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STUBBINGTON STUDY CENTRE, STUBBINGTON, HAMPSHIRE

GEOTECHNICAL REPORT ON THE GROUND INVESTIGATION

Report No. LW23157 August 2012

Report prepared for the benefit of:

Hampshire County Council
The Castle
Winchester
Hampshire
SO23 8UD

ASHDOWN SITE INVESTIGATION LIMITED

STUBBINGTON STUDY CENTRE,
STUBBINGTON, HAMPSHIRE

GEOTECHNICAL REPORT
ON THE
GROUND INVESTIGATION

Report No. LW23157 August 2012

FACTUAL REPORT	
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GEOTECHNICAL ASSESSMENT	
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STUBBINGTON STUDY CENTRE, STUBBINGTON, HAMPSHIRE

GEOTECHNICAL REPORT ON THE GROUND INVESTIGATION

TABLE OF CONTENTS

STUBBINGTON STUDY CENTRE, STUBBINGTON, HAMPSHIRE

GEOTECHNICAL REPORT ON THE GROUND INVESTIGATION

TABLE OF CONTENTS

1. INTRODUCTION

As part of an overall renovation scheme of the Centre it is proposed to demolish the south western dormitory and the southern learning spaces and construct three new dormitories to the east of the games room and dining room and one to the north west of the games room. A link way is proposed between the dormitory buildings and the existing games room. The proposed new structures are single storey and anticipated to be of a timber construction.

Ashdown Site Investigation Limited was commissioned to carry out a ground investigation and geotechnical assessment of the site by the:

Geotechnics Group
Economy, Transport and Environment Department
Hampshire County Council
The Castle
Winchester
Hampshire
SO23 8UD

The scope of the investigation, specified by Hampshire County Council, and information relating to the Geotechnical Work Package was received from the client on the 16th June 2012. A purchase order (No. 9002443904) for the works was received, dated 24th July 2012. The information provided by Hampshire County Council pertaining to the project is presented in Appendix C.

The purpose of the works was to:

i. assess ground and shallow groundwater conditions prevailing at the site;
ii. undertake preliminary contamination testing;
iii. investigate the form of foundations to existing structures; and
iv. provide information to assist others undertaking design of spread foundations, ground floors and soakaways.

The analysis and discussions and information contained in this report is based on the ground conditions encountered during the recent site work together with the findings from a programme of laboratory analyses. The possibility of a variation in ground and groundwater conditions away from the positions investigated should not be overlooked. Groundwater conditions can vary both seasonally and due to other effects.

The investigation was undertaken and the report prepared for Hampshire County Council and their consultant partners. It is noted that the investigation was undertaken specifically for the proposed development identified in the Geotechnical Work Package and the recommendations given may not be appropriate to alternative schemes. The copyright for the report and licence for its use shall remain vested in Ashdown Site Investigation Limited (the Company) who disclaim all responsibility or liability (whether at common law or under the express or implied terms of the Contract between the Company and the Client) for any loss or damage of whatever nature in the event that this report is relied on by a third party, or is issued in circumstances or for projects for which it was not originally commissioned, or where the exploratory hole records and test results contained therein are interpreted by anyone other than the Company.

2. FACTUAL REPORT

2.1 Site Details

The site was visited by Ashdown Site Investigation Ltd and a representative from the study centre on the 4th July 2012 in order to ascertain site conditions in advance of the ground investigation works.

The site comprises an approximately 5 Ha irregularly shaped plot of land located to the west of Stubbington Lane, Stubbington, Hampshire and is centred on the approximate Ordnance Survey national grid reference SU 551 019. A site location plan and site plan are presented as Figure 1 and Figure 2 respectively.

2.1.1 Site History

An examination of historical Ordnance Survey maps was undertaken by Hampshire County Council to determine the history of the site and surrounding areas and is provided as Appendix C. A brief summary of the review is included below.

The review states that the site was undeveloped agricultural land until the publication of the contemporary maps. The area surrounding the site is shown to be undeveloped on the Epoch 1 maps (1843 to 1893). However, the Epoch 2 maps (1891 to 1912), show an ‘old gravel pit’ to have been present approximately 110m to the south west of the Site. The later epoch maps show the gradual expansion of the gravel pit, and that at no time did it encroach onto the site. The review goes on to state ‘that there was no evidence that the pit had been in filled and no landfill whether former or current is identified’.

2.1.2 Walkover Survey

The site is occupied by a field study centre consisting of a number of single storey timber structures comprising dormitories, kitchens, offices, and teaching spaces. These are set in approximately 5 Ha of relatively level, soft landscaping. It is bounded to the north, east and south by residential properties and the west by open parkland.

The area of the proposed development is located across the centre of the site, adjacent to the existing grass surfaced car park, asphalt surfaced tennis courts and the area to the north west of the games room.

A site plan is provided as Figure 2.

2.2 Geology Hydrogeology and Hydrology

2.2.1 Expected Geology

The stratigraphic succession expected to underlie the site is presented in the following table. In preparing the table reference has been made to the British Geological Survey 1:50,000 series scale map of the area, information provided by Hampshire county Council and the British Geological Survey lexicon of named rock units.

