Report No	Date
WE00298/R2 Rev A	September 2010

Project

Pinhoe Clay Pit
Geotechnical Assessment; Interpretative Report

Client

Pinhoe Quarry LLP

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2	**Issue**	
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1.0 INTRODUCTION

1.1 Instruction

Clarke Bond (South West & Wales) Ltd (CB) was commissioned by Pinhoe Quarry LLP to undertake a ground investigation to provide parameters for slope stability and geoenvironmental assessment at the former Pinhoe Clay Pit in Exeter.

1.2 Background

The Pinhoe Clay Pit, formerly owned and operated by Ibstock Brickworks, together with two fields to the north east and one field to the north west have been acquired by Pinhoe Quarry LLP. In order to assess development options and opportunities it is necessary to understand the ground conditions at the site. An intrusive ground investigation was undertaken by Clarke Bond (SW&W) Ltd the report reference is:

- WE00298/R1 Rev A September 2010 – Pinhoe Clay Pit – Ground Investigation; Factual Report

Previous slope stability assessments were reviewed by CB and the findings presented in a letter report dated 22nd September 2009. The reports included the following:

- GEOPLAN LTD June 2000: Pinhoe Quarry Slope Stability Assessment – Ibstock Property and Mineral Services.
- James Associates December 2002: Geotechnical Site File – Ibstock Westbrick Ltd, Pinhoe Quarry Exeter.

1.3 Objectives

The objective of the investigation is to identify ground related risks, uncertainty and mitigation measures to allow development opportunities to be explored. The ground investigation works included the following:

- Surface mapping of the structure and lithology of the rock exposure.
- Rotary boreholes to obtain rock cores and install monitoring equipment.
- Installation of instruments including piezometers, inclinometers and gas monitoring standpipes.
- Cable Percussive drilling and trial pitting to assess a land-raise in south west corner of site.
- Trial pitting in the base of quarry, stockpiles and field to north east.

The purpose of this geotechnical assessment is to identify site-specific risk and uncertainty associated with potential development options. Specific details are required in respect of the following:

- Slope stability in relation to infilling and re-profiling works.
- Long-term slope stability in relation to the proposed development platform.
- Re-use, recycling, recovery and disposal options for existing materials within quarry.
- Re-use and recovery of materials obtained via the development options for the site.
- In-filling and backfilling requirements to achieve proposed development platform.
- Potential long-term risks and mitigation measures associated with proposed development platform.

1.4 Report Layout

Section 2 of this report provides details of the site. Section 3 provides details of the intrusive investigation.

The following sections detail specific geotechnical issues associated with the site redevelopment:

4. Geotechnical risk assessment associated with proposed development platform.
5. Slope stability
6. Re-use, recycling, recovery and disposal options for existing materials.
7. Infilling and backfilling requirements to achieve proposed development platform.
8. Groundwater and ground gases.

Conclusions and recommendations are presented in Section 9.

1.5 Methodology

A review of previous slope stability assessment reports was undertaken as an initial desk top study of the ground related risks associated with the site. This included a geotechnical risk assessment which was used to focus the ground investigation on particular areas of the site.

Further detailed rock mass mapping was undertaken on accessible slopes in order to provide a better understanding of the slope stability parameters.

The intrusive ground investigation included, trial pitting, cable percussive boreholes, rotary open hole drilling and rotary coring techniques to observe and sample the substrata. Ground monitoring instruments were installed and monitored over a period, to date, of one month.

Systematic soil and rock descriptions provide a basis for determining the ground model at the site. Laboratory testing has been scheduled based on the strata observed to allow both geotechnical and geo-environmental classification of materials for re-use options.

1.6 Limitations

Subsoils are inherently variable and by their very nature are hidden from view such that no investigation can be exhaustive to the extent that all soil conditions are revealed. Conditions may therefore be present beneath the site that were not apparent from the limited number of exploratory hole locations.

Groundwater level fluctuation may occur outside the range observed during the monitoring period.

2.0 SITE DETAILS

2.1 Site Location

The site is located at the former clay pits in the Pinhoe area of Exeter, Devon. The site is located at the approximate Ordnance Survey grid reference of SX 954 946.

A site location plan is presented as Figure 1.

2.2 Site Description

The site is approximately 520m long east to west by 390m wide north to south. The site comprises a former quarry and includes outlying fields to the north east and woodland to the west. The general topography of the area drops from 93mAOD in the north west to 50mAOD in the south west. The sites topographic low is located in the pond in the base of the quarry (located towards the south east corner of the site) at approximately 35mAOD.

The site has been divided into twelve zones for description, this includes the following areas:

- Quarry Base (centre of site)
- Quarry Faces (north west, north, north east, east, south east, south and west)
- Quarry Rim
- Fields and Stables (north east)
- Quarry tailings stockpiles and land-raise (south west)
- Woodland and scrubland (west)
- Compound and access roads (south west)

Quarry Base

The quarry base, located in the centre of the site, is rectangular in shape and measures approximately 250m long and 130m wide. The ground levels vary from 48mAOD in the south-western corner to approximately 35mAOD in the base of the pond in the south east corner. The ground surface of the base is a mixture of exposed rock and clayey shale gravel. Vegetation including bullrushes, grass and algae are located around the inflowing streams emanating from the north half of the site and the two ponds and other areas of standing water.

The two ponds are located on the eastern side of the quarry base. The smaller of the two ponds measures 25m by 12m and is located above the larger pond with a recorded water level of approximately 43.6mAOD. The larger pond measures 78m by 67m and acts as a collection point for the intermittent stream (sourced as highway run-off to the north) coming from the fields to the north (via the waterfall) and the water seepage out of the north-western face which flows across the quarry base via the smaller pond. The recorded water level of the larger pond is 37.5mAOD, however this fluctuates due to variations in pumping rates. The pond serves as a pumped sump from which the collected water is discharged to a consented discharge in the south east corner of the site.

A stockpile of extracted shale is located in the centre of the quarry base. The square shaped stockpile measures 75m by 75m and is approximately 6m high. The stockpile has been in-situ for a number of years and has been colonised with vegetation including grass and buddleia.

Two buildings are located towards the western end of the quarry base. They comprise of a brick built conveyor-house from which a conveyor belt connects to a steel framed open shed located on the south elevation; the whole system appears to have been a loading facility presumably to facilitate transport of shale from the quarry to the brickworks. A brick rubble access ramp is located to the north east of the building.

Quarry Faces
The following table presents the quarry face information:

Quarry Face	Top-Base mAOD	General. Height (m)	Overall Gradient	Comments
North (West Section)	87-47	40	28 degrees	Strata inclined into face generally at 30-70 to north, Surface is of gravel of shale with occasional Buddleia
North (East Section)	82-45	37	32 degrees	Strata inclined into face generally at 30-70 to north, Surface is of gravel of shale with occasional Buddleia A stream runs down the eastern end of the face and forms a small waterfall.
East	63-37	26	27 degrees	Strata inclined into face generally at 30-70 to north, Surface is covered with bricks.
South	38-60	22	Lower slope 26 degrees Upper slope 36 degrees	Slope cut in two by track. The lower slope is soil covered and scarcely vegetated. The upper slope is formed by a sandstone band, Strata inclined generally at 50-70 to the north,
West/north west	78-48	30	37degrees	Strata inclined into face generally at 30-70 to north, Surface is of gravel of shale with occasional Buddleia

Note – Gradients quoted above are a general average of the subject slopes. Steeper and flatter gradients are present on sections of all slopes.

Quarry Rim
A track runs along the crest of the northern quarry face. Small plateaux are located above the north-western and north-eastern corners of the quarry. Topsoil Bunds are also located here.

A narrow plateau is also present along the southern crest of the quarry face. The plateau is generally overgrown however access is possible along the western end.

A former pump house is located in the south east corner of the site. Next to this is an area where a pipe discharges water pumped from the large pond into a ditch that runs in a south-easterly direction off-site.

Field and Stables
In the north east corner of the site are two large fields, and a stable block with paddock.

