (preliminary and updated). These were built up through consideration of potential sources, pathways and receptors. The following conceptual model has been developed, which draws heavily from the previous GEL conceptual modelling.

4.2.1 Sources

The considered potential sources of contamination at the site are shown below (in Table 17). The likelihood of these sources being present can be (partly) assessed using the results of the previous investigation.

Table 17: Potential sources of contamination (identified in GEL report)			
Location	*Source*	*Comments*	*Example contaminants*
On site	Drainage / buildings	Effluent from leaking drains Asbestos within existing buildings	Metals, asbestos, pesticides
	Sub-soils	Contamination within made ground	Metals, PAHs, asbestos
	Above ground tanks	Leaking tanks and pipework	Petroleum hydrocarbons (TPH)
	Soil gas	From made ground or natural geology	Carbon dioxide & methane
Off-site	Ground in adjacent land	Migrating contamination	Metals, PAHs, TPH

4.2.2 Receptors and pathways

The identified receptors of potential contamination are as shown below (in Table 18). These receptors include humans, controlled waters, ecology and buildings/structures.

Table 18: Potential receptors and pathways		
Pathway	*Receptors*	*Comments*
Ingestion, direct contact, inhalation of dust	Future site users	Mainly in gardens, possible also in soft landscaping areas.
Inhalation of vapours		E.g. volatile compounds
Ingestion, direct contact, inhalation of dust	Construction workers	Competent contractors using standard safe working practices should reduce exposure, if present
Inhalation of vapours		E.g. volatile compounds
Leaching and migration downwards	Bedrock aquifer	The underlying bedrock is Oxford Clay Formation which is of low permeability and not aquiferous
Lateral migration	Surface waters	Underlying natural soils are low permeability. Groundwater table is probably shallow (e.g. c. 1m bgl)
Dust		Drains are located next to the site
Direct contact	Ecology	Site vegetation appears to be in excellent health
Direct contact or leaching and migration	Buried concrete	Buried concrete will be designed to BRE. Water pipes will meet water authority requirements.

4.3 Preliminary risk assessment

The desk study review, findings of the previous GEL investigation have been used to carry out a qualitative preliminary risk assessment, as shown below (in Table 19) using a *source-pathway-receptor* model of potential pollution linkages.

Table 19: Preliminary contamination risk assessment

Conceptual site model			*Risk Assessment*			
Potential source	*Potential pathway*	*Potential receptor*	*Probability*	*Severity*	*Risk rating*	*Comments*
Drainage / Buildings **Metals, asbestos, pesticides**	Direct contact, ingestion or dust inhalation	Future site users	Low likelihood	Medium	Low/ moderate	These risks should be mitigated through the safe removal of the existing buildings, e.g. with a suitable asbestos survey and demolition carried out by competent persons. Previous GEL investigation did not identify any elevated contamination. However, much of nursery site was not investigated
		Construction workers	Low likelihood	Medium	Low/ moderate	
	Vapour inhalation	Future site workers	Unlikely	Mild	Very low	
		Construction workers	Unlikely	Mild	Very low	
	Direct contact	Ecology	Low likelihood	Mild	Low	
	Leaching & migration	Surface waters	Low likelihood	Mild	Low	
	Leaching & migration	Bedrock aquifer	Unlikely	Mild	Very low	
	Direct contact	Buildings/ structures	Unlikely	Mild	Very low	
Sub-soils / Made ground **metals, PAHs and asbestos**	Direct contact, ingestion or dust inhalation	Future site users	Low likelihood	Medium	Low/ moderate	Previous GEL investigation did not identify any made ground or any elevated contamination. However, much of the nursery site was not investigated
		Construction workers	Low likelihood	Medium	Low/ moderate	
	Vapour inhalation	Future site workers	Unlikely	Medium	Low	
		Construction workers	Unlikely	Medium	Low	
	Direct contact	Ecology	Low likelihood	Mild	Low	
	Leaching & migration	Surface waters	Unlikely	Mild	Very low	
	Leaching & migration	Bedrock aquifer	Unlikely	Mild	Very low	
	Direct contact	Buildings/ structures	Unlikely	Mild	Very low	
Above ground tanks **Petroleum hydrocarbons**	Direct contact, ingestion or dust inhalation	Future site users	Low likelihood	Medium	Low/ moderate	Previous GEL investigation did not identify any elevated petroleum contamination even in the one location (WS5, to 2.0m bgl) where a possible hydrocarbon odour was identified. However, much of the nursery site was not investigated including two identified former tank locations
		Construction workers	Low likelihood	Medium	Low/ moderate	
	Vapour inhalation	Future site workers	Low likelihood	Medium	Low/ moderate	
		Construction workers	Low likelihood	Medium	Low/ moderate	
	Direct contact	Ecology	Low likelihood	Mild	Low	
	Leaching & migration	Surface waters	Low likelihood	Mild	Low	
	Leaching & migration	Bedrock aquifer	Unlikely	Mild	Very low	
	Direct contact	Buildings/ structures	Unlikely	Minor	Very low	
Soil Gas **CO2, methane**	Inhalation / Combustion	Future site users	Low likelihood	Medium	Low/ moderate	Previous GEL investigation identified some organic soils within the TFD.
		Construction workers	Low likelihood	Medium	Low/ moderate	
Off-site Sources **Metals, PAHs, TPHs**	Direct contact, ingestion or dust inhalation	Future site users	Unlikely	Medium	Low/ moderate	Previous GEL investigation did not identify any made ground or any elevated contamination around the boundary of the site
		Construction workers	Unlikely	Medium	Low/ moderate	
	Vapour inhalation	Future site workers	Unlikely	Medium	Low	
		Construction workers	Unlikely	Medium	Low	

Some of the considered potential contamination linkages have been assessed to be *very low* or *low* risk and are generally not considered significant. However, some linkages have been given unacceptably high risk ratings (i.e. low/moderate or higher) and, therefore, require further consideration.

Many of the unacceptably high risks have arisen due to the limitations on the previous GEL investigation. This did not sample much of the north part of the nursery site so there is a risk of contamination in this area. Notably, there are two former tank locations, one being a suspected above-ground fuel tank and the other being of unknown type, shown only in historical mapping. Also, the previous investigation did not investigate the deeper TFD soils or include any gas monitoring to assess the potential for hazardous ground gases.

Some of the significant contamination-related risks should be mitigated through standard construction practices. The risk from contaminants within the existing building, for example, asbestos, should be mitigated by suitable demolition by competent persons. Risks to construction workers should be managed by the site contractors through suitable safe working practices. Possible harm to buried concrete from sulphates should be prevented by adherence to BRE Special Digest 1. Risks from permeation of contaminants into water pipes should be mitigated through compliance with the local water authority's pipework specifications.

However, the risks from the following contamination sources are still considered to be unacceptable.

- Sub-soils / Made ground: metals, PAHs and asbestos
- (Above ground) tanks: Petroleum hydrocarbons
- Soil Gas: CO2, methane

These sources are associated with harm to future site users. The pathways from sources to receptors include ingestion and direct contact (e.g. via garden areas) and inhalation of gases and vapours (e.g. within dwellings).

Further investigation is desirable to collect more information about the nature and extent of the identified unacceptable contamination risks. This is expected to include gas monitoring, soil sampling and TPH testing near previous tanks and asbestos testing of near surface soils. The previous investigation did not include any sampling and testing within the main active nursery area in the north part of the site or any gas monitoring or asbestos testing.

5 Site Investigation Works

The procedures adopted for this site investigation are based on BS 5930 (2015) – Code of Practice for Site Investigations and BS 10175 (2011 + A2, 2017). The soils and rocks encountered have been described based on BS5930 (2015), BS EN ISO 14688-1 (2018), BS EN ISO 14688-2 (2018) and BS EN ISO 14689-1 (2018).

5.1 Fieldwork

Fieldwork took place between on the 15th June and 24th June 2020. Gas monitoring is currently ongoing.

Approximate locations of the site works are shown in the exploratory hole location plan which is presented later (in Appendix A).

The positions of the exploratory holes were established relative to existing site features and using GPS locating equipment. The depths to sub-strata and groundwater were measured from existing ground level.

Logs for the exploratory holes are presented later (in Appendix B).

5.1.1 Shell and auger boreholes (BH1 to BH3)

Three shell and auger boreholes (BH1 to BH3) were drilled between 22nd June and 24th June 2020 by SIS Drill (UK) Ltd of Willoughton, Lincolnshire. The rig extracted disturbed samples as well as some U100 samples. The borehole was about 150mm in diameter and reached depths of between 15m and 16m below ground level (bgl). Standard penetration tests (SPTs) were carried out at frequent intervals.

5.1.2 Windowless sampler boreholes

Ten windowless-sampler boreholes (WS101 to WS110) were drilled by HML between 15th and 17th June 2020 with a Dando Terrier rig to a maximum depth of sampling of 3.0m below ground level (bgl). The rig extracted consecutive 1m-long tubes of soil material for logging and sampling. The diameter of the borehole decreased from about 100mm to about 75mm. Standard penetration tests (SPTs) were carried out at frequent intervals. Shear vane testing was carried out on suitable cohesive material within the windowless-sampler sampling tubes using a calibrated Impact shear vane.

5.1.3 DCP testing

Super heavy (SH) dynamic cone penetrometer (DCP) testing was carried out at five locations adjacent to windowless sampler boreholes using the Dando Terrier rig.

Standard penetration tests (SPTs) were carried out at frequent intervals. The DCP locations were at WS103, WS105, WS106, WS108 and WS109. The DCP testing reached depths of between 8m and 13m bgl.

5.2 Laboratory testing

Geotechnical testing was undertaken at the HML UKAS accredited laboratory. The scope of geotechnical testing comprised the following.

- 4 number moisture content (MC) tests
- 4 number plasticity index (PI) tests
- 3 consolidation tests
- 6 particle size distribution tests (wet gradings)
- 8 CBR tests on remoulded samples (in a soaked condition)
- 1 quick unconsolidated undrained (UU) triaxial test

Geochemical testing was undertaken by Chemtech Environmental, a UKAS and MCERTS accredited laboratory. The scope of geochemical testing comprised the following.

- 4 number tests for pH and sulphates
- 4 number metals suites
- 4 number PAHs suites (by US EPA 16)
- 4 number asbestos identification tests
- 3 number pesticides suites
- 1 number test for TPH by CWG
- 4 number soil organic matter (SOM) content tests

A schedule of testing is shown below (in Tables 20 and 21). Results for all the laboratory testing undertaken are included later in this report (in Appendix C).

Table 20: Laboratory testing on shell and auger borehole samples (BH1 to BH3)

Sample hole	*Sample location*	*HML Sample ref. (S/__)*	*Chemtech sample ref.*	*Depth (m bgl)*	*PI*	*MC*	*Consol-idation*	*Particle size distn*	*UU Triaxial*	*pH and sul-phates*	*SOM*
BH1	BH1-1	56401	-	1.2-1.65	✓	✓					
	BH1-6	56402	-	5.0-5.45				✓			
	BH1-8	56403	87075-1	7.5-7.95			✓				✓
	BH1-9	56404	87075-2	7.95-8.1	✓	✓				✓	
	BH1-12	56405	87075-3	12.0-12.45						✓	
BH2	BH2-2	56406	-	2.0-2.45			✓				
	BH2-9	56434	87075-5	9.0-9.45							✓
	BH2-10	56407	-	10.5-10.95					✓		
BH3	BH3-2	56408	-	2.0-2.45				✓			
	BH3-5	56409	-	5.0-5.45				✓			
	BH3-7	56410	87075-4	7.5-7.95			✓				✓

Table 21: Laboratory tests on windowless sampler borehole samples (WS101 to WS110)														
Sample hole	*Sample location*	*HML Sample ref. (S/__)*	*Chemtech sample ref.*	*Depth (m bgl)*	*Metals suite*	*PAHs by USEPA16*	*Asbestos ID*	*Pesticides*	*CBR (soaked)*	*TPH by CWG*	*Particle size distn*	*PI & MC*	*pH and sulphates*	*SOM*
WS101	WS101-1	56319	86882-1	0.1-0.3	✓	✓	✓	✓						
	WS101-2	56324	-	0.5-1.0					✓					
	WS101-4	56326	86882-8	1.3-1.7						✓				
WS102	WS102-2	56330	-	0.5-1.0					✓		✓			
WS103	WS103-1	56320	86882-2	0.2-0.4	✓	✓	✓							
	WS103-2	56331	-	0.5-1.0					✓					
	WS103-3	56332	-	1.1-1.2								✓		
WS104	WS104-2	56334	-	0.55-0.9					✓					
WS105	WS105-1	56321	86882-3	0.15-0.5	✓	✓	✓	✓						
	WS105-2	56322	86882-4	1.2-1.7								✓	✓	
WS106	WS106-1	56323	86882-5	0.1-0.4	✓	✓	✓	✓						
	WS106-2	56336	-	0.5-1.0					✓					
	WS106-3	56338	-	1.1-1.6							✓			
WS107	WS107-2	56339	-	0.5-1.0					✓					
WS108	WS108-2	56324	86882-6	0.5-1.0					✓				✓	
	WS108-3	56340	-	1.2-1.8							✓			
	WS108-5	56325	86882-7	2.6-3.0										✓
WS109	WS109-2	56341	-	0.5-1.0					✓					

6 Ground Conditions & Soil Properties

6.1 Sequence of sub-strata

The sub-strata as revealed during the site works (in BHs 1 to 3 and WSs 101 to 110) are summarised below (in Table 22 and 23). Copies of the borehole logs are presented later (in Appendix B).

Table 22: Ground conditions in windowless sampler boreholes (WS101 to WS110)

Strata:	*Topsoil*	*Made ground*	*Tidal Flat Deposits*					*Comments*
			Cohesive (SILT/ CLAY	*Granular (SILT / fine SAND*	*Cohesive (SILT/ CLAY)*	*Granular (SILT fine SAND)*	*Cohesive (SILT/ CLAY*	
Hole	*Depth to base of strata (m bgl)**							
WS101	-	0.3 (0.3)	-	1.15 (0.80)	1.30 (0.15)	3.0+ (1.70+)	-	
WS102	-	0.1 (0.1)	-	1.55 (1.45)	1.63 (0.08)	3.0+ (1.37+)	-	
WS103	-	0.2 (0.2)	0.45 (0.25)	1.05 (0.6)	1.25 (0.2)	1.7 (0.45)	3.0+ (1.3+)	
WS104	0.55 (0.55)	-	-	0.9 (0.35)	1.0 (0.1)	3.0+ (2.0+)	-	Organic odour >2.4mbgl
WS105	0.15 (0.15)	1.0 (0.85)	2.0+ (1.0+)	-	-	-	-	
WS106	0.45 (0.45)	-	-	3.0+ (2.55+)	-	-	-	
WS107	0.33 (0.33)	-	-	3.0+ (2.63+)	-	-	-	
WS108	0.32 (0.32)	-	-	3.0+ (2.68+)	-	-	-	Organic odour >2.0mbgl
WS109	0.30 (0.30)	-	-	3.0+ (2.70+)	-	-	-	
WS110	0.30 (0.30)	-	-	1.25 (0.95)	2.0+ (0.75+)	-	-	

*Strata thicknesses shown in parentheses

The windowless sampler boreholes generally show that the Tidal Flat Deposits (TFD) soils straddle the boundary between cohesive and granular soil. Some cohesive silts and clays were logged. However, most TFD soils were logged as granular silts and fine sands.

The ground conditions were relatively uniform around the site. However, more granular silts and fine sands (and less cohesive silts and clays) were noted in the south part of the site, as was reported previously (in the GEL boreholes, WS1 to WS20).