Table 1. Expected Geological Strata

Type	**Stratum**	**Age**
Superficial Deposits	River Terrace Deposits	Quaternary
Bedrock Geology	Selsey Sand Formation (Bracklesham Group) Undifferentiated Earnley Sand Formation and Marsh Farm Formation (Bracklesham Group)	Eocene

River Terrace Deposits

The River Terrace Deposits generally comprise well graded sandy fine to coarse gravel of flint. Locally sand or gravel strata may predominate. Gravels normally include a high proportion of subangular to angular flint.

The deposits mark the outlines of an earlier east-west flowing river (the ‘Solent River’). The River Terrace Deposits are commonly referred to as the Plateau Gravel, to distinguish the strata from the lower Terrace Gravel that is more closely related to the existing drainage pattern.

Selsey Sand Formation

The Selsey Sand Formation is shown to outcrop over the southernmost part of the site only. Typically it comprises silty sand, silty clay and sandy clayey silt, that is glauconitic, bioturbated and locally calcareous. The Selsey Sand Formation rests on the Marsh Farm Formation at a sharp contact on a burrowed surface overlain by glauconitic sandy silt with a few pebbles.

Undifferentiated Earnley Sand Formation and Marsh Farm Formation

The Earnley Sand Formation typically comprises glauconitic silty sands and sandy silts. Pebble beds may be present towards the base of this sequence. The Marsh Farm Formation typically comprises laminated clay, wavy to lenticular bedded sand interbedded with clay and fine to medium grained sparsely glauconitic sand with laminae and intercalations of clay.

Previous investigations undertaken at the site are reported to have generally encountered Topsoil or made Ground over a succession of clay with gravel (River

Terrace Deposits) in turn overlying a sequence of interbedded sands and clays (Brackelsham Group).

Radon

The Health Protection Agency (HPA) and its predecessor the National Radiological Protection Board (NRPB) have recommended to government that excessive exposures to radon should be reduced. The HPA advise that the Action Level for radon in homes should be 200 Bq/m3, averaged over a year. Radon affected areas are defined as those areas with 1% or more of homes above this action level. Monitoring is only recommended above the Action Level.

The HPA has published Atlas maps showing estimated radon potential. These maps present the percentage of homes above the Action Level at a resolution of 1km square. The maps include consideration of British Geological Survey (BGS) data relating levels to underlying geology. These Atlas maps are indicative rather than definitive. More detailed information is available as a geographical information dataset licensed by the BGS.

Inspection of the relevant HPA Atlas map of the area shows that less than 1% of the houses are above the action level. Based on reference to this indicative document, no specific form of protection is therefore likely to be needed in connection with radon gas. However, as the definitive dataset has not been consulted, it is recommended that the local authority and/or any relevant building control organisation be contacted to establish their requirements or alternatively more detailed information regarding the likely radon levels particular to the site obtained.

2.2.2 Hydrogeology

Aquifer Classification

The Environment Agency designates aquifers in accordance with the Water Framework Directive. These designations not only reflect the importance of aquifers in terms of groundwater as a resource (drinking water supply) but also their role in supporting surface water flows and wetland ecosystems.

The aquifer designation data is based on geological mapping provided by the British Geological Survey. Reference has been made to Aquifer Designation Maps available on the Environment Agency website (http://www.environment-agency.gov.uk).

The River Terrace Deposits and the Bracklesham Group deposits expected beneath the site are classified as Secondary A Aquifers. Secondary A Aquifers are characterised by permeable layers capable of supporting water supplies at a local rather than strategic scale, and in some cases forming an important source of base flow to rivers. These are generally aquifers formerly classified as Minor Aquifers.

Source Protection Zones (SPZs)

The site does not lie within an Environment Agency Source Protection Zone with regard to the protection of the quality of groundwater that is abstracted for potable supply. No potable water abstraction licenses are indicated within the area of the site.

2.2.3 Hydrology

The nearest significant surface water feature is the Solent, located approximately 200m to the south of the Site. The site does not lie within an Environment Agency Zone 2 or Zone 3 floodplain.

2.3 Ground Investigation

2.3.1 Introduction

The ground investigation comprised the excavation of a series of dynamic sampler boreholes together with hand excavated soakage pits and foundation inspection pits. The fieldwork was carried out between the 10th and 11th July 2012. The exploratory hole locations are shown on the Site Plan (Figure 2).

Descriptions of the strata encountered and comments on groundwater conditions are shown in the exploratory hole records given in Appendix A. Notes to assist in the interpretation of the records are also contained in the appendix.

2.3.2 Methodology

2.3.2.1 Dynamic Sampler Boreholes

Nine boreholes (designated WS1 to WS9) were drilled to depths of between 2.2m and 4.0m below ground level.

Dynamic sampler boreholes are formed by a series of 1.0m long hollow steel tubes, successively reducing in diameter from 100mm to 35mm, driven into the ground by means of a track-mounted drop weight. The sampler is extracted from the ground using a hydraulically operated jack and the enclosed samples recovered in 1.0m long plastic liners. The system enables sub-samples to be taken for detailed examination and laboratory testing.