The western of the two fields is rectangular in shape and measures 113m by 92m. The highest point in the field is in the north west corner and is at approximately 93mAOD and the lowest is in the south west corner at approximately 80mAOD. The field is grass covered and is used as grazing land for horses.

A hedge with a gate at the southern end separates the two fields. The eastern field is also rectangular in shape and measures 110m by 85m. The highest point on the field is in the north east corner at 91mAOD and the lowest is in the south west corner at approximately 70mAOD. An intermittent stream runs from north to south down the field and has formed a small valley; the bottom of the valley is quite boggy. The field is grass covered and is used as grazing land for horses.

A stable block is located on the western boundary with the access road leading in from the south west. There are two buildings located in this area, one is a long single storey building that is used as a stable, the other is a small single storey building used for storage.

A small triangular paddock is located to the south of the stable block. The paddock appears heavily used and is quite rutted.

Landfill Area
In the south west corner of the site is a partially restored area comprising a backfilled former part of the quarry working. The area is triangular in shape and drops from 66mAOD in the north to 50mAOD in the south west corner. Access to the area is difficult, however a track that runs up the north-eastern boundary of the land-raise allows access to its northern portion

A public footpath path runs along the western boundary of the land-raise.

Wooded and Scrub Area
The far western portion of the site comprises a roughly rectangular area of land measuring approximately 130m by 80m. The topographic high in the area is located in the north west corner at 79mAOD and the low is located in the centre of the southern boundary at 59mAOD. A valley has been cut through the middle of the area with a winterbourne stream running from north to south. The valley sides are vegetated by trees, scrub and bracken.

Quarry Compound and Access Routes
The main access to the quarry is off Harrington Lane through the gated entrance on the southern site boundary. A concrete road leads north through the gate and down to the two buildings in the quarry base. Immediately inside the gate and to the west of the road is a corrugated iron fenced compound and a red brick building. The building contains a toilet, control room with water tanks and pumps for a wheel wash system, and a room containing electrical equipment.

A clayey gravel track runs north east parallel with the concrete road for approximately 100m before turning east. This track circles the quarry.

Approximately 40m north of the compound to the west of the concrete road is a vegetated stockpile. This stockpile is believed to contain quarry waste. On the southern edge of the stock pile is a small disused brick building and concrete chamber (allegedly 4m deep).

Surrounding land uses are presented in the table below:

Direction	Boundary
North	Church, Fields, House.
East	Fields and housing
South	Housing, Commercial units, Brickworks.
West	Fields and housing

A site layout and exploratory hole location plan is presented in Figure 2.

2.3 Geology

The published British Geological Survey mapping (Sheet 325) 1:50,000 series indicates the site to be in an area underlain by the Carboniferous Crackington Formation. No Drift deposits are shown on the mapping however given the sites' history as a clay pit it is likely a layer of clay Head deposits mantle the solid geology.

2.4 Hydrology

Water features within the property include two streams and two ponds. The winterbourne stream to the west of the quarry discharges to the Pin Brook. The intermittent stream in the north east part of the quarry appears to be largely sourced as highway drainage from the lane to the north of the site; this discharges to the larger of the two ponds. Water is pumped from the large pond into a ditch which then flows off site in a southerly direction and discharges to the Pin Brook to the south. Some water seepage emanates from the toe of the north west quarry face.

2.5 Hydrogeology

Information provided by the Environment Agency indicated that the Crackington Formation is classed as a minor aquifer.

The Environment Agency mapping indicates that the site in not located in Source Protection Zone.

3.0 INTRUSIVE INVESTIGATION

3.1 Introduction

A targeted / judgemental intrusive investigation has been undertaken. The investigation has included slope inspection, trial pits, cable percussion boreholes, rotary open boreholes and rotary cored boreholes with installation of ground monitoring instruments.

3.2 Slope Inspection

Between the 4th and 6th of December 2009 four sections on the north and north western faces were mapped. The mapping involved recording the angle of the slope, the dip, dip direction, the fracture orientation and direction and the ratio of shale to sandstone beds, at 1m intervals.

The record of the mapping and photographic record is presented in Report WE00298/R1 Appendix A.

3.3 Trial Pitting

Twenty one trial pits were excavated on the site between 18th and 20th of December 2009 to depths ranging from 0.3m to 4.0m. The trial pits were excavated using a 13 tonne tracked excavator. Trial pits terminated at the maximum reach of the excavator or where hard ground or spalling of the pit sides precluded further excavation.

Trial pits were excavated to a depth of 1m and, where safe to do so, were entered for detailed logging, sampling and in-situ testing. Pits were then extended to final depth with sampling and testing being carried out from surface. Soils were described in accordance with BS5930: 1999 nomenclature.

The following table presents the trial pits and their targets.

Trial Pit	Target
TP1	Land raise
TP2	Land raise
TP3	Land raise
TP4	Land raise
TP5	Quarry base
TP6	Stockpile
TP7	Quarry base
TP8	Quarry base
TP9	Quarry base
TP10	North quarry face
TP11	General ground conditions
TP12	Clay stockpile
TP13	Clay stockpile
TP14	Field
TP15	Field
TP16	Not undertaken – unable to reach proposed location
TP17	Stockpile
TP18	Stockpile
TP19	Land raise
TP20	Land raise
TP21	Land raise

Trial Pit records and photos are presented in Report WE00298/R1 Appendix B.

3.4 Cable Percussion Boreholes

Five cable percussion boreholes (CP1 to CP4) were drilled between the 16th and 20th of December 2009 to depths ranging from 5.5mbgl to 13mbgl.

The soil recovered was observed directly for logging purposes. Sub-samples were taken for subsequent laboratory analysis.

Three of the five boreholes were completed as monitoring wells in order to enable soil gas and groundwater sampling to be undertaken.

The following table presents the borehole details:

Borehole	Location	Depth (m)	Well Installation
CP1	South west area of land raise	12.70	None
CP2	North area of land raise	5.50	None
CP2i	North area of land raise	11.00	50mm standpipe GL-3m plain pipe 3m-9m slotted pipe
CP3	South area of land raise	12.00	50mm standpipe GL-1m plain pipe 1m-9.5m slotted pipe
CP4	South east area of land raise	7.00	50mm standpipe GL-1m plain pipe 1m-6m slotted pipe

The cable percussion borehole records are presented in Report WE00298/R1 Appendix C.

3.5 Rotary Open Hole Boreholes

Fourteen rotary open hole boreholes were drilled between the 17th of November and 7th of December 2009 to depths ranging from 1m bgl to 57m bgl using a Commachio 205 track mounted rotary drill with air-flush.

Ten of the boreholes were completed as monitoring wells (MW1-MW7 and (RC1S-RC3S) in order to enable soil gas and groundwater sampling to be undertaken.

Two of the boreholes were completed with inclinometer tubing (INC1 and INC2).

The remaining two holes (VWP1 and VWP2) had a string of five vibrating wire piezometers installed at specified depths throughout the hole.

The following table presents the borehole details:

Borehole	Location	Depth (m)	Instruments / Installation
MW1	Western end of north quarry face	7	50mm standpipe GL-5m plain pipe 5m-57m slotted pipe
MW2	Western end southern boundary	30	50mm standpipe GL-4m plain pipe 4m-30m slotted pipe
MW3	Southeast corner of site	30	50mm standpipe GL-4m plain pipe 4m-30m slotted pipe
MW4	Midway along southern boundary	30	50mm standpipe GL-3m plain pipe 3m-30m slotted pipe
MW5	Western boundary	45	50mm standpipe GL-6m plain pipe 6m-45m slotted pipe
MW6	Midway along northern boundary	57	50mm standpipe GL-6m plain pipe 6m-57m slotted pipe
MW7	Eastern boundary	45	50mm standpipe GL-6m plain pipe 6m-45m slotted pipe
INC1	Western end of north quarry face	40	40m of Inclinometer access tubing
INC2	Western end of north quarry face	40	40m of Inclinometer access tubing
VWP1	Western end of north quarry face	40	Vibrating wire Piezometers at 10m, 20m, 25m, 30m and 40m
VWP2	Northeast corner of Northeast field	40	Vibrating wire Piezometers at 10m, 20m, 25m, 30m and 40m
RC1A	Northwest of quarry base	1	50mm standpipe GL-0.5m plain pipe 0.5m-1m slotted pipe
RC2A	Northeast of quarry base	1	50mm standpipe GL-0.5m plain pipe 0.5m-1m slotted pipe
RC3A	South of quarry base	1	50mm standpipe GL-0.5m plain pipe 0.5m-1m slotted pipe

The rotary open hole boreholes records are presented in Report WE00298/R1 Appendix D.

