TP111	0.3-0.5		Black staining of natural clayey silt arisings
TP112	0.2-0.4		Black stained sand and gravel made ground fill
TP115	0.9-1.9	Moderate to strong organic ‘manure’ odour	
TP115	1.9		Partially black stained natural silt at surface of strata
TP116	1.2-1.5	Faint organic odour	Black staining within natural clay
TP124	0.8-1.4		Black staining of reworked clay soils
TP126A	0.9	Moderate to strong organic ‘manure’ odour within natural silt soils	
TP126B	0.8		Black staining encountered within the made ground
TP146	0.45-0.6		Black staining encountered within the made ground
WS106	0.47-0.52	Slight organic odour within reworked clay soils	
WS108	0.3-1.6	Slight organic odour within natural clay soils	
WS115	0.3	Slight organic odour encountered within made ground soils	

Non-aqueous phase product (NAPL) / strong hydrocarbon odours were not encountered in any of the trial holes.

13.5 Other Observations

On the 19th January 2021 ground gas was observed to be ‘bubbling’ from around the headworks of BH106 (the internal section of the cover was flooded during a period of heavy rain), and an audible flow could be heard when the valve was opened. A ‘spot’ reading was undertaken to initially characterise the gas concentrations emanating from the borehole. A flow rate of 104L/hr with 13.6% carbon dioxide, 0% methane and 1.0% oxygen concentrations were recorded. The gas flow rate was high for at least 24 hours, but was found to gradually decrease over 7 days, as the borehole was visually and acoustically monitored.

The high flow rates were only observed after a period of very heavy rain. The borehole was located away from known services and no voids were recorded when the borehole was drilled. The response zone for the monitoring well extended from 11.3 m to 20.3 m within the Upper

Tunbridge Wells Sand Formation. A relatively thin layer of made ground ~0.50 m thick was encountered in BH106. An additional ground gas monitoring well (WS106) was installed within 2.0 m of BH106 to characterise the shallow ground gas conditions at this position.

During the Phase III site investigation the two lodge houses at the site entrance were demolished. The demolition included grubbing out of below ground foundations and a brick wall using heavy plant and machinery. This caused disturbance to otherwise natural ground.

Boggy soft wet ground was encountered at the surface within the footprint of former Building 12.

Table 11 below summarises obstructions encountered during this phase of investigation.

Table 11: Summary of obstructions encountered during investigation

Hole ID	Depth (m)	Comment
TP116	2.0	No further progress due to concrete slab obstruction encountered. Possible remnant foundation.
TP118	1.4-1.6	Concrete foundation footing encountered within the northern wall of the trial pit.
TP125	1.4	No further progress due to encountered obstruction within the made ground.
TP132	2.1	No further progress due to hard ground. Base of trial pit obscured by groundwater, not possible to confirm if due to concrete or refusal on natural ground.
TP136	2.3	No further progress due to hard ground. Base of trial pit obscured by groundwater, not possible to confirm if due to concrete or refusal on natural ground.
WS104	0.8	Borehole refused on an assumed concrete obstruction. Bottom of the pit not visible due to water ingress.
WS107	2.4	Borehole refused on an obstruction.

Tunnels or voids were not encountered during this phase of investigation, however the presence of tunnels or voids across the site cannot be discounted.

The record of obstructions encountered during previous investigations by others is outlined below, and a summary of historical exploratory hole locations is provided as Figure 4, Appendix B.

Table 12: Summary of obstructions encountered during previous investigations

Hole ID	Investigation	Depth (m)	Comment
WS19A	SKM July 2013	0.55	Terminated due to concrete at base of inspection pit.
WS20	SKM July 2013	1.0	Terminated due to obstructions in sides and base of inspection pit.
WS22	SKM July 2013	0.65	Terminated due to dense gravel and obstruction at the base.
WS24	SKM July 2013	0.4	Terminated due to concrete obstruction.
WS25	SKM July 2013	0.55	Terminated due to concrete obstruction.
WS29	SKM July 2013	0.6	Terminated due to concrete obstruction.
WS39	SKM July 2013	0.6	Terminated due to brick and concrete obstructions.
WS39A	SKM July 2013	0.9	Terminated due to brick and concrete obstructions. Second attempt at WS39,
WS45	SKM July 2013	0.7	Concrete obstruction recorded across base of the inspection pit. Pit extended but the obstruction continued.
WS60	Jacobs October 2014	0.3	Concrete with rebar noted across the base of the inspection pit.
WS60A	Jacobs October 2014	0.5	Concrete with rebar noted across the base of the inspection pit.
WS61	Jacobs October 2014	0.75	A metal obstruction encountered.
WS70	Jacobs October 2014	1.0	Concrete noted across the base of the pit.
TP35	KDC June 2016	3.7	Trial pit terminated due to two concrete sidewalls encountered causing restricted excavator bucket movement.
TP47	KDC June 2016	0.8	Trial pit terminated upon reaching an obstruction visually described as a concrete slab. Potential cover of a rainwater drain.
TP-CP1	KDC June 2016	1.2-1.5	Encountered an obstruction noted as rows of bricks, possible former oil storage tank foundation.