Table 23: Ground conditions in shell and auger boreholes (BH1 to BH3)						
Strata:	*Tidal Flat Deposits*				*Glacial Till*	
	(very) Soft clayey SILT	Granular silts / fine sands	(Very) soft slightly organic sandy SILT / CLAY	SAND and GRAVEL	Firm to stiff CLAY	(very) stiff CLAY
Hole	*Depth to base of strata (m bgl)**					
BH1	4.0 (4.0)	6.6 (2.6)	8.4 (1.8)	-	10.5 (2.1)	15.0+ (4.5+)
BH2	4.2 (4.2)	9.9 (5.7)	-	-	10.95 (1.05)	16.0+ (5.05+)
BH3	1.2 (1.2)	7.0 (5.8)	8.7 (1.7)	9.0 (0.3)	10.5 (1.05)	15.0+ (4.5+)

The shell and auger borehole logs show that the Tidal Flat Deposits (TFD) are underlain by Glacial Till, as expected. The transition depth is between about 8.4m and 9.9m bgl. They also show a 1.7m- to 1.8m-thick band of slightly organic (i.e. slightly peaty) silts and clays at the base of the TFD in two locations (BH1 and BH3).

6.2 Topsoil

Topsoil was found across most of the site just below the ground surface. It was logged in seven of the 10 windowless sampler boreholes to depths of between 0.15m and 0.55m bgl.

6.3 Made ground

A layer of made ground – usually thin – was revealed in some parts of the northern portion of the site. It was logged in four boreholes (WSs 101, 102, 103 and 105). In three boreholes (WSs 101 to 103), the made ground was found to be very thin (0.1m to 0.3m in thickness), but in WS105, it was found to reach 1.0m bgl.

The made ground was typically found to comprise almost entirely of natural sand or sand and gravel. In WS101 and WS102, the thin sand made ground was underlain by a geotextile membrane. In WS103, the made ground comprised the sub-base to the hardstanding in the main vehicle loading / parking area in the northwest corner of the site. In WS105, the made ground comprised reworked sands to about 0.95m bgl, underlain by a thin layer of pea gravel (possibly installed as a French drain).

Generally, there was no anthropogenic material (e.g. bricks, tile, concrete) found within the made ground and there was no visual or olfactory evidence of contamination. Nevertheless, there was a slight organic odour within WS105.

6.4 Tidal Flat Deposits (TFD)

Thick Tidal Flat Deposits (TFD) were revealed across the site. They were logged in all exploratory holes at shallow depth and were found to reach depths of between 8.4m (in BH1) and 9.9m (in BH2) bgl.

As discussed, the TFD were found to be close to the borderline between cohesive (i.e. clayey and mouldable) and granular (i.e. sandy and cohesionless) soil. Nevertheless, most of the logged soils were recorded as granular (silts and fine sands).

Interestingly, in the north half of the site, the shallow TFDs were found to vary locally between cohesive silts/clays and granular silts/sands. However, in the south half of the site, the TFD soils were found to almost entirely consist of (non-shrinkable) granular silts and sands, as shown in previous GEL boreholes (e.g. WSs 10 to 20) and recent windowless sampler boreholes (e.g. WSs 106 to 109).

The deeper TFD soils in the south half of the site also appear to be dominated by granular silts and fine sands. The deep shell and auger borehole in the far southeast of the site (BH3) revealed exclusively granular TFD soils except for a basal layer of very soft slightly organic silt/clay (7.0m to 8.7m bgl). BH2, near the middle of the site, revealed only granular TFD soils at depths greater than 4.2m bgl.

Interestingly, a basal layer of very soft slightly organic silt/clay (similar to the one in BH3) was also revealed in BH1. It was found to have a similar thickness (1.7m) and depth (6.6m bgl). Therefore, a deep band of very soft slightly organic silt/clay could extend across much of the site (although no such layer was logged in BH2).

6.4.1 Cohesive TFD soils

Cohesive TFD soils include the clayey silt, silt/clay and silty clay revealed around the site (above the Glacial Till). With such fine-grained soils, it is the undrained strength that tends to be more critical (rather than the drained frictional strength). The long-term consolidation compressibility is also a factor as is the plasticity and moisture content.

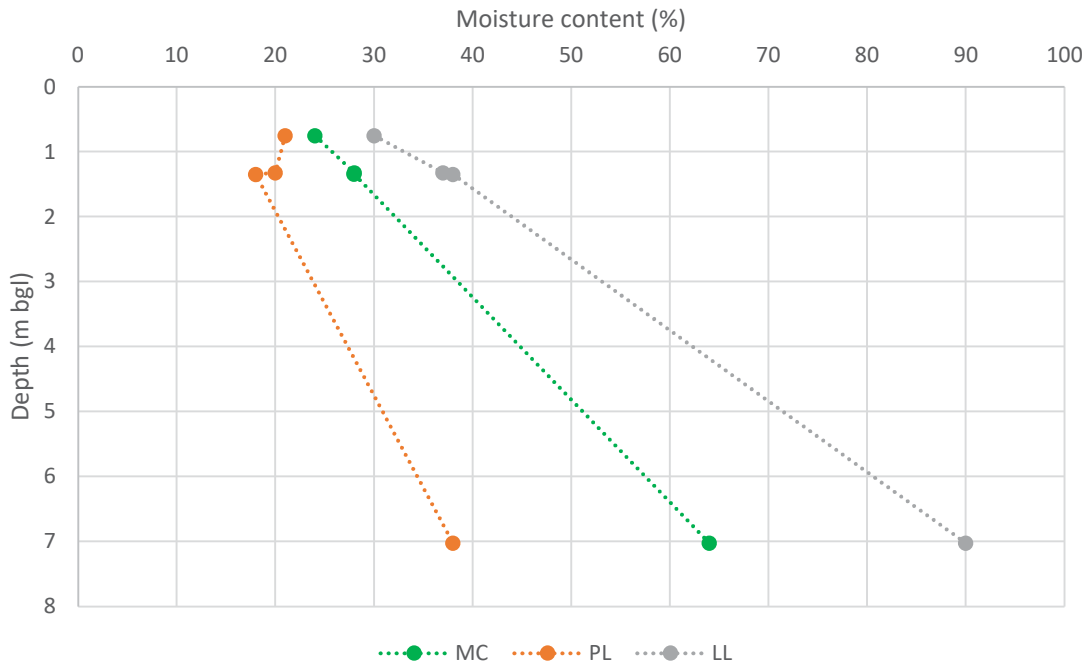
Plasticity and moisture content

A total of four samples of the cohesive TFD soils were tested for plasticity index and moisture content, the results of which are shown later (in Appendix C). A summary is shown below (in Table 24). These tests can determine the shrinkability of the TFD clays.

Table 24: Plasticity and moisture content test results for GLD clays									
Sample geology	*Sample location*	*Depth (m bgl)*	*HML Sample reference*	*Natural moisture content (%)*	*Plastic limit (%)*	*Liquid limit (%)*	*Plasticity index (%)*	*% of sample passing 425µm sieve*	*Modified plasticity index (%)*
TFD clay	WS103-2	0.5-1.0	56322	24	21	30	11	100	11
	WS105-2	1.2-1.7	56322	28	18	38	20	100	20
	BH1-1	1.2-1.65	56401	28	20	37	17	100	17
	BH1-9	7.95-8.1	56404	64	38	90	52	100	52

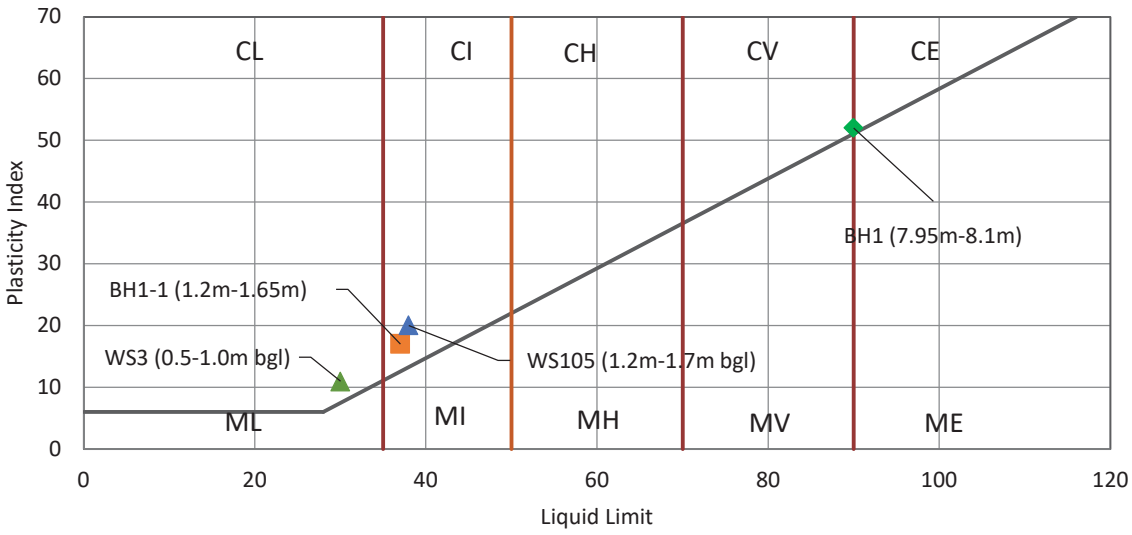
The moisture content of the TFD clay samples appears to increase with depth, as shown below (in Figure 2). The moisture content is between 24% and 28% at depths less than 1.7m bgl. However, it increases to 64% at about 8m bgl. Perhaps the slightly organic silts/clays at greater depth have higher water contents.

***Figure 2:* Moisture content against depth**



The liquid limit and plasticity index of the deeper (very soft) slightly organic clay appears to be much greater than in the shallow (soft / firm) TFD clays. The plasticity index at shallow depth is generally between 11% and 20%. However, the sample of deeper peaty clays at about 8m bgl was found to have a plasticity index of about 52%. The plasticity chart below (in Figure 3) shows that the shallow samples of TFD silts/clays would be classed as low to intermediate plasticity (silty) clay while the deeper peaty sample would be classed as very to extremely high plasticity silt/clay. However, it is normal for deeper, softer (more organic) clays to have much higher moisture contents and plasticity indices.

Figure 3: **Plasticity data for TFD cohesive soils**



The calculated modified plasticity index (i.e. the plasticity index multiplied by the fraction of sample passing a 425um sieve) of the shallow TFD cohesive silt/clay samples is between 11% and 20%. Thus, the shallow TFD cohesive material is shrinkable and could be categorised as being of low volume change potential (based on NHBC Standards).

However, previous testing in the GEL report shows some samples of TFD clays were of medium shrinkability. Therefore, it would be prudent to assume medium volume change potential.

Undrained shear strength, c_u

Undrained shear strength in the TFD cohesive silts/clays can be determined in several ways using the site test data. This includes shear vane test results in U100 samples and windowless sampler tube material. The results of penetration tests (e.g. SPTs and DCP tests) can also be used to give a rough estimation of undrained shear strength.

Measured hand vane shear strengths in the TFD cohesive silts/clays decrease quite dramatically with depth, as shown below (in Table 25). The raw results range from up to 130kPa (at 0.9m bgl) down to 4kPa (at 2.2m or 7.6m bgl).

The shear vane results should be corrected for plasticity. Using a formula from Morris and Williams (1994), the corrected values range from 2kPa to 78kPa, as shown below (in Table 25).

Table 25: Shear vane results in clays			
Test location	*Depth (m bgl)*	*Raw results (kPa)*	*Corrected results (kPa)*
BH1	7.6	4	**2**
BH2	2.2	4	**2**
BH3	7.6	8	**5**
WS102	1.9	14	**8**
	2.9	23	**14**
WS104	0.9	130	**78**
Note: correction based on formula from Morris and Williams (1994) with a plasticity of 50%			

Some of the SPTs were mainly carried out in the TFD granular silts, sands and gravels, but some mostly tested the cohesive silts/clays of the TFD, as shown below (in Figure 4). SPT N values mainly in the silty clays ranged between 0 and 3 (blows per 300mm penetration).