The inclinometer graphs are presented in Report WE00298/R1 Appendix E.

The vibrating wire piezometer results are presented in Report WE00298/R1 Appendix F.

3.6 Rotary Core Boreholes

Four rotary cored boreholes (RC1D to RC3D and HRC1) were drilled between the 17th of November and 7th of December 2009 to depths ranging from 5m bgl to 15m bgl. The single 15m borehole was a horizontal hole whereas the remaining three holes were 5m deep vertical cores.

Cores were drilled using a T6-116 thin wall triple tube barrel to produce cores of 92mm diameter. Cores were retained in plastic core-liner and extruded into wooden core boxes for subsequent logging and photography.

All three of the vertical boreholes were completed as monitoring wells in order to enable soil gas and groundwater sampling to be undertaken. The following table presents the borehole details:

Borehole	Location	Depth (m)	Installation
RC1	North west of quarry base	5	50mm standpipe GL-4.5m plain pipe 4.5m-5m slotted pipe
RC2	North east of quarry base	5	50mm standpipe GL-4.5m plain pipe 4.5m-5m slotted pipe
RC3	South of quarry base	5	50mm standpipe GL-4.5m plain pipe 4.5m-5m slotted pipe

The rotary core borehole records and core photos are presented in Report WE00298/R1 Appendix G.

3.7 Sampling

Sample Collection and Analysis

Soil Samples
All soil samples were collected using either clean stainless steel utensils or clean disposable gloves and placed directly into clean containers provided by the laboratory.

Water Samples
Groundwater samples were collected using either dedicated disposable bailers or a Grundfos MP1 electric submersible pump with clean dedicated HDPE tubing. The samples were placed directly into clean containers provided by the laboratory.

Groundwater and surface water samples were placed in the following containers following appropriate purging of the wells to provide a representative sample:

- 1000ml green glass jar for organics including PAHs, EPHs and inorganics.
- 40ml glass vial for VOCs, GRO (must be filled to the top).

Samples were stored in cool boxes with cool packs for onward transmission to the laboratory.

Groundwater and Soil Gas Monitoring
Groundwater depths were recorded in each monitoring well using an electronic dip meter. The depth was recorded from the top of the monitoring well cover.

Soil gas monitoring was undertaken using an ATEX Phocheck 3000+ Photo-ionisation detector (PID) to measure Volatile Organic Compounds (VOCs) and an ATEX approved GA2000 landfill gas analyser with flow pod to measure the following:

- Methane
- Carbon Dioxide
- Oxygen
- Atmospheric Pressure

The monitoring of each position was undertaken for a minimum of 90 seconds (unless water was detected in the equipment) with readings recorded every 30 seconds. This continued until readings stabilised.

Groundwater levels and soil gas levels were measured on the 3rd of December 2009, 15th of December 2009 and 29th of January 2010. On the 21st of January 2010 just groundwater levels were measured. The Groundwater and Soil Gas monitoring results are presented Report WE00298/R1 Appendix H.

3.8 Chemical Testing

Soils

Samples obtained during the investigation were subjected to geochemical testing at Scientific Analysis Laboratories Ltd to determine:

- 18 No. Metals suite comprising arsenic, boron (water soluble), barium, beryllium, cadmium, copper, chromium, lead, mercury, nickel, selenium, vanadium, zinc.
- 18 No. Speciated Poly Aromatic Hydrocarbons (PAH)
- 18 No. Total Petroleum Hydrocarbon Criteria Working Group (TPH CWG)
- 18 No. Total Sulphate, Sulphide and Sulphur
- 18 No. Free Cyanide, Thiocyanate and pH
- 18 No. Phenols (Mono)

The analyses were undertaken on the following samples:

Trial Pit	Sample Depth (m)	Metals	PAH	TPH	Phenols	Total Sulphate, Sulphur Sulphide	Free Cyanide, Thiocyanate and pH
TP1	2.5	X	X	X	X	X	X
TP2	0.9	X	X	X	X	X	X
TP3	0.3	X	X	X	X	X	X
TP4	4.0	X	X	X	X	X	X
TP5	2.1	X	X	X	X	X	X
TP6	0.6	X	X	X	X	X	X
TP7	0.2	X	X	X	X	X	X
TP9	1.0	X	X	X	X	X	X
TP11	0.5	X	X	X	X	X	X
TP13	0.5	X	X	X	X	X	X
TP14	2.0	X	X	X	X	X	X
TP15	0.6	X	X	X	X	X	X
TP17	0.9	X	X	X	X	X	X
TP18	1.0	X	X	X	X	X	X
TP19	3.5	X	X	X	X	X	X
TP20	1.0	X	X	X	X	X	X
TP21	1.0	X	X	X	X	X	X
TP22	2.8	X	X	X	X	X	X

Waters

Water samples taken from the monitoring wells were scheduled for the following:

- 2 No. Metals suite comprising arsenic, boron (water soluble), barium, beryllium, cadmium, copper, chromium, lead, mercury, nickel, selenium, vanadium, zinc.
- 2 No. Speciated Total Petroleum Hydrocarbons (TPH)
- 2 No. Semi-Volatile Organic Compounds
- 2 No. Total Petroleum Hydrocarbon Criteria Working Group (TPH CWG)
- 2 No. Ammonia Low
- 2 No Mercury Dissolved
- 12 No. Ammonia
- 12 No. pH and Electrical Conductivity (EC)
- 11 No. Chemical Oxygen Demand (COD)
- 12 No. Chloride
- 12 No. Total Dissolved Solids (TDS)

The analyses were undertaken on the following samples:

Borehole	Ammonium	Metals	SVOC	TPH	Ammonia Low	Chloride and COD	TDS, pH and EC
MW1	X					X	X
MW2	X					X	X
MW3	X					X	X
MW4	X					X	X
MW5	X					X	X
MW6	X					X	X
MW7	X					X	X
RC1	X					X	X
RC2	X					X	X
RC3	X					X	X
CP3	X	X	X	X	X	X	X
CP4	X	X	X	X	X	X	X

The chemical laboratory test results are presented in Report WE00298/R1 Appendix I.

3.9 Geotechnical Testing

Samples obtained during the investigation were sent to Geo Testing Laboratories Ltd for geotechnical tests.

The following testing was undertaken

- 34 No. Moisture Content.
- 7 No. Liquid and Plastic Limits
- 28 No. Particle Size Distribution
- 2 No. Remoulded California Bearing Ratio (CBR)
- 2 No. Permeability in a triaxial cell
- 7 No. Dry Density / Moisture Content Relationship
- 3 No. Slake Durability Tests.
- 10 No BRE SD1 suite
- 9 No Hoek rock core shear box tests.

