

Appendix A



Ref: J23-037-R01

Date: 20th June 2023

FAO: Chris Owen-Hughes

Paul Basham Associated

The Bothy
Cams Hall Estate
Fareham
PO16 8UT

Dear Chris,

Re: Infiltration Testing – 78 Havant Road, Emsworth, PO10 7LH

GCC was commissioned by Paul Basham Associates on behalf of Baytree Securities Limited to undertake infiltration testing to support the proposed drainage design at the site at 78 Havant Rd, Emsworth, PO10 7LH.

Available geological maps for the area shows the site/area to be overlain by River Terrace Deposits (RTD) over London Clay Formation (LC).

Encountered ground conditions, as part of a geotechnical site investigation undertaken in June 2023, confirms the presence of River Terrace Deposits beneath the site comprising various layers of clay, silt and sand overlying London Clay Formation. Groundwater was encountered during the investigation works and groundwater levels were measured in installed monitoring wells prior to commencing the infiltration testing works at 2.2mbgl and 2.29mbgl. (boreholes logs, Figure 1: SI plan and groundwater monitoring data attached).

Two trial-pits (SA101 & SA102) were excavated on the 20th June 2023 at the site (locations shown on Figure 1). The ground conditions encountered in the trial-pits are shown on the attached calculation sheets.

Infiltration testing was conducted in accordance with the procedures detailed in BRE DG365 and comprised rapidly pouring water into an open trial hole from a 2000 litre bowser, the drop in measured head over time was then recorded to give an indication of soakage potential. BRE DG365, 2016 states that for an accurate infiltration rate to be obtained, a soakage pit needs to be filled three times in quick succession. Each test is completed once 75% of the water present has drained away.

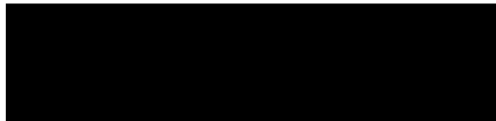
The infiltration calculation sheets are appended and the rates are summarised below.

Location	Stratum	Trial Pit Base Depth (m)	Infiltration Rate (m/sec)	Notes
SA101 Test 1	RTD	1.3	1.68×10^{-6}	One partial test completed. Final data point extrapolated. Infiltration rate is indicative only



Location	Stratum	Trial Pit Base Depth (m)	Infiltration Rate (m/sec)	Notes
SA102 Test 1	RTD	1.35	9.43×10^{-7}	One partial test completed. Final data point extrapolated. Infiltration rate is indicative only

Yours sincerely

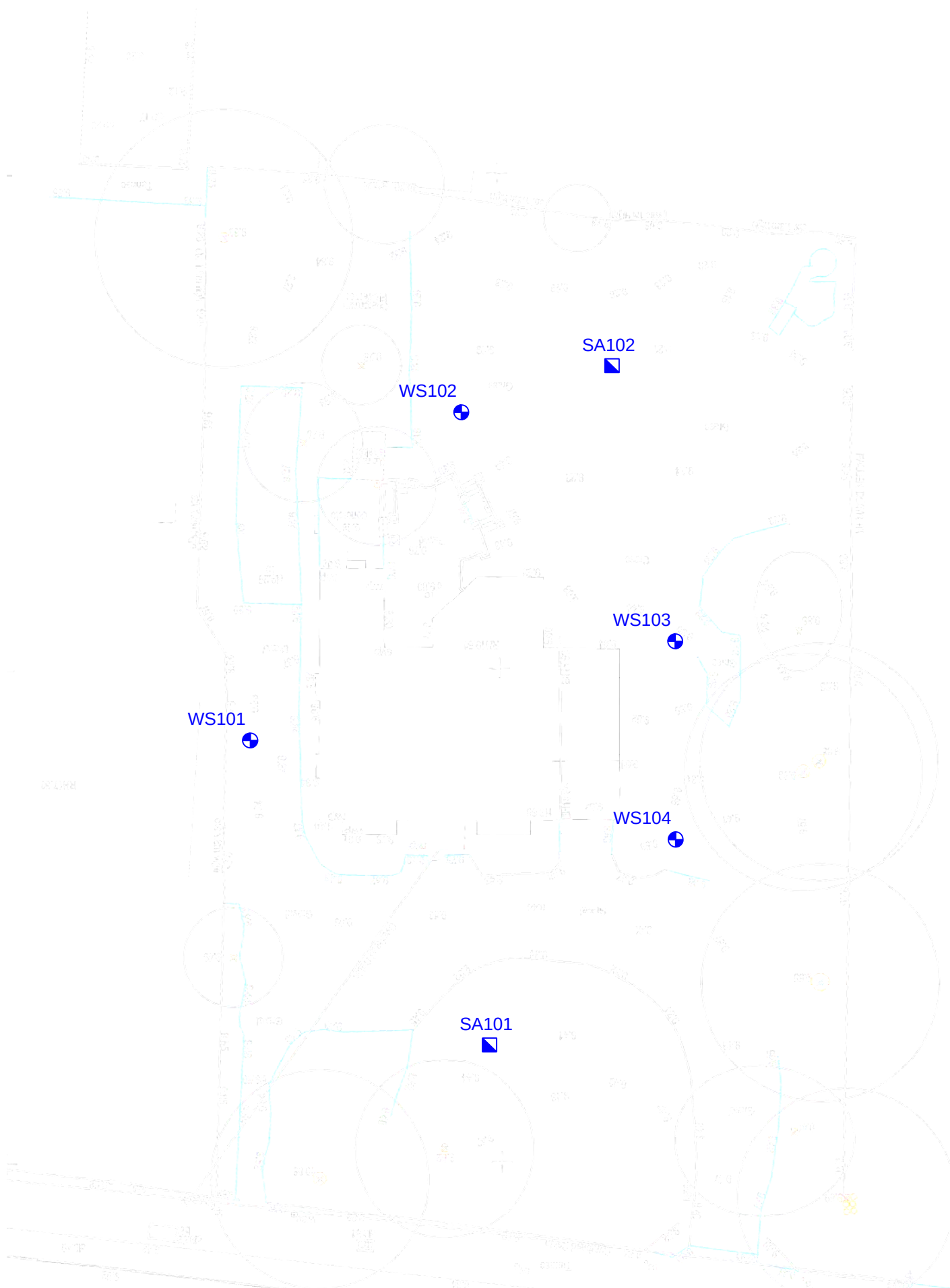


ANTONY PLATT
Director
For Ground Condition Consultants Ltd



RASMUS PALMGREN
Director
For Ground Condition Consultants Ltd

Encl: Figure 1: Location plan
Borehole logs
Groundwater monitoring data
Infiltration Test Results



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client PAUL BASHAM ASSOCIATES

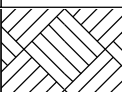
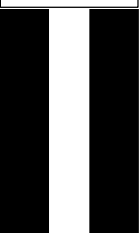
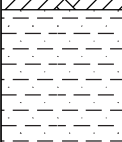
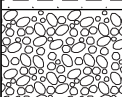
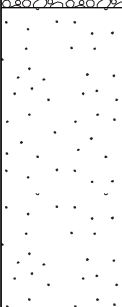
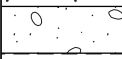
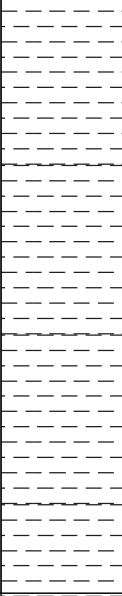
site 78 HAVANT ROAD, EMSWORTH

title SITE INVESTIGATION PLAN

scale NTS | drawn by DP | checked by RP

date 20 JUNE 2023 | job no J23-037

drawing number FIGURE 1 | rev. 1

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GROUND GAS AND GROUNDWATER MONITORING RESULTS



SITE/REF: 78 Havant Road, Emsworth						General Site comments				
DATE: 13/06/2023		1... of 1...								
OPERATOR(S): DAP										
Ambient Conditions:		Barometric Pressure (mb)	CH ₄ (%v/v)	CO ₂ (%v/v)	O ₂ (%v/v)				PID (ppm)	
Before Monitoring							EQUIPMENT			
After Monitoring										
Atmospheric Pressure Trend:							Instrument / Model Type		Serial Number	Comments
Ground Conditions:							GA2000 Gas Analyser		GA12820	
Weather Conditions:							Standard Dipmeter		79301	

Ground Gas Note: "0" readings to be recorded as instrument detection limit

Borehole ID	Time	Flow (l/hr)	Relative Pressure (mb)	Steady CH ₄ (%v/v)	CO ₂ (%v/v)	O ₂ (%v/v)	Peak CH ₄ (%v/v)	% of Lower Explosive Limit	CO (ppm)	H ₂ S (ppm)	Balance (%)	PID (ppm)	Response Zone		Response Strata	Gas screening value CH ₄	Gas screening value CO ₂
													From (mbgl)	To (mbgl)			

Groundwater

Borehole ID	Standing Water Level (mbgl)	Installation Base Dip (mbgl)	Free Phase LNAPL Thickness (m)	Free Phase DNAPL Thickness (m)	Water Quality Indicators					Well Volume (l)	Purge Volume (l)	Comments (samples)
					Eh	EC	pH	DO	Temp			
WS102	2.16	5.05										
WS104	4.34	5.01										

Well volume (l) = $(3.14 \times (\text{hole diameter (m)} / 2)^2 \times (\text{base dip (m)} - \text{standing water level (m)})) \times 1000$ (2l/m in 50mm well)

Gas screening value = gas concentration (%) x gas flow rate (l/hr)

%v/v - Percentage volume by total volume; mbgl - metres below ground level; ppm - parts per million; mb - milibars; l/hr - litres per hour; vv - water vial; gb - 1l glass bottle; pb - 1l plastic bottle

GROUND GAS AND GROUNDWATER MONITORING RESULTS



SITE/REF: 78 Havant Road, Emsworth						General Site comments				
DATE: 20/06/2023		1... Of 1...								
OPERATOR(S): RP										
Ambient Conditions:		Barometric Pressure (mb)	CH ₄ (%v/v)	CO ₂ (%v/v)	O ₂ (%v/v)				PID (ppm)	
Before Monitoring							EQUIPMENT			
After Monitoring										
Atmospheric Pressure Trend:							Instrument / Model Type		Serial Number	Comments
Ground Conditions:		Dry					GA2000 Gas Analyser		GA12820	
Weather Conditions:		Overcast					Standard Dipmeter		79301	

Ground Gas Note: "0" readings to be recorded as instrument detection limit

Borehole ID	Time	Flow (l/hr)	Relative Pressure (mb)	Steady CH ₄ (%v/v)	CO ₂ (%v/v)	O ₂ (%v/v)	Peak CH ₄ (%v/v)	% of Lower Explosive Limit	CO (ppm)	H ₂ S (ppm)	Balance (%)	PID (ppm)	Response Zone		Response Strata	Gas screening value CH ₄	Gas screening value CO ₂
													From (mbgl)	To (mbgl)			

Groundwater

Borehole ID	Standing Water Level (mbgl)	Installation Base Dip (mbgl)	Free Phase LNAPL Thickness (m)	Free Phase DNAPL Thickness (m)	Water Quality Indicators					Well Volume (l)	Purge Volume (l)	Comments (samples)
					Eh	EC	pH	DO	Temp			
WS102	2.2	4.78										
WS104	2.29	4.72										

Well volume (l) = (3.14 x (hole diameter (m)/2)² x (base dip (m) - standing water level (m)))x1000 (2l/m in 50mm well)

Gas screening value = gas concentration (%) x gas flow rate (l/hr)

%v/v - Percentage volume by total volume; mbgl - metres below ground level; ppm - parts per million; mb - milibars; l/hr - litres per hour; ww - water vial; gb - 1l glass bottle; pb - 1l plastic bottle

Soakaway Infiltration Calculation - Derived in accordance with BRE Digest 365

Soakaway Test ID:	SA101	GCC
Contract:	Emsworth	10 Waldegrave Close
Contract No:	J23-037	Southampton
Date:	20 June 2023	SO19 9RY

Test location details:

Depth: 1.30 m
 Width: 0.40 m
 Length: 1.50 m
 Gravel backfill: NO

Trial Pit Log:

0.0-0.2 Turf over light brown grey clayey SILT (TS)
 0.2-0.4 Soft light brown grey silty CLAY with occasional flint gravel and red brick fragments (MG)
 0.4-0.9 Soft brown slightly silty CLAY
 0.9-1.3 Brown silty gravelly CLAY

Test Data:

Test 1				Test 2				Test 3			
Depth to water (m)	Elapsed Time (min)	Head of Water (% of Ho)	Head of Water (m)	Depth to water (m)	Elapsed Time (min)	Head of Water (% of Ho)	Head of Water (m)	Depth to water (m)	Elapsed Time (min)	Head of Water (% of Ho)	Head of Water (m)
0.83	0	100	0.47								
0.84	10	98	0.46								
0.85	20	96	0.45								
0.86	30	94	0.44								
0.88	60	89	0.42								
0.89	90	87	0.41								
0.90	120	85	0.4								
0.93	180	79	0.37								
0.95	240	74	0.35								
0.97	300	70	0.33								
1.19	1200	23	0.11								
Test 1 Depth:		1.30 m		Test 2 Depth:		m		Test 3 Depth:		m	
Max Water Depth:		0.47 m		Max Water Depth:		m		Max Water Depth:		m	
Max Water Volume:		0.28 m³		Max Water Volume:		m³		Max Water Volume:		m³	
ap 50 (test area):		1.49 m²		ap 50 (test area):		m²		ap 50 (test area):		m²	
Depth at 75% Ho:		0.9475 m		Depth at 75% Ho:		m		Depth at 75% Ho:		m	
Depth at 25% Ho:		1.1825 m		Depth at 25% Ho:		m		Depth at 25% Ho:		m	
Duration at 75 Ho%:		233 min		Duration at 75 Ho%:		min		Duration at 75 Ho%:		min	
Duration at 25 Ho%:		1169 min		Duration at 25 Ho%:		min		Duration at 25 Ho%:		min	
Vp 75-25:		0.141 m³		Vp 75-25:		m³		Vp 75-25:		m³	
tp 75-25:		937 min		tp 75-25:		min		tp 75-25:		min	
Infiltration Rate (f):		1.68E-06 m/s		Infiltration Rate (f):		m/s		Infiltration Rate (f):		m/s	