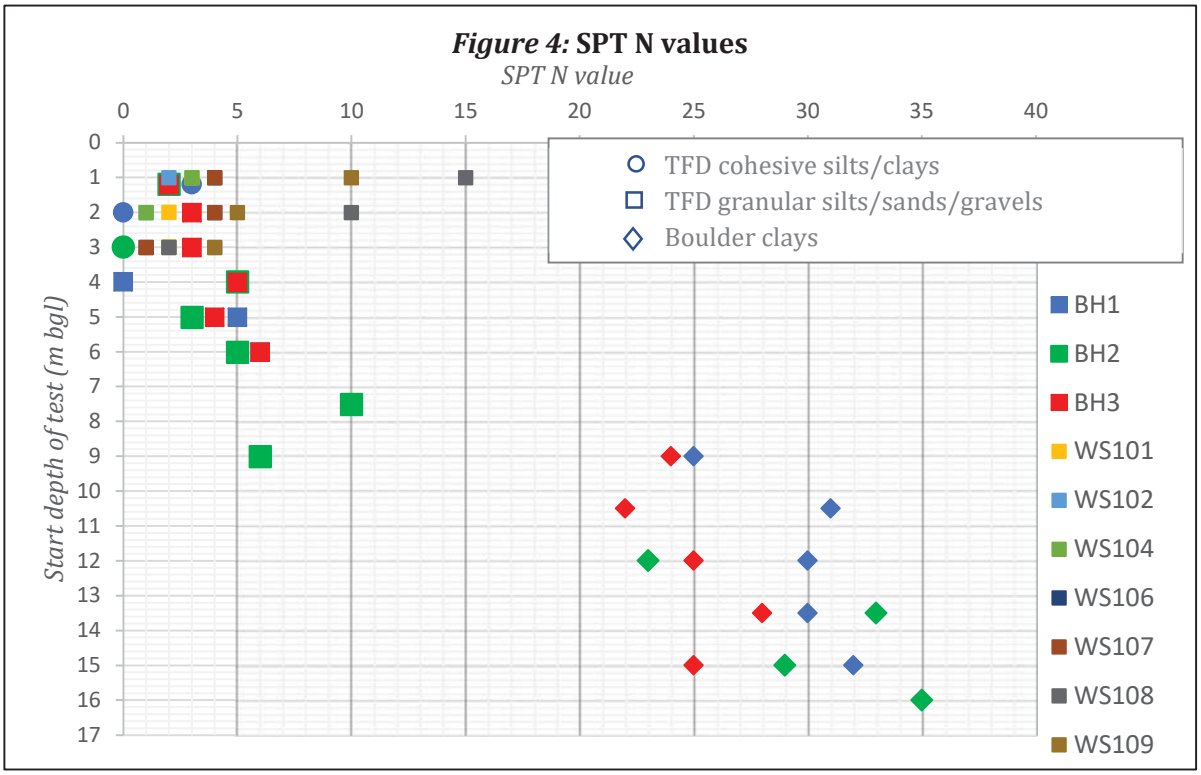


***Figure 4:* SPT N values**

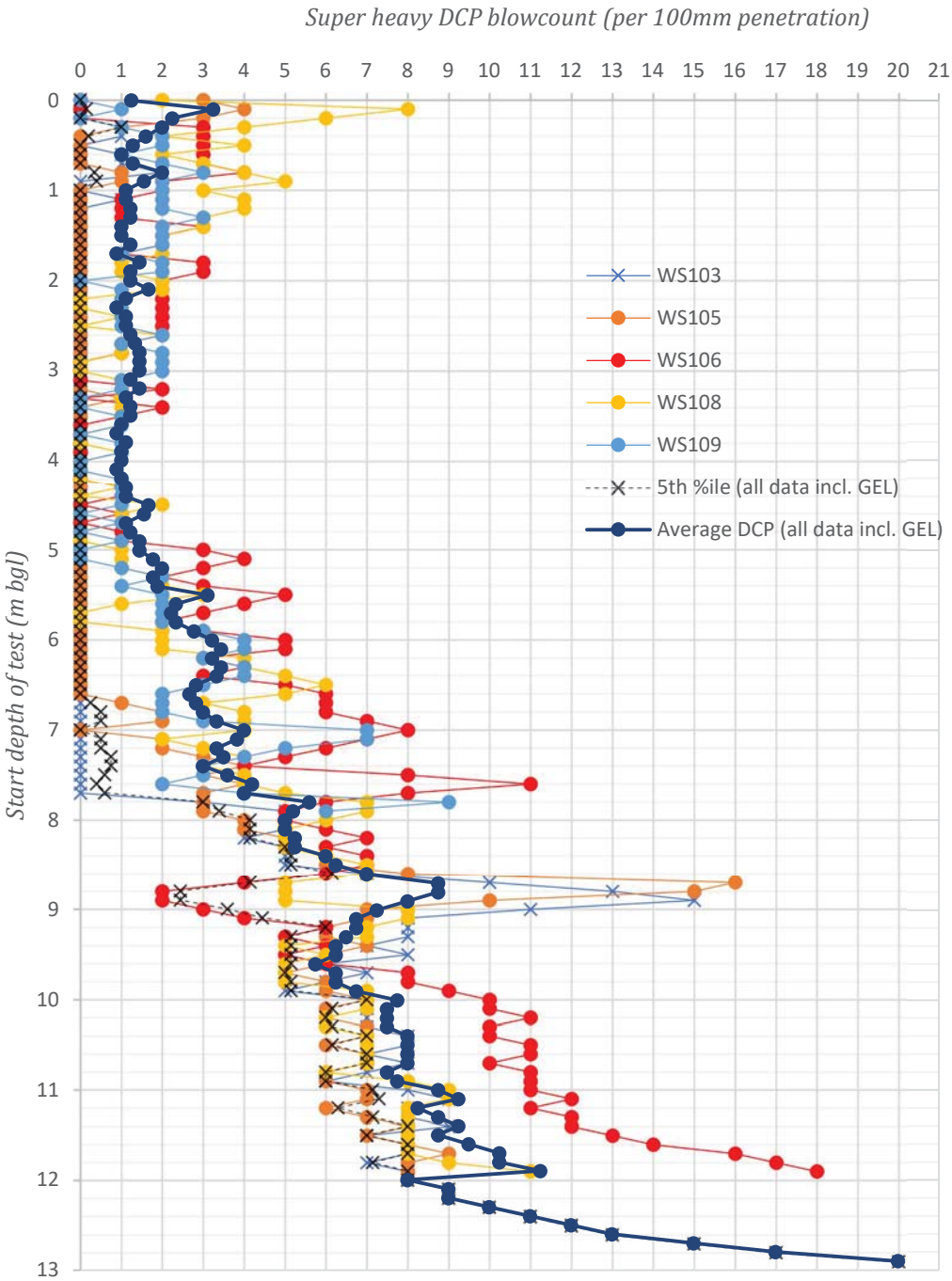
The undrained shear strengths of the underlying TFD cohesive silts/clays can be roughly correlated from SPT N values (e.g. using Stroud, 1976), as shown below (in Table 26). The estimations show that the undrained shear strength could be about 10kPa to 15kPa at about 1.5m bgl and decrease to less than 5kPa at depths greater than about 2.3m bgl.

Table 26: Estimated strength of TFD cohesive soils from SPT correlations			
Test location	*Average SPT depth (m bgl)*	*SPT N value*	*Estimated undrained shear strength (kPa)*
BH1	1.5	3	15
	2.3	0	<5
BH2	1.5	2	10
	3.0	0	<5
Note: uses a rule-of-thumb factor of 5 x N value = c_u (kPa). Average depth is 0.3m plus starting test depth.			

Super heavy DCP testing is not a very reliable way to determine undrained shear strengths of clays, especially at extremely low to very low strengths. However, the results can be used to provide rough indications of strength and can be used to support other means of strength assessment.

The DCP results indicate that the TFD soils have very little resistance to penetration between depths of about 2m and 5m bgl, as shown below (in Figure 5). There is a blowcount of mostly between 0 and 3 (blows / 100mm) to a depth of about 2m bgl. The 5th percentile of the combined data (including previous DCP results from the GEL investigation) show a possible characteristic DCP blowcount of about zero blows/100mm between ground surface and about 7m bgl.

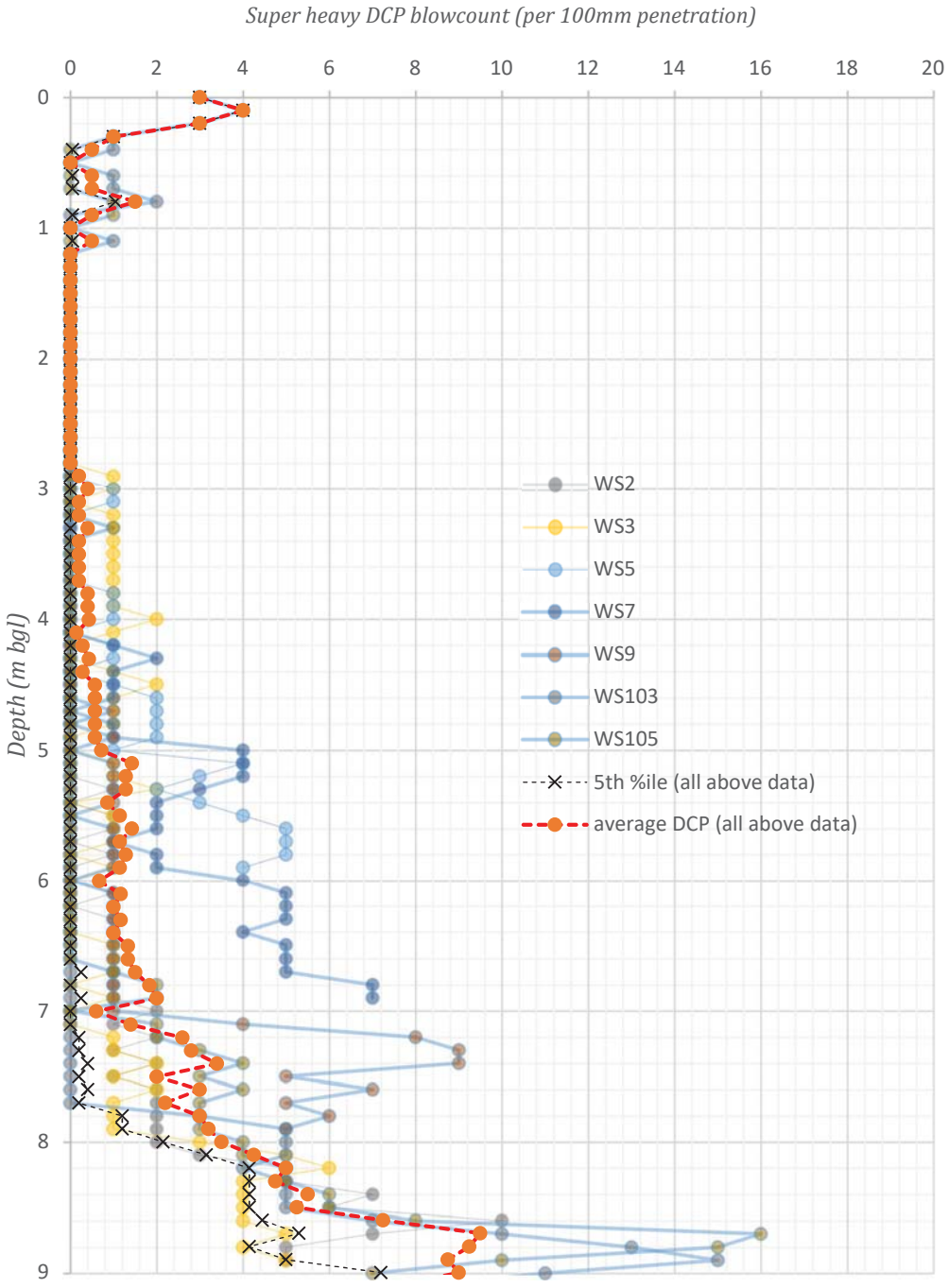
***Figure 5: DCP* results in recent investigation**



The undrained shear strength of the TFD cohesive soils can also be roughly determined using the results of DCP testing. The boreholes where the ground conditions were mainly of cohesive silts and clays tended to be in the north area of the site. Thus, the results of DCP testing in the north part of the site (in WS103, WS105, WS2, WS3, WS5, WS7, WS9) show that the average blowcount is 0 to 4 between 1m and 1.3m bgl and zero from 1.3m to 7m bgl, as shown later (in Figure 6). This suggests the undrained shear strength was

probably low to a depth of about 1.3m bgl and then extremely/very low (e.g. <15kPa) at greater depth.

Figure 6: **Combined DCP results in north part of the site**



A characteristic value is a cautious or conservative value that is suitable for design calculations (e.g. according to Eurocode 7). A suitable characteristic undrained shear strength for the TFD cohesive silts/clays to a depth of about 1.5m bgl could be about 20kPa, but this could decrease to <15kPa for depths greater than about 1.5m bgl. In fact,

the deeper TFD silts/clays – anticipated in the north part of the site – could have extremely low strengths (<10kPa) at depths greater than about 1.5m bgl.

Coefficient of volume compressibility, mv

The results of consolidation testing (as shown later, in Appendix C) suggest that the coefficient of volume compressibility, m_v, of the TFD cohesive silts/clays is greater in the deeper very soft slightly organic soils than in the shallower very soft silts and clays. For instance, the m_v value for a 50kP to 100kPa stress range is about 0.54m²/MN in the shallow soft clays (about 2m bgl) but is about 1.2m²/MN in the deep very soft clays (about 7.5m bgl), as shown below (in Table 27).

The measured m_v values are reasonably typical for tidal, estuarine or alluvial clays. According to Carter and Bentley (1991), m_v values for (not highly organic) alluvial silts/clays tend to be between 0.3 m²/MN and 1.5m²/MN. Highly organic silts/clays often have m_v values higher than 1.5m²/MN.

Table 27: Summary of consolidation test results on TLD silts/clays

Borehole:	BH1		BH2		BH3	
Sample depth (m bgl):	7.5 – 7.95		2.0 – 2.45		7.5 – 7.95	
Pressure range (kPa)	m_v *in* MN/m^2	c_v *in* $m^2/year$	m_v *in* MN/m^2	c_v *in* $m^2/year$	m_v *in* MN/m^2	c_v *in* $m^2/year$
0 – 12	9.2	3.0	3.7	7.8	6.6	3.1
12 – 24	3.2	2.4	1.2	4.6	2.5	2.6
24 – 47	2.0	2.6	0.83	9.7	1.7	2.9
47 – 95	1.2	2.3	0.54	11.4	1.2	2.6
95 – 189	0.56	2.3	0.29	16	0.54	2.6
189 – 378	0.33	2.6	0.13	17	0.31	2.9
378 – 95 (unloading)	0.07	4.2	0.028	30	0.08	4.8

The consolidation test samples from BH1 and BH3 were from the deep very soft slightly organic silts/clays. These results can be combined to predict likely m_v values in this deep soil layer, as shown below (in Figure 7). Figure 7 shows, for example, that if the soil were loaded from 50kPa to 75kPa, an m_v value of about 1.15m²/MN could be suitable.

The consolidation test samples from BH2 were from the shallower (very) soft silts/clays (2.0m to 2.45m bgl). The results can predict likely m_v values in this shallow soft layer, as shown below (in Figure 8). Figure 8 shows, for example, that if the soil were loaded from 50kPa to 75kPa, an m_v value of about 0.6m²/MN could be suitable.

Figure 7: **Deep TFD very soft peaty silt/clay - m_v values against vertical stress (during loading)**

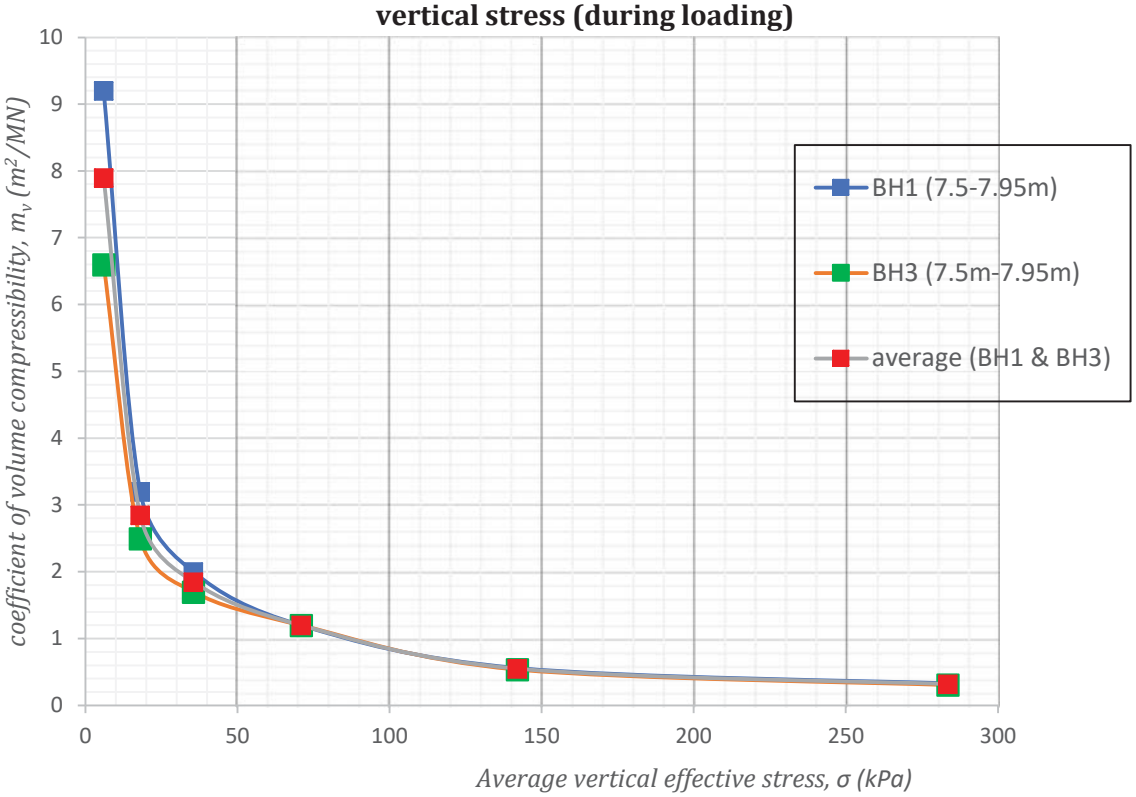
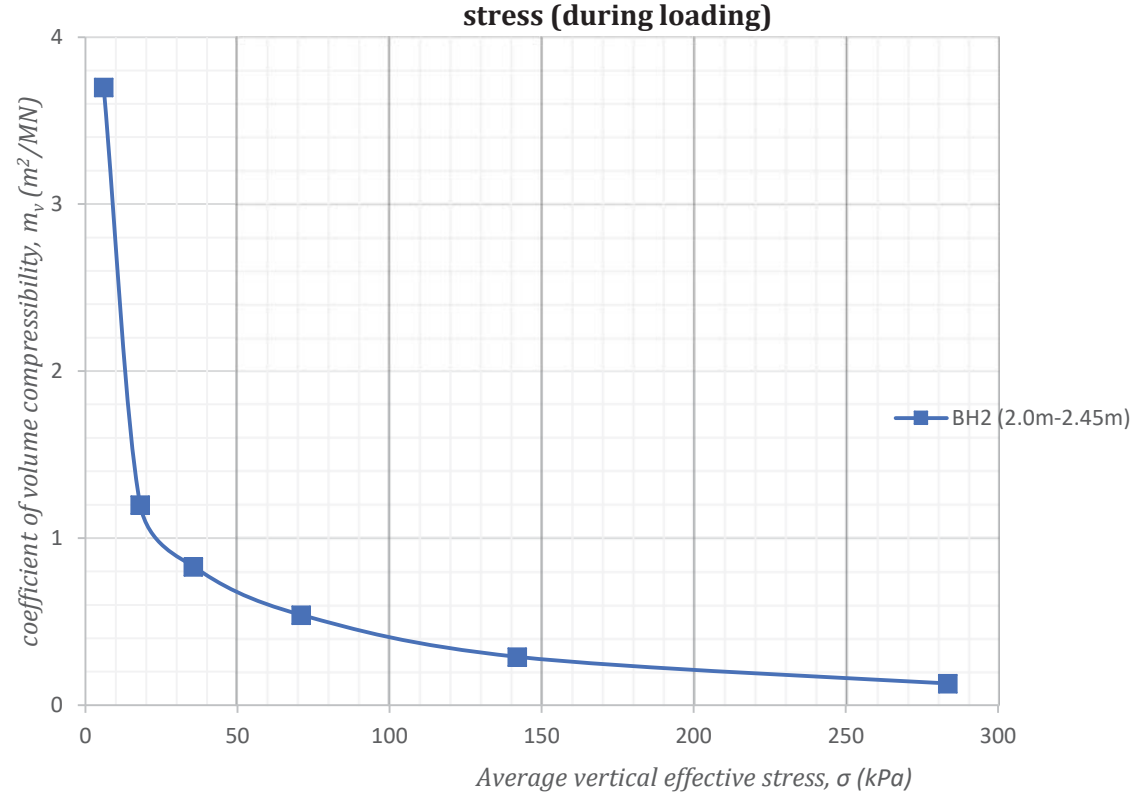


