



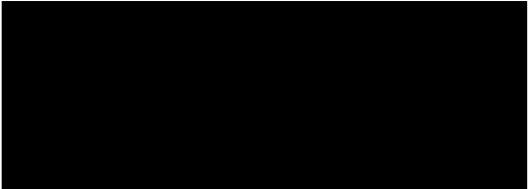
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BELLWAY HOMES WESSEX

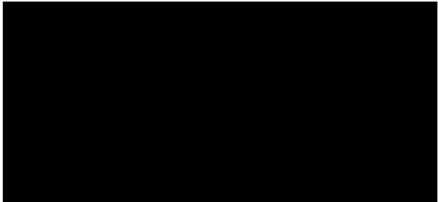
**REPORT
on
GROUND INVESTIGATION
NYTON ROAD
WESTERGATE**

**NOVEMBER 2015
REPORT NO: 730794**

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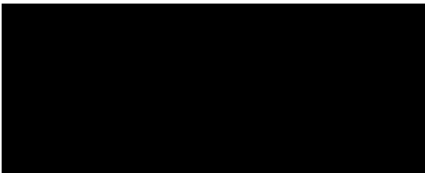


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1 INTRODUCTION

This investigation was carried out on the instructions of and on behalf of Bellway Homes Wessex (the Client). It is proposed to develop 268 residential 2/3 storey houses on farmland enclosed by residential suburbs.

Structural Soils Limited has undertaken a previous investigation of this site, which is detailed in our report of May 2013: Interpretive Report on Site Investigation at Westergate no.727897 (see References).

The purpose of this work was to further investigate ground conditions and provide additional information for foundation design, and to provide information for additional contamination assessment purposes.

The work included an intrusive investigation, laboratory testing and the preparation of this report, which contains a description of the site and the works carried out, the exploratory hole logs, in-situ and laboratory testing results. The report gives recommendations relating to geotechnical aspects such as foundation design. It presents an appraisal of geoenvironmental aspects such as soil contamination and gives recommendations on risk reduction. It should not be assumed that these would meet the requirements of the local authority, whose advice should be sought regarding planning permission.

The ground investigation has been carried out using intrusive ground investigation techniques in general accordance with the recommendations of BS5930: 1999 *Code of Practice for Site Investigations* (including Amendment A2, 2010). Whilst every attempt is made to record full details of the strata encountered in the exploratory holes, techniques of hole formation and sampling will inevitably lead to disturbance, mixing or loss of material in some soils and rocks.

The investigation has been carried out in accordance with BS10175: 2011 *Investigation of Potentially Contaminated Sites: Code of Practice* (including Amendment A1, 2013).

All information, comments and opinions given in this report are based on the ground conditions encountered during the site work, and on the results of laboratory and field tests performed during the investigation. However, there may be conditions at the site that have not been taken into account, such as unpredictable soil strata, contaminant concentrations and water conditions between or below exploratory holes. It should be noted that groundwater levels, gas concentrations and gas flows usually vary due to seasonal, atmospheric and/or other effects and may at times differ to those measured during the investigation.



This report was prepared by Structural Soils Limited for the sole and exclusive use of Bellway Homes Wessex in response to particular instructions. Any other parties using the information contained in this report do so at their own risk and any duty of care to those parties is excluded. No liability will be accepted after a period of 6 years from the date of the report.



2 SITE DESCRIPTION

2.1 Location and Topography

The site is located to the north of Nyton Road in Westergate, approximately 10 kilometres east of Chichester (see Site Location Plan in Appendix A). The British National Grid Reference of the site is SU 941 057.

The site, irregular in shape but broadly rectangular, is approximately 600m east to west by 375m north to south along its longest dimensions (~10ha). The site principally comprises farmer's fields, defined by hedgerows and fences, and is generally flat and grass covered, gently sloping downwards to the south west. At the time of the investigation a disused garden centre nursery was present in the south west of the site and a farmer's compound was present to the east of the centre of the site. The remainder of the site was open farmland (see Exploration Hole Location Plan in Appendix A).

The nursery comprises a large disused greenhouse measuring approximately 100m by 65m and several sheds, trees, flower beds and a pond. A house and a single storey dwelling with a large garden measuring approximately 100m × 100m also occupy this section of the site. The houses, sheds and the greenhouse are serviced by an asphalt driveway that is in a poor state of repair.

The farmer's compound is approximately square, measuring around 50m by 50m, with a large shed measuring 25m by 15m in the centre. At the time of the investigation numerous agricultural machines in a poor state of repair and miscellaneous piles of construction material occupied the western side of the compound.