Soakaway Infiltration Calculation - Derived in accordance with BRE Digest 365

Soakaway Test ID:	SA102	GCC
Contract:	Emsworth	10 Waldegrave Close
Contract No:	J23-037	Southampton
Date:	20 June 2023	SO19 9RY

Test location details:

Depth: 1.35 m
 Width: 0.40 m
 Length: 1.60 m
 Gravel backfill: NO

Trial Pit Log:

0.0-0.2 Turf over light brown grey clayey SILT (TS)
 0.2-0.8 Soft light brown grey silty CLAY with occasional flint gravel
 0.8-1.1 Brown silty gravelly CLAY
 1.1-1.35 Brown grey silty fine SAND

Test Data:

Test 1				Test 2				Test 3			
Depth to water (m)	Elapsed Time (min)	Head of Water (% of Ho)	Head of Water (m)	Depth to water (m)	Elapsed Time (min)	Head of Water (% of Ho)	Head of Water (m)	Depth to water (m)	Elapsed Time (min)	Head of Water (% of Ho)	Head of Water (m)
0.79	0	100	0.56								
0.80	5	98	0.55								
0.81	10	96	0.54								
0.82	20	95	0.53								
0.83	30	93	0.52								
0.84	60	91	0.51								
0.88	120	84	0.47								
0.91	180	79	0.44								
0.92	240	77	0.43								
0.93	300	75	0.419								
1.21	2100	25	0.139								
Test 1 Depth:		1.35 m		Test 2 Depth:		m		Test 3 Depth:		m	
Max Water Depth:		0.56 m		Max Water Depth:		m		Max Water Depth:		m	
Max Water Volume:		0.36 m³		Max Water Volume:		m³		Max Water Volume:		m³	
ap 50 (test area):		1.76 m²		ap 50 (test area):		m²		ap 50 (test area):		m²	
Depth at 75% Ho:		0.93 m		Depth at 75% Ho:		m		Depth at 75% Ho:		m	
Depth at 25% Ho:		1.21 m		Depth at 25% Ho:		m		Depth at 25% Ho:		m	
Duration at 75 Ho%:		295 min		Duration at 75 Ho%:		min		Duration at 75 Ho%:		min	
Duration at 25 Ho%:		2094 min		Duration at 25 Ho%:		min		Duration at 25 Ho%:		min	
Vp 75-25:		0.1792 m³		Vp 75-25:		m³		Vp 75-25:		m³	
tp 75-25:		1799 min		tp 75-25:		min		tp 75-25:		min	
Infiltration Rate (f):		9.43E-07 m/s		Infiltration Rate (f):		m/s		Infiltration Rate (f):		m/s	



Ground Condition Consultants

78 HAVANT ROAD, EMSWORTH, PO10 7LH

GEOTECHINICAL DESIGN REPORT

JULY 2023

Prepared for
Baytree Securities Ltd
72 Havant Road
Emsworth
PO10 7LH

Prepared by
Ground Condition Consultants Ltd
10 Waldegrave Close
Southampton
Hampshire
SO19 9RY

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www.groundcc.com

VERSION CONTROL

Project Name 78 Havant Rd, Emsworth – GDR

Project No. J22-037

Report No. J22-037-R01

Site Investigation Factual Report	
Prepared by: David Pethica - Director	Reviewed by: Rasmus Palmgren - Director
Geotechnical Design Report	
Prepared by: David Fall – Geotechnical and Environmental Consulting Ltd	Reviewed by: Rasmus Palmgren - Director

Version	Date	Description
1.0	07.07.2023	Final

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APPENDIX 5: GEOTECHNICAL CALCULATION WORKSHEETS

0. EXECUTIVE SUMMARY

Brief	The brief was to undertake a site investigation and assessment to identify geotechnical conditions that may affect the proposed development. The geotechnical sections of this report provide a detailed assessment of the materials present beneath the site and present characteristic parameters that should be used in design of all geotechnical facets of the proposed development.
Current Site Status	The is located at 78 Havant Road, Emsworth, PO10 7LH and comprises a residential dwelling with associated parking and garden areas. The site is situated in a residential area with residential properties surrounding the site on all sides.
Geology	Available geological maps for the area shows the site/area to be overlain by River Terrace Deposits over London Clay Formation.
Investigation Scope	The site investigation works were undertaken by GCC on the 13 th January 2023 and comprised: 4no. windowless sample boreholes (WS101-WS104) to a maximum depth of 5.45mbgl to obtain samples for geotechnical laboratory testing.
Conclusions and Recommendations	The ground investigation has shown the site to be underlain by Topsoil / Made Ground over a mixture of fine- and coarse-grained River Terrace Deposits (RTD) in turn overlying the silty CLAY of the London Clay Formation. Geotechnical testing has shown that the coarse-grained RTD has greater than 25% coarse-grained content leading to a similar effective shear strength parameters being appropriate for both the coarse- and fine-grained RTD. The fine-grained RTD has been identified as being of medium volume change potential and therefore a minimum foundation depth of 0.90mbgl should be adopted for all foundations outside the zone of influence of any existing trees or proposed future planting. ULS bearing resistance calculations have been carried out for both the fine-coarse-grained RTD in accordance with Eurocode 7 and give a single ULS bearing resistance of 120kN/m², which is considered appropriate for all foundations up to 1.00m wide regardless of the composition of the RTD. It should be noted that groundwater correction will need to be considered if foundation deepening places the foundation within B of the standing groundwater, where B is the foundation breadth. It is recommended that a suspended floor slab design is adopted due to medium volume change potential of the fine-grained RTD. Chemical testing of ground aggressivity has shown the ground to be consistent with Design Sulphate Classification DS-1 and an ACEC of AC-1 assuming mobile groundwater conditions within the coarse-grained RTD.
<i>This sheet is intended to provide a summary only of the initial indicative assessment study of the site in relation to ground contamination. It does not provide a definitive engineering analysis for the purposes of costing or construction and is subject to the limitation of the agreed brief.</i>	

1. INTRODUCTION

1.1 INSTRUCTION

Ground Condition Consultants Ltd. (GCC) was commissioned by Baytree Securities Limited in June 2023 to undertake a Ground Investigation and Geotechnical Design Report for a site at 78 Havant Road, Emsworth, PO10 7LH.

The brief was to undertake a ground investigation and assessment to identify geotechnical conditions to inform the design of the foundations for the proposed residential development. The report provides a detailed assessment of the materials present beneath the site and present characteristic parameters that should be used in design of all geotechnical facets of the proposed development.

This report is prepared in line with the agreed brief and is subject to the report conditions shown in Appendix 1.

1.2 PROPOSED USE

It is currently understood that a block of apartments with associated parking areas and landscaped area is proposed for the site.

1.3 REPORT SCOPE AND LIMITATIONS

This report is based upon the recent site investigation data detailed herein. The report presents an interpretation of the borehole and laboratory data provided by the GCC ground investigation undertaken on the 13th June 2023. In addition, this report outlines the basic ground conditions encountered in the exploratory holes.

This information has been collated, processed and used to provide an interpretation of the ground conditions, with recommendations for foundation design for the proposed development.

The recommendations and opinions expressed in this report are based on the strata observed in the exploratory holes and the results of the site and laboratory tests. GCC takes no responsibility for conditions that have not been revealed by the exploratory holes, or which occur between them.

Whilst every effort has been made to interpret the conditions between investigation locations, such information is only indicative and liability cannot be accepted for its accuracy. Information provided from other sources is taken in good faith and GCC cannot guarantee its accuracy.

The information contained in this report is intended for the use of Baytree Securities Limited and GCC can take no responsibility for the use of this information by any other party or for uses other than that described in this report.

1.4 SITE LOCATION AND DESCRIPTION

The site is located at 78 Havant Road, Emsworth, PO10 7LH and comprises a residential dwelling with associated parking and garden areas. The site is situated in a residential area with residential properties surrounding the site on all sides with Havant Road located directly to the north.

At the time of investigation, the site was noted to be largely overgrown with tall grasses and dense vegetation in areas. A small above ground air raid shelter was noted in the rear garden to the south of the property. Mature trees are noted to fringe the site.

1.5 PUBLISHED GEOLOGY

Available geological maps for the area shows the site/area to be overlain by River Terrace Deposits (silt, sand and gravel) over London Clay Formation.

2. FIELDWORK SUMMARY OF SCOPE AND RATIONALE

2.1 RATIONALE AND SUMMARY OF SCOPE

The site investigation works were undertaken by GCC on the 13th June 2023. The scope and rationale of the site investigation are presented below.

- 4no. windowless sample boreholes (WS101-WS104) to a maximum depth of 5.45mbgl to obtain samples for geotechnical laboratory testing.

The layout of the exploratory hole positions is presented in Figure 1.

2.2 SAMPLING

Soil sampling was undertaken at various depth for geotechnical testing.

2.3 SITE INVESTIGATION STANDARDS

Methods employed during the site investigation were generally undertaken in accordance with BS10175, BS EN 1997-2 and BS5930.

3. GROUND CONDITIONS ENCOUNTERED

3.1 SOIL CONDITIONS

Ground conditions encountered during the site investigation were in agreement with those presented on the BGS maps and comprised Made Ground/Topsoil over River Terrace Deposits over London Clay Formation. Variations in strata thickness are summarised in Table 3.1 and engineering logs, are presented in Appendix 2.

Table 3.1. Summary of encountered ground conditions

Location	Made Ground/Topsoil		River Terrace Deposits		London Clay Formation	
	From (mbgl)	Thickness (m)	From (mbgl)	Thickness (m)	From (mbgl)	Thickness (m)
WS101	GL	0.25	0.25	2.65	2.90	>2.55
WS102	GL	0.40	0.40	2.50	2.90	>2.55
WS103	GL	0.35	0.35	2.85	3.20	>2.25
WS104	GL	0.30	0.30	3.10	3.40	>2.05

3.1.1. Made Ground/Topsoil

Ground Investigations encountered either a flint gravel surface or topsoil comprising a pale brown slightly gravelly fine to medium sand, to a maximum depth of 0.40mbgl.

3.1.2. River Terrace Deposits

River Terrace Deposits were encountered directly beneath the Made Ground or Topsoil and generally presented as a sequence of sandy gravelly silt or clay (0.60-1.05m thick), over a band of gravel (0.40-0.50m thick, absent in WS101), in turn overlying a fine sand becoming gravelly toward its base (1.50-1.80m thick). At WS101 and WS104, a thin band of gravelly clay was also encountered at the base of the River Terrace Deposits, overlying the London Clay Formation. River Terrace Deposits were encountered to a maximum depth of 3.40mbgl.

3.1.3. London Clay Formation

The London Clay Formation was encountered between 2.90m and 3.40mbgl and generally comprised a firm to stiff grey silty clay. The London Clay Formation was the basal stratum in all of the windowless sampler borehole locations.

3.2 OBSTRUCTIONS

No obstructions were encountered during the investigation.

3.3 GROUNDWATER CONDITIONS

Groundwater was encountered at 2.00-2.50mbgl in all locations except WS104. A secondary water strike was encountered at 4.00mbgl in WS102. It is assumed that the deeper water strike, noted within the London Clay Formation is confined to a fracture or sandy horizon, although neither was noted in the arisings.

Groundwater levels were monitored on completion of siteworks and on 20th June 2023 in the two installed wells in (WS102 and WS104) and the results are presented in Appendix 3. Data across the monitoring and ground investigation indicates natural equilibrium for groundwater was between 2.00m and 2.50mbgl during this time.

3.4 VISUAL/OLFACTORY EVIDENCE OF CONTAMINATION

No visual or olfactory evidence of contamination was observed in the soil during the investigation.

3.5 CHEMICAL TESTING

Chemical testing or contamination assessment purposes did not form part of the scope of this investigation.

4. MATERIAL PROPERTIES

4.1 INTRODUCTION

The geotechnical assessment and the subsequent foundation design recommendations will use the findings of the ground investigation and the results of the in-situ and geotechnical laboratory testing carried out in the boreholes and on representative samples of the material encountered.

4.2 GEOTECHNICAL TESTS

A programme of geotechnical laboratory testing was carried out on representative samples collected from the strata encountered during the ground investigation.

Details of the specific procedure used in each case are shown below in Table 4.1 and the geotechnical test certificates are presented in Appendix 4.

Table 4.1. Summary of geotechnical and chemical testing

Test	Standard (BS1377:1990) unless otherwise indicated	No.
Classification Tests		
Atterberg Limits	Part 2, Clauses 4.4, 4.5 & 5.4	6
Particle Size Distribution – Wet Sieve	Part 2, Clause 9.2	6
Shear Strength (Total Stress) Tests		
Unconsolidated Undrained Triaxial Compression	Part 7, Clause 8.4	3
Chemical Tests		
Sulphate Content 2:1 soil water extract	Part 3, Clause 5.3	3
pH value	Part 3, Clause 9.0	4
BRE SD1	Various	1

4.3 MATERIAL PARAMETERS

This section discusses the key geotechnical characteristics of each encountered stratum as determined from field observations, in-situ and laboratory geotechnical testing.

4.3.1. Topsoil / Made Ground

Made Ground / Topsoil was encountered at all exploratory hole locations to a maximum depth of 0.40mbgl.

No geotechnical testing was undertaken on the Topsoil / Made Ground as all foundations will be constructed beneath this highly variable material.

4.3.2. River Terrace Deposits

River Terrace Deposits (RTD) were encountered immediately beneath the Topsoil / Made Ground and attained thicknesses of between 2.50 and 3.10m. The RTD generally comprised a superficial layer of sandy gravelly CLAY / SILT overlying a coarse-grained sandy fine to coarse angular to subangular GRAVEL in turn overlying silty fine SAND becoming gravelly towards its base.