Figure 8: **Shallow very soft TFD silt/clay - m_v values against vertical stress (during loading)**



Coefficient of consolidation, c_v

The coefficient of consolidation, c_v, was found to generally range between 2 and 4 m^2/year for the deeper very soft slightly organic cohesive soils, as shown above (in Table 27). Higher c_v values of between 5 and 30m^2/year were recorded in the shallower firmer silts/clays. Lower values tend to be associated with longer times for consolidation.

The c_v values can be used to determine the time required to achieve a specific degree of consolidation. For instance, assuming a maximum drainage path of 4m to the closest drainage layer, it would take a layer of clay with a c_v value of 4.0 m^2/year about 3.4 years to achieve 90% consolidation.

Permeability

The consolidation test results indicate that the permeability of the clay samples was between about 1 x 10^{-10} m/s and 9 x 10^{-9} m/s. These results are less reliable than standard permeability tests and should be used with caution. Nevertheless, the results are reasonably typical for a silty clay soil.

6.4.2 TFD granular silts/sands

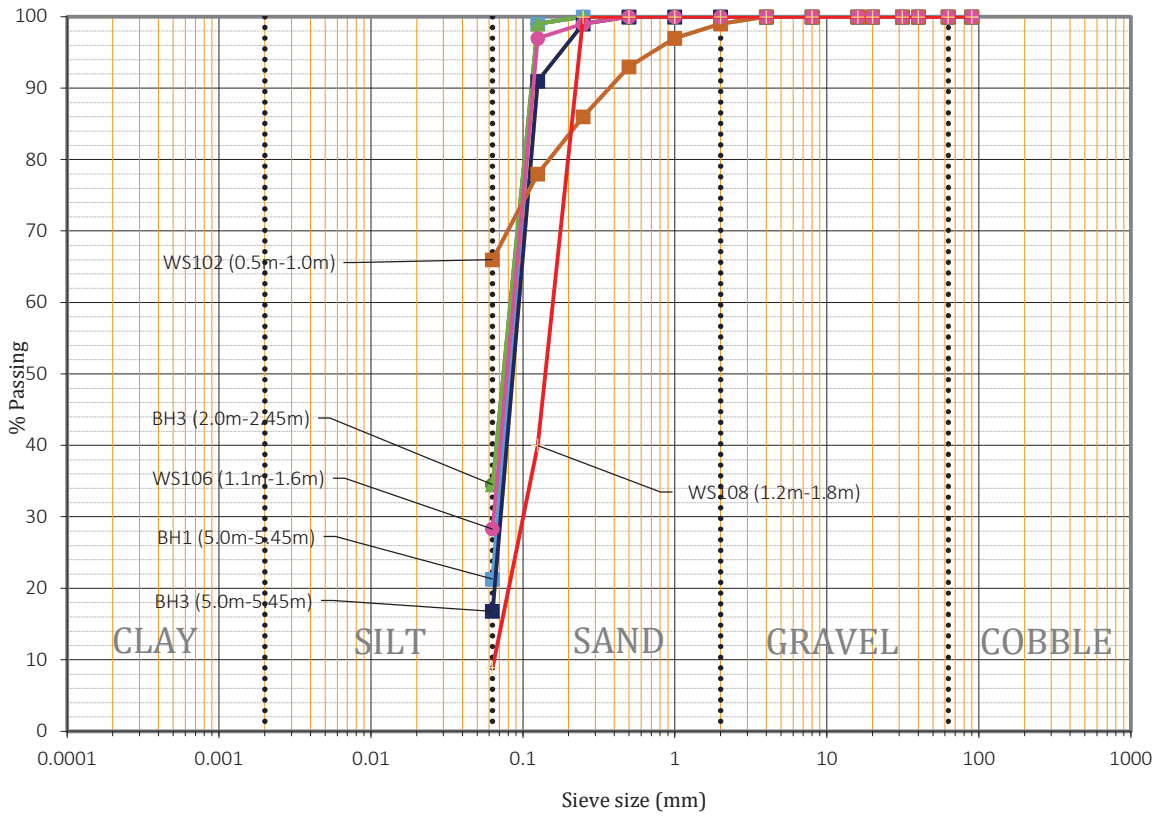
Important engineering features of sands and granular silts tend to be the particle size distribution, relative density and particle angularity. These properties will usually determine the strength and stiffness.

Particle size distribution

Particle size distribution tests (namely wet gradings) were carried out on six samples of the TFD granular silts and sands. The complete results are presented later (in Appendix C). The findings, as shown below (in Figure 9), indicate the samples roughly contained 0% gravel, 34% to 91% sand and 9% to 66% fines (of silt or clay). The TFD granular soils are generally shown to range between a sandy SILT and a very silty fine SAND.

The gradings indicate all the test samples are granular in nature except for the sample from WS102 (from the north part of the site). This confirms the findings of the previous GEL investigation (see Table 7 above) which found **the shallow TFD soils in the south half of the site are comprised entirely of granular (non-shrinkable) soil**.

Figure 9: **Grading curves for granular silts and fine sands**



The findings of the grading curves suggest the TFD silts and sands have a narrow spread of particle size. The soils are mostly very closely- or poorly-graded.

Relative density and SPT N value

The relative density of the revealed TFD granular silts and sands was generally very loose or loose, as shown in the borehole logs (in Appendix B). SPT N values in the granular TFD soils ranged between 0 and 15, as shown below (in Figure 10).

There appears to be some variation in recorded SPT N values with depth in the TFD granular soils, as shown below (in Figure 10). However, a generally suitable characteristic (i.e. cautious value for) N value could be 2 (blows / 300mm) while an average SPT N value is about 4 (blows/300mm) at shallow depth.

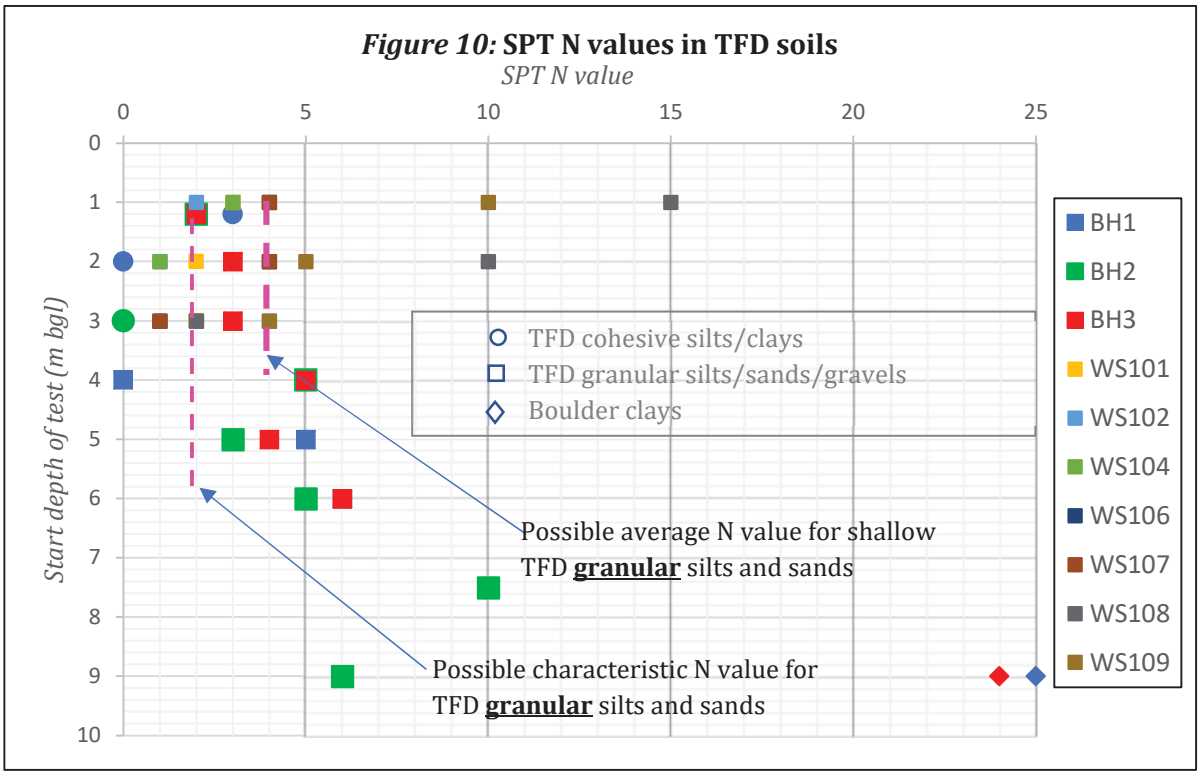


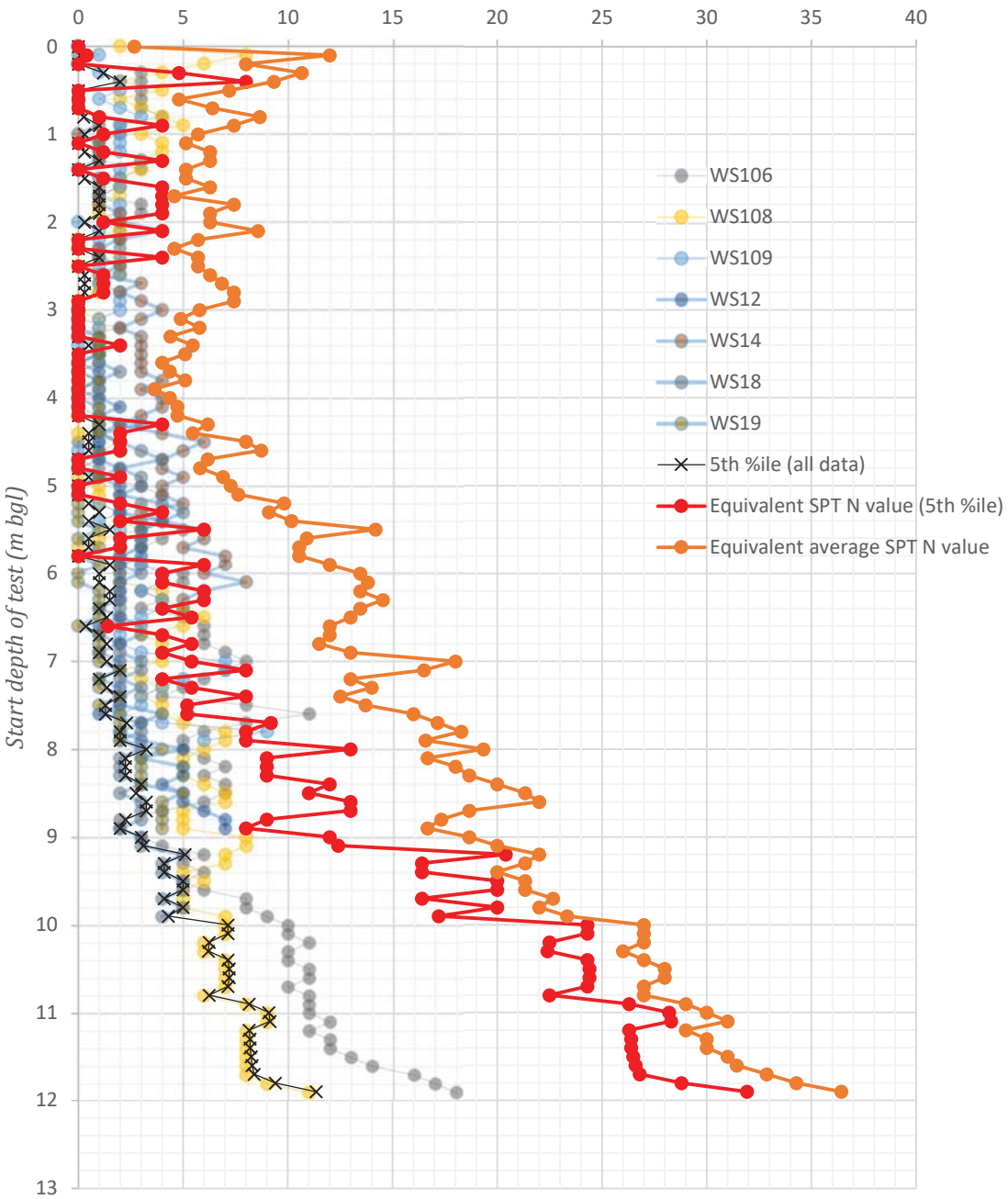
Figure 10: **SPT N values in TFD soils**

The relative density of the TFD granular soils can also be evaluated from the results of DCP probe testing. The combined DCP test results from the south part of the site – where granular silt and sand ground conditions were found to predominate – indicate that the blowcount ranged between 0 and 8 (per 100mm) between ground level and about 7.5m bgl, as shown below (in Figure 11).

At depths greater than about 7.5m bgl, the DCP blowcounts tended to increase. This suggests the TFD soils were becoming more resistant to penetration or the material was starting to change into a different stratum (e.g. Glacial Till).

Figure 11 (below) also shows equivalent SPT N values (using the findings of Cearns and McKenzie, 1988) for the 5th percentile DCP blowcounts. This shows that the minimum relative density could occur between 3m and 4m bgl, where the equivalent (5th percentile) SPT N value is about zero. Nevertheless, an approximate overall suitable characteristic equivalent SPT N value between 1m and 7m bgl could be 1 blow per 300mm. Similarly, an average SPT N value overall in the TFD granular silts/sands could be about 6 (blows/300mm).