2.3.2.2 Trial Pits

Three trial pits (designated Soak Pit 1 to Soak Pit 3) were excavated to depths of between 0.3m and 0.4m below ground level, for the purpose of conducting soakage testing.

2.3.2.3 Foundation Inspection Pits

Two pits (designated TP1 and TP2) were excavated using hand-tools to depths of 0.35m and 0.40m below ground level, respectively, to enable inspection of the foundations to existing structures.

2.3.3 Sampling

Undisturbed samples of soil were taken at the depths shown in the exploratory hole records and collected in either plastic bags, plastic tubs or amber jars fitted with gas tight lids. On collection amber jars were stored in cool boxes with cooling blocks to maintain temperatures below 4°C and transferred to refrigerators upon return to the office until forwarded to the external accredited laboratory.

2.3.4 In Situ Testing

The depths of in situ testing, together with the test results, are given on the exploratory hole records or are summarised separately in Appendix A. Notes providing additional information on the tests that were performed are also included in Appendix A.

DPSH Dynamic Probe (Super Heavy) Testing

Dynamic probe testing was carried out adjacent to dynamic sampler boreholes WS1, WS3, WS5, WS7, WS8 and WS9. The dynamic probing was undertaken in accordance with BS EN ISO 22476-2:2005 using a super heavy probing geometry.

The DPSH configuration is similar to that of the standard penetration test (SPT) except that the blow counts are recorded over 100mm increments rather than 300mm, as is the case for the SPT.

Perth Penetrometer Tests

Perth penetrometer tests were undertaken in the coarse grained soils encountered in the trial pits. Perth Pentrometer testing was carried out in accordance with Australian Standard AS 1289:6.3.3-1997, Method of Testing Soils for Engineering Purposes; no equivalent European or British Standard having been published to date.

Undrained Shear Strength

Undrained shear strength determinations were made within samples of the fine grained soils held in the dynamic sampler liners using a hand penetrometer. Although samples taken by the window sampling technique cannot be regarded as being undisturbed for testing purposes, penetrometer testing can provide a useful indication of the strength of the material.

Soakage Testing

Falling head soakage testing was carried out within Soak Pit 1 to Soak Pit 3 at depths of 0.3m and 0.4m, within the River Terrace Deposits. The tests were carried out in general accordance with Building Research Establishment (BRE) Digest 365 (1991), other than the pits were filled only once or twice rather than the 3 times suggested by the digest.

2.3.5 Installations

Groundwater monitoring standpipes were installed to depths of between 2.0m and 3.0m in boreholes WS1, WS4 and WS9. Descriptions of the installations are shown in the exploratory hole records given in Appendix A.

The depths to groundwater were recorded on one occasion on the 26th July 2012. Groundwater levels were also recorded continuously over a fifteen day period using dataloggers installed in borehole WS4. The readings obtained by the monitoring are presented in Appendix A.

2.4 Laboratory Testing

Laboratory testing was scheduled by Ashdown Site Investigation Ltd and approved by Hampshire County Council. Results from the laboratory tests are provided in Appendix B.

2.4.1 Geotechnical Testing

Geotechnical testing was undertaken by Ashdown Site Investigation Ltd in accordance with the methods given in BS1377:1990 Parts 1 to 8 'Methods of test for soils for civil engineering purposes'. Notes to assist with the interpretation of the tests are contained within Appendix B.

The types and numbers of tests carried out are detailed in the following table. The test results are discussed in Section 3.

Table 2. Geotechnical Testing

Type of test	No. of samples tested
Moisture Content	15
Atterberg Limits	15
Particle Size Distribution (Wet Sieve)	16
Particle Size Distribution (Wet Sieve and Sedimentation)	2

2.4.2 Chemical Testing

Chemical testing of selected samples was undertaken by a laboratory with recognised (UKAS and MCERTS) accreditation for quality control.

The types and numbers of tests undertaken are detailed in the following table. Brief comment on the test results is provided in Section 4.

Table 3. Chemical Testing

Determinand	No. of soil samples tested
Arsenic	6
Cadmium	6
Chromium	6
Lead	6
Mercury	6
Nickel	6
Copper	6

Determinand	No. of soil samples tested
Zinc	6
Selenium	6
Hexavalent Chromium	6
Water Soluble Boron	6
pH	6
Total Sulphate	6
Total Cyanide	6
Free Cyanide	6
Sulphide	6
Total Sulphur	6
Thiocyanate	6
Total Monohydric Phenols	6
Ammonium	6
Total Extractable Petroleum Hydrocarbons	6
Asbestos	6
Speciated PAH	6
BRE SD1	6
WAC Suite 2	1

2.5 Ground Conditions

2.5.1 Stratigraphy

2.5.1.1 Surface Covering

In general, each of the exploratory holes was excavated through a surface cover of Topsoil some 150mm to 350mm in thickness.

Within borehole WS7, which was excavated on the car park, asphalt some 120mm in thickness was encountered overlying made ground soils extending to a depth of 300mm below the surface.