E GEOTECHNICAL INFORMATION

14 Strata Encountered

The encountered conditions of the natural soils of the Upper Tunbridge Wells Sand was largely uniform across the site. However, the made ground encountered was noted to vary between the eastern and western site of the site. On this basis the ground model for the made ground is split between the two sections, as per below. The following laboratory geotechnical tests have been undertaken:-

Table 13 Geotechnical Laboratory Testing

Test	Number of tests						
	Clay	Silt	Sand	Siltstone	Mudstone	Sandstone	Made Ground
Moisture Content	12	13					
Atterberg Limit Test	12	13					
Particle Size Distribution Test	1	4					1 reworked[18] silt
Sulphate Determination	6	5		1			1 reworked[18] silt, 1 reworked clay and 1 made ground fill
pH	6	5		1			1 reworked[18] silt, 1 reworked clay and 1 made ground fill
Undrained Triaxial Test	1						
Point Load Strength Test				11	27	33	
Uniaxial Compressive Strength Test				2	13	9	
California Bearing Ratio		1					

14.1 Made Ground – Western Area

The made ground soils encountered on the western side of the site comprised blacktop hardstand areas and underlying fill, building demolition rubble and 'reworked' soils occasionally with anthropogenic inclusions of fine to coarse brick and concrete. The demolition fill on the western side of the site was encountered up to 1.0m within previous footprints of buildings 18 and 38, comprising fine to coarse brick and concrete with occasional fragments of vinyl flooring, plastic and ceramic. The central blacktop hardstand area was encountered to be underlain by red fine to coarse brick sandy gravelly fill up to 0.4 m. The deepest area of made ground on the western side was encountered towards the south west as reworked clayey gravelly silts in BH105 up to 2.9m.

Table 14: Summary of the limited Geotechnical Testing on the Western side made ground

Test	Range
SPT 'N value'^	9 - 28
Undrained shear strength (kN/m^2) – Hand Penetrometer Field Test within reworked made ground	58 - 115

^SPT-N Values are uncorrected

14.2 Made Ground – Eastern Area

Much of the eastern side of the site is underlain by made ground generally comprising demolition rubble fill made up of brick and concrete with occasional anthropogenic inclusions of ceramic, plastic, vinyl flooring, rare wood, cables, scrap metal, rebar, wire, fragments small metal and plastic pipes. The made ground was generally deepest within the footprint of the demolished buildings, extending to depths generally between 1-2m and up to 4.05 m in the south eastern corner.

Made ground or fill is by nature highly variable in both composition and bearing capacity and can be subject to large differential settlements when loaded. It is therefore generally unsuitable for use as a bearing stratum. In addition, made ground may contain contaminated and/or putrescible material. It can therefore be potential source of contamination and landfill gas.

Table 15: Summary of the limited Geotechnical Testing on the Eastern side made ground

Test	Range
Sulphate Content (g/l)	0.15 - 0.89
pH	8.1 – 11.7
SPT 'N value'^	4 - 66
Undrained shear strength (kN/m^2) – Hand Penetrometer Field Test within reworked made ground	15 - 158

^SPT-N Values are uncorrected

14.3 Clay and Silt – Weathered Upper Tunbridge Wells Sands

The clays and silt soils encountered extended to depths up to 3.7m in the eastern section and to depths up to 3.1m in the western section and were underlain by extremely weak interbedded mudstone, siltstone and sandstone. The results of the geotechnical laboratory testing are summarised in Table 16 below.