Figure 11: **Combined DCP results in south part of the site**



The peak effective angle of friction, φ'_{pk}, of granular soil depends on (a) angularity of particles, (b) particle size distribution and (c) relative density. The *characteristic* peak effective angle of shearing resistance, $\varphi'_{pk,k}$, is a cautious estimate of the actual φ'_{pk}.

Given the logged descriptions of angularity and the results of particle size analyses and SPT N values, 28 degrees should be a suitably conservative value of $\varphi'_{pk,k}$ for the revealed

TFD granular silts and sands. A *constant volume* angle of friction, $\varphi'_{cv,k}$, of 28 degrees should also be acceptably conservative.

6.5 Glacial Till (boulder clay)

Glacial Till was revealed at depth below the site in all three shell and auger boreholes (BHs 1 to 3). It was found below depths ranging between 8.4m (BH1) and 9.9m (BH2) bgl and was logged to the full depth of exploration (of 15m to 16m bgl).

The Glacial Till was encountered in the form of boulder clay. It was typically recorded as: "Stiff to very stiff brown mottled grey silty CLAY with occasional stone gravel."

6.5.1 Plasticity and moisture content

No plasticity testing was carried out on the boulder clay material. Nevertheless, typical plasticity indices in the local boulder clay are known to typically range between 15% and 30%.

A moisture content test was carried out on a triaxial sample from BH2 (10.5-10.95m bgl). The water content was measured at 20% which is – unsurprisingly – less than the water content in the overlying much softer TFD clays.

6.5.2 Undrained shear strength, c_u

The undrained shear strength of the boulder clay can be assessed from the results of site testing, lab testing and material descriptions. SPTs and DCP tests can be used to give a rough indication of strength while triaxial compressive tests could give a more reliable estimation. The logged descriptions of stiffness should also give a general idea of strength.

SPT N values in the boulder clay ranged between 22 and 35 blows per 300mm at depths of between 9m and 16m bgl, as shown above (in Figure 4). This suggests that the undrained shear strength could be between about 110kPa and 175kPa. Below a depth of 13m bgl, the SPT N value was at least 25 (blows/300mm) which suggests the strength was at least 125kPa.

A quick triaxial compressive strength test (i.e. undrained unconsolidated test) was carried out on a U100 boulder clay sample from between 10.5m and 10.95m bgl in BH2. The results (in Appendix C) show the strength was measured at 189kPa.

The logged descriptions of the boulder clay indicate the material is initially stiff becoming stiff to very stiff. This latter description suggests undrained strengths of at least 150kPa.

Determinations of undrained shear strength have been calculated using the DCP test results, as shown later (in Appendix D). The findings indicate that any stiff cohesive clays at depths greater than 9m bgl should have an undrained shear strength of 150kPa. At depths greater than 12m, it should have a strength of at least 200kPa.

Accordingly, a suitable characteristic shear strength for the boulder clay could be about 75kPa down to a depth of about 9m bgl. At depths between 9m and 12m bgl, a suitable characteristic undrained shear strength (or c_u value) could be 150kPa increasing to 200kPa below 12m bgl.

6.6 Groundwater observations

Groundwater was encountered during site works within all ten of the windowless sampler boreholes (WS101 to WS110). The recorded groundwater level ranged between 0.89m and 1.84m bgl, as shown below (in Table 28).

No final groundwater levels were recorded in the deeper shell and auger boreholes (BH1 to BH3). This is probably due to the use of casing, which seals off the boreholes during their advancement.

Table 28: Summary of groundwater observations

Location	*Prevailing ground conditions*	*Water level after drilling (m bgl)*	*During monitoring (dd/mm/yyyy)*					
			03/07/ 2020	*17/07/ 2020*	*30/07/ 2020*	*13/08/ 2020*	*28/08/ 2020*	*11/09/ 2020*
WS101	Sandy SILT	1.30	1.26	1.27	2.32	1.78	0.97	1.38
WS102	Sandy SILT	1.20	1.10	1.06	2.54	1.38	0.71	1.13
WS103	Clayey SILT	1.34	-	-	-	-	-	-
WS104	Sandy SILT	1.87	-	-	-	-	-	-
WS105	Clayey SILT	0.89	-	-	-	-	-	-
WS106	Silty SAND	1.80	-	-	-	-	-	-
WS107	Silty SAND	1.84	1.83	1.88	2.29	2.18	1.91	1.82
WS108	Silty SAND	2.12	N/A	2.12	2.20	2.27	2.27	2.28
WS109	Silty SAND	1.68	0.74	1.80	-	-	-	-
WS110	Sandy SILT	1.82	-	-	-	-	-	-
BH1	Clayey SILT	-	-	-	-	-	-	-
BH2	SILT	-	-	-	-	-	-	-
BH3	SAND/SILT	-	-	-	-	-	-	-

Groundwater monitoring has indicated groundwater levels of between 0.71m and 2.54m bgl, as shown above (in Table 28).

Accordingly, it seems groundwater levels are between about 0.8m and 2.5m below existing ground levels. The site is very flat, so the discrepancy in groundwater levels suggests that groundwater is of limited mobility and interconnectivity. That is, groundwater is likely to generally flow quite slowly around the site.

The groundwater monitoring results seem more constant in the silty sands in WS107 and WS108 (in the south half of the site) with recorded levels between 1.82m and 2.29m depth. More varied results were observed in the sandy silts in WS101 and WS102 where levels ranged between 0.71m and 2.54m bgl. Overall, the groundwater table could be between about 1.8m and 2.3m bgl, as indicated in the sands in the south part of the site. The more varied levels found in the north part of the site could reflect the likely lower permeability ground conditions and presence of perched water.

Groundwater levels are often subject to seasonal or annual variations. Nevertheless, the recently recorded groundwater levels broadly coincide with the previous levels (of 1.2m to 1.9m bgl) reported in the GEL investigation.

6.7 CBR test results

The results of CBR laboratory tests (on remoulded samples in a soaked condition) are presented below (in Table 29). The complete results are in Appendix C.

Table 29: Summary of CBR test results

Location:		WS101	WS102	WS103	WS104	WS106	WS107	WS108	WS109
Natural or soaked:		S	S	S	S	S	S	S	S
Sample depth:		0.5-1.0	0.5-1.0	0.5-1.0	0.5-1.0	0.5-1.0	0.5-1.0	0.5-1.0	0.5-1.0
Soil type:		SILT/ CLAY	Silty SAND	Silty SAND	Silty SAND	SILT/ CLAY	Silty SAND	Silty SAND	SILT/ CLAY
CBR (%)	*Top*	**0.5**	**1.1**	0.6	**20**	**2.0**	**2.1**	**35**	**9.2**
	Bottom	**0.8**	**1.5**	**0.6**	**28**	**4.7**	**5.0**	**37**	**31**
Moisture content (%)	*Top*	21	23	23	22	22	25	23	22
	Bottom	21	22	21	21	21	26	23	22
Swelling (mm)		0.57	0.44	0.83	0.04	0.12	0.27	0.3	0.14

Many of the CBR test samples (e.g. from WS106 to WS109) are from the south part of the site where sandy ground conditions tend to prevail. This includes the samples from. CBR values for the soils in the south part of the site range between 2.0% and 37%.

Many of the CBR test samples (e.g. from WS101 to WS104) are from the north part of the site where more clayey ground conditions tend to prevail. CBR values for the soils in the north part of the site range between 0.5% and 28%.

The recent results tend to coincide with the findings from the previous GEL investigation, as shown previously (in Table 8).

7 Settlement analysis

7.1 Design methodology

The settlement analyses include two basic elements which are assumed to combine to roughly equal the total settlement:

- Consolidation settlement of cohesive silts/clays
- Long term settlement of granular sands/silts

Therefore, the settlement analysis for each ground model considers the cohesive and granular soil layers separately.

7.1.1 Foundations in cohesive silts/clays

With shallow strip or pad foundations in reasonably firm cohesive silts and clays, the common design approach is to determine a safe bearing capacity using the undrained shear strength, c_u, of the *in situ* clays. If a suitably large factor of safety is used (typically, 3), then settlements are usually assumed to be tolerable (e.g. less than 25mm). Thus, the safe bearing capacity can be adopted as the 'allowable bearing pressure' (or design bearing resistance).

On the other hand, with wide (raft) foundations, it is often settlement which determines the allowable bearing pressure. This is because a wider foundation has a deeper zone of influence (than with narrow strip foundations) and can cause settlement down through a large thickness of soil strata. Therefore, with wide (raft) foundations, the settlement often needs to be directly evaluated to check the foundation will not move excessively.

Settlements in clays tend to take place over a lengthy time period. This is because clay usually needs substantial time to consolidate. Pore water within clays need to drain out to allow the clay particles to adopt a more compact structure, but the pore water can only drain very slowly. Longer term creep settlement can also occur.

Total settlements within a clay stratum can be estimated using Terzaghi's one-dimensional consolidation analysis, as summarised by the equation below. Research shows that such analyses are reasonably accurate provided the applied stress is not much greater than about one-third of the ultimate bearing capacity (Burland, 2012). However, if the one-third bearing capacity limit is significantly exceeded, greater settlements should be expected compared to those predicted by Terzaghi's method.

$$Settlement, \rho = \sum \Delta\sigma_v m_v \Delta h$$

where

- $\Delta\sigma_v$= change in vertical stress in a soil layer
- m_v = coefficient of volume compressibility
- Δh = soil layer thickness

7.1.2 Foundations in granular soils (e.g. sand)

It is usually the settlement in sandy granular soils that limits the allowable bearing pressure, although settlements in sand are generally lower and occur almost instantly, compared to settlements in clays. With foundations in sand, the safe bearing capacity is often high due to the greater inter-particle frictional strength of sands, compared with foundations in cohesive silts and clays. Therefore, the common approach in sands – for all types of shallow foundation – is to calculate the expected settlements, typically from the results of *in situ* strength tests (e.g. SPTs), to determine the allowable bearing pressure.

7.1.3 Foundation dimensions and stress influence factors

The possible raft foundations are assumed to be founded at shallow depth not far beneath the existing ground surface. The increase in soil stress at depth below the rafts caused by the loading from the proposed structure can be calculated using Boussinesq theory (which assumes an elastic semi-infinite soil behaviour). This method tends to be reasonably representative of *in situ* stresses unless there is a stiff shallow soil stratum, in which case it can overestimate deeper soil stresses.

Vertical stress influence factors have been calculated for foundation dimensions based on Boussinesq theory, as shown below (in Table 30). The considered foundation arrangements include a 12m by 12m wide raft as well as a 12m by 6m wide raft.

The increase in vertical stress at a depth below the centre of a raft can be calculated by multiplying the (assumed uniformly distributed) foundation loading (near the surface) by the influence factor, I_f, as shown in the equations below (within Table 30). For example, the vertical pressure increase 2.0m below the centre of a 12m wide square raft with a 100kPa load would be 97.6kPa (i.e. $\Delta\sigma_v$ x I_f = 100 x 0.976).

Table 30: Stress Influence factors below centre of a raft foundation												
Plan area foundation dimensions (m)	*Depth below footing level, z (m)*											
	0.5	*1.0*	*1.5*	*2.0*	*3.0*	*4.0*	*5.0*	*6.0*	*7.0*	*8.0*	*9.0*	*10.0*
	Vertical stress influence factor, I_f											
12 x 12	**1.000**	**0.997**	**0.989**	**0.976**	**0.930**	**0.863**	**0.783**	**0.700**	**0.622**	**0.549**	**0.484**	**0.428**
12 x 6	**1.000**	**0.985**	**0.956**	**0.913**	**0.800**	**0.680**	**0.572**	**0.481**	**0.405**	**0.343**	**0.293**	**0.252**

$$\Delta\sigma_v = qI_f$$

Where: $\Delta\sigma_v$ = change in vertical stress beneath centre of footing
q = imposed load on footing
z = depth below footing
I_f = vertical stress influence factor

The vertical soil stresses below a rectangular (12m by 6m) raft will be significantly lower than beneath a square (12m by 12m) raft with the same foundation loading. For example, at a depth of 8m, the increased soil stress will be only 34.3% of the imposed foundation load for the 12m by 6m raft but 54.9% of the foundation load for the 12m by 12m raft.

7.2 Adopted ground models

The following general ground models are based on consideration of the available information about the *in situ* ground conditions, as shown above (in Section 5). Different ground models are adopted for the north and south areas of the site to reflect the different ground conditions revealed. These have been selected to represent some of the more conservative ground conditions in the two halves of the site. For settlement analysis, reasonably representative average values have been employed.

7.2.1 North area of the site

Table 31 below shows the assumed ground model for the north area is of the site. This includes much soft and very soft cohesive clay layers at shallow depth, e.g. as revealed in BH1.

Table 31: North area: ground model for settlement analysis						
Layer	*Depth to top of stratum (m bgl)*	*Material*	*Layer thickness (m)*	*Unit weight (kN/m3)*	*in situ overburden at base (kPa)*	*Soil properties*
1	0.0	Soft clay (dry)	1.0	17	17.0	**m_v = 0.3m²/MN**
2	1.0	Soft clay	0.5	17	20.5	
3	1.5	Very soft clay	2.7	16	36.7	**m_v as per Figure 8**
4	4.2	Sandy SILT	2.4	18 (dry) 20 (sat.)	60.7	**Average SPT N value = 6**
5	6.6	Very soft SILT/CLAY	1.8	16	71.5	**m_v as per Figure 7**
6	8.4	Firm to stiff clay	1.6	19	85.9	**m_v = 0.075m²/MN**
7	10.0	Stiff clay	>10	20	-	
Initial vertical overburden effective stress is based on groundwater level of 1.0m bgl The adopted unit weights are relatively conservative values taken from those presented by Bond (2013). An m_v value for 8.4m to 10m bgl is assumed to be 0.075m²/MN which is considered average for a stiff clay An m_v value for 0.0m to 1.5m bgl is assumed to be 0.3m²/MN which is typical for a soft/firm clay Average SPT N value of 6 based on inspection of DCP and SPT results.						

The shallow (soft to) firm clays are assumed to have a coefficient of volume change (i.e. m_v value) of 0.3m²/MN as no consolidation testing was carried out in this material. This value was adopted based on an average for soft to firm clay, e.g. as indicated in Carter and Bentley (1991).

7.2.2 South area of the site

Table 32 below shows the assumed ground model for the north area of the site. This includes granular silts and sands clay layers at shallow depth, e.g. as revealed in BH3.

Table 32: South area: ground model for settlement analysis						
Layer	*Depth to top of stratum (m bgl)*	*Material*	*Layer thickness (m)*	*Unit weight (kN/m3)*	*in situ overburden at base (kPa)*	*Soil properties*
1	0.0	Very loose to loose silt/sand (dry)	1.0	18 (dry) 20 (sat.)	18.0	**Average SPT N value = 6**
2	1.0	Very loose to loose silt/sand	6.0	18 (dry) 20 (sat.)	78.0	
3	7.0	Very soft SILT/CLAY	1.8	16	88.8	**m_v as per Figure 7**
4	8.8	Firm to stiff clay	1.6	19	103.2	**m_v = 0.075m2/MN**
5	10.4	Stiff clay	>10	20	-	
Initial vertical overburden effective stress is based on groundwater level of 1.0m bgl An m_v value for 8.8m to 10.4m bgl is assumed to be 0.075m²/MN which is considered average for a very stiff clay The adopted unit weights are relatively conservative values taken from those presented by Bond (2013). Average SPT N value of 6 based on inspection of DCP and SPT results.						

7.3 Results of settlement analyses

The results of settlement analyses are presented later (in Appendix D) and summarised below (in Table 33).