The site is bounded by housing to the south and further housing and Northfield Lane to the west. The northern boundary comprises further farmland, and a scrapyards and Fontwell Avenue make up the eastern boundary. A stream flows north south along the western perimeter of the site.

Three access points were used during the ground investigation; access to the east of the site via Fontwell Avenue, access to the south west of the site via Nyton Road and access to the north west of the site via Northfields Lane.

No evidence of contamination was noted.

2.2 Geology

Information on the geology of the site was obtained from the following sources published by the the British Geological Survey (BGS):



- BGS survey map (sheet 317/322, scale 1:50,000 published 1996).
- The BGS digital geology map, which utilises the most up to date names for geological units (www.bgs.ac.uk/data).
- The BGS Lexicon of Named Rock Units, which provides typical descriptions for most geological units (www.bgs.ac.uk/lexicon).

The site is shown to be underlain by the following descending sequence of strata:

TABLE 1: SUMMARY OF SITE GEOLOGY	
Geological Unit Name	Description
River Terrace Deposits (Undifferentiated) – partial coverage in the north east and south west of the site.	<i>Sand, Silt and Clay; sand and gravel, locally with lenses of silt, clay or peat.</i>
Head Deposits	<i>Gravel, Silt, Sand and Clay; impure clays, silts and sand, locally gravelly; chalky and flinty in dry valleys.</i>
London Clay Formation	<i>Clay, Silt and Sand - bioturbated or poorly laminated, blue-grey or grey-brown, slightly calcareous, silty to very silty clay, clayey silt and sometimes silt, with some layers of sandy clay.</i>

Note: Information obtained from BGS digital records © NERC.

The British Geological Survey map shows the site to be underlain by Head Deposits (*gravel, sand, silt and clay*) which are in turn underlain by London Clay Formation (*clay, silt and sand*). The published map describes the Head Deposits as variable deposits of *impure clays, silts and sand, locally gravelly; chalky and flinty in dry valleys*.

It is noted that two areas of River Terrace Deposits (Undifferentiated) (*Sand, Silt and Clay*) are recorded on the map; one in the south west of the site, and the second to the north east of the site. These deposits are referred to as Brickearth (*mainly silts, impure contaminated with gravel*) on the 1996 map; however they have more recently been reclassified as River Terrace Deposits.

2.3 Hydrogeology

The Environment Agency (EA) website has classified the site's Drift Deposits (Head and River Terrace Deposits) as a Secondary 'A' Aquifer (variably permeable).

Secondary' aquifers include a wide range of rock layers or drift deposits with an equally wide range of water permeability and storage. Secondary 'A' Aquifers are 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.

The Environment Agency (EA) website has classified the site's solid geology (London Clay Formation) as an Unproductive Strata. 'Unproductive Strata' are rock layers or drift deposits with low permeability that have negligible significance for water supply or river base flow.

The site does not lie within any Source Protection Zones.

2.4 Hydrology

The nearest surface water to the site is a stream flowing north south along the western perimeter of the site. Groundwater contained within the drift deposits is likely to be in potential continuity with this feature.

2.4.1 Site History from Other Sources

Structural Soils Limited has previously conducted a ground investigation at Nyton Road, Watergate (Structural Soils Limited Report No. 727897, May 2013). The ground investigation comprised 2 no. cable percussion boreholes and 5 no. machine dug trial pits which were completed between 18 and 26 March 2013.

The ground investigation revealed a thin layer of topsoil which overlies generally firm slightly sandy slightly gravelly clay to depths ranging from 0.35m to 2.50m. This is in turn underlain by brown very clayey sandy gravel to a depth of 3.50m. Underlying this gravel is a mixture of clay and sand soils to a maximum proven depth of 10m.

A preliminary risk assessment (PRA) that included a historical desk study of the site, together with chemical contamination testing of soil samples was also completed as part of the work.



3 FIELDWORK

3.1 Scope of Works

The following works were completed between 21 and 23 September 2015 at locations shown on the Exploratory Hole Location Plan in Appendix A:

TABLE 2: SCOPE OF INTRUSIVE WORKS AND IN-SITU TESTING		
Quantity	Exploratory Hole or In-Situ Test Type	Hole / Test Numbers
2	Cable Percussion Boreholes.	BH01 & BH02
4	Window Sample Holes.	WS01 to WS04
6	Machine Dug Trial Pits.	TP01 to TP06
3	Soakaway Infiltration Tests in trial pits.	TP01A to TP03A

The exploratory holes typically encountered the following descending sequence of soils: Head Deposits comprising firm to stiff orangish brown sandy gravelly clays and orangish brown clayey gravelly sand over stiff to very stiff grey sandy gravelly clay of the London Clay Formation.