2.5.1.2 River Terrace Deposits

Underlying the surfacing, the investigation progressed into undisturbed firm clays and dense sand and gravel deposits extending to depths of between 2.0m and 3.3m below ground level. The full depth of these deposits was not proven at the location of borehole WS9 that was aborted at a depth of 2.2m below ground level due to collapsing ground.

These soils are considered to represent the River Terrace Deposits indicated on the published geological map. The deposits appear to be banded with the generalised succession comprising layers of sandy gravelly clays some 0.5m to 1.0m in thickness, overlying up to 2.0 metres of sand and gravel beds formed of fine to coarse angular gravel of flint and medium to coarse sand.

2.5.1.3 Undifferentiated Earnley Sand Formation and Marsh Farm Formation

Beneath the River Terrace Deposits, where penetrated, the dynamic sampler boreholes encountered generally firm to stiff sandy clays and interbedded clays and silts to the full depth of investigation.

These soils are considered to be representative of the Undifferentiated Earnley Sand Formation and Marsh Farm Formation soils indicated on the published geology map.

2.5.2 Stability and Groundwater Conditions

Instability was recorded locally within the granular soils encountered. Running sand conditions were also noted where groundwater was encountered at shallow depths within the sand deposits, see below.

Groundwater seepages were encountered within the dynamic sampler boreholes at depths of between 1.0m and 2.0m originating within the sand and gravel beds of the River Terrace Deposits.

Standing water depths of between 0.5m and 1.3m were recorded in the boreholes on completion of drilling. Standing water was noted at ground level within borehole WS7 on completion of drilling, this was most probably due to extremely heavy rainfall during drilling and surface runoff entering the borehole.

Standing water depths of between 1.0m and 1.70m were recorded within the standpipes during the monitoring visit. The data logger installed in borehole WS4 recorded depths to groundwater ranging between 0.86m and 1.76m below ground level over the monitoring period that extended from 11th to 26th July 2012.

2.5.3 Existing Foundations

Details of the foundations encountered within the foundation inspection pits are included as Figure 3 of this report.

The foundations appeared to comprise mass concrete pad foundations bearing at a depth of 0.35m below ground level. From measurements of the foundations exposed it is estimated that the pad dimensions are likely to be approximately 0.6m by 0.6m.

The foundations appeared to be supported within the River Terrace Deposits.

3. GEOTECHNICAL ASSESSMENT

The geotechnical assessment has been prepared in connection with the development proposals shown on the drawings contained within Appendix C.

In summary these drawings indicate the proposed development to comprise demolition of the structures within the southern part of the site, refurbishment of structures to be retained and the construction of four new single storey dormitories.

At the time of writing, no details were available concerning the specific loads likely to be applied to the foundations.

3.1 Foundations

3.1.1 Soil Shrinkage/Heave Potential

The fine grained soils of the River Terrace Deposits have been classified as clays of low to intermediate plasticity and with modified plasticity indices in the range of 13% to 22% the soils may be expected to exhibit a low to medium volume change potential.

The fine grained soils of the Bracklesham Group have been classified as clays of low to very high plasticity. Modified plasticity indices in the range 11% to 40% have been calculated for these soils, indicative of low to high volume change potential. However it is noted that only a single sample was recorded to have a modified plasticity of 40% and as such it is considered that a medium volume change potential may be assumed for the fine grained Bracklesham Group soils.

The undisturbed sand and gravel soils of both strata may be considered to be non-plastic and hence non shrinkable.

It is recommended that precautions against shrinkage and heave for any foundation system constructed within the fine grained soils should assume a medium volume change potential and take into account current guidance such as that given by the Building Research Establishment (BRE) or National House Builders Council (NHBC).

Whilst this report has been prepared to provide advice to assist designers in undertaking detailed design, the report itself does not represent a detailed design statement. All detailed foundation design including assessment of minimum founding depths for spread foundations, requirements for sleeving or reinforcing of piled foundations and requirements for placement of void formers must take into account the volume change potential of the soil and the presence of trees (previous, present and proposed).

Whilst a tree survey was beyond the scope of the present investigation it is noted that mature and immature trees were located on the site in the general vicinity of the proposed structures. As recommended within the desk study prepared by Hampshire County council, it is strongly recommended that a detailed arboricultural survey should be undertaken for the site. The survey should be extended to include a review of historical photographs and detailed site plans (if available) to establish the species and location of any felled trees that may affect foundation design. The information obtained from the arboricultural survey, information on proposed planting schemes and the findings of this report should be provided to the structural engineers responsible for design of foundation systems.

3.1.2 Spread Foundations

Any made ground or soils disturbed by the construction or removal of any previously existing foundations and services should be regarded as potentially variable in nature and state of compaction and, as such, unsuitable as a founding medium for shallow footings. New footings should be constructed below such deposits to bear within undisturbed natural soils.

For design purposes, a net allowable bearing capacity of $100kN/m^2$ may be assumed for the construction of spread (pad or strip) foundations up to 1.0m across bearing within the River Terrace Deposits of at least firm consistency or medium dense relative density. The quoted bearing capacity is expected to limit settlement to less than 25mm. Whilst a minimum foundation depth of 0.90m below ground level is recommended where clay soils are present, foundations that are within the zone of influence of trees should be appropriately deepened where clay soils are (or are likely to be) present beneath foundations.