Table 16: Summary of Geotechnical Test Results for the clays and silts

Test	Range
SPT 'N value'^	6 – 64
Moisture Content (%)	13.3 - 36.4
Liquid Limit (%)	26 - 49
Plastic Limit (%)	16 - 27
Plasticity Index (%)	7 - 25
Undrained shear strength (kN/m^2) – Hand Penetrometer Field Test	35 - >300
Undrained shear strength (kN/m^2) – Undrained Triaxial Test	16
California Bearing Ratio (%)	6.2 – 7.1
Sulphate Content (g/l)	<0.010 – 0.34
pH	5.7 – 10.7

^SPT-N Values are uncorrected

The results of Atterberg limit testing completed on the shallow soils indicate they comprise low to intermediate plasticity clays and silts. The full plasticity testing results are illustrated in an A line plot in Figure 11 Appendix B.

One undisturbed sample of the clay was recovered from BH102 at 1.50m and undrained triaxial testing was undertaken. The results returned an undrained shear strength of 16 kPa indicative of soft clay. Discussions with the testing laboratory regarding the low shear strength suggested that the sample was fissured and that the low strength related to failure along one of these pre-existing planes rather than failure of the intact soil matrix. Furthermore, it should be noted that the sample was at ~ 300mm below the base of the overlaying made ground. As such the shear strength recorded may also relate to slight disturbance/reworking of the soil from placement of the overlying made ground. It is therefore considered that the recorded shear strength from the triaxial testing is not indicative of the wider soil strengths on site.

Uncorrected SPT N values recorded within the shallow soils ranged from N = 6 to N = 64 indicative of firm to very stiff/hard clays and silts. It was noted that the test yielding the lowest SPT value was completed at 1.0m bgl within WS108, described as a grey clay and recorded firm (indicative) N values and a soft i(ndicative) undrained shear strength of 35 kPa, which was also the lowest reading recorded using a hand penetrometer test.

This test was completed within soft becoming firm grey clays which were noted in only two locations on site, namely WS108 and BH102. Stroud (1974)[20] found that a good correlation exists between the undrained shear strength of over-consolidated clays and their corresponding SPT blow count N value, where:

$$C_u = f_1 \times N$$

f_1= factor between 4-6 relating to the plasticity of the clay.

SPT blow counts of N=6 to 64 were recorded within the shallow soils. Using Stroud's factor f_1 = 5 for the soils encountered, the corresponding undrained shear strengths vary from 35-495kPa, indicative of soft to very stiff soils. Disregarding the low results from WS108 this range reduces to 55kPa to 495kPa indicating firm to very stiff soils.

Table 17: Summary of PSD test results for the clays and silts

BH/TP Ref	Depth (m)	Clay (%)	Silt (%)	Sand (%)	Gravel (%)	Cobbles (%)
TP102	1.8	10.5	85.8	2.4	1.3	0
TP111	1.7	44.1	54	1.3	0.6	0
TP124	2.3	18.7	62.7	3.3	15.3	0
TP135	1.6	25.6	67.7	5.9	0.8	0
WS106	1.9	18.5	68.4	5.9	7.2	0

The results of particle size distribution testing indicate that the shallow weathered portion of the Upper Tunbridge Wells Sand deposits comprise slightly gravelly slightly sandy clayey silts.

14.4 Interbedded Siltstone, Mudstone and Sandstone - Upper Tunbridge Wells Sand

The clays and silts were underlain by interbedded grey mudstones and siltstones and sandstones to the full depth of investigation. The results of laboratory and *insitu* testing undertaken on the rocks at depth are summarised below.

[20] Stroud M A "The Standard Penetration Test in Insensitive Clays and Soft Rocks" Proceedings of European Symposium on Penetration Testing, Stockholm 1974

Table 18: Summary of Geotechnical Test Results for interbedded Upper Tunbridge Wells Sand formation units

Test	Range
SPT 'N value'	29 - >50
Uniaxial Compressive Strength (MPa)	0.377 – 27.3
Point Load Index $I_{s(50)}$ (MPa)	0.01 – 2.54
Sulphate Content (g/l)	<0.010
pH	6.6

The results of Uniaxial Compression Strength (UCS) testing ranged from 0.377 MPa to 27.3 MPa indicating that the intact Upper Tunbridge Wells Sand units on site range from extremely weak to moderately strong. However, only one sample recorded UCS values indicative of moderately strong sandstone namely BH106 at 17.8m. It is also noted that just three samples, BH101 at 11.7m, BH102 at 3.5m and BH106 at 9.0m recorded UCS values indicative of extremely weak rock. Furthermore, all three of these samples were taken from the mudstone, whilst all samples of the siltstone and sandstone tested exhibited very weak to weak compressive strengths. A plot showing the UCS results vs depth is available in Figure 12 Appendix B.