The scope of investigation and choice of investigation equipment was decided by Structural Soils Limited in consultation with Bellway Homes Wessex. Sampling and in-situ testing details were specified by Structural Soils Limited.

The positions were selected by Structural Soils Limited, set out by Structural Soils Limited and adjusted where necessary to take account of buried or overhead services, or other restrictions.

The positions were chosen to target existing or proposed features as follows:

TABLE 3: RATIONALE FOR THE POSITIONING OF EXPLORATORY HOLES	
Exploratory Hole	Reason for Selection of Position
Cable Percussion Boreholes	General site coverage for geotechnical data including the previously inaccessible nursery area. Installation of groundwater monitoring wells.
Trial Pits	General site coverage for geotechnical and geoenvironmental data and soakaway testing.
Window Sample Holes	Geoenvironmental sampling of the previously inaccessible areas; the nursery and the farm compound. Installation of groundwater monitoring wells.



The exploratory holes were logged by an engineer in accordance with the recommendations of BS5930: 1999 (Amendment A2, 2010, which incorporates the requirements of BS EN ISO 14688-1, 14688-2 and 14689-1). Detailed descriptions, together with relevant comments, are given in the logs included in Appendix B.

Prior to the commencement of any exploratory hole or intrusive test all positions were checked for buried services by a specialist utility surveyor using a cable avoidance tool (CAT), signal generator ('genny'), and ground penetrating radar (GPR).

Inspection pits were hand dug at exploratory locations where noted on the relevant exploratory hole logs or in-situ test results.

On completion of the works, a survey of the exploratory hole locations was undertaken using specialist Global Positioning System (GPS) equipment. The coordinates of each exploratory hole were measured relative to British National Grid, and the level relative to Ordnance Datum. These are shown on the exploratory hole logs contained in Appendix B which have been printed with a reduced level column.

3.2 Cable Percussion Boreholes

The boreholes were drilled using a cable tool percussion drilling rig and were 150mm in diameter. The depths of the boreholes were 10.45m. Small and bulk disturbed soil samples were taken from the boreholes at regular intervals.

100mm diameter undisturbed samples were recovered from suitable strata in accordance with BS EN ISO 22475-1 using a thin walled UT100 sampler. Standard Penetration Tests (SPT) were carried out at regular intervals (see In-Situ Testing, below).

3.3 Window Sampling and Dynamic Probing

The window sample exploratory holes were drilled using a tracked rig with chain driven drop weight to drive the sample tubes. Steel sample tubes containing a plastic sample liner were used. The holes reduce in diameter with depth, as reported on the logs. The depths of the holes were 5.0m to 5.45m.

Small and bulk disturbed soil samples were subsampled from the window samples at regular intervals. Hand vane or hand penetrometer tests were carried out at selected intervals on samples of intact cohesive soils, the results of which are included on the logs contained in Appendix B. Standard Penetration Tests (SPT) were carried out at selected intervals in the window sample



holes using the rig's integral drop weight/hammer (see In-Situ Testing, below). Photographs were taken of the recovered window samples, and these are included in Appendix B.

3.4 Trial Pits

The machine dug trial pits were excavated using a JCB 3CX wheeled mechanical excavator and were approximately 2.25m x 0.65m in plan and were up to 3.5m deep.

Small and bulk disturbed soil samples were taken from the trial pits at regular intervals. Hand vane and/or hand penetrometer tests were carried out in the cohesive strata (within the pit where safe to do so, else on excavated material). Photographs of the trial pits are contained in Appendix B.

3.5 Backfill, Monitoring Wells and Installations

On completion 50 or 40mm diameter groundwater monitoring wells and 19mm diameter standpipe piezometers were installed in selected exploratory holes the design having been decided by Structural Soils Limited. The installation details are shown on the exploratory hole logs in Appendix B.

The well details are also summarised below:

TABLE 4: SUMMARY OF MONITORING WELL INSTALLATIONS					
Location	Well Diameter (mm)	Well Depth (m bgl)	Well Response Zone (m bgl)	Type of Protective Cover	Notes
BH01	50	5.0	3.0 to 5.0	Flush protective cover	
BH02	50	5.0	3.0 to 5.0	Flush protective cover	
WS02	40	4.5	3.0 to 4.5	Flush protective cover	
WS03	40	4.5	3.0 to 4.5	Flush protective cover	
WS04	19	4.5	3.0 to 4.5	Flush protective cover	



On completion the remaining boreholes were backfilled with arisings and bentonite pellets. The trial pits were backfilled with arisings, compacted in layers by the excavator bucket.