It is noted that groundwater was recorded at relatively shallow depth at the time of the investigation (that corresponded with an unusually wet summer). As discussed under Section 3.2 construction of foundations extending beneath the groundwater table is likely to prove problematic as excavations are expected to be very unstable with potential for running sand conditions to be encountered. Notwithstanding the high groundwater table recorded at the time of the investigation, foundations constructed onto clay soils and within the zone of influence of trees should be deepened to the depths recommended by NHBC or BRE guidance documents, as the effects of any heave are likely to manifest during periods of extended dry weather when groundwater levels may be significantly lower than those recorded by this ground investigation.

Given the potential for mixed soil conditions to be present beneath footings, it is recommended that they be suitably reinforced both top and bottom to minimise the potential for cracking to occur due to differential settlement.

Care must be taken to ensure that new foundations do not undermine or impose additional loading (vertical or lateral) on existing foundations. Similarly, the construction of new foundations must be carefully undertaken and monitored in those situations where temporary works may affect existing foundations.

3.1.3 Piled Foundations

Consideration could be given to supporting the proposed development on piled foundations, which could be sleeved to minimise the risk of damage through soil shrinkage/heave. Pile bores could be supported with temporary steel casing to overcome difficulties of constructing foundations through unstable ground. Further ground investigation works, including deep cable percussion boreholes, would be required to provide detailed advice for piled foundation design.

3.2 Groundwater

The groundwater conditions encountered by the ground investigation, detailed in Section 2.5.2, highlight the potential for a high groundwater table to be encountered during the construction of foundations.

Excavations beneath the water table, which may be required for spread foundations within the zone of influence of trees or for services, will require groundwater control to maintain adequately dry working conditions and excavation stability, particularly in coarse grained soils. Where significant excavations extending below the groundwater table are required, it is recommended that trial pumping from proposed formation depths be undertaken in advance of detailed design to fully establish requirements for groundwater control at this site.

3.3 Stability of Excavations

All excavations must be assumed to be subject to short term instability. Excavations below the water table are likely to be problematic without positive groundwater control as discussed above.

Where personnel access is required to any excavation its stability must be assessed by a suitably qualified and experienced responsible person. Entry into unsupported excavations should only be permitted where they have been assessed as safe to enter but in any case it is advised that personnel access to unsupported excavations greater than 1.2m depth should be prohibited.

Particular attention must be paid to ensuring the stability of adjacent structures.

3.4 Aggressivity to Concrete

In consideration of the soils encountered beneath the site and its historical usage it is considered that ‘natural ground conditions’ may be assumed for the purposes of assessing the aggressivity of the chemical environment for concrete classification (ACEC class). Given the noted occurrence of groundwater ‘mobile groundwater’ conditions should reasonably be assumed.

Chemical analysis of the soil indicates a sulphate content falling into Design Sulfate Class DS-1 of Table C1 of the Building Research Establishment Special

Digest No 1 “Concrete in aggressive ground”, 2005. The results of the pH tests indicate that the underlying soils are neutral.

In accordance with the BRE digest, a DS-1 Design Sulfate Class and an AC-1 ACEC classification should be assumed as a minimum for the design of concrete in contact with the ground at the site.

3.5 Ground Floors

In view of the presence of soils of up to medium volume change potential it is recommended that ground floors be suspended for all sensitive structures.

Ground bearing floor slabs may, however, be considered where made ground soils are not present or are to be removed from beneath building footprints and the depth of foundations required to protect against seasonal soil volume changes close to trees is less than 1.5m. In such circumstances further works should be undertaken to demonstrate, that close to the time of construction, no significant soil desiccation is present. If ground bearing floors are adopted it is recommended that the potential for differential movement both between the floor slab and walls and across the floor slab itself should be anticipated. Such floors should be fully debonded from walls and should be suitably reinforced to enable spanning of soft spots. Formations should be adequately proof rolled and any excessively soft materials excavated and replaced with a suitable engineered fill. The depth of any engineered fill should be limited to a maximum of 600mm.

The detailing of services through, or under, ground-bearing floors should incorporate flexible connections and, where appropriate, enhanced falls.

3.6 Pavement Design

The former Department of the Environment, Transport and the Regions Design Manual for Roads and Bridges, Volume 7 (Pavement Design & Maintenance), Section 2, Part 2 1994 provides a useful correlation between soil type and equilibrium (long term) CBR values. This guidance suggests a design equilibrium CBR value of 4% to 5% applicable to the natural soils for the construction of pavement in average construction conditions and assuming a high groundwater table.

Based upon review of the quoted guidance it is suggested that for the River Terrace Deposits a CBR value of 4% may be adopted for preliminary pavement design.

All formations should be proof rolled and any very loose, bulky, soft, degradable or otherwise unsuitable materials thus identified should be removed and replaced with well compacted coarse grained fill. Prepared subgrades should be protected from severe adverse weather by ensuring they are graded to falls to prevent ponding, and they should be reasonably protected from trafficking during construction.

The subgrade may be assumed not to be susceptible to frost heave.