Point load testing was also completed on samples of the intact Upper Tunbridge Wells strata. The results ranged from 0.01 MPa to 2.54 MPa. Using a conservative factor of C = 10 then the approximate UCS can be estimated using the following conversion:

$$UCS = L_{s(50)} \times C$$

Using the above conversion the $L_{s(50)}$ results equate to an approximate UCS range 0.3 MPa to 9.7 MPa for the mudstones and 0.1 MPa to 25.4 MPa for the sandstone. These point load testing results are in general accordance with the UCS testing and indicate that the mudstones are extremely weak to weak and the sandstones range from extremely weak to weak with one sample verging on moderately strong.

F GEOTECHNICAL APPRAISAL

The foregoing geotechnical appraisal does not constitute a Geotechnical Design Report in accordance with BSEN1997. The following recommendations are for preliminary design purposes only.

For the detailed design, the short-term and long-term design situations must be considered. Where relevant, the following limit states should be considered:-

- Loss of equilibrium of the structure or the ground, considered as a rigid body, in which the strengths of structural material and the ground are significant in providing resistance (EQU)
- Internal failure or excessive deformation of the structure or structural elements in which the strength of structural materials is significant in providing resistance (STR)
- Failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance (GEO)
- Loss of equilibrium of the structure or the ground due to uplift by water pressure or other vertical actions (UPL)
- Hydraulic heave, internal erosion and piping in the ground caused by hydraulic gradients (HYD)

The following factors should also be considered.

- Overall stability and ground movements
- Nature and size of the proposed construction including the design life
- Conditions with regards to the surroundings (e.g. neighbouring structures, traffic, utilities, vegetation, contamination etc.)
- Ground and groundwater conditions
- Influence of the environment

15 Swelling and Shrinkage

Based on the laboratory test results in Section E, an overall classification of NHBC **MEDIUM** Volume Change Potential (VCP) is recommended for the clay soils.

Foundations will therefore require deepening in accordance with NHBC Chapter 4.2 where shallow clay soils are encountered near trees. Foundation depths should be calculated based on the mature height of the tree, however, the existing height is relevant for trees which are to be removed. Deepening may be terminated where the mudstone, siltstone or sandstone are encountered at depth.

16 Sulphates

16.1.1 Made Ground Soils

Three made ground soil samples were tested for sulphate content and pH. The limited geotechnical testing carried out on the made ground soils classified the material as sulphate design class DS-2, and, assuming a mobile groundwater table, the ACEC class is AC-2 (using approaches described by BRE).

16.1.2 Tunbridge Wells Sand Formation

One siltstone sample at depth and eleven samples of silts and clays were tested for sulphate content and pH. In accordance with the BRE[21] methodology and the number of samples tested, the site has been classified on the mean of the highest 20% of the measured sulphate content.

Based on the results detailed in Section E and the above, the natural clays and silts and the Upper Tunbridge Well Sandstone Formation are classified by the BRE as sulphate design class DS-1, and, assuming a static groundwater table, the ACEC class is AC-1.

16.1.3 Groundwater

Based on the pH and sulphate results gathered from the groundwater monitoring programme detailed in Section 12.4, the deep groundwater of the site is classified by the BRE as sulphate design class DS-1, and, assuming a static groundwater table, the ACEC class is AC-1.

17 Groundwater

A total of 41 No. groundwater strikes / seepages were reported during this phase of investigation. The majority of these occurred at depths ranging from 0.60m to 1.2m bgl and generally coincided with the contact between the base of the made ground and the natural clay/silt soils below. Deeper groundwater strikes were recorded within the rotary boreholes at depths ranging from 1.3m to 19m bgl. Groundwater monitoring has recorded the groundwater table within the deep monitoring wells at between 7.0 to 16.80 m bgl (49.93 to 40.76 m AOD) with shallower depths to groundwater recorded on the southern side of the site. The groundwater within the shallow monitoring wells was encountered from 0.10 to 1.80 m bgl (57.38 to 54.89 mAOD) with shallowest depths in areas of made ground.