The testing was undertaken at the following locations:

Exploratory Hole	Sample Depth (m)	Moisture Content	Liquid and Plastic limits	Particle Size Distribution	California Bearing Ratio	Perneability in triaxial cell	Dry Density / Moisture Relationship	Slake Durability Test	BRE SD1	Hoek Shear Box
TP1	0.8	X		X			X			
TP1	2.5	X		X						
TP2	0.8	X		X	X				X	
TP3	0.8	X		X						
TP4	0.8	X					X			
TP4	3.5	X							X	
TP8	0.2	X		X			X	X		
TP9	0.9	X		X				X		
TP9	2.0	X		X					X	
TP10	0.8	X		X				X		
TP11	0.7	X			X					
TP12	1.0	X		X		X			X	
TP14	0.8	X	X	X		X	X			
TP14	1.8	X		X			X		X	
TP18	1.8	X		X					X	
TP18	2.0	X								
TP19	2.5	X		X						
TP20	3.0	X		X						
TP21	2.0	X		X						
TP22	1.8	X		X						
CP1	1.7	X		X					X	
CP1	3.7	X		X			X			
CP1	5.7	X	X							
CP1	6.8	X		X						
CP1	9.7	X	X	X						
CP2	0.2	X		X						
CP2	4.7	X		X						
CP2i	0.2	X	X	X						
CP2i	2.7	X	X	X						
CP2i	4.7								X	
CP2i	6.8			X						
CP2i	9.5	X								
CP3	0.5	X								
CP3	1.7	X		X			X			
CP3	5.7	X							X	
CP3	6.8	X	X							
CP3	8.0	X		X						
RC1	1.24									X
RC2	2.56									X
RC2	3.40									X
RC3	2.38									X
RC4	1.88									X
RC4	3.30									X
RC4	6.00									X
RC4	11.78									X
RC4	12.60									X

The geotechnical laboratory test results are presented in Report WE00298/R1 Appendix J.

3.10 Rising Head Permeability Testing

Rising head permeability tests were undertaken in RC1 and RC2 on the 29th of January 2010. A test was not undertaken in RC3 as the water level in this hole was too low.

The tests involved pumping the water from the borehole and reducing the water level. The reduced water level was then recorded and the recovery in water level recorded at timed intervals as the water level rises. This data has been used to calculate an infiltration factor for each borehole location.

The following table presents the calculated infiltration factors:

Borehole	Location	Infiltration Factor
RC1	North west of quarry base	6.84 E-07 $m.s^{-1}$
RC2	North east of quarry base	7.77 E-09 $m.s^{-1}$

The data and calculations are presented in Report WE00298/R1 Appendix K.

4.0 GEOTECHNICAL RISK ASSESSMENT

4.1 Introduction

The geotechnical risk assessment methodology is based on DETR Document *'Managing Geotechnical Risk'*. This provides the following definition: *"Geotechnical Risk is the risk to building and construction work created by the site ground conditions. Ground related problems can adversely affect project cost, completion times, profitability, health and safety, quality and fitness for purpose, and can lead to environmental damage".* Risk can be taken to mean: *the chance or possibility of danger, loss, injury or other adverse consequences.* It is therefore normal to identify particular hazards, consequences and identify the likelihood and effect to determine the degree of risk. A qualitative score is given to the likelihood and effect to provide an indication of the degree of risk. This provides the basis for the initial geotechnical risk screening process. Risk Scores of greater than 10 indicate a potential master planning development constraint, requiring further consideration. The following table provides a basis for assessing the degree of geotechnical risk.

Likelihood (L)	Description	Probability	Effect (E)	Description	Increase in cost and time
5	Almost certain	>70%			
4	Probable	50-70%	4	Very high	>10%
3	Likely	30-50%	3	High	4-10%
2	Unlikely	10-30%	2	Low	1-4%
1	Negligible	>10%	1	Very low	<1%
Risk (R)	**Risk Level**	**Action**			
1-5	Trivial	None			
6-10	Moderate	Undertake appropriate mitigation measures to reduce the risk level by appropriate on-site practice at little additional cost.			
>10	Significant	Designers should take such risks into account and avoid or reduce risk level to acceptable levels. Additional resources required.			

The degree of risk is used to inform decision making in terms of focusing resources towards significant risks. The geotechnical risk assessment will cover three distinct conditions:

- Existing hazards i.e. pre-development
- Construction related hazards
- Post construction hazards

The main generic hazards requiring attention associated with the site are as follows:

- Slope stability
- Settlement resulting from vertical compression
- Groundwater
- Ground gas
- Potentially contaminated materials

4.2 Existing Conditions

The following table presents the geotechnical risk assessment for the existing site:

Hazard	Consequence	L	E	R	Mitigation	Residual risk
Existing quarry side slopes - Strain in existing slopes mobilising drained constant volume effective stress strength parameters.	Initially stable slopes become unstable as ground strains in order to mobilise drained shear strength and equalisation of porewater pressures. Resulting in failure of oversteep slopes.	3	2	6	Re-grade slope to safe angle. Reduce groundwater levels. Reinforce slope by mechanical means.	Greater land take. Long-term pumping and maintenance. High cost may be prohibitive.
Settlement of existing fill materials.	Self weight compression of fill materials resulting in large settlements.	3	1	3	None	None
Rising groundwater	Flooding at the base of the quarry.	3	2	6	Pumping of water from the quarry base.	Long-term maintenance.
Ground gases	Decomposition of organic matter within the quarry infill material generating carbon dioxide and methane.	3	2	6	Monitoring of ground gas conditions to determine Gas Screening Value. Removal of primary source.	Reduction in land value if levels warrant special precautions.
Potentially contaminated land.	Materials deemed to pose a risk to human health or Controlled Waters and associated eco-systems that may blight the land.	3	2	6	Investigation and assessment of source-pathway-receptor scenarios associated with the infill material and remediation where necessary.	Remediation to protect existing conditions may not be suitable for future development options.
Expansive fill material.	Re-use of such material may have long-term consequence to foundations.	2	3	6	Assess the fill material characteristics to determine potential for expansive materials.	Fill material identified as expansive may require pre-treatment or disposal.

The above table provides a baseline of existing conditions at the site.

4.3 Proposed Development

The following table presents the geotechnical risk assessment for the construction phase:

Hazard	Consequence	L	E	R	Mitigation	Residual risk
Existing quarry side slopes - Strain in existing slopes mobilising drained constant volume effective stress strength parameters.	Initially stable slopes become unstable as ground strains to mobilise drained shear strength and equalisation of porewater pressures. Resultant failure of oversteep slopes.	3	4	12	Re-grade slope to safe angle. Reduce groundwater levels. Reinforce slope by mechanical means.	Smaller development platform. Long-term pumping and maintenance. High cost may be prohibitive.
Settlement of existing fill materials.	Surcharge compression of fill materials resulting in large settlements of development platform. Potential settlement of structures.	3	4	12	Appropriate selection of engineering fill placed and compacted in accordance with method specification. Strict quality control. Pre-loading surcharge of development platform.	Long-term self weight settlement of development platform requiring a limitation to be placed in terms of bearing capacity for foundations. Obstructions may preclude certain methods of piling.
Differential settlement across quarry boundaries	Knife edge effect for example on west boundary of pond resulting is large rotational displacement and differential settlement.	3	4	12	Break out and remove 'knife edge' effects, for example excavate out west boundary wall of pond and create a wedge profile.	Vertical settlements have same differential, however, rotational displacement reduced.
Rising groundwater	Saturation of fill material resulting in inundation settlement.	3	4	12	Place fill material to appropriate specification that is wet of optimum.	Long-term settlement of development platform.
Ground gases	Decomposition of organic matter within the quarry infill material generating carbon dioxide and methane.	3	4	12	Monitoring of ground gas conditions to determine Gas Screening Value. Removal of primary source.	Reduction in land value if levels warrant special precautions for development.
Radon	Naturally occurring radon may migrate into dwellings resulting in inhalation of alpha particles.	2	3	6	BGS Geological Assessment confirms the site is not in an area requiring radon protection.	No radon protection measures required.
Potentially contaminated land.	Materials deemed to pose a risk to human health or controlled waters and associated eco-systems that may blight the land.	3	3	9	Investigation and assessment of source-pathway-receptor scenarios associated with the in-fill material and remediation where necessary.	Remediation standard should be protective of end development usage.
Expansive fill material.	Re-use of such material may have long-term consequence to foundations.	2	4	8	Assess the fill material characteristics to determine potential for expansive materials.	Fill material identified as expansive may require pre-treatment or disposal.
Foundation construction.	Excessive long-term settlement.	3	4	12	Appropriate selection and compaction of engineering fill. Monitoring or settlement during and post construction. Pre-loading surcharge prior to foundation construction. Removal of obstructions to allow piling.	Developers may select low risk high cost foundation options when assessing development value of the site. Dwellings to be constructed in groups of 2-3 plots with movement joints. Terraces should be constructed in "short" runs and must include movement joints or, preferably physical breaks. Large buildings should not span across 'knife edges' or cross from natural to made ground.