The distribution of the fine- and coarse-grained RTD is presented in Table 4.2.

Table 4.2. Distribution of fine- and coarse-grained River Terrace Deposits

Exploratory Hole	Fine-Grained RTD Sandy gravelly CLAY/SILT		Coarse-Grained			
			Sandy fine- to coarse- GRAVEL		Silty fine SAND gravelly at base	
	From (mbgl)	To (mbgl)	From (mbgl)	To (mbgl)	From (mbgl)	To (mbgl)
WS101	0.25	1.30	Not present		1.30	2.70
	2.70	2.90				
WS102	0.40	1.00	1.00	1.40	1.40	2.90
WS103	0.35	1.20	1.20	1.60	1.60	3.25
WS104	0.30	0.90	0.90	1.40		
	1.40	1.65			1.65	2.90
	2.90	3.40				

In-Situ Testing

Standard penetration tests (SPT) were undertaken within the RTD in the 4no. WS boreholes. The resultant SPT N-values are presented on the engineering logs and summarised in Table 4.3.

Table 4.3. Results of the standard penetration testing – River Terrace Deposits

BH No.	Depth (mbgl)	SPT N Value	Main Constituents	BH No.	Depth (mbgl)	SPT N Value	Main Constituents
WS101	1.00	11	gravelly SILT	WS103	1.00	16	Crossing between SILT to GRAVEL
	2.00	16	silty SAND		2.00	6	silty SAND
WS102	1.00	16	sandy GRAVEL	WS104	1.00	16	Sandy GRAVEL
	2.00	14	silty SAND		2.00	4	Silty SAND

The results of the SPTs returned N-values of between 4 and 16 for the RTD. Only one N-value was recorded entirely within the fine-grained material and returned a value of 11. Of the remaining results, the lowest values N = 4 and 6 were recorded with the fine silty SAND. All the SPT N results have been included on a plot against depth and form Figure 3 of this Report.

Classification Testing

Consistency limits

3no. Atterberg limits tests were undertaken on samples of the fine-grained RTD. The results are presented in Table 4.4 and the laboratory test certificates are located in Appendix 4.

Table 4.4. Results of the Atterberg limits testing – River Terrace Deposits

Location	Depth (mbgl)	Liquid Limit w_L (%86)	Plastic Limit w_P (%)	Plasticity Index I_P (%)	% Passing the 425 μ m sieve	Corrected Plasticity Index (%)
WS101	0.80-1.00	58	22	36	97	35
WS102	0.50-0.70	51	24	27	80	22
WS103	0.60-1.00	50	21	29	71	21

The Atterberg limits tests returned plasticity indices of 27 and 36% with liquid limits of 50 to 58%, which are consistent with a fine-grained material of high plasticity. After correction, to account for the >425 μ m content, the corrected range of plasticity indices is 21 to 35%, which is consistent with a fine-grained soil of medium volume change potential.

For design purposes the fine-grained RTD should be treated as being of medium volume change potential with a characteristic plasticity Index (I_{Pk}) of 30%.

Particle Size Distribution

3no. representative samples of the coarse-grained RTD underwent particle size distribution analysis using the wet-sieving method. The results of these analyses are presented in Table 4.5 and the laboratory test certificates and grading envelopes are located in Appendix 4.

Table 4.5. Results of the Particle Size Distribution Analyses – Coarse-Grained River Terrace Deposits

Location	Depth (mbgl)	Granulometric Composition (%)				Uniformity Coefficient	Curvature Coefficient
		Fines	Sand	Gravel	V Coarse		
WS101	1.20-2.00	32	68	0	0	>1.4	Not Calculated
WS102	1.00-1.30	23	15	62	0	>130	Not Calculated
	1.40-2.40	46	54	0	0	>1.3	Not Calculated
WS103	1.60-2.60	32	68	0	0	>1.4	Not Calculated
WS104	1.60-2.60	45	10	45	0	>76	Not Calculated
	1.70-2.70	42	57	0	0	>1.3	Not Calculated

The results of the particle size distribution analyses show 5no. of the 6no. samples tested had fine-grained contents >25%. confirm the coarse-grained composition of the samples tested with fine-grained contents all less than 24%. BS8004 2015 4.3.1.3.11 states; the characteristic angle of shearing resistance (ϕ_k) for coarse soils with fines contents exceeding 25% should be determined as for fine soils.

Therefore, in the following section a single characteristic effective angle of friction will be applied to both the fine and coarse grained RTD.

Shear Strength – Fine-Grained RTD

Undrained Shear Strength

The undrained shear strength of the fine-grained RTD has been determined from the result of the SPT and the engineering description of the fine grained RTD.

Using the widely accepted empirical relationship with SPT N; where:

$$c_u = f_1 \times \text{SPN}$$

Where f_1 is a correlation factor based on the characteristic plasticity index.

Using a correlation factor of 4.5 and the SPT N of 11 gives a characteristic undrained shear strength (c_{uk}) of 50kN/m².

Drained (Effective) Shear Strength – Fine and Coarse-Grained RTD

The widely accepted relationship between drained shear strength and plasticity index for remoulded clays has been used to determine the effective shear strength parameters, where:

$$\phi_k = 42 - (12.5 \text{ Log } I_{pk})$$

Taking the characteristic plasticity index of 40% will give a characteristic effective angle of shearing resistance (ϕ'_k) of 24°.

In terms of effective stress, the shear strength of a fine-grained soil can be considered as frictional, such that $c'_k = 0$ kN/m².

Other Geotechnical Parameters – Fine-Grained River Terrace Deposits

Modulus of Deformation (E_k)

Using an E_k/c_{uk} ratio of 500 for the fine-grained RTD gives an undrained Modulus of Deformation (E_{uk}) of 25MN/m².

CIRIA R 143 suggests that the drained modulus (E'_k) should be taken as $0.75E_{uk}$. Therefore, an E'_k of 19MN/m² has been attributed to the fine-grained RTD.

Modulus of Volume Compressibility

The coefficient of volume compressibility (m_v) has been determined from the empirical relationship with the plasticity index (I_{pk}) and published data for similar materials giving an m_{vk} of 0.18m²/MN, which is indicative of a material of medium compressibility.

Other Geotechnical Parameters – Coarse-Grained River Terrace Deposits

Modulus of deformation (E'_k)

The determination of the modulus of deformation for square and long strip foundations can be obtained by multiplying the static cone resistance by a factor of 2.5 or 3.5 respectively. Where only SPT data is available the static cone resistance can be determined by multiplying the N-value by an empirical factor. For sands / Gravels a value of 5 is recommended. The determination of the deformation modulus is presented in Table 4.6.

Table 4.6. Deformation Modulus for the Coarse-Grained River Terrace Deposits

Foundation Configuration	Characteristic SPT N Value	Equivalent Static Cone Resistance (kg/cm ²)	Static Cone Resistance (MN/m ²)	Deformation Modulus E _d (MN/m ²)
Strip	11	55	5.5	19
Pad				14

4.3.3. London Clay Formation

The London Clay Formation (LCF) was encountered beneath the RTD at 2.90 to 3.40mbgl. The LCF comprised firm to stiff grey silty CLAY and attained a maximum proven thickness of > 2.55m.

In-Situ Testing

Standard penetration tests (SPT) were undertaken within the LCF in the 4no. WS boreholes. The resultant SPT N-values are presented on the engineering logs and summarised in Table 4.7.

Table 4.7. Results of the standard penetration testing – London Clay Formation

BH No.	Depth (mbgl)	SPT N Value	Main Constituents	BH No.	Depth (mbgl)	SPT N Value	Main Constituents
WS101	3.00	12	silty CLAY	WS103	3.00	6	silty CLAY
	4.00	15			4.00	11	
	5.00	19			5.00	16	
WS102	3.00	5	silty CLAY	WS104	3.00	10	silty CLAY
	4.00	13			4.00	12	
	5.00	19			5.00	18	

The results of the SPTs returned N-values of between 5 and 19 for the LCF and have been included in Figure 3. Careful inspection of Figure 3 reveals an increase in N-Value with depth from N=8 at 3.00mbgl to N=18 at 5.00mbgl. This range of N-Values will be adopted to represent the characteristic SPT N value for the LCF.

Classification Testing

Consistency limits

3no. Atterberg limits tests were undertaken on samples of the LCF. The results are presented in Table 4.8 and the laboratory test certificates are located in Appendix 4.

Table 4.8. Results of the Atterberg limits testing – London Clay Formation

Location	Depth (mbgl)	Liquid Limit w_L (%86)	Plastic Limit w_P (%)	Plasticity Index I_P (%)	% Passing the 425 μ m sieve	Corrected Plasticity Index (%)
WS101	3.50-4.00	73	28	45	100	45
WS102	3.50-4.00	73	30	43	100	43
WS103	3.50-4.00	70	27	43	100	43

The Atterberg limits tests returned plasticity indices of 43 and 45% with liquid limits of 70 to 73%, which are consistent with a fine-grained material of very high plasticity. No correction, to account for the >425 μ m content, was required therefore the range of plasticity indices of 43 to 45% is consistent with a fine-grained soil of high volume change potential.

For design purposes a the LCF should be treated as being of high volume change potential with a characteristic plasticity index (I_{pk}) of 44%.

Shear Strength

Undrained Shear Strength

Three (3no.) representative samples of the LCF underwent laboratory unconsolidated undrained triaxial compression tests to determine their undrained shear strengths. The results are presented in Table 4.9 and the laboratory test reports are located in Appendix 4.

Table 4.9. Results of the UU Triaxial Compression Testing – London Clay Formation

Location	Depth (mbgl)	Bulk Density (Mg/m ³)	Cell Pressure (kN/m ²)	Undrained Shear Strength c_u (kN/m ²)
WS101	3.50-4.00	1.94	75	72
WS102	3.50-4.00	1.95	75	88
WS103	4.50-5.00	1.91	95	124

The result of the UU triaxial compression tests give undrained shear strengths (c_u) of 72 to 124kN/m² which are consistent with high undrained shear strength.

The laboratory test result has been augmented with undrained shear strengths determined using the widely accepted empirical relationship, where:

$$c_{uk} = \text{SPT } N \times f_1$$

Where f_1 is a correlation factor based on the plasticity index

All the undrained shear strength data has been plotted against depth (Figure 4). Close inspection of Figure 3 shows an increase in undrained shear strength with depth from 40kN/m² at 2.50mbgl to 100kN/m² at 5.00mbgl. This range of values has been adopted to represent the characteristic undrained shear strength of the LCF.

Drained (Effective) Shear Strength

The widely accepted relationship between drained shear strength and plasticity index for remoulded clays has been used to determine the effective shear strength parameters. In terms of effective stress, the shear strength of a fine-grained soil can be considered as frictional, such that $c'_k = 0\text{kN/m}^2$.

Taking the characteristic plasticity index of 44% will give a characteristic effective angle of shearing resistance (ϕ'_k) of 22°.

Other Geotechnical Parameters

Modulus of Deformation (E_k)

Butler (1974) suggests a E_{uk} / c_{uk} ratio of 400 for the LCF giving an undrained modulus of deformation of 16MN/m² at 2.50mbgl rising to 40MN/m² at 5.00mbgl. CIRIA R 143 states the drained modulus should be taken as 0.75 x E_{uk} . Therefore, a drained modulus of 12MN/m² at 2.5mbgl rising to 30MN/m² at 5.00mbgl is considered appropriate for the LCF.

Coefficient of Volume Compressibility

The coefficient of volume compressibility (m_v) has been determined from the empirical relationship with the plasticity index (I_p) and published data for similar materials giving an m_{vk} of 0.15m²/MN which is indicative of a material of medium compressibility.

4.4 AGGRESSIVE GROUND SOIL CHEMISTRY

Chemical testing was carried out on three (3 no.) samples of the encountered materials in accordance with Box C10, BRE Special Digest 1. The results are summarised in Tables 4.10 and 4.11.

Table 4.10. Summary of the results of chemical testing for concrete classification

Test	WS101	WS102	WS104
	0.80-1.00	1.00-1.30	0.50-1.40
	RTD	RTD	RTD
pH	8.2	8.4	8.1
Soluble Sulphate (g/l)	0.0028	0.015	0.011

Table 4.11. Results of the BRE SD1 testing

BH	Depth (mbgl)	Stratum	pH	Total Sulphate AS (% SO ₄)	Total Sulphur TS (%)	Total Potential Sulphate TPS (%)	Oxidisable (Sulphate) (%)
WS103	0.60-1.00	RTD	7.7	0.018	0.008	0.024	0.006

Based on the maximum determined water-soluble sulphate and total potential sulphate content, Table C1 of BRE SD1 gives a Design Sulphate Classification of DS-1 and an Aggressive Chemical Environment for Concrete (ACEC) Class of AC-1 assuming mobile groundwater in the coarse-grained horizons of the RTD.

4.5 CHARACTERISTIC GEOTECHNICAL PARAMETERS

Based on visual observations and published guidance characteristic geotechnical parameters, which will be used in the subsequent geotechnical and foundation design calculations, are presented in Table 4.12.