Table 33: Results of settlement analyses for rafts

Area of site	*Load (kPa)*	*Foundation dimensions*			*Calculated immediate granular settlement (mm)*	*Calculated total settlement (mm)*
		Length, L (m)	*Width, B (m)*	*Depth, D (m)*		
North	20	12	12	Minimal (e.g. <1.0m bgl)	1.6	**78.2**
			6		0.0	**61.0**
South	20	12	12		13.5	**41.3**
			6		8.3	**12.4**
	31	12	6		12.9	**39.8**

7.3.1 12m by 6m raft

The findings seem to support the claim in the previous GEL report that a nominal 12m by 6m raft in the firm clay soils should have a maximum safe bearing capacity of 20kPa with an anticipated settlement of approximately 40mm. The above calculated settlement of 61.0mm (in Table 33) for such a raft in the clayey conditions in the north part of the site could be a slight overestimate as the firmer clay at shallow depth could absorb more of the vertical stress than has been assumed (using Bousinnesq theory). Therefore, 40mm could be a reasonable prediction of actual settlement.

On the other hand, the calculated settlements indicate a higher allowable bearing pressure could be suitable in the sandy conditions in the south part of the site. Calculations indicate a 12m by 6m raft might only settle by about 12.4mm under a load of 20kPa. Such a raft might be able to withstand 31kPa before a settlement of 40mm is likely.

7.3.1 12m by 12m raft

Unfortunately, wider rafts tend to settle by greater amounts as they impart greater loads at deeper depths. Consequently, a 12m by 12m raft loaded to about 20kPa in the clayey conditions (in the north part of the site) might experience total settlements of up to about 78.2mm. However, such a raft might only settle by about 41mm founded in the sands in the south part of the site.

7.4 Acceptable settlement

Terzaghi and Peck (1967) reportedly stipulated a maximum total settlement of 50mm for rafts. However, 40mm is often cited as a blanket acceptable settlement for rafts.

Nevertheless, slightly higher allowable settlements are sometimes allocated to foundations on clays than on sands. Tomlinson (2001: 63) reports that: *"For foundations on **sands** ... Skempton and MacDonald [1956] concluded ... the limiting total settlement is ... **40-65mm for raft foundations**", while, "for foundations on **clays** [they] similarly prescribed ... design limits for total settlement of ... **65-100mm for rafts**."* Such limits are expected to limit the angle of distortion caused by differential settlement to about 1 in 500 and thus restrict cracking in walls and partitions.

Limiting the angle of distortion to 1 in 500 should enable the design to conform to Eurocode 7 recommendations. EC7 advocates agreement with structural designers to identify the limiting values of settlements or movements.

7.5 Preloading

Additional surcharge loading could be placed across the site to compress the underlying soft clay layers which appear to be normally consolidated (from the results of consolidation testing). After compression, the stiffness and the strength of the underlying clay material should improve and increase the allowable bearing pressures.

The time taken for the consolidation to occur is related to the coefficient of consolidation c_v (as measured during recent consolidation testing on samples of the soft peaty clays). As indicated above, the results show that values of 2.3m^2/year to 30m^2/year were measured for c_v.

Time taken for consolidation is also related to the thickness of clay layers between drainage layers. The drainage path length could be as much as about 9m for the deeper very soft peaty clays. *In situ* bands of sand may be able to provide continuous drainage layers but this is difficult to confidently confirm and vertical sand (or geotextile) drains are often installed prior to preloading to decrease the consolidation time.

Thus, the time taken for 90% consolidation could be about 30 years, as shown below (in Table 34). If sand drains (or similar prefabricated vertical drains, PVDs) were installed on a regular 2m grid pattern, the time could potentially be reduced to about 6 months.

Table 34: Time required for consolidation of 9m thick clay layer

Based on: *coefficient of consolidation, c_v = 2.3 m^2/year; drainage path length = 9m*

Degree of consolidation, Uv (%)	**0**	**10**	**20**	**40**	**50**	**75**	**90**	**95**	**99**
Calculated non-dimensional time factor, T_v	**0.0**	**0.0079**	**0.031**	**0.126**	**0.196**	**0.477**	**0.848**	**1.129**	**1.781**

8 Engineering Issues

8.1 Foundations

All foundations should be designed by a competent engineer with consideration to all factors and the information within this report.

8.1.1 Shallow raft foundation

The revealed ground conditions comprise much shallow silty/clayey soils in the north part of the site, while granular silty sands dominate in the south part of the site. Settlement calculations suggest that rafts are more likely to be feasible within the sands.

Allowable bearing pressures (ABPs) of raft foundations depend on the raft dimensions as well as the ground conditions. An ABP of 31kPa should be available to shallow rafts (12m by 6m) installed in the loose granular silts and sands revealed in the south part of the site, which should limit anticipated total settlements to about 40mm. However, an ABP of only 20kPa might be available to a 12m by 12m raft as this loading is expected to induce about 40mm of total settlement.

An ABP of 20kPa could be available to a 12m by 6m raft founded at shallow depth in the soft/firm clays. Such loading is expected to induce about 40mm to 60mm of total settlement, which should be acceptable in clay soils.

8.1.2 Strip footings

Sands

Calculations indicate allowable bearing pressures of between about 33kPa and 61kPa for strip foundations in the granular silts and sands revealed in the south part of the site, as shown below (in Table 35). The complete calculations are presented later (in Appendix D). These are based on an acceptable total settlement of 25mm, which is a typical limiting value for strip foundations.

Table 35: South area: allowable bearing pressures in TFD granular sands/silts

	Depth (m bgl)	*Foundation width (m)*			
		0.45	*0.6*	*0.9*	*1.5*
Allowable bearing pressure	0.5	**48**	**50**	**46**	**33**
	0.75	**61**	**61**	**46**	**33**

Clays

Foundations in firm clays at 0.9m bgl could provide an allowable bearing pressure of 40kPa for footings no greater than 0.6m in width, as discussed in the previous GEL

report. This might only be sufficient for very lightly loaded structures. Unfortunately, the deeper very soft clays mean that if foundations need to be deepened (e.g. due to the presence of trees), then much lower ABPs would be appropriate.

8.1.3 The influence of trees on foundation depth

The Tidal Flat Deposits (TFD) silts and clays have been found to be of potentially medium volume change potential. Therefore, foundations will need to reach below the zone of possible shrinkage and swelling caused by seasonal moisture changes and exacerbated by the presence of nearby trees. Shallow TFD silts/clays have been mainly revealed in the north part of the site.

Typically, rafts in medium volume change potential clays should be founded in granular fill material which reaches a depth of at least 0.45m bgl. However, if shallow clays are found to be present with trees, the raft foundation level may also need to be deepened.

Some trees are present at or along the boundaries of the site. It would be prudent to undertake a tree survey to determine their species, water demand and mature height, notably in the north and central parts of the site, if shallow foundations are selected. Along with details of proposed tree planting, the survey information should help determine the required foundation depth for shallow foundations (e.g. strips or rafts) in clay soils.

8.1.4 Piled foundations

The deeper glacial till consists of boulder clay which has good strength and stiffness properties and should be able to provide a suitable bearing stratum for deep foundations (i.e. piles). As discussed, the boulder clays deeper than 10m should have an undrained shear strength of at least 150kPa, while those deeper than 12m bgl should have a c_u value of at least 200kPa.

Possible bored (i.e. cast in-situ) pile *ultimate* load resistances have been estimated, as shown below (in Table 36). The calculations are for preliminary design purposes only. Negative skin friction has been ignored. The adopted geotechnical properties are intended to be suitably cautious. Different, more suitable values may need to be used in the final design to comply with relevant design standards and the final design should be undertaken by a suitable piling specialist.

Table 36: Preliminary ultimate (cast in-situ) pile load resistances					
Stratum	*Depth range (m bgl)*	*Soil type*	*Adopted geotechnical parameters*	*Friction or adhesion (kPa)*	*End bearing resistance (kPa)*
1	0.0 – 1.5	Soft clay-	γ=17kN/m³, γ'=7kN/m³	-	-
2	1.5 – 4.2	Very soft clay	c_u=5kPa, α=0.7, γ'=6kN/m³	3.5	45
3	4.2 – 10.0	Sandy SILT	K.tanδ=0.2, Nq=10, γ'=10N/m³	13.1	varies
4	10.0 – 12.0	Stiff to v. stiff clay	c_u=150kPa, α=0.3, Nc=9, γ'=8kN/m³	45	1350
5	12.0+	Very stiff clay	c_u=200kPa, α=0.3, Nc=9, γ'=10kN/m³	60	1800
End bearing: clays: $q_p = N_c \times c_u$; sands: $q_p = N_q \times \sigma'_{vo}$ where $\sigma'_{vo} = \gamma' \times D_c$ (10B to 20B) Friction/Adhesion: clays: $q_a = \alpha c_u$; sands: $q_f = K tan\delta \times \sigma'_v$					

The above resistances have been used to estimate the *ultimate* load for single piles of different lengths and widths, as shown below (in Table 37). *Working* loads have also been estimated based on a factor of safety of 2.5. As discussed, these are indicative and for preliminary design purposes only.

Table 37: Preliminary estimated bored pile loads			
Pile length (m)	*Pile diameter (mm)*	*Estimated ultimate load (kN)*	*Estimated working load based on 2.5 global factor of safety (kN)*
11	300	218	**87**
	450	399	**160**
14	300	406	**162**
	450	704	**282**

The preliminary estimations suggest that an 11m-long 300mm-diameter bored pile might be able to provide a working load in the region of 87kN, as shown above (in Table 37). Longer, wider piles should have greater carrying capacity. These values are subject to re-evaluation and confirmation by a specialist piling engineer.

Typically, a pile and beam arrangement would provide a suitable foundation solution for low-rise dwellings combined with either a ground-bearing or suspended floor solution. Piles are usually distributed beneath load-bearing walls and the steel within the pile reinforcement lapped into a cast-in situ beam.

Alternative types of pile (e.g. driven/displacement piles) may also be suitable at the site. However, noise and vibration could be an issue with some installation methods.

8.2 Slab design

Heave may be an issue near existing (or recently removed) trees where shallow clays are present, especially if the trees are high water demand. A tree survey can help determine if heave is likely to be significant.

If heave is determined to be potentially significant, a suspended floor solution (e.g. beam and block) may be required. Otherwise, a ground-bearing floor slab could be feasible.

8.3 Excavations and groundwater control

Based on the findings of the site works, conventional mechanical excavation plant should be suitable at the site. However, the presence of unforeseen underground obstacles cannot be ruled out.

Shallow excavations in the soft to firm clay are likely to remain stable for considerable time. The granular silts and sands are more likely to required shoring. However, appropriate safety precautions should be taken with any open excavations, especially where man-entry is involved.

The groundwater table at the site could be between about 1.1m to 1.9m bgl based on recorded groundwater observations. However, localised groundwater levels as shallow as 0.74m bgl have been measured.

Therefore, groundwater inflows should be anticipated in excavations deeper than about 1m bgl. However, localised perched water may be encountered at even shallower depths.

Accordingly, some groundwater control measures (e.g. sump pumping) may be required. Care should be taken when dewatering in the vicinity of existing structures to avoid inducing adverse settlement.

The formation for any foundations should be dry and firm. Where groundwater has softened the foundation soil, excavations may need to be re-bottomed to find firmer material. However, deepening the foundations risks loading the softer clays at greater depths. Therefore, it might be sensible to initially underdig footings and then deepen (up to 0.9m depth) prior to concrete pouring.

8.4 Buried concrete

The results of sulphate (SO_4) and pH tests (full copies of which are included in Appendix C) provide information about the required sulphate resistance of buried concrete within the ground conditions at the site. The buried concrete should be designed in accordance with BRE Special Digest 1 (BRE, 2005) using these results.

The (shallow) ground conditions have been found to be largely of cohesive silt/clays and granular silts/sands and some groundwater has been observed. Therefore, it would be prudent to assume a *mobile* groundwater regime for the specification of buried concrete.

A maximum concentration of 57mg/kg of water-soluble sulphate was detected in the ***shallow ground*** (in WS108), as shown below (in Table 38). The soil samples were not acidic (pH was 8.6 to 8.8) and no significant quantity of sulphides was present (>0.3%). Therefore, no special sulphate resistant concrete should be required in the shallow ground conditions which agrees with the findings of the GEL report. Based on the BRE

Special Digest 1 (2005), a DS-1 Design Sulfate and an AC-1 ACEC classification should be suitable in the shallow soils (e.g. to a depth of about 3m bgl).

Table 38: Soil sulphate content

Stratum	*Sample location*	*HML sample ref. (S/_)*	*Chem-tech sample ref.*	*Depth (m bgl)*	*Water-soluble sulfate (mg/l)*	*pH*	*Total sulfur, [TS] (%)*	*Total (acid-soluble) sulfate, [AS] (%)*	*Total potential sulfate, [3TS] (%)*	*Oxidisable sulfides (%) [3TS-AS]*
Clayey SILT	WS105-2	56322	86822-4	1.2-1.7	57	8.6	0.048	0.029	0.14	0.11
Silty SAND	WS108-2	56324	86822-6	0.5-1.0	38	8.8	0.071	0.027	0.21	0.19
Peaty CLAY	BH1-9	56404	87075-2	7.95-8.10	820	7.8	1.54	0.28	4.61	4.33
Boulder CLAY	BH1-12	56405	87075-3	12.0-12.45	473	8.1	0.83	0.13	2.49	2.36

However, within the ***deeper ground***, a maximum concentration of 820mg/kg of water-soluble sulphate was detected in the ***slightly peaty clays*** (in BH1), as shown above (in Table 38). The soil samples were not acidic (pH was 7.8 to 8.1). Significant quantities of oxidizable sulphides were present (>0.3%), but the significant depth of these soil strata means there is not expected to be any opportunity for these to oxidise into sulphates. Therefore, based on the BRE Special Digest 1 (2005), a DS-2 Design Sulfate and an AC-2 ACEC classification should be suitable for concrete (e.g. in piles) within the deeper soil strata.

8.5 Road pavement design

Lab tests indicate (soaked) CBR values ranging between 0.5% and 31%%, as shown later (in Appendix C) and summarised previously (in Table 29). The poorer subsoils appear to be in the north part of the site. CBR values tend to be no less than 2% in the south part of the site.

The CBR values can be used to inform the road pavement design, e.g. for proposed access roads and parking areas. The values may also be used to determine approximate moduli of sub-grade modulus.

8.6 Soakaway design

Soakaways are not expected to be feasible at the site for two main reasons. Firstly, groundwater levels are shallow. Secondly, the ground conditions are of (very) low permeability.

9 Ground Gas Assessment

9.1 Ground gas monitoring results

Gas monitoring has been carried out at the site on six dates, as shown below (in Table 39). In summary, 0.0% methane was detected and a maximum steady concentration of 5.0% carbon dioxide was recorded (in SLR BH5), which was also the maximum peak carbon dioxide concentration. A maximum steady flow rate of -0.6 l/hr and peak flow of -7.6 l/hr were detected.

Table 39: Summary of recent HML gas monitoring results

Visit	*Date*	*Peak flow*		*Carbon dioxide*		*Methane*		*Comments*
		Max. flow (l/hr)	*Hole*	*Max. steady conc. (%)*	*Hole*	*Max. steady conc. (%)*	*Hole*	
1	07/07/20	0.0	All	4.1	WS107	0.0	All	Max. hexane 0.017%. No H2S or CO detected. PID Cf – 1.0.
2	17/07/20	0.0	All	4.1	WS102, WS107	0.0	All	Max hexane 0.021%. No H2S or CO detected. PID Cf – 1.0.
3	30/07/20	0.0	All	4.4	WS102, WS107	0.0	All	Max hexane 0.016%. No H2S or CO detected. PID Cf – 1.0.
4	13/08/20	0.0	All	5.0	WS102	0.0	All	Max hexane 0.016%. No H2S or CO detected. PID Cf – 1.0.
5	28/08/20	0.0	All	4.7	WS108	0.0	All	Max hexane 0.017%. No H2S or CO detected. PID Cf – 1.0.
6	11/09/20	-7.6	WS108	4.0	WS102	0.0	All	Max hexane 0.017%. No H2S or CO detected. PID Cf – 1.0.
No steady borehole flow detected apart from on visit 6: -0.6 *l/hr* in WS108								

The complete results of the gas monitoring are presented later (in Appendix C).