The above table provides a base-line of likely development phase geotechnical hazards.

5.0 SLOPE STABILITY

5.1 Methodology

The assessment of slope stability requires the use of an appropriate methodology in order to appreciate scenario, model and parameter uncertainty and to understand the applicability of any model output.

Rock Slopes
The principal factors controlling rock slope stability are effective stress and discontinuity spacing and orientation. It is therefore necessary to take these into account when selecting 'characteristic' parameters for design.

Intact pieces of very weak rock generally provide unconfined compressive strengths (UCS) of 0.6-1.5MPa. If the rock mass was considered as a continuum with such high UCS values rock slope failures would be unlikely to occur. Rock is however, not a continuum, but contains numerous breaks in the rock mass known as discontinuities. These breaks in the mass form planes of weakness which tend to result in planar, wedge or toppling failure mechanisms if the interaction of slope and discontinuity orientations are unfavourable.

It is therefore necessary to identify all persistent discontinuities i.e. bedding planes, cleavage planes, foliation planes, joints, faults and fractures and their associated dip and dip-direction. These may be plotted on a stereographic projection and compared with the dip and angle of the proposed rock slope. Dip vectors will show the points at which the discontinuities daylight and hence identify the slope face orientations prone to a particular failure mechanism. Where planar slides or wedge failure blocks are identified further visual assessment and calculation of block size may be made and the factor of safety against sliding may be calculated and the required reinforcement to prevent sliding determined.

Circular failure conditions may occur when the material is very weak, as in a soil slope, or when the rock mass is very heavily jointed or broken, as in a waste rock dump.

Soil Slopes
The stability of soil slopes is based on particulate or soil mechanics principals, where the soil comprises solid particles, pore-air and porewater. The grain size of the particles dictates the behaviour due to the mineralogy and permeability.

The soil behaviour in the short-term will be dictated by the permeability of the soil. Coarse grained soils will respond instantly to a change in effective stress resulting in an immediate dissipation of excess porewater pressure i.e. the drained condition. Fine-grained low permeability soils will display a hydrodynamic time lag to changes in effective stress, which will result in an immediate undrained condition where part of the vertical stress will be taken by the porewater, which will alter with equalisation of porewater pressure through the partially drained to the fully drained condition as porewater pressures dissipate due to applied stress.

The assessment of fine-grained soil under short-term conditions may be based on undrained shear strength and total stress. Therefore groundwater pressures will not be applicable to this situation. This type of analyses is applicable to unsaturated zone conditions, where a temporary cut slope may be cut to a steep angle to allow construction of a retaining wall for example.

The assessment of long-term soil conditions requires selection of appropriate drained parameters. This situation applies to both a saturated and unsaturated slope, where pre-consolidation may have 'built in' suction, which takes an equally long-time period to equalise to the drained condition. This allows clay slopes to stand at oversteep angles for long-periods. The London Clay provides a well documented example of this phenomenon. This is a stiff clay which has a reported stand up time of approximately 70-80 years prior to equalisation of negative porewater pressures and failure. The drained condition is controlled by the internal angle of friction, the total stress and the porewater pressure, which provides the effective stress. The realistic drained cohesion (c') intercept for long-term conditions rarely exceeds 1-2kN/m^2 and for design purposes is typically assumed to be zero.

Analysis of layered rock slopes may warrant the inclusion of tensional characteristics resulting in long-term c' values that are higher than those typically employed for soil slope stability assessment.

5.2 Existing & Proposed Slopes

The existing quarry slopes provide excellent exposure of the rock mass. The orientation of the slopes provide a reasonable indication of the three dimensional nature of the rock mass.

Mapping of the existing exposures has been undertaken on the north side of the quarry at four scan line locations. The findings of this exercise may be simplified as follows:

- Shale with sandstone layers at a ratio of 9:1 to 3:1 encountered above 60mAOD on slope surface.
- Shale with sandstone layers at ratios of 7:3 to 3:2 encountered below 60mAOD on slope surface.
- Bedding dips towards the north, the folding of the strata has resulted in both steep and shallow beds dipping to the north. Northern limb is shallow and upright. Southern Limb is steep and inverted.
- A joint set (Joint Set 1) within the sandstone was observed on the north face, which is perpendicular to the bedding. This dips towards the south and creates a potential saw tooth failure surface.
- Folds exhibit a plunge of 15 degrees and trend of 080 degrees to the east.
- Joints are steep (75 degrees below horizontal) with a dip direction to both the east (Joint Set 2) and west (Joint Set 3).

Field mapping and measurement involved four scan lines on the accessible north side of the quarry. 465 discontinuity readings were obtained. These comprised identification of bedding or joints and the measurement of the dip direction and dip of the discontinuities.

- The quarry slopes on the north side typically dip towards the south (180 degrees) at average angles measured from the scan lines of; Scan Line 1 - 26 degrees; Scan Line 2-30 degrees; Scan Line 3 -30 degrees; Scan Line 4 -33 degrees.
- Scan Lines 1 and 3 indicate an average slope angle of 28 degrees.
- Scan line 2 shows an average angle of 30 degrees, however, the upper section of slope stands at 28 degrees, the mid section at 39 degrees and the lower section at 22 degrees.
- Scan line 4 consists of a slope standing at 36 degrees in the upper section and 30 degrees at the lower section.

The data has been plotted on a stereographic projection. The data has been presented in two formats; poles to the bedding and dip vectors.

- The pole plot shows a distinct cluster of bedding planes with a large scatter of joint planes.

- The dip vector plot is more useful in assessing slope stability. The dip vectors for the bedding confirm the beds dipping towards the north with dip angles ranging from 10-90 degrees. This reflects the folded limbs, where the inverted southern limb stands steeply at angles >45 degrees and the less steeply dipping beds of the northern limb which range from 10-45 degrees. The fold hinges obviously exhibit a bedding plane that rotates around the fold hinge and therefore the dip direction and dip reflects this locally. These have been observed to plunge 15 degrees and trend 080 degrees.
- Joint Set 1 can be observed within the sandstone layers and can be seen to be perpendicular to the bedding and parallel to the fold hinges. These joints typically display a dip direction of 180 degrees and a measurable dip of 11 to 86 degrees. This joint set creates a stepped weakness within the rock mass, where slopes that dip towards the south i.e. 180 degrees are prone to this potential sliding surface where it daylights out of the slope surface. This joint set is likely to have produced the slope profile on the north west side of the quarry, which stands at a shallow gradient compared to the other slopes, where the proportion of shale and sandstone is typically 9:1 to 3:1.
- Two intersecting joint sets that dip steeply to the east and west are also present. Centres of clusters include dip direction and dip angles of 280/80 and 100/75 degrees. A greater proportion of measurable joints were observed dipping towards the east (Joint Set 2), this is likely to reflect the slope cutting and ability to measure these particular joints.

The stereographic projection can also be used to assess which discontinuities daylight out of particular slope orientations. The existing slopes may be assessed to determine the validity of the model:

- North slope dipping towards the south 180 degrees at a slope angle of 28 degrees as represented by Scan Lines 1 and 3 has been cut at the limit of safety for planar slide within joint set 1 for shale to sandstone ratios of 9:1 to 3:1.
- North slope dipping towards the south 180 degrees at a slope angle of 33 degrees as represented by Scan Line 4 has been cut at the likely limit of safety for planar slide within joint set 1 for shale to sandstone ratios of 7:3 to 3:2.
- West slope dipping towards east 090 degrees represented by Scan Line 2 stands at an average angle of 30 degrees, however, the upper slope rests at 28 degrees mid slope at 39 degrees and lower slope at 22 degrees. Again the plot confirms the slope angle is likely to be controlled by Joint Set 2, which dips to the east.
- East slope dipping towards west 270 degrees is unlikely to be affected by the main discontinuity sets.
- South slope dipping towards north 350 degrees. Bedding planes provide a potential planar failure mechanism if the slope is pushed further south and intercepts less steep north dipping limbs of the folded beds.