The full groundwater monitoring results/observations are given in Section D. On the basis of these results, groundwater within the Tunbridge Wells Sand is anticipated to be at below 9-10m depths across the majority of the site, but is shallower in the south east (BH101) of the

[21] Building Research Establishment Special Digest 1: 2005. Concrete in aggressive ground. Part 1: Assessing the aggressive chemical environment.

site at around ~7m. However, significant quantities of shallow perched water should be anticipated within the made ground across much of the site. As such allowance should be in place for pumping of excavations with side support where excavation in granular, reworked or made ground material is required. It is advised that a more comprehensive dewatering long term strategy of the made ground soils is considered as opposed to a method where pumping out is undertaken in a single excavation.

It is noted that the large volume of made ground in the east could be acting as a sump for infiltration and may also be collecting surface water run-off from historic channels and/or damaged surface water drains. It is recommended that the surface water drainage system is camera surveyed to confirm its integrity and to ensure that no historic connections are feeding water into the made ground. Following that, and any necessary repairs, it is may also be necessary to undertake more extensive control of perched groundwater in the made ground (e.g. by recompacting existing made ground to reduce pore space and protecting the surface) in order to allow excavations to be safely constructed in the made ground soils.

18 Soakage Potential

18.1 Test Results

3 No. soakage tests were undertaken on site in accordance with BRE365[22]. Given the time restrictions, it was only possible to fill and test each pit once. TP126A and TP147 soakage test pits were installed within natural and reworked clays and silts, which were underlying the demolition made ground present at surface, whereas TP138 was installed mainly within clay and siltstone. The results are attached in Appendix G and are summarised in Table 19 below:

Table 19: Summary of soakage tests

Test Location	Soakage Rate		Comments
	m/s	l/m²/min	
TP126A	1.67E-07	0.0088	Water level did not fall to 75% of the maximum water depth and as such the results are not compliant with BRE DG365 requirements.
TP138	-		Water level remained unchanged for 5 hours at which point the test was abandoned.

[22] Building Research Establishment DG365: 2016. Digest Soakaway Design

TP147	-	Water level remained unchanged for 5 hours at which point the test was abandoned.

Prior to testing standing water was noted in all tests pits at depths ranging from 0.37m bgl in TP147 to 1.8m bgl in TP126A. Due to the elevated water levels recorded within TP138 and TP147 it was not possible to fill the pits with a sufficient input volume to undertake a representative test. However, water levels within the pits were recorded over a 5-hour period during which no level changes were recorded.

In order to complete the testing in TP126A the pit was filled to 1.0m bgl with the intention to monitor water levels until they returned to resting water levels at ~1.8m bgl. The soils encountered at the monitored depth range were recorded slightly gravelly clayey silts. The test was undertaken over a period of 6 hours and a soakage rate of 1.47E-07 m/s (0.0088 l/m^2/min) was recorded.

18.2 Recommendations

Based on the results of soakage testing completed to date along with the shallow groundwater levels recorded across the site it is considered that the soils on site are not suitable for soakaway drainage and allowance should be in place for surface water to be piped off site.

19 Bearing Capacity and Foundations

19.1 Shallow Foundations

19.1.1 Eastern Area - Commercial

Based on the consistent thicknesses of made ground >2.0m on the eastern half of the site it is considered that shallow foundations are not suitable and that piled foundations would be required in this area. Vibro compaction may be a viable foundation solution, although some of the made ground may be difficult to penetrate and foundation trenches difficult to excavate where groundwater levels are very shallow.

19.1.2 Western Area - Residential

Except for the area of previous footprints of buildings 18 and 38 and land south of Building 3/36 (including WS103,WS107, TP107, TP108 and BH105) made ground on the western portion of the site was found to either not be present or only extend to ~1.20m bgl. As such it is considered that traditional strip foundations would be appropriate in this portion of the site for structures up to 3-4 storeys. All loads should be transferred beneath any topsoil, made ground, loose, soft, low strength, desiccated or disturbed soils and transferred onto the stiff clays at depth. For low rise housing, based on local experience and BS8103[23] Table 8, an allowable bearing capacity of 100 kPa is recommended for shallow foundations on the stiff clays below about 1.2m. However, foundations will require deepening in the clay soils near trees to NHBC Medium VCP precautions.