3.6 Chemical Contamination Sampling

Samples for contamination testing were taken from the exploratory holes where indicated on the exploratory hole logs, recorded as sample types ES for soil or EW for water.

Water samples were also taken from the monitoring wells as described in Monitoring and Post Fieldwork Environmental Sampling, below.

All samples were placed in appropriate contamination sample containers that were supplied by the laboratory. Containers for volatiles testing of soil samples were filled to capacity. Any water sample containers (with preservatives where required) were filled to capacity to minimise volatilisation. All samples were then kept in cool boxes with ice packs and were transported to the laboratories under Chain of Custody documentation, as promptly as possible to maintain sample integrity.

3.7 In-Situ Testing

Standard Penetration Tests (SPT) were carried out in the exploratory holes, where noted in the preceding sections, in accordance with BS EN ISO 22476-3 using hammers which had been calibrated for efficiency. The calibration certificate(s) is included in Appendix C. Seating drives have been recorded in increments of 75mm in accordance with recommended UK practice.

The SPT N-values are reported on the exploratory hole logs, on which the serial number of the hammers used is recorded. The full results are presented in tabular format on the Summary of Standard Penetration Tests in Appendix C, on which the normalised N_{60} values are also reported, which are the equivalent N-value for a hammer delivering 60% of the theoretical drop energy. Plots showing both N and N_{60} values versus depth and elevation are also included.

The Soakaway tests were undertaken in trial pits in general accordance with recommended practice given in BRE Digest 365. Three fillings of the pits were not undertaken in some of the tests due to slow infiltration rates and instability of the pits. The results are contained in Appendix C.

Infiltration rates have not been calculated where standing groundwater was present in the exploratory hole prior to commencement of the test, where the fall in water level was too low to sensibly calculate a soil infiltration rate, or where a rise in water level occurred during the test.



In cases where the test water level did not fall to the 25% full (or both the 75% and 25% full) level, but sufficient readings were obtained, a soil infiltration rate has been extrapolated from the available data as noted on the test results. Results extrapolated in this way should be considered tentative.

3.8 Monitoring and Post Fieldwork Environmental Sampling

Groundwater levels were recorded in the monitoring wells on 8 October 2015. The results together with the temporal (weather) conditions are tabulated in Appendix F. Water samples were taken from the wells for testing during the visit.



4 LABORATORY TESTING

Samples for potential geotechnical testing were returned to one of the Company's UKAS accredited laboratories, and those for potential contamination testing were sent to a sister company Envirolab Limited, a MCERTS and UKAS accredited chemical testing laboratory. Laboratory tests were scheduled by Structural Soils Limited.

4.1 Geotechnical Laboratory Testing

Geotechnical laboratory testing was generally carried out in accordance with BS1377: 1990, *Methods of Test for Soils for Civil Engineering Purposes*, Parts 1 to 8. The number of tests completed and the test methods used are summarised below. Where non-standard procedures have been undertaken, this is recorded on the report sheet. The results are reported in tabular and/or graphical form and included as Appendix D of this report.

TABLE 5: SUMMARY OF GEOTECHNICAL LABORATORY TESTING			
Number of tests	Test	Test Method	Notes
Classification Tests			
8	Moisture content.	BS1377: Part 2.	
8	Liquid and plastic (Atterberg) limits.	BS1377: Part 2.	
9	Particle size distribution by sieving.	BS1377: Part 2.	
Shear Strength - Total Stress			
1	Single stage unconsolidated undrained triaxial compression tests without the measurement of pore pressure.	BS 1377: Part 7.	
Chemical Tests: Soil			
5	Water soluble sulphate content and pH value.	BRE SD1*.	

4.2 Contamination

TABLE 6: SUMMARY OF CONTAMINATION LABORATORY TESTING*			
Numbers of tests	Description		Notes
SOIL			
6	SSL	HHB Human Assessment suite	Comprises arsenic, cadmium, chromium (total), lead, mercury, selenium, copper, nickel, zinc, speciated polycyclic aromatic hydrocarbons (PAH), total petroleum hydrocarbons (TPHCWG (speciated)), organic matter, soluble sulphate and pH.
6	Asbestos presence screen.		Identification was undertaken if/where asbestos fibres were detected.
4	Pesticides		Organochlorine & organophosphorous pesticides
WATER			
2	SSL	GWS Groundwater Screening suite.	Comprises arsenic, cadmium, chromium (total), lead, mercury, selenium, copper, nickel, zinc, speciated polycyclic aromatic hydrocarbons (PAH), total petroleum hydrocarbons (TPHCWG (speciated)), sulphate, hardness and pH.