The following generic observations may be made:

- Slopes consisting of shale with sandstone layer ratios of 7:3 to 3:2 will stand at steeper angles than slopes consisting of shale with sandstone layer ratios of 9:1 to 3:1.
- The north slope angles are therefore determined by the ratio of shale to sandstone layers. A large slip is evident on the north slope where the shale to sandstone ratio ranges from 9:1 to 3:1. Steeper sections of the north slope exhibit higher proportion of sandstone beds. The standing angle of these slopes appears to be controlled by Joint Set 1.
- The south slope cut angles will be dictated by the location of the steep overturned southern limbs of sandstone, which can be observed. Where the sandstone layers are absent in terms of the proposed cut slope the angle is likely to be dictated by the heavily fractured shale, which is likely to behave as a frictional soil.
- Existing slopes facing east are controlled by the joint set orientation and the reinforcement of the sandstone layers.
- The west facing slope is likely to be controlled by the ratio of shale to sandstone with less influence from discontinuities.

Specialised rock shear strength testing has been carried out using the Hoek shear box to provide specific data in respect of shear strength along and across discontinuities. Nine rock specimens were selected. Five of the specimens were tested parallel to the bedding planes and four were tested cutting across the bedding planes.

The rock shear strength is normally presented in terms of normal stress and shear stress, where the normal stress applied to the cell is plotted on the x-axis and the share stress plotted on the y-axis. The normal stress values are typically based on the likely normal stress operating within the slope, in addition values at half and twice overburden pressure are also measured. This provides a plot of three points, which form a curved line, however this is simplified in order to derive apparent cohesion and internal angle of friction.

The following observations may be made:

- Normal stress <115kN mean phi 37º, with minimum and maximum values of 30 º and 49 º and the mean c' 72kPa, with min. and max. values of 33kPa and 102kPa.
- Normal stress >115kN mean phi 31 º, with min. and max. values of 23 º and 45 º and the mean c' 90kPa, with min. and max. values of 49kPa and 128kPa.

The above results confirm the curved relationship between shear stress and normal stress, where the corresponding phi value decreases with increasing stress.

The relationship between the coefficient of friction (i.e. tan phi) value measured at <115kN and that measured at >115 is 1.25. This value is equivalent to the relationship between peak and ultimate strength parameters, where the factor is typically related to the stress strain properties of the material, which in this case ranges from 1.0 to 1.5.

The ultimate or constant volume phi values are appropriate for long-term slope stability design. The relationship between peak and ultimate strength parameters is 1.1-1.5. This shows good correlation and indicates the peak phi values at normal stress values above 115kPa are suitable values for assessment of relatively shallow slope failures i.e. <5-10m depth, or put another way the peak values measured at normal stress values of <115kPa require a suitable mobilisation factor for use in design.

The strength parameter results for failure planes parallel to the bedding indicate the following design phi values:

- Shale to sandstone ratio 9:1-3:1 mean Phi = 27 degrees.
- Shale to sandstone ratio 7:3 to 3:2 mean Phi = 27 degrees.
- Sandstone mean Phi = 27 degrees.

The strength parameter results for failure planes that cross cutting the main bedding orientation indicate the following design phi values:

- Shale to sandstone ratio 9:1-3:1 mean Phi = 32 degrees.
- Shale to sandstone ratio 7:3 to 3:2 mean Phi = 36 degrees.
- Sandstone mean Phi = 40 degrees.

The measured cohesion values are as follows:

- Normal stress <115kN mean, min. and max. values of 72, 33, 102kPa. Application of a typical partial factor of 1.5 to the appropriate characteristic value indicates a design value of 22kPa.

The cohesion intercept is a function of the assumed linear relationship. The cohesion intercept will therefore be higher at higher normal stresses with corresponding lower friction angles. In practice the curve tends towards zero at very low normal stress. It is therefore only appropriate to rely on values of cohesion >5kPa for cemented and intact rock masses. The strength and stability of such rock masses is controlled by the discontinuities within the rock mass.

Existing slope on west half of the north side of the quarry (shale to sandstone ratio 9:1 to 3:1)
The north slope in the west half of the site contains a number of tension cracks at the crest of the slope. This area was therefore specifically targeted to assess the porewater pressures and measure the magnitude of slope movement associated with the tension cracks. The current monitoring data from the vibrating wire piezometer VWP1 indicate the following:

VWP	Piezometer Level (mAOD)	Head Height (m)	Head Level (mAOD)
1	77.3	1.74	79.04
2	67.3	9.57	76.87
3	62.34	12.83	75.17
4	57.34	23.56	80.9
5	47.34	26.39	73.73
Mean			77.14

The above table indicates porewater pressures within VWP ranging from 73.7-80.9mAOD. These high porewater pressures will generate a significant reduction in the effective stress conditions for the slopes below the mean level. This will therefore reduce the long-term stable slope angle unless drainage measures are introduced.

Inclinometers were installed within the zone of tension crack labelled IC1 and IC2. IC2 indicates horizontal movement to the south (i.e. downslope) of some 10mm displacement at ground level to zero below a depth of 12.5m below ground level. Further monitoring is required, however, this suggests a shear zone at the crest of the slope 87mAOD down to a level of 74.96mAOD. The slope is on average 27 to 30 degrees below horizontal and slope failure is evident. The zone of shale to sandstone ratio of 9:1-3:1 in this area extends to a level of 60mAOD on the slope. The toe of the failure appears to be above this level, which again indicates the failure is within the shale material.

Back analysis of the existing failed slope, which by definition has a factor of safety of <1 has been used to determine representative effective stress strength parameters taking account the porewater pressures in the slope. The following table presents the results of the analysis:

Run	Analytical Model	Unit Weight	Slope angle	Water level	Phi'	c'	Factor of Safety
Units		kN/m^3	degrees	mAOD	degrees	kPa	Ratio
1	Mohr-Coulomb	23	26	77	29	0	0.86
1A	Anisotropic	23	26	77	29	0H/5V	0.968
2	Mohr-Coulomb	23	26	77	35	0	1.085
3	Mohr-Coulomb	23	26	77	32	0	0.969

The above table provides a reasonable range of effective stress strength parameters, where c' ranges from 0-5kPa and Phi' ranges from 29-32 degrees. The lower bound value is entirely consistent with the likely mobilised shear strength method. The peak phi value is likely to be 35 degrees. The mobilised shear strength parameter is likely to be tan35/1.25=29 degrees. This shows good correlation with the mean Phi values of 27 degrees recorded in the rock shear strength testing.

On-site measurements of sliding angles in the shale ranged from 22 to 33 degrees with an average value of 28 degrees on planar surfaces. Again, this shows good correlation with the mean Phi values of 27 degrees recorded in the rock shear strength testing. In reality planar surfaces are unlikely to be present, therefore higher frictional values are likely where the failure plane crosses the bedding.

For a dry slope the following must be satisfied, tanPhi'/tanβ >1 where phi' represents the effective friction angle and β represents the slope angle. Three slope angles will be considered 1:2 (26.56 degrees), 1:2.5 (21.8 degrees), 1:3 (18.4 degrees).

- Slope angle of 26 degrees will provide a factor of safety of 1.13.
- Slope angle of 22 degrees will provide a factor of safety of 1.37.
- Slope angle of 18 degrees will provide a factor of safety of 1.7.

This does not take account of the porewater pressure, therefore adequate drainage will be required to control pore pressure; the required factor of safety must be adequate to take account of inevitable variations in porewater pressures with time.