Where foundations are stepped or span different soil types, allowance should be made for nominal reinforcement.

19.1.3 Construction Supervision

Foundation excavations should be inspected by a suitably qualified engineer prior to pouring concrete to ensure competent soils, most notably to ensure made ground is fully penetrated and that no soft grey silty clays or any other low strength soil is present at the bases of foundation formations.

19.2 Piling

Piled foundations are likely to be required for taller residential structures and all buildings on deeper made ground. For the ground conditions encountered on site, driven, bored or continuous flight augered (cfa) piles may be considered. However, given the proximity of adjacent buildings and railway surrounding the site, driven piles may be excluded due to the vibrations induced during installation and the effect these may have on existing buildings. Therefore, conventional bored or cfa piles may be more suitable. Discussions with previous site users indicate that several, below ground structures including services ducts and drainage culverts were previously present on site. Significant loss of concrete can occur where cfa piles encountered voids. Therefore, if cfa piles are to be adopted, probing of the proposed pile locations is recommended and allowance may need to be made for casing.

Given the potentially unstable nature of the made ground at depth, particularly below the water table, careful monitoring of the bored piles will be required when withdrawing temporary casings or the flight auger, to prevent necking of the pile shafts. This problem should be discussed with the specialist contractor and a program of integrity testing may be required.

Mudstone, siltstone and sandstone should be anticipated underlying the made ground and the clays and allowance should be made for drilling through layers of rock.

BS8103-1:2011 Structural design of low-rise buildings. Code of practice for stability, site investigations, foundations, precast concrete floors and ground floor slabs for housing

Discussions should be held with the specialist piling contractors to assess the technical and financial merits of their various systems for the ground conditions encountered, and the depth and size of pile required, with an adequate safety margin.

The final pile design will be determined by the selected piling contractor as the method of pile formation will affect the bearing capacity. For the purpose of preliminary design, the following soil model may be adopted:

Table 20: Soil model for preliminary pile design

Depth to base (m)	Soil Type	Characteristic Shear Strength (kPa) / angle of shearing resistance (°) / UCS (MPa)
3.45	Made Ground	Ignore Skin Friction
4.0	Clay / Silt	150 kPa
6.0	Mudstone/Sandstone/Siltstone	1 MPa
>20	Mudstone/Sandstone/Siltstone	10 MPA

Given the ground conditions on site and the presence of mudstone/sandstone at depth, rock socketed piles may be considered. The final design must be carried out by the specialist contractor in accordance with their preferred method of formation. Piling schemes which limit or negate the need for subsurface ground beams may also be considered beneficial in areas where groundwater levels are very shallow.

20 Floor Slabs

With reference to NHBC Standard 5.2, suspended floor slabs are recommended for residential properties (west of site) where:-

- Where the depth of fill exceeds 600mm
- Where foundations are deepened below 1.5m in accordance with NHBC Standards Chapter 4.2
- Where desiccated soils are encountered
- Where vibratory ground improvement techniques have been used
- Where foundations have been piled

In the east of the site, given the thickness of the made ground encountered, the medium VCP of the cohesive soils and the recommendation for piled foundations, suspended floor slabs are also recommended for the proposed commercial properties. Should larger storage /

warehouse buildings be proposed, consideration could be given to ground bearing slabs supported on vibro-compacted or re-engineered made ground.

21 Roads

CBR testing across much of the site was considered impractical given the thickness and variability of the made ground, which would prohibit road formation level founding on the natural soils. As such the limited laboratory CBR testing and DCP CBR testing were focussed on the western portion of the site, where natural soils appeared to be at surface and where made ground thicknesses were shallower.

21.1.1 Test Results

A single laboratory CBR test was undertaken on samples of the shallow clayey silts from TP127 on the eastern half of the site and returned CBR values in the range of 6.2% to 7.1%. The results are attached in Appendix F.

A series of 6 No. dynamic cone CBR tests were carried out in the north western portion of the site (where natural soils had been encountered at shallow depths). The results ranged from 0.3% to 126.8%. However, this very high CBR value was only encountered within one location DCP 1 and it is considered that this represents the probe impacting an obstruction (likely a flint, brick or concrete cobble) at depth rather than a true representation of the bearing ratio of the soils. Away from this anomalous result the CBR values recorded ranged from 0.3% to 13.1%. Assuming nominal formation depth of between 400mm and 600mm bgl the tests results range from 2.9% to 13.1%.