3.7 Drainage

In situ soakage testing was carried out in general accordance with the requirements of BRE 365 ‘Soakaway Design’. From the results of the soakage tests, calculations were made to determine the infiltration rate that could be expected for soakaways constructed into the underlying River Terrace Deposits.

The soil infiltration rates (f) have been calculated in the following way:

For each test the volume of water lost between 25% and 75% of the test depth was divided by the sum of the average surface area of the sides of the pit during the test monitoring period, and its base area. This figure was then divided by the duration of the test in seconds to give the soil infiltration rate in metres per second.

The following infiltration rates have been derived from the tests.

Soak Pit 1	**f = 3.45 x 10^{-6} m/s**
Soak Pit 2	**f = 7.29 x 10^{-6} m/s**
Soak Pit 3 (Test 1)	**f = 3.52 x 10^{-5} m/s**
Soak Pit 3 (Test 2)	**f = 6.47 x 10^{-6} m/s**

The value ‘f’ is equivalent to the soil infiltration coefficient ‘q’ quoted in the Construction Industry Research and Information Association (CIRIA) Report 156.

It is considered that the soakage rates determined are relatively low. However it is noted that the tests were conducted within the uppermost part of the River Terrace Deposits where fine grained soils were recorded to generally predominate. It is expected that infiltration rates into the underlying coarse grained soils of the River Terrace Deposits would be greater than those recorded. However it is also noted that the high groundwater levels recorded at the site would serve to limit storage capacity. In view of the above the detailed arrangement of any proposed soakaway would need careful consideration to ensure that soakaways are compliant with the requirement to half-empty within a 24 hour period.

To minimise the risk of subsidence, soakaways should be constructed a minimum of 5.0m away from proposed or existing buildings.

In the event that discharge to ground via soakaways is proposed, it is recommended that designers refer to Pollution Prevention Guidance Notes (PPGs) published by the Environment Agency for advice on legal requirements and good practice to reduce risks to groundwater. In particular reference should be made to PPGs 1 and 3, providing general advice on the prevention of pollution and the use and design of oil separators in surface water drainage systems, respectively.

As an alternative to soakaways the construction of peak flow storage tanks connected via attenuated drainage pipes to mains surface water drainage could be considered, subject to approval of the Local Authority and/or relevant water company.

4. CONTAMINATION TEST RESULTS

A full Phase 1 and Phase 2 contamination assessment of the Site was beyond the brief of this investigation, however limited contamination testing was undertaken on selected samples.

Six soil samples were tested for a range of commonly occurring contaminants at the request of Hampshire County Council. The levels of contaminants determined are not considered to be significantly elevated and do not exceed typical Soil Screen Values and Generic Assessment Criteria for the proposed end use.

It is advised that the local authority has ultimate jurisdiction over contamination risk assessment for developments within their jurisdiction and may, by way of imposing planning conditions, require a full risk assessment to be undertaken. Similarly, warrantors may require additional risk assessment to be undertaken.

5. CONCLUSIONS

The following conclusions present a summary of the main findings of the investigation. However, no reliance should be placed on any point of the conclusions until the whole of the report has been read as other sections of the report may put into context the information contained herein.

- The ground investigation confirmed the underlying soils to comprise a shallow thickness of topsoil or made ground, overlying firm clays and dense sand and gravel of the River Terrace Deposits. These soils were, in turn, underlain by firm to stiff sandy clays and interbedded clays and silts of the Bracklesham Group to the full depth of investigation.
- Groundwater was recorded at depths ranging between ground level and 1.70m during the course of the ground investigation works and subsequent monitoring period.
- All excavations must be assumed to be subject to short term instability. Excavations below the water table are likely to be problematic. Entry into unsupported excavations should only be permitted where they have been assessed as safe to enter but in any case it is advised that personnel access to unsupported excavations greater than 1.2m depth should be prohibited.
- Where significant excavations extending below the groundwater table are required, it is recommended that trial pumping from proposed formation depths be undertaken in advance of detailed design to fully establish requirements for groundwater control at this site.
- It is recommended that precautions against shrinkage and heave for any foundation system constructed within the fine grained soils should assume a medium volume change potential and take into account current guidance such as that given by the Building Research Establishment (BRE) or National House Builders Council (NHBC).
- A net allowable bearing capacity of 100kN/m^2 may be assumed for pad or strip foundations up to 1.0m in breadth that are constructed directly onto the River Terrace deposits.
- Construction of foundations extending beneath the groundwater table is likely to prove problematic. Notwithstanding the high groundwater table, foundations constructed onto clay soils and within the zone of influence of trees should be deepened to the depths recommended by guidance documents, as the effects of any heave are likely to manifest during periods of extended dry weather when groundwater levels may be significantly lower than those recorded by this ground investigation.
- Consideration could be given to supporting the proposed development on piled foundations. Further ground investigation works, including deep cable percussion boreholes, would be required to provide detailed advice for piled foundation design.
- A DS-1 Design Sulfate Class and an AC-1 ACEC classification should be assumed as a minimum for the design of concrete in contact with the ground at the site.