Table 4.12. Characteristic geotechnical parameters

Stratum	Geotechnical Parameter				Notes	Characteristic Value
Topsoil / Made Ground		Not provided as material will be removed prior to construction				
River Terrace Deposits	Coarse-Grained	Unit Weight γ			BS8004 2015	18kN/m ³
		Shear strength	c'_k	BS8004 2015	0kN/m ²	
			ϕ'_k		24°	
		Deformation Modulus	Strip	E_k	SPT Correlation with Q	19MN/m ²
			Pad			14MN/m ²
	Fine-Grained	Unit Weight γ			BS8004 2015	19kN/m ³
		Undrained Shear Strength	c_{uk}		Correlation with SPT	50kN/m ²
			ϕ_{uk}			0°
		Drained (effective) Shear Strength	c'_k		Relationship with I_{pk}	0kN/m ²
			ϕ'_k			24°
		Undrained Deformation Modulus	E_{uk}		500 x c_{uk}	25MN/m ²
		Drained Deformation Modulus	E'_k		0.75 x E_{uk}	19MN/m ²
	Modulus of Volume Compressibility	m_{vk}		Relationship with I_{pk} and SPT N	0.18m ² /MN	
London Clay Formation		Unit Weight γ			Lab UUT	19kN/m ³
		Undrained Shear Strength	c_{uk}		Lab UUT & SPT Cor.	40kN/m ² at 2.50mbgl rising to 100kN/m ² at 5.00mbgl
			ϕ_{uk}			0°
		Drained (effective) Shear Strength	c'_k		Relationship with I_{pk}	0kN/m ²
			ϕ'_k			22°
		Undrained Deformation Modulus	E_{uk}		400 x c_{uk}	16MN/m ² at 2.50mbgl rising to 40MN/m ² at 5.00mbgl

Stratum	Geotechnical Parameter		Notes	Characteristic Value
	Drained Deformation Modulus	E'_k	$0.75 \times E_{uk}$	12MN/m ² at 2.50mbgl rising to 30MN/m ² at 5.00mbgl
	Modulus of Volume Compressibility	m_{vk}	Relationship with I_{pk} and SPT N	0.15m ² /MN

5. FOUNDATION AND GROUND ENGINEERING

5.1 INTRODUCTION

The geotechnical assessment below relates to the details of the proposed development. It is currently understood that a replacement block of apartments is to be constructed on the site.

4no. windowless sample boreholes were undertaken across the site and indicated ground conditions to comprise Topsoil / Made Ground over River Terrace Deposits (RTD) in turn overlying the London Clay Formation (LCF) to the full depth of the ground investigation. The RTD comprised a fine-grained layer of gravelly SILT / CLAY over coarse-grained material comprising sandy GRAVEL / silty SAND and gravelly SAND. London Clay Formation formed the basal unit of the ground investigation and was present immediately below the RTD from 2.90 to 3.20mbgl.

Groundwater was encountered at 2.00-2.50mbgl in 3no. of the 4no. WS boreholes during the ground investigation. Standing groundwater will be placed at 2.00mbgl for geotechnical design purposes.

5.2 SHALLOW FOUNDATIONS

Given the proposed development and the ground conditions encountered a shallow foundation is considered appropriate dependant on the magnitude of the structural loadings, which are not available at this time.

5.2.1. Depth of Foundations

All foundations should be constructed beneath the Topsoil / Made Ground within the underlying RTD.

For foundations constructed within the RTD consideration will need to be given to its medium volume change potential identified from the consistency limits testing.

NHBC guidance offers two foundation depth scenarios dependant on future planting regimes (Table 5.1).

Table 5.1. Minimum depth of foundations for medium volume change soil (after NHBC 2020)

Depth of Foundation	Description	No Tree Planting Zone (m)		
		High Water Demand	Moderate Water Demand	Low Water Demand
0.90	No tree planting within Zone of Influence	1.25 x mature height	0.75 x mature height	0.50 x mature height
1.25	Restricted tree planting within zone of influence	1.00 x mature height	0.50 x mature height	0.20 x mature height

A minimum foundation depth of 0.90m is recommended, deepened if any of the proposed structures impinge upon the zone of influence of existing or recently removed trees (see Chart 3, Chapter 4.2, NHBC 2020).

5.2.2. Ultimate Limit State Bearing Resistance

Bearing resistance analysis has been carried out in order to check the Ultimate Limit State (ULS) bearing resistance of the RTD. The RTD was identified as comprising both fine- and coarse-grained materials, although due to the fine-grained content being greater than 25%, the effective shear strength parameter ϕ'_k is the same for both materials. Although the effective shear strength parameters for the coarse-grained RTD was determined by treating it as a fine-grained material, it is coarse-grained and therefore the ULS bearing resistances of both the fine-grained and coarse-grained RTD have been determined. All ULS bearing resistance calculations have been carried out in accordance with Eurocode 7.

In the bearing resistance analyses, a minimum foundation depth of 0.90m has been used with groundwater at 2.00mbgl (Table 5.2).

Table 5.2 Ground Model used in the ULS bearing resistance calculations

Ground Model	Stratum	Thickness (m)	Foundation Depth	Shear Strength Parameters			
				Total		Effective	
				c_{uk}	ϕ_{uk}	c'_k	ϕ'_k
GM1	Topsoil	0.25	0.90mbgl	Not used in foundation design			
	Fine-grained RTD	3.00		50	0	0	24°

Ground Model	Stratum	Thickness (m)	Foundation Depth	Shear Strength Parameters			
				Total		Effective	
				c_{uk}	ϕ_{uk}	c'_k	ϕ'_k
GM2	Topsoil	0.25	0.90mbgl				
	Coarse-grained RTD	3.00		Granular		0	24°
Groundwater at 2.00mbgl – 1.10m blow foundation base							

The UK has implemented Design Approach 1 (DA1) of BS EN1997-1:2004 (Eurocode 7: Geotechnical Design), which requires two design combinations to be satisfied:

Combination 1 (C1): A1 “+” M1 “+” R1

Combination 2 (C2): A2 “+” M2 “+” R1

Where A, M and R refer to groups of partial factors relating to actions, materials and resistance, respectively.

Table 5.3. Partial Factors Applied to the Bearing Resistance Analysis

	Combination 1			Combination 2		
	A1	M1	R1	A2	M2	R1
Tangent of the Angle of Shearing Resistance ($\tan \phi$)		1.00			1.25	
Cohesion Intercept (c')		1.00			1.25	
Undrained Cohesion c_u		1.00			1.40	
Permanent Action	1.35			1.00		
Variable Action	1.50			1.30		
Resistance			1.00			1.00

The summary of the bearing checks is presented in Table 5.4 and the geotechnical calculation sheets are located in Appendix 5.

Table 5.4. Summary of ULS Bearing Capacity Checks

Ground Model	Foundation type and dimensions	Foundation Material	Design Approach 1	Bearing Resistance (kN/m²)	
				Undrained	Drained
GM1	0.60m Wide Strip	Fine-grained RTD	C1	430	208
			C2	312	126
	1.00m Wide Strip		C1	392	237
			C2	285	140
GM2	0.60m Wide Strip	Coarse-grained RTD	C1	198	
			C2	120	
	1.00m Wide Strip		C1	226	
			C2	133	

The ultimate limit state bearing resistances presented in Table 5.4 represent the maximum load that can be supported by the foundation without failure occurring within the soil mass. Table 5.4 indicates that, regardless of whether the founding stratum is fine- or coarse-grained RTD, the application of combination 2 factoring produces the limiting values of 120 and 133kN/m² for a strip foundation of 0.60 and 1.00m width under effective stress conditions.

It is recommended that a single ULS bearing resistance of 120kN/m² is adopted for the entire thickness of the RTD, regardless of composition. Furthermore, it should be noted that if foundation deepening is required there is a possibility that foundation bases may impinge upon the zone of influence of the standing groundwater (foundation depth >1.40m & >1.00m for 0.60 and 1.00m wide foundation). However, it can be demonstrated that provided the foundation base remains above groundwater, the reduction in ULS bearing resistance associated with a groundwater correction is countered by an increase in bearing resistance associated with a deeper foundation. Therefore, a ULS bearing resistance of 120kN/m² is considered appropriate for all foundations constructed above the standing groundwater.

5.2.3. Serviceability Limit State

Eurocode 7 states that calculations of settlement must always be carried out on footings on soft clays - and should be carried out on footings on firm to stiff clays when the risk is anything other than negligible. Therefore, in strict accordance with EC7 a full SLS settlement analysis has been undertaken.

Verification of serviceability limit state (SLS) is demonstrated by satisfying the inequality:

$$s_{Ed} = s_0 \leq s_{Cd}$$

where s_{Ed} is the total settlement; and

s_{Cd} is the limiting value of that settlement

A limiting value of settlement (s_{Cd}) of 25mm is considered appropriate for the type of construction under consideration.

Settlement analyses have been carried out on both the 0.60 and 1.00m wide strip foundations to demonstrate the effect of widening the foundation.

The settlement analyses have been carried out using the Rockscience software programme Settle 3D.

The settlement worksheets are located in Appendix 5 of this Report and précised in Table 5.5.

Table 5.5. Results of the settlement analyses

Founding Stratum	Depth (mbgl)	Foundation Width (m)	Applied Foundation Loading (kN/m ²)	Settlement (mm)		
				Immediate	Consolidation	Total
Founding Stratum – Fine-grained RTD						
Fine-grained RTD	0.90	0.60	120	5	14	19
		1.00	120	7	17	24
Coarse-grained RTD		0.60	120	5	6	11
		1.00	120	8	8	16

Note all loads unfactored for Serviceability Limit State analysis

The results of the SLS settlement analyses indicate that settlements will remain under the SLS limit of 25mm for all strip foundations up to 1.00m wide.

5.3 OTHER GEOTECHNICAL CONSIDERATIONS

5.3.1. Floor Slabs

The medium volume change potential of the fine-grained RTD means a suspended floor slab option should be adopted. Minimum void spaces below suspended floors are presented in Table 7 of the NHBC Guide 2020, reproduced below as Table 5.6.

Table 5.6. Void dimensions (from NHBC 2020)

Volume change Potential	Void dimension against side of foundation and ground beam	Void dimensions under ground beams and suspended in- situ concrete ground floor	Void dimension under suspended precast concrete and timber floors ⁽¹⁾
Medium	25mm	100mm	250mm

¹ Under suspended floor the void dimension is measured from the underside of beam or joist to the ground surface and includes a 150mm ventilation allowance

5.3.2. Excavation Stability

No open excavations were undertaken during the ground investigation; therefore, the following assessment is based on the stability of the WS boreholes and visual description of the encountered materials. Due to the cohesive nature of the fine-grained RTD and the fine-grained content of the coarse-grained, foundation excavations should remain stable in the medium time provided they are completed above the standing groundwater level.

However, in line with good engineering practice, all foundation excavations should remain open for the minimum time possible prior to the placement of foundation concrete. If it becomes apparent that there will be a delay between excavation and placement of concrete, it is recommended that foundation excavations are left short to avoid deterioration of the founding stratum.

5.4 CHEMICAL ATTACK ON BURIED CONCRETE

Laboratory chemical testing of pH and sulphate concentrations were undertaken on samples of the RTD. Based on the results of these tests a Design Sulphate class of DS-1 and ACEC classification of AC-1 is considered appropriate for foundation concrete.

6. CONCLUSIONS

6.1 GROUND CONDITIONS

The site investigation has shown that ground conditions at the site comprise up to 0.40m of Topsoil / Made Ground over River Terrace Deposits (RTD) in turn overlying the London Clay Formation, which formed the basal unit of the ground investigation. The RTD comprised both fine and coarse-grained materials; however, geotechnical testing has shown the coarse-grained material to comprise > 25% fine-grained clay and silt such that the effective shear strength characteristic values are the same as those for the coarse-grained material.

Groundwater was set at 2.00mbgl for geotechnical design purposes.

6.2 SHALLOW FOUNDATIONS

6.2.1. Foundation Depths

All foundations should be constructed within the RTD, which was shown to be of medium volume change potential. Therefore, based on NHBC Guidance a minimum foundation depth of 0.90m is required away from the influence of any existing or recently removed trees or other vegetation (Chart 3, Chapter 4.2, NHBC 2020).

6.2.2. Bearing Resistance

ULS bearing resistance calculations have been carried out for both the fine-grained and coarse-grained RTD in accordance with Eurocode 7 and give a ULS bearing resistance of 120 and 133kN/m² for the range of foundation configurations analysed (Table 5.4). Based on these results a single ULS bearing resistance of 120kN/m² is recommended in both the coarse and fine-grained RTD.

Serviceability limit state settlement analyses show that total settlements will be less than 25mm for all strip foundations up to 1.00m wide founded in the RTD and loaded to the ULS bearing resistance of 120kN/m². Therefore, an SLS bearing resistances of 120kN/m² is considered appropriate for foundations up to 1.00m wide.

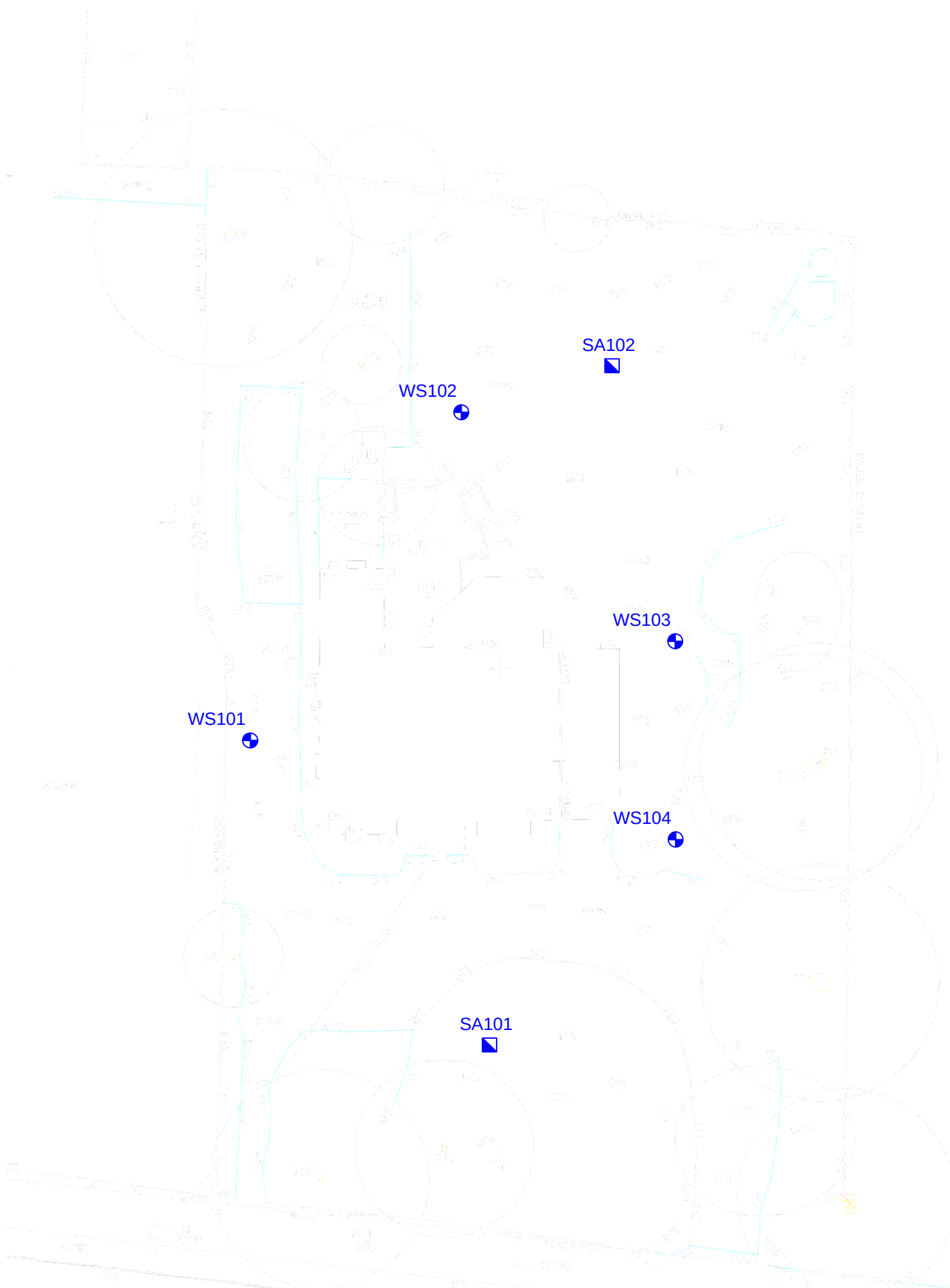
6.3 GROUND FLOOR SLABS

A ground bearing ground floor slab is not considered appropriate due to the medium volume change potential of the fine—grained RTD.