In accordance with the CIRIA C665 (Wilson et al, 2007) methodologies, a conservative Gas Screening Value (GSV) has been calculated using a worst-case scenario. That is, the highest gas concentration has been multiplied by the highest flow rate.

9.1.1 Methane (CH4) assessment

No methane and a maximum of –(7.2) l/hr (peak) borehole gas flow has been detected so far. Therefore, the (highly conservative) calculation of the methane borehole gas flow rate is as follows:

(0.1%/100) x 7.2 l/hr = 0.0072 l/hr.

The GSV indicates a CIRIA C665 Characteristic Situation 1 (CS1), which is *Very Low Risk* (Wilson et al., 2007), as shown below (in Table 40).

Table 40: CS by site characteristic GSV

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

[A)] The figures used in this column are empirical
NOTE The CS is equivalent to the characteristic GSV in CIRIA C665

9.1.2 Carbon dioxide (CO_2) assessment

The highest measured percentage of carbon dioxide has been 5.0% with a maximum of -0.6*l/hr* borehole steady gas flow detected. Therefore, the calculation of the carbon dioxide borehole gas flow rate is as follows:

CO_2 borehole flow rate = (5.0%/100) x 0.6 l/hr = 0.03 l/hr

The calculation indicates a CIRIA C665 Characteristic Situation 1 (CS1), which is *Very Low Risk* (Wilson et al., 2007).

9.2 Gas sources and pathways

Degradable organic material within the underlying Tidal Flat Deposits (TFD) natural soils are a potential source of hazardous ground gas. Typically, bands of peat within these can produce carbon dioxide (in aerobic conditions) or methane (in anaerobic conditions).

No peat was revealed during the site investigation. However, some slightly organic silts and clays were revealed, notably in BH1 (6.6m – 8.4m bgl) and in BH3 (7.0m – 8.7m bgl). Local slight organic odours were also noted in the shallow TFD soils.

Organic matter tests were carried out on the most peaty/organic samples of TFD soils, the results of which are presented later (in Appendix C). The results are summarised below (in Table 41).

Table 41: Soil organic matter (SOM) content						
Stratum	*Sample location*	*HML sample ref. (S/_)*	*Chemtech sample ref.*	*Depth (m bgl)*	*TOC (% w/w)*	*Estimated SOM (% w/w)*
Slightly organic sandy SILT/CLAY	BH1-8	56403	87075-1	9.0-9.45	2.1	3.6
Slightly organic sandy clayey SILT	BH3-7	56410	87075-4	7.5-7.95	1.8	3.1
Grey sandy SILT	BH2-9	56434	87075-5	9.0-9.45	0.5	0.9
Silty SAND (with sl. organic odour)	WS108-5	56325	86882-7	0.5-1.10	0.5	0.9

The detected levels of organic material in the peatier bands of revealed TFD soils are generally low and unlikely to generate any significant quantities of harmful ground gases. The estimated soil organic matter (SOM) contents range from 0.9% to 3.6% and means only the two deeper samples from BH1 and BH3 would even be classed as "slightly organic" (based on BS5930:2015) as they have SOMs of between 2% and 6%. Therefore, the gas generating potential of the underlying natural soils is likely to be very low.

The ground conditions around the site are generally of very low to low permeability. The high silt and clay content of nearly all the revealed ground conditions means that gas flow is severely restricted.

The possibly proposed piled foundation solutions could pass through the peatiest of the revealed silts/clays. However, the overlying soils are very soft (or very loose) and should collapse around the piles to provide a tight low permeability seal. Therefore, there is considered to be minimal risk of the piles providing a conduit for deeper ground gases.

9.3 Gas protection measures

As discussed in the previous GEL report, the risk from hazardous ground gases is very low. The revealed natural strata do not contain any highly organic soils (only slightly organic soils). Gas monitoring has revealed no gas flows and only small concentrations of carbon dioxide (a maximum of 5.0%). The very soft clayey and silty soils should restrict gas flows and provide a good seal around all piled foundations.

Therefore, the gas regime at the site should be given a classification of CS1 (e.g. as per CIRIA C735) or Green (as per the NHBC/RSK traffic light system). Accordingly, no ground gas protection measures are anticipated at the site.

10 Analysis of soil contamination test data

Assessment criteria (in mg/kg) have been taken from the *generic assessment criteria* (GAC) published in the *Land Quality Management Ltd/Chartered Institute of Environmental Health suitable for use levels* (LQM/CIEH S4ULs) report (LQM/CIEH, 2015). The exception to this is the GAC for lead which is taken from the Category 4 Screening Level (C4SL) data published in a report by Defra (2014). All the GAC are based on the context of a *residential* development ***with*** the potential for consumption of home-grown produce (i.e. with plant uptake).

10.1 Metals and metalloids

Table 42 (below) gives the GAC for metal (and metalloid) contaminants considered for the site.

Table 42: Generic assessment criteria for metals - *Residential with plant uptake*

Element	***(mg/kg)***
Arsenic	37
Boron	290
Cadmium	11
Chromium III	910
Chromium VI	6
Copper	2400
Lead**	200
Mercury element	1.2
Mercury inorganic	40
Mercury methyl	11
Nickel	130
Selenium	250
Vanadium	410
Zinc	3700
* from LQM/CIEH (2015) except: ** from Defra (2014)	

Four near surface soil samples were subjected to a suite of metals and metalloids tests. Two samples were of near surface topsoil (from WS101 and WS106). One sample was silt from below the hardstanding (in WS103). The other sample was of reworked sandy silt made ground (from WS105). Most samples were in the northwest area which was not sampled and tested during the previous GEL investigation.

The test results show that there are no elevated concentrations of any metals or metalloid compounds. All concentrations were well below GAC screening values.

The complete test results are presented later (in Appendix C).

10.2 Polycyclic aromatic hydrocarbons (PAHs)

Table 43 (below) gives the GAC (from LQM/CIEH, 2015) for speciated PAH contaminants. These vary with soil organic matter (SOM) content. At this stage, the most onerous SOM of 1% is assumed, which should be reasonably conservative.

Table 43: Soil GACs for PAH - Residential with plant uptake

Element	*SOM of 1%*	*SOM of 2.5%*	*SOM of 6%*
Acenaphthene	210	510	1100
Acenaphthylene	170	420	920
Anthracene	2400	5400	11000
Benzo(a)anthracene	7.2	11	13
Benzo(a)pyrene	2.2	2.7	3
Benzo(b)fluoranthene	2.6	3.3	3.7
Benzo(k)fluoranthene	77	93	100
Benzo(ghi)perylene	320	340	350
Chrysene	15	22	27
Dibenzo(ah)anthracene	0.24	0.28	0.3
Fluoranthene	280	560	890
Fluorene	170	400	860
Indeno(123-cd) pyrene	27	36	41
Naphthalene	2.3	5.6	13
Phenanthrene	95	220	440
Pyrene	620	1200	2000
BaP as Surrogate marker	0.79	0.98	1.1

Four near surface soil samples were subjected to a suite of PAH tests (speciated by US EPA 16). Two samples were of near surface topsoil (from WS101 and WS106). One sample was silt from below the hardstanding (in WS103). The other sample was of reworked sandy silt made ground (in WS105). Most samples were in the northwest area which was not sampled and tested during the previous GEL investigation.

The test results show that there are no elevated concentrations of any PAH species. All concentrations were well below GAC screening values and most were below the limit of detection.

The complete test results are presented later (in Appendix C).

10.3 Asbestos

Four near surface soil samples were subjected to an asbestos identification test. Two samples were of near surface topsoil (from WS101 and WS106). One sample was silt from below the hardstanding (in WS103). The other sample was of reworked sandy silt made ground (in WS105). Most samples were from the northwest area which was not sampled and tested during the previous GEL investigation.

The test results show that there no asbestos fibres were detected.

The complete test results are presented later (in Appendix C).

10.4 Petroleum hydrocarbons (TPH & BTEX)

Table 44 (below) gives the GAC (from LQM/CIEH, 2015) for speciated TPH fractions. These vary with soil organic matter (SOM) content. At this stage, the most onerous SOM of 1% is assumed, which should be reasonably conservative.

Table 44 - Total Petroleum hydrocarbons (TPHs) generic acceptance criteria – residential with plant uptake

Compound		*1% SOM*	*2.5% SOM*	*6% SOM*
TPH Aliphatic	**C5-6**	42	78	160
	C6-8	100	230	530
	C8-10	27	65	150
	C10-12	130 (48)	330 (118)	760 (283)
	C12-16	1100 (24)	2400 (59)	4300 (142)
	C16-35	65000 (8.5)	92000 (21)	110000
	C35-44	65000 (8.5)	92000 (21)	110000
TPH Aromatic	**C5-7**	70	140	300
	C7-8	130	290	660
	C8-10	34	83	190
	C10-12	74	180	380
	C12-16	140	330	660
	C16-21	260	540	930
	C21-35	1100	1500	1700
BTEX	**Benzene**	0.087	0.17	0.37
	Toluene	130	290	660
	Ethylbenzene	47	110	260
	O-xylene	60	140	330
	M-xylene	59	140	320
	P-xylene	56	130	310

Note: values in parentheses indicate vapour saturation limit (v) or limit of solubility (s)

One sample was tested for a suite of petroleum test (TPH by CWG). The sample was from WS101 (1.3m to 1.7m bgl) adjacent to a suspected former above-ground fuel tank.

The results of TPH testing show that there are no exceedances in TPH contamination above the GAC for a residential development with the potential for plant uptake. Nearly all the fractions of hydrocarbons were found to be below the detection limits.

No TPH testing was undertaken on the soil samples from within WS101, the location of a former historical tank (of unknown type). However, no olfactory or visual evidence of

contamination was noted within WS101 and the PAH testing on a shallow sample from WS101 did not indicate any petroleum-related PAH contamination.

10.5 Pesticides (organophosphates, and organochlorines)

Three near surface soil samples were tested for the presence of pesticides (organophosphates and organochlorines). These were taken from WS101, WS105 and WS106 in the central and northern parts of the site.

Test results show no elevation (or presence) of pesticide contamination. No concentrations were found above the detection limits.

11 Revised Contamination Assessment

Based on initial assessment, findings of the site works and analysis of the contamination test data, a revised combined CSM and risk assessment has been developed, as shown below (in Table 45), based on a *source-pathway-receptor* consideration of potential pollution linkages.

A risk rating of *very low* or *low* is not considered significant while a rating of *moderate / low* (or greater) could be potentially hazardous and the associated source-pathway-receptor taken to be a plausible contamination linkage and in need of further consideration. However, the revised risk assessment below has not identified any unacceptable risks associated with potential contamination.

Nonetheless, some contamination-related risks might be considered significant, but they should be mitigated through standard construction practices, as discussed previously. The risk from contaminants within the existing buildings and infrastructure, for example, asbestos, should be mitigated by suitable demolition by competent persons. Risks to construction workers should be managed by the site contractors. Possible harm to buried concrete from sulphates should be prevented by adherence to BRE Special Digest 1. Risks from permeation of contaminants into water pipes should be mitigated through compliance with the local water authority's pipework specifications.

Any soils and waste removed off site should be sent to a suitably licensed landfill operator. Consignment and waste transfer notes should be kept along with site records of quantities. The landfill operator may require additional testing (e.g. WAC testing) in addition to a copy of this report in order to classify the waste (as inert, hazardous, etc.).

There may be scope to re-use some of the existing materials on site. For instance, the stone sub-base beneath the existing hardstanding might be suitable for re-use below proposed access roads. The in-situ concrete might also be useful below hardstanding areas, once suitably grubbed up. The tarmac hardstanding material will require phenols and PAH testing (or PAK spray testing as a minimum) before it can be considered suitable for re-use.

Any unforeseen contamination (for example, hydrocarbon odours or staining) encountered during the development of the site should be monitored and reported to the local authority. Work should be halted in any area of the revealed potential contamination and the contamination investigated and assessed to the satisfaction of the local authority.

Table 45: Revised contamination risk assessment						
Conceptual site model			*Risk Assessment*			
Potential source	*Potential pathway*	*Potential receptor*	*Probability*	*Severity*	*Risk rating*	*Comments*
Drainage / Buildings **Metals, asbestos, pesticides**	Direct contact, ingestion or dust inhalation	Future site users	Low likelihood	Medium	Low	These risks should be mitigated through the safe removal of the existing buildings, e.g. with a suitable asbestos survey and demolition carried out by competent persons. Neither this nor the previous GEL investigation found any elevated contamination
		Construction workers	Low likelihood	Medium	Low	
	Vapour inhalation	Future site workers	Unlikely	Mild	Very low	
		Construction workers	Unlikely	Mild	Very low	
	Direct contact	Ecology	Low likelihood	Mild	Low	
	Leaching & migration	Surface waters	Low likelihood	Mild	Low	
	Leaching & migration	Bedrock aquifer	Unlikely	Mild	Very low	
	Direct contact	Buildings/ structures	Unlikely	Mild	Very low	
Sub-soils / Made ground **metals, PAHs and asbestos**	Direct contact, ingestion or dust inhalation	Future site users	Low likelihood	Medium	Low	Neither this nor the previous GEL investigation found any elevated contamination
		Construction workers	Low likelihood	Medium	Low	
	Vapour inhalation	Future site workers	Unlikely	Medium	Low	
		Construction workers	Unlikely	Medium	Low	
	Direct contact	Ecology	Low likelihood	Mild	Low	
	Leaching & migration	Surface waters	Unlikely	Mild	Very low	
	Leaching & migration	Bedrock aquifer	Unlikely	Mild	Very low	
	Direct contact	Buildings/ structures	Unlikely	Mild	Very low	
Above ground tanks **Petroleum hydrocarbons**	Direct contact, ingestion or dust inhalation	Future site users	Low likelihood	Medium	Low	Neither this nor the previous GEL investigation found any elevated contamination. The only noted olfactory evidence was in the previous WS5, but no contamination was detected. No evidence of contamination was found at WS101 or WS103 the locations of suspected former tanks
		Construction workers	Low likelihood	Medium	Low	
	Vapour inhalation	Future site workers	Low likelihood	Medium	Low	
		Construction workers	Low likelihood	Medium	Low	
	Direct contact	Ecology	Low likelihood	Mild	Low	
	Leaching & migration	Surface waters	Low likelihood	Mild	Low	
	Leaching & migration	Bedrock aquifer	Unlikely	Mild	Very low	
	Direct contact	Buildings/ structures	Unlikely	Minor	Very low	
Soil Gas **CO2, methane**	Inhalation / Combustion	Future site users	Low likelihood	Medium	Low	Soils are low organic content, soft and of low permeability. No harmful gases found.
		Construction workers	Low likelihood	Medium	Low	
Off-site Sources **Metals, PAHs, TPHs**	Direct contact, ingestion or dust inhalation	Future site users	Unlikely	Medium	Low/ moderate	Neither this nor the previous GEL investigation did not identify any made ground or any elevated contamination around the boundary of the site
		Construction workers	Unlikely	Medium	Low/ moderate	
	Vapour inhalation	Future site workers	Unlikely	Medium	Low	
		Construction workers	Unlikely	Medium	Low	

12 SUMMARY & CONCLUSIONS

12.1 Ground conditions

The findings of the investigation indicate the following sub-strata at the site.