Note: Tests carried out in accordance with MCERTS/UKAS standards where noted on the results sheets.



5 GROUND CONDITIONS

5.1 General

The exploratory holes were logged by an engineer and the ground conditions encountered are detailed on the logs contained in Appendix B and summarised in the following table:

TABLE 7: SUMMARY OF GROUND CONDITIONS			
Strata	Exploratory holes encountered in	Depth to top of stratum m bgl	Thickness (m)
Topsoil	BH01, WS02, TP01 to TP06	Ground Level	0.2 to 0.25
Made Ground	BH02, WS01, WS03 & WS04	Ground Level	0.1 to 0.3
River Terrace Deposits	BH01 & BH02, WS01 to WS04, TP01 to TP06	0.1 to 0.3	0.3 to 1.0
Head Deposits	BH01 & BH02, WS01 to WS04, TP01 to TP06	0.5 to 1.2	7.1 & 5.3 in BH01 & BH02. All other positions terminated in the Head Deposits.
London Clay Formation	BH01 & BH02	6.5 to 8.3	>3.95

The ground conditions are summarised in more detail below.

5.2 Topsoil

The topsoil comprised soft to firm dark brown slightly sandy slightly gravelly clay in all the exploratory holes to depths ranging between 0.20m and 0.25m. The gravel fraction included fine to coarse, subrounded to subangular flint.

5.3 Made Ground

Made ground was encountered in Window Sample WS01 at the nursery and in Window Sample WS03 & WS04 at the farmer's compound. At the nursery the made ground comprised of grass over firm to stiff high strength brown sandy gravelly clay. The gravel fraction included fine to coarse flint with occasional concrete and brick with abundant glass fragments. At the farmer's compound the made ground comprised black, dark grey and grey slightly clayey slightly sandy gravel with a medium to high cobble content. The gravel fraction included flint, brick, concrete, asphalt and limestone ballast. The cobble fraction included angular limestone ballast and flint.

5.4 River Terrace Deposits (Undifferentiated) (previously Brickearth)

Firm high strength brown slightly sandy gravelly clays or silt with a low cobble content were recorded in all the exploratory holes underlying the topsoil or made ground to depth ranging from 0.5m to 1.2m.



A summary of the geotechnical parameters for this stratum is presented below:

TABLE 8: SUMMARY GEOTECHNICAL PARAMETERS FOR RIVER TERRACE DEPOSITS			
Parameter	Number of results	Range of results	Comments
Undrained Shear Strength (c_u) from in-situ Hand Vane, kN/m^2 .	11	66 to 124	Medium strength to very high strength
Consistency term.	15	Firm to stiff	
Moisture Content %	2	20 to 21	
Liquid Limit %	2	35 to 43	
Plastic Limit %	2	19 to 28	
Plasticity Index %	2	15 to 16	
Modified (for $425\mu\text{m}$ content) Plasticity Index %	2	51 to 90	

5.5 Head Deposits

Head Deposits were recorded in all the exploration positions. These deposits generally comprised a mixture of yellowish brown, greyish brown and brown very clayey very sandy gravel soft to firm creamish white sandy gravelly clay and orangish brown sand.

A summary of the geotechnical parameters for this stratum is presented below:

TABLE 9: SUMMARY GEOTECHNICAL PARAMETERS FOR HEAD DEPOSITS			
COARSE SOILS (SAND / GRAVEL)			
Parameter	Number of results	Range of results	Comments
SPT N-value.	31	3 to 24	
SPT N_{60} value.	31	4 to 28	
Density term.	14	Very Loose to Medium Dense	
FINE SOILS (CLAY / SILT)			
Parameter	Number of results	Range of results	Comments
Undrained Shear Strength (c_u) from in-situ Hand Vane, kN/m^2 .	14	Low strength to very high strength	
Consistency term.	31	Very Soft to Stiff	
Moisture Content %	6	14 to 23	
Liquid Limit %	6	29 to 43	
Plastic Limit %	6	15 to 19	
Plasticity Index %	6	13 to 19	
Modified (for $425\mu\text{m}$ content) Plasticity Index %	6	43 to 100	



5.6 London Clay Formation

The Superficial Deposits were underlain by stiff to very stiff grey to dark grey clay of the London Clay Formation.