6.4 CONCRETE AGGRESSIVITY

Chemical tests carried out on the RTD shows it to be consistent with Design Sulphate Class 1. Based on the assumed mobile groundwater within the coarse-grained RTD, an ACEC classification of AC-1 should be adopted for all sub-surface concrete.

FIGURE 1: SITE INVESTIGATION PLAN



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client PAUL BASHAM ASSOCIATES

site 78 HAVANT ROAD, EMSWORTH

title SITE INVESTIGATION PLAN

scale NTS | drawn by DP | checked by RP

date 20 JUNE 2023 | job no J23-037

drawing number FIGURE 1 | rev. 1

FIGURE 2: SPT N V DEPTH

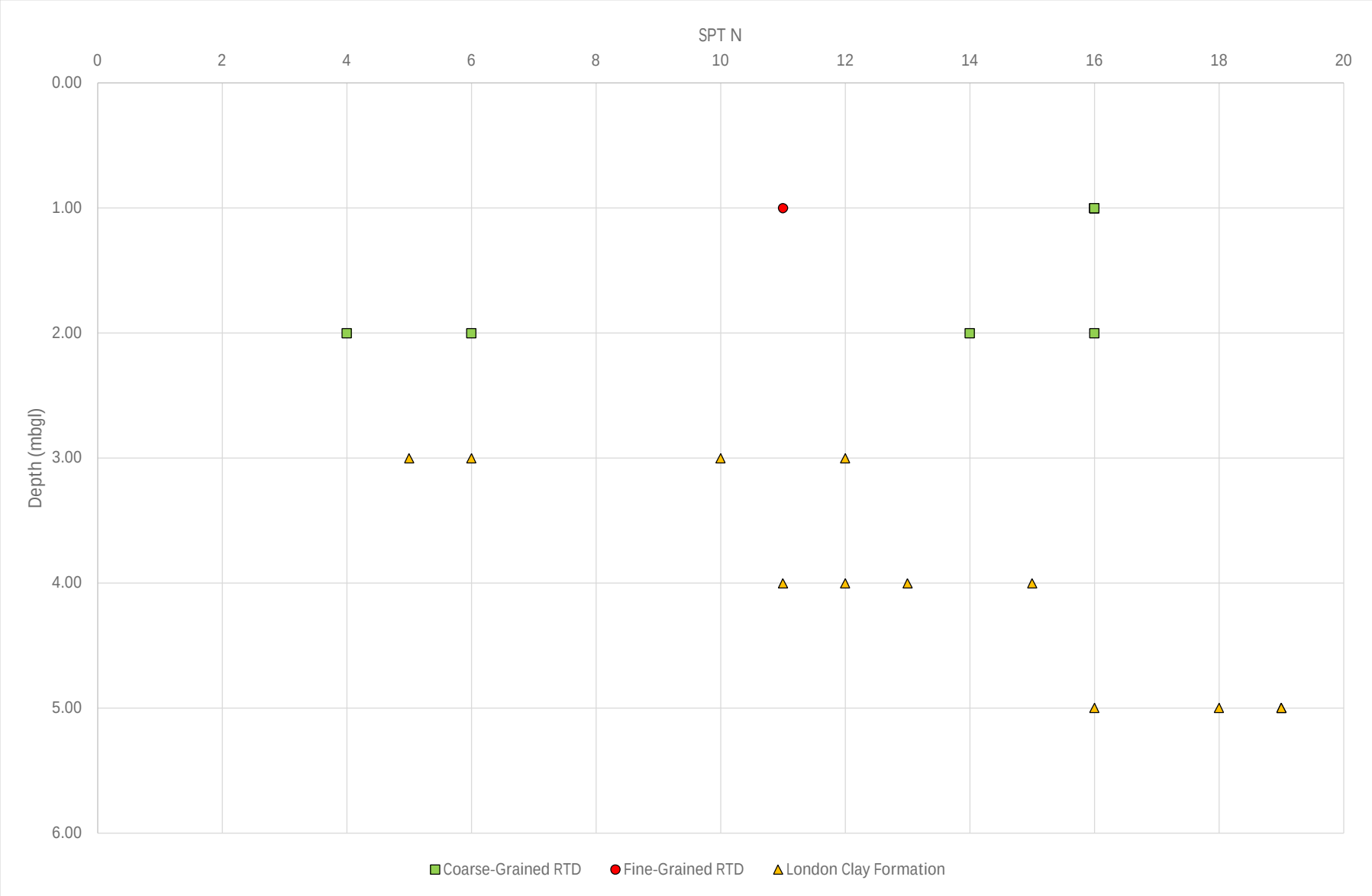
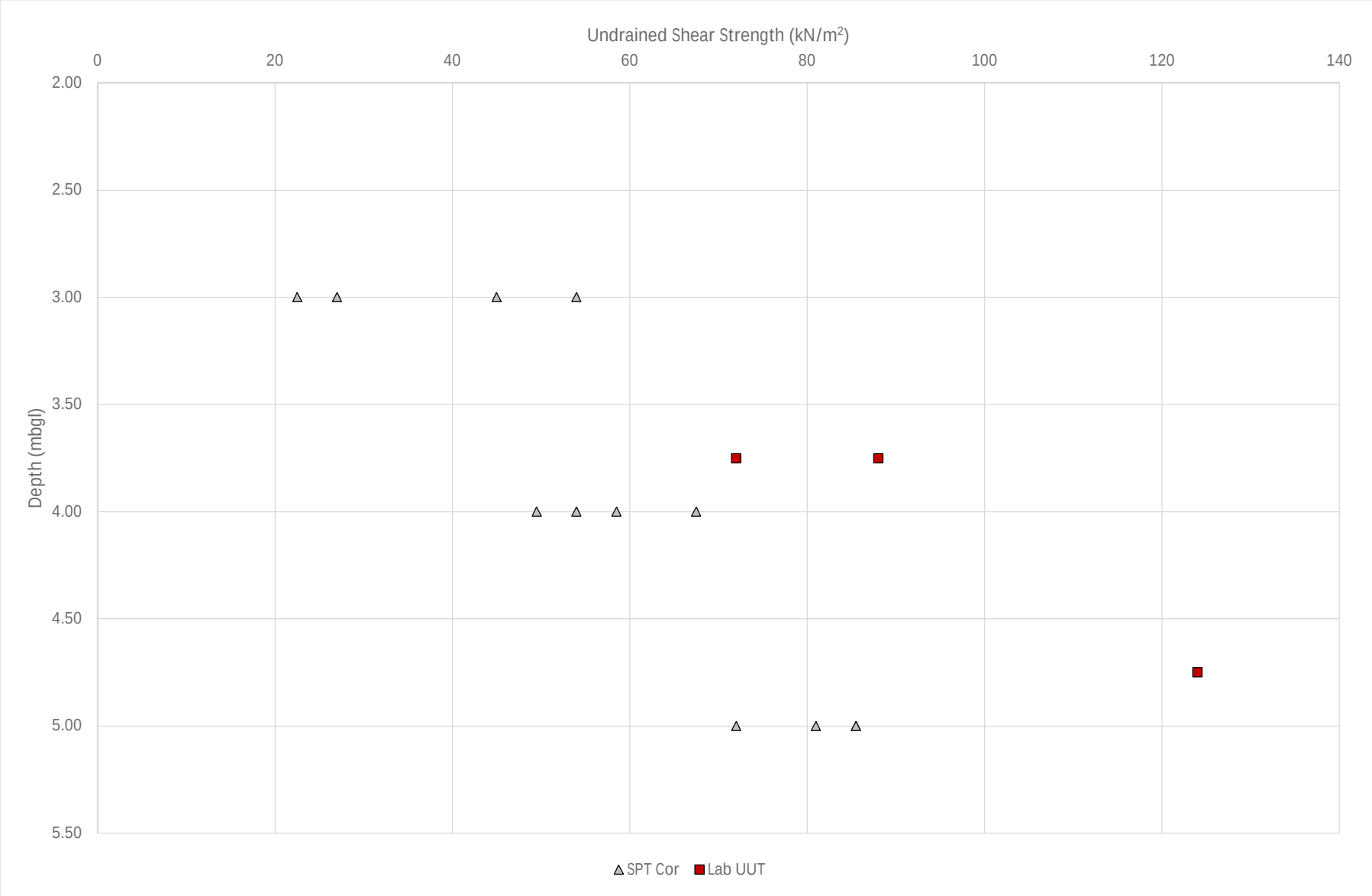


FIGURE 3: UNDRAINED SHEAR STRENGTH V DEPTH



APPENDIX 1: REPORT CONDITIONS

This report is produced solely for the benefit of **Baytree Securities Limited** and no liability is accepted for any reliance placed on it by any other party unless specifically agreed in writing otherwise.

This report refers, within the limitations stated, to the condition of the Site at the time of the inspections. No warranty is given as to the possibility of future changes in the condition of the Site.

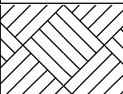
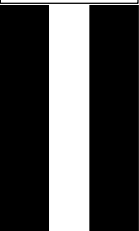
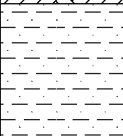
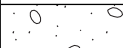
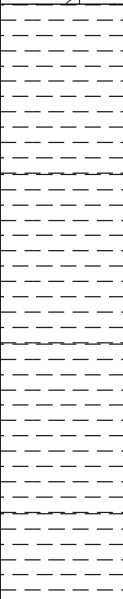
This report is based on a visual Site inspection, study of readily accessible referenced historical records, information supplied by those parties noted in the text and preliminary discussions with local and Statutory Authorities. Some of the opinions are based on unconfirmed data and information and are presented in good faith without exhaustive clarification. Where test results undertaken by others have been made available these can only be regarded as a limited sample.

This report is prepared for the proposed uses stated in the report and should not be used in a different context without reference to Ground Condition Consultants Ltd. In time improved practices or amended legislation may necessitate a re-assessment.

The report is necessarily limited to those aspects of geotechnical assessment reported on and no liability is accepted for any other aspect. The opinions expressed cannot be absolute due to the limitations of time and resources within the context of the agreed brief and the possibility of unrecorded previous use and abuse of the Site and adjacent Sites. The report concentrates on the Site as defined in the report and provides an opinion on surrounding Sites

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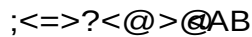
APPENDIX 2: ENGINEERING LOGS

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APPENDIX 3: GROUNDWATER MONITORING DATA

GROUND GAS AND GROUNDWATER MONITORING RESULTS



SITE/REF: 78 Havant Road, Emsworth						General Site comments				
DATE: 13/06/2023		1... of 1...								
OPERATOR(S): DAP										
Ambient Conditions:		Barometric Pressure (mb)	CH ₄ (%v/v)	CO ₂ (%v/v)	O ₂ (%v/v)				PID (ppm)	
Before Monitoring							EQUIPMENT			
After Monitoring										
Atmospheric Pressure Trend:							Instrument / Model Type		Serial Number	Comments
Ground Conditions:							GA2000 Gas Analyser		GA12820	
Weather Conditions:							Standard Dipmeter		79301	

Ground Gas Note: "0" readings to be recorded as instrument detection limit

Borehole ID	Time	Flow (l/hr)	Relative Pressure (mb)	Steady CH ₄ (%v/v)	CO ₂ (%v/v)	O ₂ (%v/v)	Peak CH ₄ (%v/v)	% of Lower Explosive Limit	CO (ppm)	H ₂ S (ppm)	Balance (%)	PID (ppm)	Response Zone		Response Strata	Gas screening value CH ₄	Gas screening value CO ₂
													From (mbgl)	To (mbgl)			

Groundwater

Borehole ID	Standing Water Level (mbgl)	Installation Base Dip (mbgl)	Free Phase LNAPL Thickness (m)	Free Phase DNAPL Thickness (m)	Water Quality Indicators					Well Volume (l)	Purge Volume (l)	Comments (samples)
					Eh	EC	pH	DO	Temp			
WS102	2.16	5.05										
WS104	4.34	5.01										

Well volume (l) = $(3.14 \times (\text{hole diameter (m)} / 2)^2 \times (\text{base dip (m)} - \text{standing water level (m)})) \times 1000$ (2l/m in 50mm well)