- In view of the presence of soils of up to medium volume change potential it is recommended that ground floors be suspended for all sensitive structures. However ground bearing floor slabs may be adopted subject to the precautions detailed within the report.
- An equilibrium CBR of 4% may be assumed for the design of pavement bearing on the River Terrace Deposits. The subgrade is not likely to be susceptible to frost heave.
- It is considered that the soakage rates determined by in situ testing are relatively low. It is also noted that the high groundwater levels recorded at the site would serve to limit storage capacity. In view of the above the detailed arrangement of any proposed soakaway would need careful consideration to ensure that soakaways are compliant with the requirement to half-empty within a 24 hour period.

Ashdown Site Investigation Limited
August 2012

APPENDIX A

Exploratory Hole Notes

In Situ Testing Notes

Exploratory Hole Records

DPSH Continuous Dynamic Probe N100 v Depth Profile

Summary of Trial Pit Falling Head Soakage Test Results

Summary of Standing Water Depths

Standing Groundwater Monitoring Results - WS4

NOTES FOR THE INTERPRETATION OF EXPLORATORY HOLE RECORDS

1 Symbols and abbreviations

Samples

U	'Undisturbed' Sample: - also known as 'U100' or 'U4' - 100mm diameter by 450mm long. The number of blows to drive in the sampling tube is shown after the test index letter in the SPT column.
Uo	Sample not obtained.
U*	Full penetration of sample not obtained.
U**	Full penetration obtained but limited sample recovered.
Pi	Piston Sample: 'Undisturbed' sample 100mm diameter by 600mm long.
D	Disturbed Sample.
R	Root Sample.
B	Bulk Disturbed Sample.
W	Water Sample.
J	Jar Sample (sample taken in amber glass jar fitted with gas tight lid)
T	Tub Sample
E	Environmental Suite (including a jar sample, tub sample and vial sample)

In situ Testing

S	Standard penetration test (SPT): In the borehole record the depth of the test is that at the start of the normal 450mm penetration, the number of blows to achieve the standard penetration of 300mm (the 'N' value) is shown after the test index letter, but the seating blows through the initial 150mm penetration are not reported unless the full penetration of 450mm cannot be achieved. In the latter case, the symbols below are added to the test index letter:-
S*	Seating blows only
S**	Blow count includes seating blows
S++	No penetration
So	'Split spoon' SPT sampler sank under its own weight.
	The test is usually completed when the number of blows reaches 50 (25 blows for seating count). The depths of both the top and bottom of the test drive are shown in the sample column on the Borehole Record. If a sample is not recovered in the sampler, a disturbed sample is over the depth of the test as boring continues.
C	Standard Penetration Test (SPT) conducted usually in coarse grained soils or weak rocks using the same procedure as for the SPT but with a 50mm diameter, 60º apex solid cone fitted in place of the sampler. Variations in test results are indicated by the same symbols as for the SPT (above).
V	Shear Vane Test: Undrained shear strength (cohesion) (kN/m^2) shown within the Vane/Pen Test and N Value column.
H	Hand penetrometer Test: Undrained shear strength (cohesion) (kN/m^2) shown within the Vane/Pen Test and N Value column.
P	Perth Penetrometer Test: See Insitu Testing Notes in Appendix C for full description. Number of blows for 300mm penetration shown under Vane/Pen Test and N Value column. In sand the number of blows is approximately equivalent to the SPT "N" value.

2 Soil Description

Description and classification of soils has been carried out using as a general basis the British Standard Geotechnical investigation and testing – Identification and classification of soil, Part 1 Identification and description (BS EN ISO 14688-1:2002) and Part 2 Principles of classification (BS EN 14688-2:2004).

Fine Grained Soils

The consistency of fine grained soils given in the report is based on visual inspection of the samples and the strength is based on results of in situ and/or laboratory undrained shear strength tests when carried out.

The consistency is determined on the following basis:

Consistency	**Manual Test**
Very Soft	Soil exudes between fingers when squeezed in hand
Soft	Soils can be moulded by light finger pressure
Firm	Cannot be moulded by finger but rolled to 3mm threads without breaking/crumbling
Stiff	Crumbles/breaks when rolled to 3mm thick threads but can be moulded into a lump again
Very Stiff	Cannot be moulded and crumbles under pressure, can be indented by thumbnail

Based on BS EN ISO 14688-1:2002

The terms used for the designation of the undrained shear strength are as follows:

Undrained Shear Strength	
Extremely to Very Low	<20 kPa
Low	20-40 kPa
Medium	40-75 kPa
High	75-150 kPa
Very High	150-300 kPa
Extremely high	300-600 kPa

Based on BS EN ISO 14688-2:2004

Note: The undrained shear strength of the soils is measured either by laboratory testing or in the field using hand penetrometer or shear vane.

It is recognised that any coarse grained soil that has in excess of approximately 35% fine grained soil (clay and silt) can often be expected to behave as a fine grained soil despite the dominance of coarse grained material within the soil mass. To reflect this, it is the soil type that dominates the behaviour of the soil mass that appears on the exploratory hole records.