Preliminary assessment of redevelopment options for the site include a development platform that will be located at 72mAOD on the north sides of the quarry. The existing piezometric surface as measured in VWP1 on the north side of the quarry ranges from 74-81mAOD. Therefore porewater pressures will reduce the effective stress conditions in the slope. Assessment of the slope angles of 26 degrees and 22 degrees have been undertaken without and with drainage.

Preliminary assessment of the drainage requirement has been carried out. This assumes the porewater pressure in the slope face below the mean piezometric level of 77.1mAOD applies from the slope surface vertically downward. Trench drains placed at 10m horizontal spacing, 4m deep and 0.5m to 1m wide will reduce the elevation head of the porewater by 2.5m. The piezometric surface has therefore been reduced to this level for the analysis with drainage.

The following table presents the results proposed slope options analysis.

Run	Analytical Model	Slope Angle	Water level	Phi' (c'=0)	FoS
Option 2	Mohr-Coulomb	26	77	29	1.09
Option 2 drained	Mohr-Coulomb	26	77	29	1.13
Option 3	Mohr-Coulomb	22	77	29	1.17
Option 3 drained	Mohr-Coulomb	22	77	29	1.34
Option 3 drained	Mohr-Coulomb	22	85	29	1.18
Option 3 drained	Mohr-Coulomb	22	77	27	1.21
Option 3 drained	Mohr-Coulomb	22	85	27	1.04

The above table demonstrates that for a low bound Phi value of 29 degrees with a slope angle of 22 degrees (1:2.5), trench drains at 10m spacing, 4m deep and 0.5m to 1m width are required to achieve a Factor of Safety appropriate to a residential redevelopment. Sensitivity analysis demonstrates the importance of the trench drains.

However, the laboratory analysis of the rock specimens indicates a mean phi value of 32 degrees. The following table provides sensitivity analysis for slope angles of 26, 24 and 22 degrees using the laboratory derived 32 degree friction angle:

Slope Angle	Water conditions	Phi' (c'=0)	FoS
26 (1:2)	Dry	32	1.3
26 (1:2)	No drainage	32	1.14
26 (1:2)	Drainage	32	1.27
24 (1:2.25)	Dry	32	1.36
24 (1:2.25)	No drainage	32	1.2
24 (1:2.25)	Drainage	32	1.33
22 (1:2.5)	Dry	32	1.52
22 (1:2.5)	No drainage	32	1.3
22 (1:2.5)	Drainage	32	1.5

The assessment therefore indicates that the most appropriate form of assessment for cut slopes within the shale with a ratio of shale to sandstone from 9:1-3:1 is an isotropic frictional analysis using effective stress strength parameters.

The slope should be cut to an angle of 22 degrees (1v:2.5h). Discrete trench drains should be placed at observed locations of spring lines to control groundwater seepages on the slope.

Existing slope on the north east & west side of the quarry (shale to sandstone Ratio 7:3 to 3:2)
The observed slopes on the north east side of the quarry and west side of the quarry exhibit a higher proportion of sandstone in ratios of 7:3 to 3:2. For example, over a distance of 1m, 60-70cm of shale will be present with 40 to 30cm of sandstone. The sandstone layers are typically 3-10cm thick. These provide significant reinforcement in terms of the slope stability parameters resulting in failure mechanisms more akin to rock mass failure i.e. planar, wedge and toppling failure, depending on the orientation of the cut slope.

The zones of high fracture frequency will be more prone to frictional failure characteristics and represent the localised areas likely to fail in the long-term. The rock mass characteristics are likely to reflect that of angular gravel and cobble size fragments of sandstone and shale. The analysis of the slope on Scan Line 4 enables parameters to be assessed:

Run	Analytical Model	Slope Angle	Water level	Phi' (c'=0)	c'	FoS
9	Mohr-Coulomb	33	77	42	0	0.91
10	Mohr-Coulomb	33	77	36	20	1.03
10A	Mohr-Coulomb	33	77	36	45	1.29
11	Mohr-Coulomb	26	77	36	20	1.3

The purely frictional approach assuming a phi' of 42 degrees and c'=0kPa indicates the slope would fail. This does not correlate with the observed slope, which does not exhibit obvious signs of movement at angles of 33 degrees where the proportion of sandstone layers is higher.

It is therefore reasonable to include a tensional strength component as indicated by the cohesion intercept within the effective stress strength parameters. The sandstone layers are likely to provide this reinforcing element, which provides tensional resistance to movement. A mobilisation factor has been included in the determination of a suitable parameter. A typical peak phi value for the sandstone is 42 degrees. This provides a design phi' value of 36 degrees. This again provides good correlation with the value of mean Phi value of 36 degrees determined by the rock shear strength testing. Analysis with a 36 degree friction angle and a drained cohesion of 20kPa indicates a factor of safety of >1 for a slope of 1:1.5 i.e. 33 degrees.

Slopes cut to angles of 1:2 in this material are likely to provide an adequate factor of safety for a residential development based on effective stress strength parameters of phi'=36 degrees and c'=20kPa. These results are supported by the rock strength testing and are considered applicable to slopes on the west, north and east sides of the quarry, where shale to sandstone ratios range from 7:3 to 3:2.

South Slopes
The slopes on the south side of the quarry typically display the steep inverted southern limb of the folded beds which occur as a planar sheet of sandstone with sole and flute marks confirming the overturned sequence of the layers. Assessment of stability in these layers requires consideration of buckling due to root fracturing, which may result in planar slides of the more persistent steeply inclined sandstone layers.

Although less obvious in outcrop, it is reasonable to assume that the less steeply dipping northern limb will be encountered in rock cuttings in this zone. This is likely to result in bedding planes daylighting out of the slope, which could result in planar failure. This is demonstrated by the stereographic projection that shows bedding planes daylighting out of the north facing cut slopes.

The horizontal core hole (HC1) was drilled initially through the 0.15m thick steeply dipping (65 degrees) sandstone beds behind which weak extremely close to closely fractured mudstone was encountered. A second sandstone layer was encountered between 1.85-2.25m, this was dipping at 40 degrees. The majority of the bedding planes dip between angles of approximately 45-50 degrees. The percentage of shale to sandstone is typically <25% i.e. 3:1 ratio. A zone between 12m and 13m revealed a ratio of 3:2 i.e. 40% sandstone, however from 13m to 15m the ratio was again less than 3:1.

On the basis of the horizontal cored hole and on-site visual observation any cutting pushed south of the existing steep inverted southern limbs exposed on the south side of the quarry is unlikely to encounter a similar layer of sandstone within the site boundary. The stability of newly cut slopes further south of the existing is therefore likely to be dictated by the long-term frictional characteristics of the shale and the porewater pressures in this zone. MW4 indicates a porewater pressure head of 54.7mAOD. The toe of the existing slope in the area of MW4 is recorded at 52.6mAOD. This is in the area of the berm of bricks placed as toe weight at the base of the slope.

Slope stability analysis has been undertaken to evaluate effective stress strength parameters for the shale and sandstone. The following table presents the results of the analysis:

Run	Analytical Model	Slope Angle	Water Level	Failure/Medium	Phi'	c'	FoS
1	Mohr-Coulomb	50	54.7	Block/Shale	29	10	1.012
1A	Mohr-Coulomb	50	54.7	Block/Shale	29	10	1.015
1B	Mohr-Coulomb	50	54.7	Block/Sandstone	36	20	2.415
1C	Mohr-Coulomb	50	54.7	Rotational	29	11	1.0
1D	Mohr-Coulomb	50	54.7	Rotational with Brick Berm	29	11	1.24
2	Mohr-Coulomb	22	54.7	Block/Shale	29	0	1.6

The above table provides back analysis of the existing slope. The parameters required to provide a factor of safety of 1 are:

- Shale Phi'=29 to 32 c'=10kPa
- Sandstone Phi'=36, c'=20kPa

Failure in the existing slope is likely to propagate through the shale rather than the sandstone layer. Block failure along the discontinuities is the more likely failure mechanism and as such provides the likely effective stress strength parameters.

The waste brick berm at the base of the slope provides an increased factor of safety of 25%. It would be prudent to maintain this bund as a precaution to protect current site users from the risk of slope failure in this area.