Gas screening value = gas concentration (%) x gas flow rate (l/hr)

%v/v - Percentage volume by total volume; mbgl - metres below ground level; ppm - parts per million; mb - milibars; l/hr - litres per hour; vv - water vial; gb - 1l glass bottle; pb - 1l plastic bottle

APPENDIX 4: GEOTECHNICAL TEST CERTIFICATES

GROUND GAS AND GROUNDWATER MONITORING RESULTS



SITE/REF: 78 Havant Road, Emsworth						General Site comments				
DATE: 20/06/2023		1... Of 1...								
OPERATOR(S): RP										
Ambient Conditions:		Barometric Pressure (mb)	CH ₄ (%v/v)	CO ₂ (%v/v)	O ₂ (%v/v)				PID (ppm)	
Before Monitoring							EQUIPMENT			
After Monitoring										
Atmospheric Pressure Trend:							Instrument / Model Type		Serial Number	Comments
Ground Conditions:		Dry					GA2000 Gas Analyser		GA12820	
Weather Conditions:		Overcast					Standard Dipmeter		79301	

Ground Gas Note: "0" readings to be recorded as instrument detection limit

Borehole ID	Time	Flow (l/hr)	Relative Pressure (mb)	Steady CH ₄ (%v/v)	CO ₂ (%v/v)	O ₂ (%v/v)	Peak CH ₄ (%v/v)	% of Lower Explosive Limit	CO (ppm)	H ₂ S (ppm)	Balance (%)	PID (ppm)	Response Zone		Response Strata	Gas screening value CH ₄	Gas screening value CO ₂
													From (mbgl)	To (mbgl)			

Groundwater

Borehole ID	Standing Water Level (mbgl)	Installation Base Dip (mbgl)	Free Phase LNAPL Thickness (m)	Free Phase DNAPL Thickness (m)	Water Quality Indicators					Well Volume (l)	Purge Volume (l)	Comments (samples)
					Eh	EC	pH	DO	Temp			
WS102	2.2	4.78										
WS104	2.29	4.72										

Well volume (l) = (3.14 x (hole diameter (m)/2)² x (base dip (m) - standing water level (m)))x1000 (2l/m in 50mm well)

Gas screening value = gas concentration (%) x gas flow rate (l/hr)

%v/v - Percentage volume by total volume; mbgl - metres below ground level; ppm - parts per million; mb - milibars; l/hr - litres per hour; ww - water vial; gb - 1l glass bottle; pb - 1l plastic bottle

APPENDIX 5: GEOTECHNICAL CALCULATION WORKSHEETS

Project Title:		Havant Road Emsworth			
Shallow Foundation		Strip Foundation			
Determination of Bearing Resistance		Calcs by	DF	Checked by	RP
Fine-Grained RTD					
Design Approach 1		Characteristic Geotechnical Parameters			
Combination 1 A1 + M1 + R1		Unit Weight	19		
		Undrained Cohesion	50		
		Cohesion	0		
		Shearing Resistance	24		
<u>Material Properties and Resistance</u>		Foundation Size			
Partial factors set M1: $\gamma_{cu} = 1$ $\gamma_{\phi} = 1$ and $\gamma_c = 1$		depth	0.90		
		breadth	0.60		
		$c_{ud} = \frac{c_{uk}}{\gamma_{cu}} =$	50		
Design shearing resistance is		$\phi_d = \tan^{-1} \left(\frac{\tan \phi_k}{\gamma_{\phi}} \right) =$	24		
Design cohesion is		$c'_d = \frac{c_k}{\gamma_c} =$	0		
<u>Drained Bearing Capacity Factors</u>					
For overburden		$N_q = \left[e^{(\pi \times \tan(\phi_d))} \times \left(\tan \left(45 + \frac{\phi_d}{2} \right) \right)^2 \right] =$	9.60		
For cohesion		$N_c = [(N_q - 1) \times \cot(\phi_d)] =$	19.31		
For self-weight		$N_{\gamma} = [2(N_q - 1) \times \tan(\phi_d)] =$	7.65		
<u>Depth and Shape Factors</u>					
Sorgado's depth factor for undrained loading:		$d_c = 1 + 0.27 \sqrt{\frac{d}{B}} =$	1.33		
Ignore depth factors for drained loading					
Sorgado's shape factor for undrained loading:		$s_c = 1 + 0.17 \sqrt{\frac{d}{B}} =$	1.21		
Shape factors are all 1.0 for drained loading and so can be ignored					
<u>Undrained Bearing Resistance</u>					
Total overburden at foundation base is		$\sigma_{vk,b} = \gamma_k \times d =$	17		
Partial factors set R1: $\gamma_{Rv} = 1$					
Ultimate resistance is		$q_{ult} = (\pi + 2) \times c_{ud} \times d_c \times s_c + \sigma_{vk,b} =$	430		
Design Resistance is		$q_{Rd} = \frac{q_{ult}}{\gamma_{Rv}} =$	430		

Project Title:		Havant Road Emsworth			
Shallow Foundation		Strip Foundation			
Determination of Bearing Resistance		Calcs by	DF	Checked by	RP
<u>Drained Bearing Resistance</u>		Depth to Groundwater		0	outside zone of influence
Effective overburden at foundation base is $\sigma'_{vk,b} = \sigma_{vk,b} - u_{k,b} = 17$					
From overburden $q'_{ult_1} = N_q \times \sigma'_{wk,b} = 164$					
From cohesion $q'_{ult_2} = N_c \times c'_d = 0$					
From self-weight $q'_{ult_3} = \left[N_\gamma \times (\gamma_k - \gamma_w) \times \frac{B}{2} \right] = 44$					
Total resistance is $q'_{ult} = \sum_{i=1}^3 q'_{ult_i} = 208$					
Design resistance is $q'_{Rd} = \frac{q'_{ult}}{\gamma_{Rv}} = 208$					
<u>Design Approach 1</u>					
Combination 2 A2 + M2 + R1					
<u>Material Properties and Resistance</u>					
Partial factors set M2: $\gamma_{cu} = 1.4$ $\gamma_\phi = 1.25$ and $\gamma_c = 1.25$					
Design undrained strength is $c_{ud} = \frac{c_{uk}}{\gamma_{cu}} = 36$					
Design shearing resistance is $\phi_d = \tan^{-1} \left(\frac{\tan \phi_k}{\gamma_\phi} \right) = 20$					
Design cohesion is $c'_d = \frac{c_k}{\gamma_c} = 0$					
<u>Drained Bearing Capacity Factors</u>					
For overburden $N_q = \left[e^{(\pi \times \tan(\phi_d))} \times \left(\tan \left(45 + \frac{\phi_d}{2} \right) \right)^2 \right] = 6.15$					
For cohesion $N_c = [(N_q - 1) \times \cot(\phi_d)] = 14.46$					
For self-weight $N_\gamma = [2(N_q - 1) \times \tan(\phi_d)] = 3.67$					

Project Title:		Havant Road Emsworth			
Shallow Foundation		Strip Foundation			
Determination of Bearing Resistance		Calcs by	DF	Checked by	RP
<u>Depth and Shape Factors</u>					
Solgado's depth factor for undrained loading:		$d_c = 1 + 0.27 \sqrt{\frac{d}{B}} =$		1.33	
Ignore depth factors for drained loading					
Solgado's shape factor for undrained loading:		$s_c = 1 + 0.17 \sqrt{\frac{d}{B}} =$		1.21	
Shape factors are all 1.0 for drained loading and so can be ignored					
<u>Undrained Bearing Resitance</u>					
Total overburden at foundation base is		$\sigma_{vk,b} = \gamma_k \times d =$		17	
Partial factors set R1: $\gamma_{Rv} = 1$					
Ultimate resistance is		$q_{ult} = (\pi + 2) \times c_{ud} \times d_c \times s_c + \sigma_{vk,b} =$		312	
Design Resistance is		$q_{Rd} = \frac{q_{ult}}{\gamma_{Rv}} =$		312	
<u>Drained Bearing Resistance</u>					
Effective overburden at foundation base is		$\sigma'_{vk,b} = \sigma_{vk,b} - u_{k,b} =$		17.10	
From overburden		$q'_{ult_1} = N_q \times \sigma'_{wk,b} =$		105	
From cohesion		$q'_{ult_2} = N_c \times c'_d =$		0	
From self-weight		$q'_{ult_3} = \left[N_\gamma \times (\gamma_k - \gamma_w) \times \frac{B}{2} \right] =$		21	
Total resistance is		$q'_{ult} = \sum_{i=1}^3 q'_{ult_i} =$		126	
Design Resistance is		$q'_{Rd} = \frac{q'_{ult}}{\gamma_{Rv}} =$		126	
Design Approach	Combination	Design Resistance			
		ULS (kN/m ²)			
		undrained	drained		
DA1	Combination 1	430	208		
	Combination 2	312	126		

Project Title:		Havant Road Emsworth			
Shallow Foundation		Strip Foundation			
Determination of Bearing Resistance		Calcs by	DF	Checked by	RP
Fine-Grained RTD					
Design Approach 1		Characteristic Geotechnical Parameters			
Combination 1 A1 + M1 + R1		Unit Weight	19		
		Undrained Cohesion	50		
		Cohesion	0		
		Shearing Resistance	24		
<u>Material Properties and Resistance</u>		Foundation Size			
Partial factors set M1: $\gamma_{cu} = 1$ $\gamma_{\phi} = 1$ and $\gamma_c = 1$		depth	0.90		
		breadth	1.00		
		$c_{ud} = \frac{c_{uk}}{\gamma_{cu}} =$	50		
Design shearing resistance is		$\phi_d = \tan^{-1} \left(\frac{\tan \phi_k}{\gamma_{\phi}} \right) =$	24		
Design cohesion is		$c'_d = \frac{c_k}{\gamma_c} =$	0		
<u>Drained Bearing Capacity Factors</u>					
For overburden		$N_q = \left[e^{(\pi \times \tan(\phi_d))} \times \left(\tan \left(45 + \frac{\phi_d}{2} \right) \right)^2 \right] =$	9.60		
For cohesion		$N_c = [(N_q - 1) \times \cot(\phi_d)] =$	19.31		
For self-weight		$N_{\gamma} = [2(N_q - 1) \times \tan(\phi_d)] =$	7.65		
<u>Depth and Shape Factors</u>					
Solgado's depth factor for undrained loading:		$d_c = 1 + 0.27 \sqrt{\frac{d}{B}} =$	1.26		
Ignore depth factors for drained loading					
Solgado's shape factor for undrained loading:		$s_c = 1 + 0.17 \sqrt{\frac{d}{B}} =$	1.16		
Shape factors are all 1.0 for drained loading and so can be ignored					
<u>Undrained Bearing Resistance</u>					
Total overburden at foundation base is		$\sigma_{vk,b} = \gamma_k \times d =$	17		
Partial factors set R1: $\gamma_{Rv} = 1$					
Ultimate resistance is		$q_{ult} = (\pi + 2) \times c_{ud} \times d_c \times s_c + \sigma_{vk,b} =$	392		
Design Resistance is		$q_{Rd} = \frac{q_{ult}}{\gamma_{Rv}} =$	392		

Project Title:		Havant Road Emsworth			
Shallow Foundation		Strip Foundation			
Determination of Bearing Resistance		Calcs by	DF	Checked by	RP
<u>Drained Bearing Resistance</u>		Depth to Groundwater		0	outside zone of influence
Effective overburden at foundation base is $\sigma'_{vk,b} = \sigma_{vk,b} - u_{k,b} = 17$					
From overburden $q'_{ult_1} = N_q \times \sigma'_{wk,b} = 164$					
From cohesion $q'_{ult_2} = N_c \times c'_d = 0$					
From self-weight $q'_{ult_3} = \left[N_\gamma \times (\gamma_k - \gamma_w) \times \frac{B}{2} \right] = 73$					
Total resistance is $q'_{ult} = \sum_{i=1}^3 q'_{ult_i} = 237$					
Design resistance is $q'_{Rd} = \frac{q'_{ult}}{\gamma_{Rv}} = 237$					
<u>Design Approach 1</u>					
Combination 2 A2 + M2 + R1					
<u>Material Properties and Resistance</u>					
Partial factors set M2: $\gamma_{cu} = 1.4$ $\gamma_\phi = 1.25$ and $\gamma_c = 1.25$					
Design undrained strength is $c_{ud} = \frac{c_{uk}}{\gamma_{cu}} = 36$					
Design shearing resistance is $\phi_d = \tan^{-1} \left(\frac{\tan \phi_k}{\gamma_\phi} \right) = 20$					
Design cohesion is $c'_d = \frac{c_k}{\gamma_c} = 0$					
<u>Drained Bearing Capacity Factors</u>					
For overburden $N_q = \left[e^{(\pi \times \tan(\phi_d))} \times \left(\tan \left(45 + \frac{\phi_d}{2} \right) \right)^2 \right] = 6.15$					
For cohesion $N_c = [(N_q - 1) \times \cot(\phi_d)] = 14.46$					
For self-weight $N_\gamma = [2(N_q - 1) \times \tan(\phi_d)] = 3.67$					