A summary of the geotechnical parameters for this stratum is presented below:

TABLE 10: SUMMARY GEOTECHNICAL PARAMETERS FOR LONDON CLAY FORMATION			
Parameter	Number of results	Range of results	Comments
SPT N-value.	4	19 to 22	
SPT N_{60} value.	4	24 to 25	
Bulk density Mg/m^3 .	1	1.97	
Undrained Shear Strength (c_u) from Triaxial Test, kN/m^2 .	1	64	
Consistency term.	3	Stiff & Stiff to Very Stiff	
Moisture Content %	1	30	

5.7 Groundwater

Groundwater was encountered during the investigation within the Head Deposits in all exploratory positions, except TP03, which was excavated on higher ground and the soakaway test pits, which were more shallow. Groundwater was encountered between 1.95m in TP05 and 4.50m in BH01.

On encountering groundwater in the window samples and cable percussion boreholes, boring operations were suspended for a period of 20 minutes and any rise in groundwater level recorded, as detailed on the borehole logs.

The boreholes had water added to them to aid drilling due to ground conditions, therefore groundwater strikes may not have been observable, and/or any standing water levels recorded may have been influenced by the addition of water.

Groundwater levels in the monitoring wells installed on site were measured (see Section 3). Standing ground water levels of 1.31m (BH02, 10.35m AOD) to 3.04m (WS03, 10.52m AOD) were recorded. The standing water level ranged between 10.11m AOD and 10.52m AOD, except in WS04 where it was higher at 11.62m AOD. The monitoring results are contained in Appendix G.

5.8 Indications of Contamination

There were no olfactory or visual indications of contamination in any of the exploratory holes.



The findings of this ground investigation are consistent with those reported in Section 5 of Structural Soils Limited (2013) *Interpretive Report on Site Investigation at Westergate* May 2013 Report No: 727879.



6 GEOTECHNICAL SITE ASSESSMENT

6.1 Proposed Development

It is proposed to develop 268 residential 2/3 storey houses on farmland enclosed by residential suburbs.

The layout of the proposed development is shown on the Proposed Development Layout Plan contained in Appendix A.

6.2 Site Preparation and Excavation

Strip or pad foundation taken to depths of around 1.0m may be used at this site, as discussed in Section 6.4, below.

The soils encountered at the site are considered suitable for excavation by standard mechanical plant such as a wheeled backhoe excavator.

Where trees are removed to facilitate construction, the root boles should be grubbed out to prevent re-growth. Any resulting voids from this operation should be backfilled with an acceptable, adequately compacted fill.

Rapid groundwater ingress into excavations is likely below depths of about 2.0m although this varies with location and elevation, the standing groundwater table at the site is likely to be encountered at about 10.5m AOD. However, it should be noted that groundwater levels may change due to seasonal or other variations. Surface water run-off from rainfall may also enter excavations. Advice on suitable dewatering techniques is given in CIRIA Report C515 *Groundwater Control – design and practice*.

All excavations should be planned and due consideration should be given to providing temporary support or suitable battering. Excavations should be regularly inspected by a competent person to ensure continued safety. Further advice on the safety of excavations is given in *Health and Safety in Construction*.

Unsupported excavations up to 2.0m depth within the River Terrace Deposits and Head Deposits, where above the water table, are likely to remain stable in the short term. Excavations below this depth within the Head Deposits may not remain stable and may require support, as would any excavations taken below the groundwater.

The investigation was undertaken at a time when the ground surface at site was firm and relatively dry. However, given the nature of the near surface soils on the site some softening



would be expected were construction undertaken during wetter parts of the year, and this should be taken into account when designing haul roads or temporary laydown areas.

6.3 Shrinkage and Swelling

Atterberg Limits tests performed on samples taken from showed the following volume change potentials with changes in moisture content, according to the criteria of NHBC Standards, Chapter 4.2 *Building Near Trees*, after correction where necessary for their >0.425mm fraction:

TABLE 11: SUMMARY OF VOLUME CHANGE POTENTIAL OF SOILS WITH CHANGES IN MOISTURE CONTENT		
Stratum	Volume Change Potential	Recommended Volume Change Potential for Design Purposes
Head Deposits	Low (7 results) Medium (1 result)	Low

Note:

1. Based on modified plasticity index, I_p'
2. $I_p' < 10\%$ = Not Shrinkable, 10% to less than 20% = low, 20% to less than 40% = medium, 40% or greater = high.

6.4 Foundations

Given the presence of competent natural soils at a relatively shallow depth it is considered that traditional shallow spread (e.g. strip or pad) foundations will be suitable for the proposed development.