12.1.1 North (and central) part of the site (e.g. BH1, BH2, WSs 101 to 105, WSs 1 to 9)

- **Topsoil**: (across much of the area) brown with rootlets *[max depth: 0.55m bgl]*
- **Made ground:** (in WSs 101, 102, 103 & 105) natural soils *[max depth: 1.0m bgl]*
- **Tidal Flat Deposits (TFD)**:
  - Bands of cohesive silts/clays and granular silts/sands.
  - Typically soft/firm to about 1.5m bgl and very soft/loose below 2m bgl.
  - Possible 1.8m thick very soft slightly organic silt/clay below about 6.6m bgl
  - Base of TFD about 8.4m to 9.9m bgl

12.1.2 South part of the site (e.g. BH3, WSs 106 to 109, WSs 10 to 20)

- **Topsoil**: (across much of the area) Brown with rootlets *[max depth: 0.45m bgl]*
- **Tidal Flat Deposits (TFD)**:
  - (Very) loose granular silts/sands (non-shrinkable).
  - Possible 1.7m-thick very soft slightly organic silt/clay below about 7.0m bgl
  - Base of deposits about 8.7m bgl

12.1.3 The whole site

- **Glacial till**: revealed as boulder clay *[proven to at least 16m bgl]*
  - Stiff becoming very stiff

12.2 Geotechnical properties

The geotechnical properties as determined from measurements, estimations and evaluations are presented below (in Table 46). Further information is provided in Section 6.

Table 46: Geotechnical properties			
Soil	*Depth (m bgl)*	*Geotechnical property*	*Value*
TFD: shallow cohesive silts/clays	0.5-1.7	Natural moisture content	24% - 28%
		Plasticity index	11% - 20%
	<1.5	Characteristic undrained shear strength, c_u	20kPa
	1.5-2.0		<15kPa
	>2.0		<10kPa
	2.0-2.45	Coefficient of volume change, m_v	0.66 m²/MN (at 50kPa)
			0.40 m²/MN (at 100kPa)
TFD: shallow granular silts/sands	0.5-5.45	Sand content	34% - 91%
		Fines content	9% - 66%
	1.0-3.0	Average SPT N value	4 (blows/300mm)
		Characteristic N value	2 (blows/300mm)
	-	Average equivalent SPT N value (DCP)	6 (blows/300mm)
	-	Angle of friction, ϕ	28°
TFD: deep slightly organic silt/clay	7.95-8.1	Natural moisture content	64%
		Plasticity index	52%
	7.5-7.95	Coefficient of volume change, m_v	1.4 – 1.5 m²/MN (at 50kPa)
			0.9 m²/MN (at 100kPa)
Glacial till (boulder clay)	10.5-10.95	Natural moisture content	20%
	10 – 12	Characteristic undrained shear strength, c_u	150kPa
	>12		200kPa

12.3 Foundations

12.3.1 Shallow raft foundation

Settlement calculations suggest that rafts are more likely to be feasible within the *granular silts and sands* in the south part of the site. An available allowable bearing pressure (ABP) could range between 20kPa (for a 12m by 12m raft) to 31kPa (for a 12m by 6m raft). These loadings should limit anticipated total settlements to about 40mm.

Higher settlements are likely in the north due to the clayey subsoils. An ABP of 20kPa could be available to a 12m by 6m raft founded at shallow depth in the *cohesive soft/firm silts/clays* in the north part of the site. Such loading is expected to induce about 40mm to 60mm of total settlement, which should be acceptable in clay soils.

12.3.2 Strip footings

Calculations indicate allowable bearing pressures of between about 33kPa (at 0.5m bgl) and 61kPa (at 0.75m bgl) for 0.6m-wide strip foundations in the *granular silts and sands* revealed in the south part of the site, as shown above (in Table 35). Anticipated total settlements are limited to about 25mm.

Foundations in firm clays at 0.9m bgl could provide an allowable bearing pressure of 40kPa for footings no greater than 0.6m in width. If foundations need to be deepened (e.g. due to the presence of trees), then much lower ABPs would be appropriate.

12.3.3 The influence of trees on foundation depth

Shallow Tidal Flat Deposits (TFD) silts and clays have been found to be of potentially medium volume change potential, mainly in the north part of the site. Rafts or strip footings may need to be deepened in these shrinkable silts/clays near any (existing/proposed) trees.

Some trees are present at or along the boundaries of the site. Details of proposed tree planting and existing trees should help determine the required foundation depth for shallow foundations (e.g. strips or rafts) in the north and central area of the site.

12.3.4 Piled foundations

The deeper glacial till consists of boulder clay which has good strength and stiffness properties and should be able to provide a suitable bearing stratum for deep foundations (i.e. piles). Stiff boulder clay has been revealed below depths ranging from 8.4m to 9.9m bgl.

Preliminary estimations suggest that an 11m-long 300mm-diameter bored pile might be able to provide a working load in the region of 87kN, as shown above (in Table 37). Longer, wider piles should have greater carrying capacity. These values are subject to re-evaluation and confirmation by a specialist piling engineer.

12.4 Slab design

Heave may be an issue near existing trees where shallow clays are present (in the north part of the site). This can be assessed with the results of a tree survey. If heave is potentially significant, a suspended floor solution (e.g. beam and block) may be required.

12.5 Excavations and groundwater control

Shallow excavations in the soft to firm clay are likely to remain stable for considerable time. The granular silts and sands are more likely to required shoring.

The groundwater table at the site could be between about 1.1m to 1.9m bgl based on recorded groundwater observations. However, localised groundwater may be present as shallow as 0.71m bgl. Therefore, some groundwater control measures (e.g. sump pumping) may be required within trench excavations.

12.6 Buried concrete

The results of sulphate (SO_4) and pH tests indicate a DS-1 Design Sulfate and an AC-1 ACEC classification should be suitable (in the shallow soils, e.g. to about 3m bgl).

However, a DS-2 Design Sulfate and an AC-2 ACEC classification should be required for concrete (e.g. in piles) within the deeper soil strata.

12.7 Road pavement design

Lab tests indicate (soaked) CBR values ranging between 0.5% and 31%%, as shown later (in Appendix C). The poorer subsoils appear to be in the north part of the site. CBR values tend to be no less than 2% in the sandier ground conditions in the south part of the site.

12.8 Soakaway design

Groundwater levels as shallow as 0.71m bgl (and typically between 1.0m and 1.9m bgl) have been recorded at the site. The ground conditions are dominated by low permeability silts. The results of previous soakaway testing have shown poor drainage with calculated infiltration values of between $1.7x10^{-6}$m/s and $2.5x10^{-6}$m/s and the calculated values are probably over-estimates of suitable design values. Therefore, soakaways are not expected to be feasible at the site.

12.9 Contamination testing

This investigation (and the previous GEL investigation) have tested numerous soil samples and found:

- No elevated levels of metals or metalloids
- No elevated levels of PAHs
- No elevated levels of petroleum hydrocarbons
- No asbestos fibres (or asbestos containing materials)
- No detected levels of pesticides (i.e. organochlorines or organophosphates)

12.10 Gas protection measures

As discussed in the previous GEL report, the risk from hazardous ground gases is very low. Gas monitoring has revealed only very slow gas flows (a maximum peak flow of -7.6 *l/hr* and a maximum steady flow of -0.6 *l/hr*) and only small concentrations of carbon dioxide (a maximum of 5.0%). The very soft clayey and silty soils should restrict gas flows and provide a good seal around all piled foundations.

Therefore, the gas regime at the site should be given a classification of CS1 (e.g. as per CIRIA C735) or Green (as per the NHBC/RSK traffic light system). Accordingly, no ground gas protection measures are anticipated at the site.

12.11 Contamination risk assessment

A revised combined CSM and risk assessment has been developed, as shown above (in Table 45), based on a *source-pathway-receptor* consideration of potential pollution linkages. The revised risk assessment below has not identified any unacceptable risks associated with potential contamination.

Some contamination-related risks might be considered significant, but they should be mitigated through standard construction practices. The risk from contaminants within the existing buildings and infrastructure, for example, asbestos, should be mitigated by suitable demolition by competent persons.

All soils and waste removed off site should be sent to a suitably licensed landfill operator. The landfill operator may require additional testing (e.g. WAC testing) in addition to a copy of this report to classify the waste (as inert, hazardous, etc.).

There may be scope to re-use some of the existing materials on site. For instance, the *in-situ* concrete might also be useful below hardstanding areas, once suitably grubbed up. Any tarmac will require further testing before it can be deemed suitable for re-use.

Any unforeseen contamination (for example, hydrocarbon odours or staining) encountered during the development of the site should be monitored and reported to the local authority. Work should be halted in any area of the revealed potential contamination and the contamination investigated and assessed to the satisfaction of the local authority.

- End of Report -

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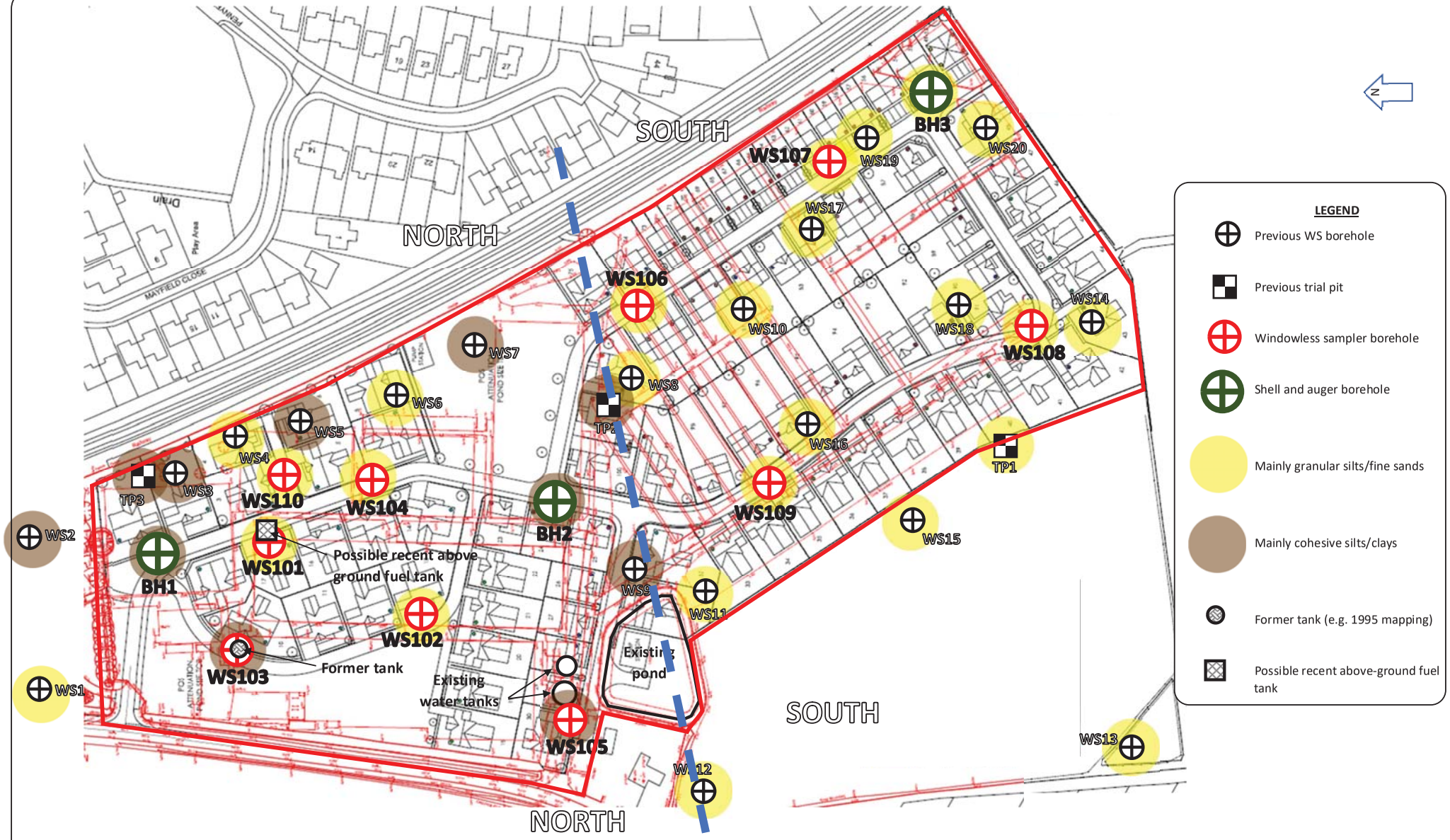
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Appendix A

Plans & Photographs



Notes:

1. Do not scale
2. Locations of all features are approximate only
3. Previous hole locations taken from Ground Engineering Limited report number C13895 dated August 2016
4. Background image taken from LDC (i.e. Linc Design Consultancy of Louth) drawing number LDC2328-03 B, dated January 2019

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Drawing Title: **Exploratory hole location plan**
Drawing No.: 1197-5174-SI-01 *Revision:* 0
Site: Keston Nurseries, Pinchbeck
Client: D Brown Building Contractors Ltd
Project No.: 1197/5174/P
Date: 30/07/2020



Site location plan *Date copied:* 21 July 2020 *Source:* openstreetmap.org



Aerial photograph: 1 *Date copied:* 21 July 2020 *Source:* Google Maps



Photo: 3 *Date:* 17 June 2020
Photo direction: East
Description: Existing building in the north part of the site

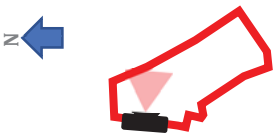




Photo: 4 *Date:* 17 June 2020
Photo direction: South
Description: North part of the site (to east of existing buildings)

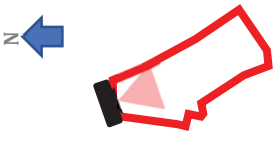




Photo: 5 *Date:* 17 June 2020
Photo direction: West
Description: Location of suspected previous above-ground fuel tank

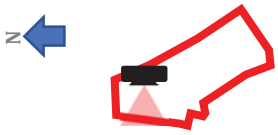
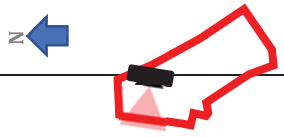




Photo: 6 *Date:* 17 June 2020
Photo direction: West



Description: Location of suspected previous above-ground fuel tank	



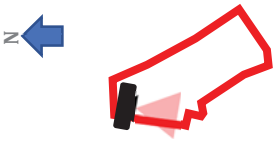
Photo: 7 *Date:* 17 June 2020 *Photo direction:* South *Description:* Existing building inside northwest boundary of the site	



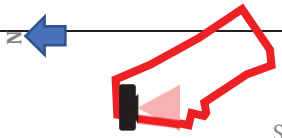
Photo: 8 *Date:* 17 June 2020	

Photo direction: South *Description:* Existing office and greenhouse buildings in northwest part of the site	



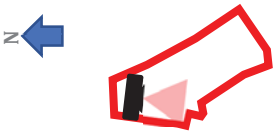
Photo: 9 *Date:* 17 June 2020 *Photo direction:* South *Description:* Existing greenhouses in the northwest part of the site	



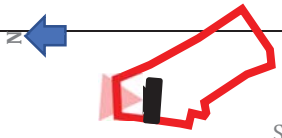
Photo: 10 *Date:* 17 June 2020	

Photo direction: North *Description:* Existing building in the north part of the site	



Photo: 11 *Date:* 17 June 2020 *Photo direction:* North *Description:* Northernmost part of the site and adjacent dwelling (in background)	