Coarse Grained Soils

The relative densities of coarse grained soils (sand and gravel) given in the report are based on field estimations and the results of the Standard Penetration Test (SPT) and equivalent correlation from other testing. The classification in terms of "N" Values is as follows:

SPT 'N' Value	**Relative Density**
0-4	Very Loose
4-10	Loose
10-30	Medium Dense
30-50	Dense
Greater than 50	Very Dense

3 Rock Description

Description and classification of rocks has been carried out using as a general basis the British Standard Geotechnical investigation and testing – Identification and classification of rock, Part 1 Identification and classification (BS EN ISO 14689-1:2003).

The description of rock mass includes the type of rock, structure, discontinuities and weathering.

The unconfined compressive strength of rock material is determined on the following basis:

Term	Field Identification	Unconfined Compressive Strength (MPa)
Extremely Weak	Indented by thumbnail	Less than 1
Very Weak	Crumbles under firm blows with point of geological hammer, peeled by pocket knife	1 to 5
Weak	Peeled by pocket knife with difficulty, shallow indentations made by firm blow with geological hammer	5 to 25
Medium Strong	Cannot be peeled or scraped with knife, can be fractured with single firm blow of geological hammer	25 to 50
Strong	Requires more than one blow of geological hammer to fracture	50 to 100
Very Strong	Requires may blows of geological hammer to fracture it	100 to 250
Extremely Strong	Can only be chipped with geological hammer	Greater and 250

The terms describing discontinuity and bedding spacing are as follows:

Bedding Thickness		**Discontinuity Spacing**	
Very Thick	>2000mm	Very Wide	>2000mm
Thick	2000-600mm		
Medium	600-200mm	Wide	2000-600mm
Thin	200-60mm	Medium	600-200mm
Very Thin	60-20mm	Close	200-60mm
Thickly Laminated	20-6mm	Very Close	60-20mm
Thinly Laminated	<6mm	Extremely Close	<20mm

Chalk

Chalk description is based on BS EN ISO 14688 and BS EN ISO 14689. The classification of chalk generally follows the guidance offered by the Construction Industry Research and Information Association (CIRIA) C574, ‘Engineering in Chalk’. This is based on assessment of chalk density, discontinuity and aperture spacing, and the proportion of intact chalk to silt of chalk. See additional chalk classification notes.

IN-SITU TESTING NOTES

1 Standard Penetration Testing

Standard penetration testing (SPT) is carried out within a cased cable percussion borehole. The test is performed using a either a split spoon (barrel) sampler in finer grained deposits, or, in coarser grained soils or weak rocks, using a 50mm diameter, 60º apex solid cone fitted in place of the sampler.

The sampler is driven into the deposits at the base of the borehole by means of a 63.5kg hammer falling freely through 760mm.

In the borehole record the depth of the test is that at the start of the normal 450mm penetration, the number of blows to achieve the standard penetration of 300mm (the "N" value) is shown after the test index letter, but the seating blows through the initial 150mm penetration are not reported unless the full penetration of 450mm cannot be achieved.

(BS EN ISO 22476-3:2005, Geotechnical investigation and testing – Field Testing, Part 3)

2 Dynamic Probe Testing

The DPH (heavy) dynamic probing rig drives a 32mm diameter rod with a 15cm^2 area, 90º end cone into the ground by means of a 50kg hammer which falls freely through a distance of 0.5m. The number of blows per 100mm penetration (N_{100}) is recorded.

The DPSH (super heavy) dynamic probing rig drives a 35mm diameter rod with a 20cm^2 area, 90º end cone into the ground by means of a 63.5kg hammer that falls freely through a distance of 0.75m. The number of blows per 100mm penetration (N_{100}) is recorded. The results can provide a useful indication of the relative strength of the material. The dynamic probing is carried out in accordance with BS EN ISO 22476-2:2005.

A tentative correlation with the Standard Penetration Test (SPT) N value can be made summing three consecutive the N_{100} blow counts.

3 Perth Penetrometer Test

In this test a hardened stainless steel rod is driven into the deposit by a 9.5kg sliding hammer falling freely through 600mm. After an initial penetration of 150mm the number of blows required to drive the rod a further 300mm is recorded. In sand the Perth blow count gives a close correlation to the "N-value" that could be expected from a standard penetration test (SPT) made in similar materials. The results are less reliable in coarser grained materials but can give an indication of their engineering properties. The perth penetrometer test is carried out in accordance with the Australian Standard AS 1289:6.3.3-1997, Method of Testing Soils for Engineering Purposes, there is no European equivalent code.

4 Undrained Shear Strength

Undrained shear strength determinations are made in situ within the fine grained soils using a Geonor hand shear vane or (usually in the case of window sampler boreholes) a hand penetrometer. The test records the undrained shear strength (cohesion) in kN/m^2. The shear vane records a maximum shear strength of 130kN/m^2 and the hand penetrometer records a maximum shear strength of 250kN/m^2.

3 California Bearing Ratio Test

In this test a hand held Farnell cone penetrometer apparatus is pushed into the deposits for the estimation of the California bearing ratio of the subgrade (for use in pavement design). The test equipment is design for the estimation of the bearing ratio of fine grained soils (clay and silt) only and is unsuitable for use in coarse grained soils and rock.