6.4.1 Shallow Spread Foundations

It is recommended that foundations be designed in accordance with the NHBC Standards. On this site the majority of the near surface Head Deposits and River Terrace Deposits comprise coarse soils – typically very clayey sandy gravel – which are not susceptible to shrinkage and swelling. The composition of these soils does vary locally to fine soils – typically medium strength slightly sandy gravelly clay which would be susceptible to shrinkage and swelling (albeit with a low volume change potential).

At a few locations, primarily in the window samples, the clay soils were described as soft. Based on all the observations a minimum undrained shear strength, c_u , of, 40kN/m² may reasonably be assumed for the clay soils at potential founding depths on this site (although this would be conservative in many areas).

Foundations on this site may therefore span both fine ('cohesive') and coarse ('granular') soils. Such changes are likely to occur gradually across the site, however if any sudden changes in



composition were found along a strip foundation consideration should be given to reinforcing the foundation to mitigate against possible differential settlement.

Where the foundation is underlain by fine soils (clay or silt) which are susceptible to shrinkage and swelling, foundation depth should be determined using the NHBC standards.

Chapter 4.2 of the NHBC Standards, *Building Near Trees*, recommends a minimum foundation depth of 0.75m for a soil of low volume change potential. Greater foundation depths may be required in proximity to trees or large shrubs, whether retained or removed. The effects of possible future plantings should also be taken into account during design.

Conventional spread foundations may therefore be constructed upon the fine (clay / silt) Head Deposits or River Terrace Deposits at a minimum depth of 0.75m below ground level, subject to consideration of shrinkage and/or swelling. It is recommended that this also be adopted as the minimum foundation depth on coarse soils (sands and gravels).

As an example of the potential significance of trees, on parts of this site, foundations are to be constructed close to mature broad leafed trees that are to be retained. Assuming a potential mature height of around 20m for trees and a high water demand the NHBC Standards would require a minimum foundation depth of around 1.8m if the foundations are to be constructed within 10m of the trees, dropping to 0.75m at 25m distance from the trees, provided the underlying soils are clay or silt. However, on most of this site the underlying soils are coarse (e.g. gravel) and shallow foundations (at 0.75m depth) would be sufficient. The foundation depths in each area should be based on findings of the nearest exploratory hole(s).

The NHBC require that precautions against heave (e.g. compressible material against the side of external foundations) be used where the foundation is within the potential zone of influence of trees, and the minimum foundation depth recommended by the NHBC Standards is greater than 1.5m based on the appropriate tree height. Again, this would potentially apply at only a few locations (e.g. WS02), if any. Where groundwater is present, the maximum foundation depth required would not exceed the depth of the groundwater.

At the recommended minimum foundation depth of 0.75m, the River Terrace and Head Deposits should offer a net allowable bearing capacity of 150kN/m^2 for strip foundations up to 1.0m wide in the gravel soils and 80kN/m^2 in the clay soils (although this will be conservative in most cases).



For the gravel soils this figure is based on a minimum SPT N-value of 13, corrected to allow for overburden pressure and assuming a water table typically at 2.0m below ground level. The allowable bearing capacity in granular soils is defined as that likely to cause a maximum settlement of 25mm. This assumes a depth of granular soil beneath the foundation of at least twice the foundation width. For lower bearing pressures the settlements will be proportionally less. For the clay soils this is based on a minimum undrained shear strength of 40kN/m^2 and an estimated coefficient of volume change, m_v , of $0.2\text{m}^2/\text{MN}$. The factor of safety against bearing capacity failure for foundations exerting the above foundation pressures should be at least 3.0.

In summary, foundation depths of 0.75m will be acceptable on most of the site. At some locations it may be necessary to deepen the foundations due to the influence of trees. Groundwater was typically encountered at around 2m depth and it will be desirable to keep foundations above the water table. The groundwater table was standing at around 10.5m AOD in the post fieldwork monitoring.

6.5 Floor Slabs

Floor slabs for the proposed residential developments should be designed in accordance with the NHBC Standards.

Ground bearing floor slabs with a suitable sub-base layer slab may be adopted for much of the proposed development. All formation level soils should be proof-rolled and all topsoil and any other loose, soft, organic or otherwise unsuitable materials should be removed and replaced with a suitable, well-compacted granular fill. Hard spots such as old foundations should also be removed.

As discussed above foundation depths in excess of 1.5m may possibly be required on parts of this site by the NHBC Standards due to the presence of trees and shrinkable soils. Suspended (or piled) floor slabs with a subfloor void as a precaution against heave should therefore be used in these areas unless it can be shown that the soil is not desiccated at the time of construction.