Project Title:		Havant Road Emsworth			
Shallow Foundation		Strip Foundation			
Determination of Bearing Resistance		Calcs by	DF	Checked by	RP
<u>Depth and Shape Factors</u>					
Solgado's depth factor for undrained loading:		$d_c = 1 + 0.27 \sqrt{\frac{d}{B}} =$		1.26	
Ignore depth factors for drained loading					
Solgado's shape factor for undrained loading:		$s_c = 1 + 0.17 \sqrt{\frac{d}{B}} =$		1.16	
Shape factors are all 1.0 for drained loading and so can be ignored					
<u>Undrained Bearing Resistance</u>					
Total overburden at foundation base is		$\sigma_{vk,b} = \gamma_k \times d =$		17	
Partial factors set R1: $\gamma_{Rv} = 1$					
Ultimate resistance is		$q_{ult} = (\pi + 2) \times c_{ud} \times d_c \times s_c + \sigma_{vk,b} =$		285	
Design Resistance is		$q_{Rd} = \frac{q_{ult}}{\gamma_{Rv}} =$		285	
<u>Drained Bearing Resistance</u>					
Effective overburden at foundation base is		$\sigma'_{vk,b} = \sigma_{vk,b} - u_{k,b} =$		17.10	
From overburden		$q'_{ult_1} = N_q \times \sigma'_{wk,b} =$		105	
From cohesion		$q'_{ult_2} = N_c \times c'_d =$		0	
From self-weight		$q'_{ult_3} = \left[N_\gamma \times (\gamma_k - \gamma_w) \times \frac{B}{2} \right]$		35	
Total resistance is		$q'_{ult} = \sum_{i=1}^3 q'_{ult_i} =$		140	
Design Resistance is		$q'_{Rd} = \frac{q'_{ult}}{\gamma_{Rv}} =$		140	
Design Approach	Combination	Design Resistance			
		ULS (kN/m ²)			
		undrained		drained	
DA1	Combination 1	392		237	
	Combination 2	285		140	

Project Title:		Havant Road Emsworth		
Shallow Foundation		Shallow Strip		
Determination of Bearing Resistance		Calcs by	DF	
Coarse-Grained RTD				
Design Approach 1		Characteristic Geotechnical Parameters		
Combination 1 A1 + M1 + R1		Unit Weight	18	
		Shearing Resistance	24	
		Cohesion	0	
<u>Material Properties and Resistance</u>		Foundation Size		
Partial factors set M1: $\gamma_\phi = 1$ and $\gamma_c = 1$		depth	0.9	
		breadth	0.6	
		length	30	
Design shearing resistance is $\phi_d = \tan^{-1}\left(\frac{\tan \phi_k}{\gamma_\phi}\right) = 24$				
Design cohesion is $c'_d = \frac{c_k}{\gamma_c} = 0$				
<u>Bearing Capacity Factors</u>				
For overburden $N_q = \left[e^{(\pi \times \tan(\phi_d))} \times \left(\tan\left(45 + \frac{\phi_d}{2}\right) \right)^2 \right] = 9.6$				
For cohesion $N_c = [(N_q - 1) \times \cot(\phi_d)] = 19.3$				
For self-weight $N_\gamma = [2(N_q - 1) \times \tan(\phi_d)] = 7.7$				
<u>Shape Factors</u>				
For overburden $s_q = \left[1 + \left(\frac{B}{L} \right) \times \sin(\phi_d) \right] = 1.01$				
For cohesion $s_c = \frac{(s_q \times N_q - 1)}{(N_q - 1)} = 1.01$				
For self weight $s_\gamma = 1 - 0.3 \times \left(\frac{B}{L} \right) = 0.994$				
<u>Bearing Resistance</u>				
Total overburden at foundation base is $\sigma_{vk,b} = \gamma_k \times d = 16.2$				
Partial factors set R1: $\gamma_{RV} = 1$				
From overburden $q_{ult_1} = N_q \times s_q \times \sigma_{vk,b} = 157$				
From cohesion $q_{ult_2} = N_c \times s_c \times c_d = 0$				

Project Title:		Havant Road Emsworth		
Shallow Foundation		Shallow Strip		
Determination of Bearing Resistance		Calcs by	DF	
From self-weight	$q_{ult_3} = \left(N_\gamma \times s_\gamma \times \gamma_k \times \frac{B}{2} \right) =$		41	
Total resistance is	$q_{ult} = \sum_{i=1}^3 q_{ult} =$		198	
Design resistance is	$q_{Rd} = \frac{q_{ult}}{\gamma_{Rv}} =$		198	
<u>Design Approach 1</u>				
Combination 2 A2 + M2 + R1				
<u>Material Properties and Resistance</u>				
Partial factors set M2: $\gamma_\phi = 1.25$ and $\gamma_c = 1.25$				
Design shearing resistance is	$\phi_d = \tan^{-1} \left(\frac{\tan \phi_k}{\gamma_\phi} \right) =$		19.6	
Design cohesion is	$c'_d = \frac{c_k}{\gamma_c} =$		0	
<u>Bearing Capacity Factors</u>				
For overburden	$N_q = \left[e^{(\pi \times \tan(\phi_d))} \times \left(\tan \left(45 + \frac{\phi_d}{2} \right) \right)^2 \right] =$		6.2	
For cohesion	$N_c = [(N_q - 1) \times \cot(\phi_d)] =$		14.5	
For self-weight	$N_\gamma = [2(N_q - 1) \times \tan(\phi_d)] =$		3.7	
<u>Shape Factors</u>				
For overburden	$s_q = \left[1 + \left(\frac{B}{L} \right) \times \sin(\phi_d) \right] =$		1.01	
For cohesion	$s_c = \frac{(s_q \times N_q - 1)}{(N_q - 1)} =$		1.01	
For self weight	$s_\gamma = 1 - 0.3 \times \left(\frac{B}{L} \right) =$		0.994	

Project Title:		Havant Road Emsworth		 Ground Condition Consultants
Shallow Foundation		Shallow Strip		
Determination of Bearing Resistance		Calcs by	DF	
<u>Bearing Resistance</u> Total overburden at foundation base is $\sigma_{vk,b} = \gamma_k \times d = 16.2$ Partial factors set R1: $\gamma_{Rv} = 1$ From overburden $q_{ult_1} = N_q \times s_q \times \sigma_{vk,b} = 100$ From cohesion $q_{ult_2} = N_c \times s_c \times c_d = 0$ From self-weight $q_{ult_3} = \left(N_\gamma \times s_\gamma \times \gamma_k \times \frac{B}{2} \right) = 20$ Total resistance is $q_{ult} = \sum_{i=1}^3 q_{ult_i} = 120$ Design resistance is $q_{Rd} = \frac{q_{ult}}{\gamma_{Rv}} = 120$				
Design Approach	Combination	Design Resistance (kN/m ²)		
DA1	Combination 1	198		
	Combination 2	120		

Project Title:		Havant Road Emsworth		 Ground Condition Consultants
Shallow Foundation		Shallow Strip		
Determination of Bearing Resistance		Calcs by	DF	
Coarse-Grained RTD				
Design Approach 1		Characteristic Geotechnical Parameters		
Combination 1 A1 + M1 + R1		Unit Weight	18	
		Shearing Resistance	24	
		Cohesion	0	
<u>Material Properties and Resistance</u>		Foundation Size		
Partial factors set M1: $\gamma_\phi = 1$ and $\gamma_c = 1$		depth	0.9	
		breadth	1	
		length	30	
Design shearing resistance is $\phi_d = \tan^{-1} \left(\frac{\tan \phi_k}{\gamma_\phi} \right) = 24$				
Design cohesion is $c'_d = \frac{c_k}{\gamma_c} = 0$				
<u>Bearing Capacity Factors</u>				
For overburden $N_q = \left[e^{(\pi \times \tan(\phi_d))} \times \left(\tan \left(45 + \frac{\phi_d}{2} \right) \right)^2 \right] = 9.6$				
For cohesion $N_c = [(N_q - 1) \times \cot(\phi_d)] = 19.3$				
For self-weight $N_\gamma = [2(N_q - 1) \times \tan(\phi_d)] = 7.7$				
<u>Shape Factors</u>				
For overburden $s_q = \left[1 + \left(\frac{B}{L} \right) \times \sin(\phi_d) \right] = 1.01$				
For cohesion $s_c = \frac{(s_q \times N_q - 1)}{(N_q - 1)} = 1.02$				
For self weight $s_\gamma = 1 - 0.3 \times \left(\frac{B}{L} \right) = 0.99$				
<u>Bearing Resistance</u>				
Total overburden at foundation base is $\sigma_{vk,b} = \gamma_k \times d = 16.2$				
Partial factors set R1: $\gamma_{RV} = 1$				
From overburden $q_{ult_1} = N_q \times s_q \times \sigma_{vk,b} = 158$				
From cohesion $q_{ult_2} = N_c \times s_c \times c_d = 0$				

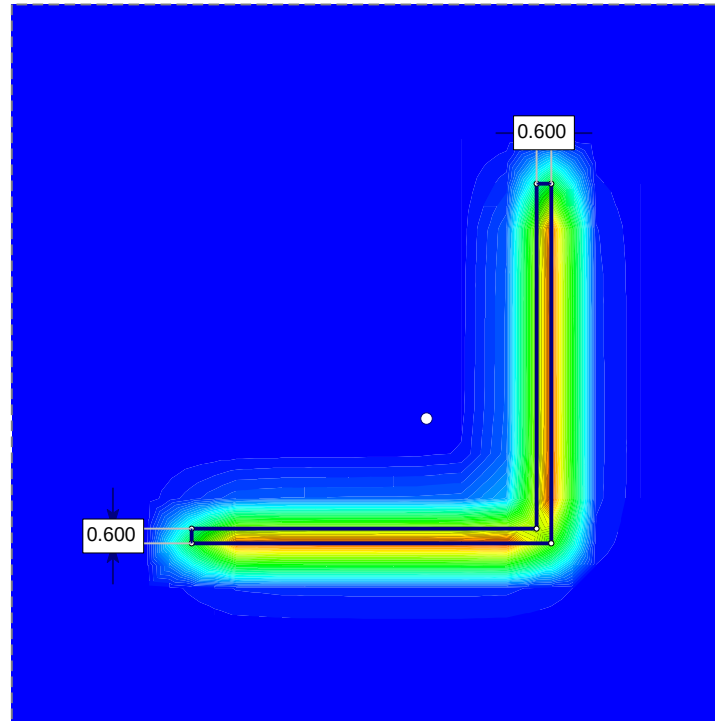
Project Title:		Havant Road Emsworth		
Shallow Foundation		Shallow Strip		
Determination of Bearing Resistance		Calcs by	DF	
From self-weight	$q_{ult_3} = \left(N_\gamma \times s_\gamma \times \gamma_k \times \frac{B}{2} \right) =$		68	
Total resistance is	$q_{ult} = \sum_{i=1}^3 q_{ult} =$		226	
Design resistance is	$q_{Rd} = \frac{q_{ult}}{\gamma_{Rv}} =$		226	
<u>Design Approach 1</u>				
Combination 2 A2 + M2 + R1				
<u>Material Properties and Resistance</u>				
Partial factors set M2: $\gamma_\phi = 1.25$ and $\gamma_c = 1.25$				
Design shearing resistance is	$\phi_d = \tan^{-1} \left(\frac{\tan \phi_k}{\gamma_\phi} \right) =$		19.6	
Design cohesion is	$c'_d = \frac{c_k}{\gamma_c} =$		0	
<u>Bearing Capacity Factors</u>				
For overburden	$N_q = \left[e^{(\pi \times \tan(\phi_d))} \times \left(\tan \left(45 + \frac{\phi_d}{2} \right) \right)^2 \right] =$		6.2	
For cohesion	$N_c = [(N_q - 1) \times \cot(\phi_d)] =$		14.5	
For self-weight	$N_\gamma = [2(N_q - 1) \times \tan(\phi_d)] =$		3.7	
<u>Shape Factors</u>				
For overburden	$s_q = \left[1 + \left(\frac{B}{L} \right) \times \sin(\phi_d) \right] =$		1.01	
For cohesion	$s_c = \frac{(s_q \times N_q - 1)}{(N_q - 1)} =$		1.01	
For self weight	$s_\gamma = 1 - 0.3 \times \left(\frac{B}{L} \right) =$		0.99	

Project Title:		Havant Road Emsworth		 Ground Condition Consultants
Shallow Foundation		Shallow Strip		
Determination of Bearing Resistance		Calcs by	DF	
<u>Bearing Resistance</u> Total overburden at foundation base is $\sigma_{vk,b} = \gamma_k \times d = 16.2$ Partial factors set R1: $\gamma_{Rv} = 1$ From overburden $q_{ult_1} = N_q \times s_q \times \sigma_{vk,b} = 101$ From cohesion $q_{ult_2} = N_c \times s_c \times c_d = 0$ From self-weight $q_{ult_3} = \left(N_\gamma \times s_\gamma \times \gamma_k \times \frac{B}{2} \right) = 33$ Total resistance is $q_{ult} = \sum_{i=1}^3 q_{ult_i} = 133$ Design resistance is $q_{Rd} = \frac{q_{ult}}{\gamma_{Rv}} = 133$				
Design Approach	Combination	Design Resistance (kN/m ²)		
DA1	Combination 1	226		
	Combination 2	133		

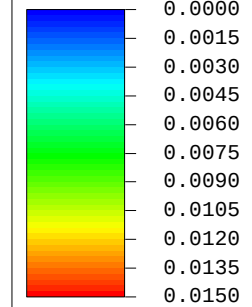
Stage 1

Data Type: Consolidation Settlement

Founding Stratum: Fine-grained RTD
Foundation at 0.90mbgl
0.60m wide
Foundation load = 120kN/m²
Total Settlement = 19mm