Based on the results of the plasticity testing (see Section E) along with the high silt content recorded during PSD testing, the soils on site are considered potentially frost susceptible.

21.1.2 Recommendations

A design CBR value of 3% is recommended for a silty clay / clay silt subgrade.

Made ground soils are by nature highly variable. Where such soils are encountered at formation level or where soft soils occur at formation level, over excavation to at least 1m below ground level, proof rolling and then engineered filling to formation level with a suitable granular fill is recommended in accordance with an appropriate specification such as The Specification for Highways Works Series 600. It is possible that the granular fill may be able to be sourced from the existing made ground subject to screening and confirmatory classification testing.

Further confirmatory testing may be required if the proposed roads are to be adopted.

22 Excavations

Excavations in the made ground and reworked natural soils are likely to be unstable and subject to collapse even over the short term. Furthermore, abundant shallow groundwater seepages were recorded in the made ground, especially at the interface between the made ground demolition fill and the underlying low permeability clays, silts or the interbedded mudstones and siltstones. As such excavations in made ground and any granular soils will be unstable. Excavations in the silty clays should be stable in the short term. However, they will be unstable in the long term and should be backfilled as soon as possible. Excavations are likely to require dewatering especially during the winter months.

Appropriate Health and Safety precautions must be adhered to where man entry into excavations is required. Even stiff clay soils at shallow depth can collapse, particularly following wet weather. Alternative foundation methods may need to be adopted where trench footings cannot be safely or satisfactorily constructed.

23 Filling

The preliminary recommendations contained within this report assume that ground levels are to remain at a similar level across the site for the proposed development, and that no significant changes in level are proposed. In the event that ground levels are to be raised, this may induce significant settlement, particularly across the areas of deep made ground and soft clays, which could adversely affect foundation design, drainage etc. Where significant changes in ground levels are proposed then further investigation may be required, particularly in areas of the infilled clays pits, to assess the impact of such earthworks on the above recommendations.

24 Slope Stability

As described in Section B the north-eastern and eastern parts of the site appears to have undergone a previous cut to create a level platform for the pharmaceutical works. Overall, the site is currently relatively gently sloping. However, there are steep slopes (circa 20˚) in the vicinity of the railway line to the east of the site, and on the boundary with Parsonage Road along the north-eastern side of the site. Detailed consideration of these slopes is outside of the brief of this investigation and will be required as part of final design. It is broadly assumed that the existing slopes will either be part retained, or reduced in height and slope as part of any landscaping design of the eastern part of the site. At this stage, and in the absence of detailed assessment, the following preliminary parameters could be adopted for initial slope stability assessment in the natural tunbridge wells sands, to be confirmed as part of any final design where slopes greater than say 7 ˚ are to remain.

$\gamma_b = 19 kN/m^2$ $\varphi' = 26° \; c'=0 \; kN/m^2$

Provided no significant level changes are proposed, no further **site wide** slope stability issues are anticipated.

25 Retaining Walls

LEAP has not been made aware that any retaining walls are proposed on site, although noting the above comments some consideration may need to be given to parts of the north eastern and eastern boundaries.

A 2m high retaining crib wall was noted on part of the eastern boundary of the site during the walkover. This structure will require further consideration if it is to be incorporated into the final design. In the meantime, care must be taken to ensure that on site activities do not have a destabilising effect on this structure. Activities with the potential to destabilise the wall include, loading of the retained soils, excavation of the soils in in front of the wall and introduction of water into the retained soils.

It is noted that existing retaining walls are present on site therefore assessment of the effect of the new development on these structures may be necessary. For this assessment, the use of the earth pressures for design is recommended based on the following equation:

$$K_0 = 1 - \sin\varphi'$$

For the natural tunbridge wells sands, a φ' of 26° can be used for preliminary design purposes.

26 Settlement

Based on the allowable bearing capacity given in Section 19, settlements of traditional foundations should be within typical tolerable limits for the low-rise development proposed. Significant settlement should be anticipated where made ground soils are loaded, and consideration should be given to pre-treatment or re-engineering of made ground where loads are to be applied.

If soft clays are encountered at formation level or below, significant settlement should be anticipated and serviceable limit state analyses will be required. Where foundations are stepped or span different soil types, differential settlement should be anticipated and allowance should be made for nominal reinforcement.