6.6 Soakaways

The soakaway testing undertaken gave the following results:

TABLE 12: SUMMARY OF SOAKAWAY TEST RESULTS					
Geology	Typical Soil Description	Test Location	Depth of Pit at time of test, m bgl	Soil Infiltration Rate m/s	Notes



Head Deposit	Very clayey slightly sandy GRAVEL.	TP01A	2.05	2.16×10^{-5}	
Head Deposits	Clayey sandy GRAVEL.	TP02A	1.32	4.89×10^{-5}	
Head Deposit	Slightly sandy slightly gravelly CLAY.	TP03A	1.73	2.88×10^{-5}	

Note:

1. Where more than one filling of a test pit was completed the lowest infiltration rate determined has been used.

Table 4.4 of CIRIA Report 156 (1996) *Infiltration Drainage – Manual of Good Practice* indicates that infiltration drainage is unlikely to be viable where the soil infiltration rate is less than 2.78×10^{-10} m/s (0.001mm/hr). Soakaways are therefore likely to be suitable for the site. BRE Digest 365 states that soakaways should not normally be constructed closer to foundations than 5m.

Groundwater should not rise above the base of soakaways under annual variations. On this site the minimum depth that groundwater was encountered was 1.95m and soakaways will need to be constructed at lesser depths. It should, however, be noted that groundwater levels can vary due to seasonal and other changes any may at times differ from those measured in this investigation.

The Environment Agency should be contacted at the design stage in order to obtain a 'consent to discharge'. This may not be forthcoming where soakage will be into or just above the water table, particularly in the Agency's sensitive aquifer protection zones. In addition, planning approval will have to be sought for their use.

Based on the test results, an infiltration rate of 2×10^{-5} may be assumed for the gravel Head Deposits. In addition to the soil infiltration rate and groundwater level, the suitability of the site for soakaway drainage depends on a number of factors including the plan size and depth of soakaways, the 'design storm' and the site area to be drained. We recommend the advice of a specialist drainage designer be sought with regard to the design of soakaways on this site.

6.7 Pavement Design

A pavement is a foundation designed to carry vehicle, pedestrian or similar loads, and would normally comprise a foundation surface such as asphalt or concrete over a granular *subbase* constructed on the *subgrade* soil. Geosynthetics such as geomembranes or geotextiles may be used to prevent mixing of the subbase with subgrade soils, or as reinforcement between layers in the subbase. Where the subgrade is poor a *capping* layer may be used to protect the subgrade.



On this site it anticipated that pavements to carry vehicular traffic or similar will be constructed with a depth to top of subgrade of between about 300mm to 500mm below ground level.

No in-situ California Bearing Ratio (CBR) tests were carried out as part of the investigation, however equilibrium CBR values may be estimated using published guidance as discussed below.

The Department for Transport, *Design Manual for Roads and Bridges Interim Advice Note (IAN) 73/06 (Draft HD25)* details the process for designing trunk roads and motorways. Local Authorities may apply the guidance to other road pavements directly, or produce their own guidance commonly based on IAN 73/06. Where no other criteria have been specified the guidance contained in IAN 73/06 may reasonably be applied to other road pavements.

IAN 73/06 states that the lower of the short and long term CBR values should be used for design; these should be derived using laboratory CBR tests and then measured in-situ at time of construction. Laboratory testing has the advantage that realistic conditions of moisture and 'disturbance' can be simulated to reproduce, as far as possible, the conditions of moisture content and density which are likely to be experienced during construction and in the completed pavement.

Where no laboratory CBR tests have been carried out the likely long term equilibrium CBR values may be estimated using Table 5.1 of IAN 73/06 (which makes reasonable allowance for moisture ingress through the pavement and assumes drainage is correctly installed as designed).

The estimated minimum equilibrium CBR value which may develop under a completed pavement at the anticipated subgrade depth is 3.5%. Using the criteria contained in IAN 73/06 this is based on a thin pavement (a layered pavement with depth to subgrade of 300mm) and a high water table of 300mm or less, i.e. the subgrade may become wetted over time.

Based on the preceding discussion, and consideration of both long and short term CBR values, it is recommended that a CBR value of 3.5% be assumed for the subgrade for design purposes.

For pavement construction all formation level soils should be proof-rolled and all topsoil and any other loose, soft, organic or otherwise unsuitable materials should be removed and replaced with well compacted granular fill where required. During construction the in-situ CBR value should be compared with the quoted design value.