Consolidation Settlement
(m)



max (stage): 0.0142 m
max (all): 0.0142 m



SETTLE3D 2.010

Project

Havent Road Emsworth

Analysis Description

Strip Foundation

Drawn By

Dr David Fall

Company

Ground Conditons Consultants

Date

03/07/2023

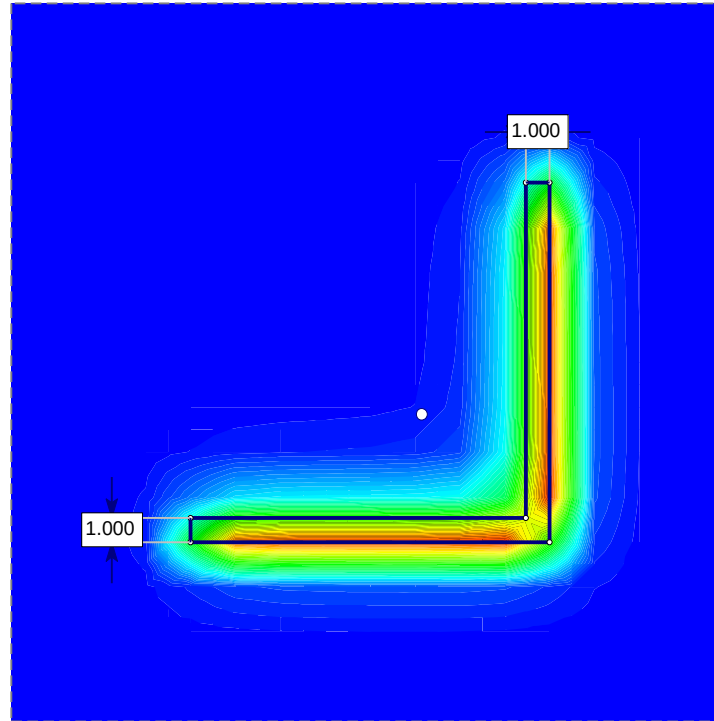
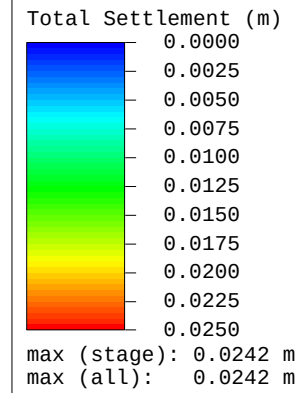
File Name

Settlement Strip 1.s3z

Stage 1

Data Type: Total Settlement

Founding Stratum: Fine-grained RTD
Foundation at 0.90mbgl
1.00m wide
Foundation load = 120kN/m²
Total Settlement = 24mm



SETTLE3D 2.010

Project

Havent Road Emsworth

Analysis Description

Strip Foundation

Drawn By

Dr David Fall

Company

Ground Conditons Consultants

Date

03/07/2023

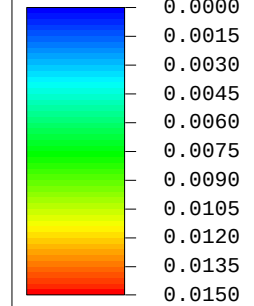
File Name

Settlement Strip 1.s3z

Stage 1

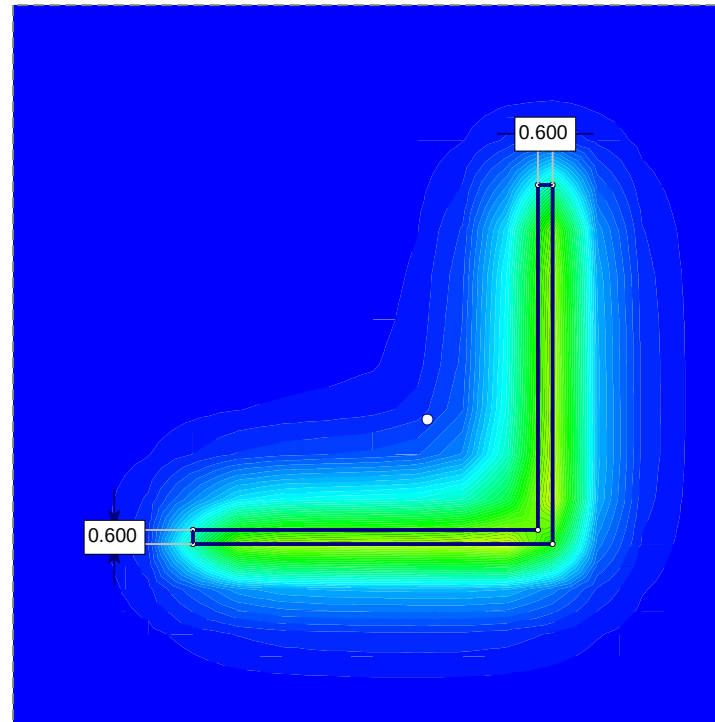
Data Type: Total Settlement

Total Settlement (m)



max (stage): 0.0109 m
max (all): 0.0109 m

Founding Stratum: Coarse-grained RTD
Foundation at 0.90mbgl
0.60m wide
Foundation load = 120kN/m²
Total Settlement = 11mm



SETTLE3D 2.010

Project

Havent Road Emsworth

Analysis Description

Strip Foundation

Drawn By

Dr David Fall

Company

Ground Conditons Consultants

Date

03/07/2023

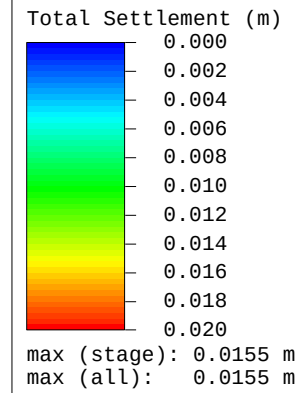
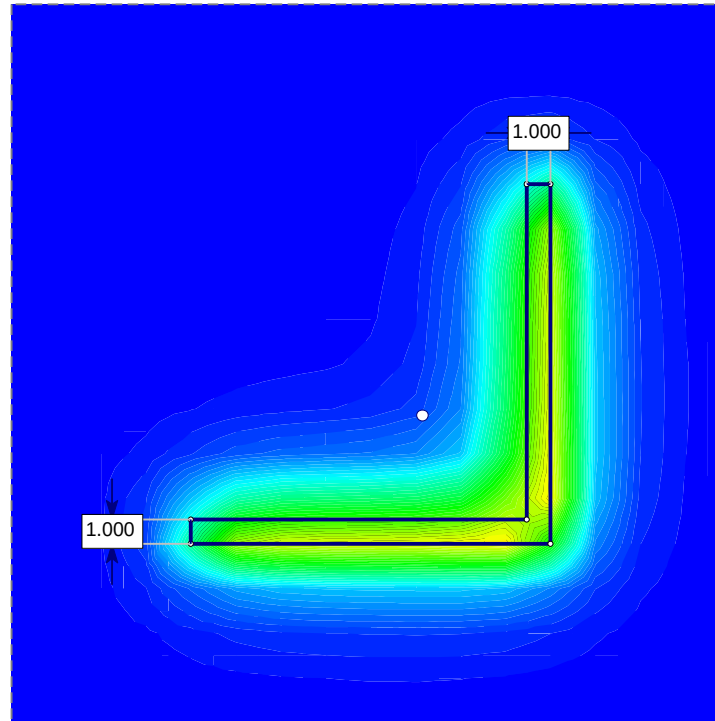
File Name

Settlement Strip 1.s3z

Stage 1

Data Type: Total Settlement

Founding Stratum: Coarse-grained RTD
Foundation at 0.90mbgl
1.00m wide
Foundation load = 120kN/m2
Total Settlement = 16mm



SETTLE3D 2.010

Project

Havent Road Emsworth

Analysis Description

Strip Foundation

Drawn By

Dr David Fall

Company

Ground Conditons Consultants

Date

03/07/2023

File Name

Settlement Strip 1.s3z

Appendix B

Flood map for planning

Your reference
<Unspecified>

Location (easting/northing)
473924/105927

Created
20 Jul 2023 9:56

Your selected location is in flood zone 1, an area with a low probability of flooding.

You will need to do a flood risk assessment if your site is **any of the following**:

- bigger than 1 hectare (ha)
- In an area with critical drainage problems as notified by the Environment Agency
- identified as being at increased flood risk in future by the local authority's strategic flood risk assessment
- at risk from other sources of flooding (such as surface water or reservoirs) and its development would increase the vulnerability of its use (such as constructing an office on an undeveloped site or converting a shop to a dwelling)

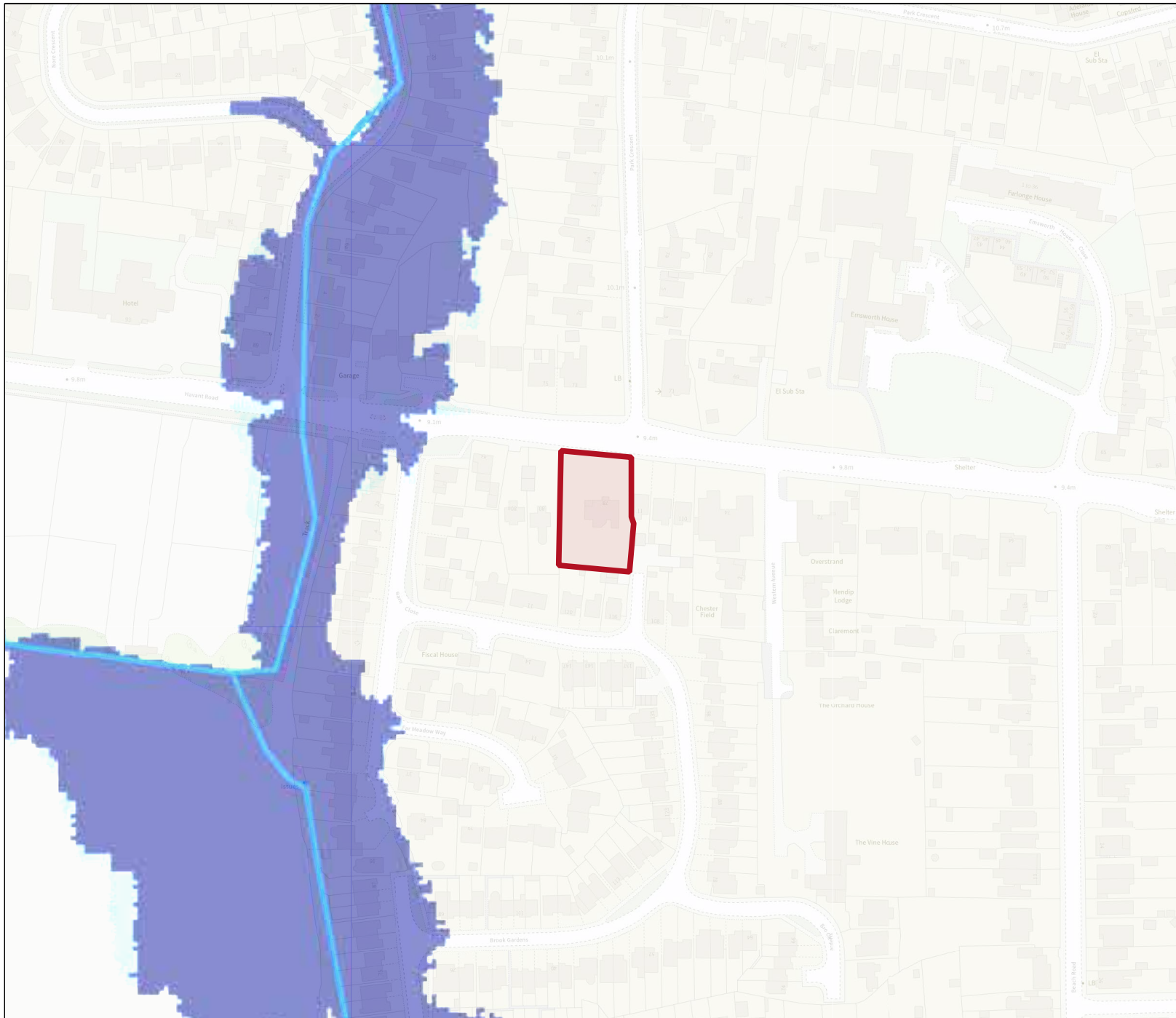
Notes

The flood map for planning shows river and sea flooding data only. It doesn't include other sources of flooding. It is for use in development planning and flood risk assessments.

This information relates to the selected location and is not specific to any property within it. The map is updated regularly and is correct at the time of printing.

Flood risk data is covered by the Open Government Licence **which** sets out the terms and conditions for using government data. <https://www.nationalarchives.gov.uk/doc/open-government-licence/version/3/>

Use of the address and mapping data is subject to Ordnance Survey public viewing terms under Crown copyright and database rights 2022 OS 100024198. <https://flood-map-for-planning.service.gov.uk/os-terms>



Flood map for planning

Your reference
<Unspecified>

Location (easting/northing)
473924/105927

Scale
1:2500

Created
20 Jul 2023 9:56

-  Selected area
-  Flood zone 3
-  Flood zone 2
-  Flood zone 1
-  Flood defence
-  Main river
-  Water storage area


0 20 40 60m

Appendix C

Calculated by:	Beth Gifford
Site name:	78 Havant Road
Site location:	

This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance "Rainfall runoff management for developments", SC030219 (2013), the SuDS Manual C753 (Ciria, 2015) and the non-statutory standards for SuDS (Defra, 2015). This information on greenfield runoff rates may be the basis for setting consents for the drainage of surface water runoff from sites.

Site Details

Latitude:	50.84816° N
Longitude:	0.95123° W
Reference:	55449485
Date:	Jul 20 2023 14:37

Runoff estimation approach

IH124

Site characteristics

Total site area (ha):	0.1675
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Methodology

Q_{BAR} estimation method:	Calculate from SPR and SAAR
SPR estimation method:	Calculate from SOIL type

Notes

(1) Is $Q_{BAR} < 2.0$ l/s/ha?

When Q_{BAR} is < 2.0 l/s/ha then limiting discharge rates are set at 2.0 l/s/ha.

Soil characteristics

	Default	Edited
SOIL type:	2	3
HOST class:	N/A	N/A
SPR/SPRHOST:	0.3	0.37

(2) Are flow rates < 5.0 l/s?

Where flow rates are less than 5.0 l/s consent for discharge is usually set at 5.0 l/s if blockage from vegetation and other materials is possible. Lower consent flow rates may be set where the blockage risk is addressed by using appropriate drainage elements.

Hydrological characteristics

	Default	Edited
SAAR (mm):	721	721
Hydrological region:	7	7
Growth curve factor 1 year:	0.85	0.85
Growth curve factor 30 years:	2.3	2.3
Growth curve factor 100 years:	3.19	3.19
Growth curve factor 200 years:	3.74	3.74

(3) Is $SPR/SPRHOST \leq 0.3$?

Where groundwater levels are low enough the use of soakaways to avoid discharge offsite would normally be preferred for disposal of surface water runoff.