The design slope on the south boundary should be cut at an angle of 1:2.5 although a substantial part of the southern boundary slope is formed by the RIGS face which currently stands at around 50 degrees and will need to be maintained at this angle. As noted in the table above the face is meta-stable at this angle and therefore site layouts will need to make provision for a rock fall area in proximity to the RIGS where a fenced of safety zone can be established.

The current measurements of groundwater suggest that drainage measures are not required. Further groundwater monitoring is suggested to confirm this view.

Summary
The discontinuities in the rock mass indicate the following:

- North side of quarry, where slope faces south Joint Set 1 provides the main discontinuity. This provides a potential failure plane where the shale to sandstone ratio is between 9:1 and 3:1 and resulting in a stepped profile where the ratio is 7:3 to 3:2.
- West side of quarry, where slope faces east Joint Set 2 provides a potential plane of weakness.
- East side of quarry, where the slope faces west Joint Set 3 provides a potential plane of weakness.
- South side of quarry where slopes face north, the steeply inclined bedding provides the potential plane of weakness.

The effective stress strength parameters for the rock cuttings are likely to be dictated by the ratio of shale to sandstone. A useful preliminary divide is a ratio of 3:1. Poor strength characteristics are associated with ratios of 9:1 to 3:1 and good strength characteristics are associated with ratios of 7:3 to 3:2.

The effective stress strength parameters for the shale to sandstone ratio of 9:1 to 3:1 are based on back analysis of the existing failure on the north west side of the quarry and rock discontinuity shear strength testing:

- Phi'= 32, c'=0.

Slope angles should be cut to approximately 22 degrees 1:2.5 and should include provision for localised discrete trench drains to intercept identified seepages of groundwater.

The effective stress strength parameters for the shale to sandstone ratio of 7:3 to 3:2 are based on back analysis and rock discontinuity shear strength testing:

- Phi'=36, c'=20kPa.

Slope angles should be cut to approximately 26 degrees 1:2. This will provide a factor of safety of >1.3. This will apply where good strength characteristics are observed.

Stereographic projections are included in Appendix A.

Slope Stability Analysis examples are presented in Appendix B.

6.0 RE-USE, RECYCLING, AND RECOVERY FOR EXISTING MATERIALS.

6.1 Introduction

The former quarry contains a number of stockpiles or areas of land that could be worked to provide fill material to create the development platform. These include:

- Land raise containing deposited quarry and brick waste in the south west corner of the site together with the stockpile adjacent the site access road.
- Clay stockpile in the central area of the quarry base.
- Field in north east corner of site.
- Western margin of the existing quarry.

The following table presents the trial pits used to target existing materials within the quarry and site area.

Exploratory Hole	Target	Depth	Made Ground	Weathered Crackington
TP1	Land raise	2.9	0-2.9	-
TP2	Land raise	1.9	0-1.9	-
TP3	Land raise	3.5	0-3.5	-
TP4	Land raise	4.2	0-4.2	-
TP5	Quarry base	2.4	0-2.4	-
TP6	Stockpile	1.4	0-1.4	-
TP7	Quarry base	1.8	0-1.3	1.3-1.8
TP8	Quarry base	0.3	0-0.1	0.1-0.3
TP9	Quarry base	1.0	0-0.1	0.1-1.0
TP10	North quarry face	0.6	-	0-0.6
TP11	General ground conditions	2.0	-	0-2.0
TP12	Clay stockpile	2.3	0-2.0	2.0-2.3
TP13	Clay stockpile	2.2	0-2.2	-
TP14	Field	2.0	-	0-2.0
TP15	Field	1.8	0	0-1.8
TP16	Not undertaken	-	-	-
TP17	Stockpile	2.9	0-2.9	-
TP18	Stockpile	2.5	0-2.5	-
TP19	Land raise	3.7	0-3.7	-
TP20	Land raise	3.2	0-3.2	-
TP21	Land raise	2.2	0-2.2	-
TP22	Land raise	2.6	0-2.6	-
CP1	Land raise	12.7	0-10.6	10.6-12.7
CP2	Land raise	5.5	0-2.7	2.7-5.5
CP2i	Land raise	11.0	0-9.3	9.3-11.0
CP3	Land raise	12	0-9.1	9.1-12.0
CP4	Land raise	7.0	0-4.8	4.8-7.0

6.2 Land Raise

The Land Raise area has been investigated using 12 exploratory hole locations. The trial pits typically terminated in the made ground as a result of collapsing and spalling sides of the excavations. The cable percussive boreholes proved made ground to depths ranging from 2.7 to 10.6m, with an average depth of 7.3m.

The upper surface of the made ground contained organics including roots and rootlets.

The made ground contained variable proportions of clay, silt and sand with gravel and cobble size fragments of brick and sandstone. Occasional waste materials including metal, tyres, rubber, conveyor belts, concrete, wire, wood and fabric was observed. Occasional boulder size fragments of sandstone were observed.

Monitoring wells were installed in Boreholes CP3 and CP4. These encountered groundwater in direct contact with the made ground at levels of 52.9mAOD and 53.12mAOD respectively.

Groundwater analysis of samples obtained from CP3 and CP4 indicate the presence of mobile petroleum hydrocarbons. The measured concentrations were 565ug/l and 2690ug/l respectively. The first tier screening value is 10ug/l. This suggests the presence of a source of petroleum in the area of CP4, which is likely to be weathered diesel. CP4 is in the area of the compound which is seen as the likely source of the hydrocarbons; as such the source is likely to be localised.

The soil gas monitoring indicates consistently high methane concentrations of CH4 at 24-29%. This indicates the degradation of organic matter by anaerobic bacteria within this zone, likely to be a result of the bio-degradation of the weathered diesel. This area would therefore pose a potential and real risk in terms of future development, resulting in blight and loss in land value.

Historic mapping confirms the first stage of excavation was in the area of the land raise. The data from the investigation indicates landfilling of the former pit has been undertaken in this zone.

This zone is unsuitable in its current condition for development due to the depth, thickness, variability and composition of the made ground. In addition the measured concentrations of petroleum hydrocarbons, likely to be weathered diesel, and measured concentrations of methane are likely to blight this area of the site.

It is recommended that the made ground in this area is excavated and segregated into stockpiles for re-use, recovery, treatment and disposal. The sooner this is undertaken the lower the likely cost constraints in terms of treatment of areas potentially contaminated with petroleum hydrocarbons.

6.3 Clay Stockpile in Central Area of Quarry Base

Trial Pits 12 and 13 were excavated within the clay stockpile. The stockpile is between 5-6m high and situated on the base of the quarry.

Trial Pit 12 was excavated through the side slope of the stockpile, which proved the expected quarry base at a level of 44.45mAOD.

Trial Pit 13 extended to a depth of 2.2m, which is a level of 51.35mAOD. The pit was terminated at this depth due to the side slopes collapsing.

Although unlikely, experience from other sites suggests the potential for made ground beneath the clay stockpile cannot be entirely ruled out. Such materials may be prone to settlement or a source of ground gas.

It is therefore recommended that this zone is excavated and segregated for re-use as an engineering fill. The base should be inspected to confirm the absence of buried made ground that may pose a longer-term risk of settlement, ground gas or blight.

6.4 Field in North East Corner of Site

Trial pits 14 and 15 were excavated in this area. Clay was encountered to depths of 1.5m and 0.7m respectively. This material is moisture susceptible therefore re-use will be subject to upper and lower limits of moisture contents. The material deemed suitable for re-use is likely to be categorised as Class 2C in accordance with the Specification for Highway Works.

The topsoil in this area should be assessed in terms of suitability for re-use for residential gardens.

6.5 Geotechnical Test Results

Samples obtained during the investigation were sent to Geo Testing Laboratories Ltd for geotechnical tests.

The following testing was undertaken

- 34 No. Moisture Content.
- 7 No. Liquid and Plastic Limits
- 28 No. Particle Size Distribution
- 2 No. Remoulded California Bearing Ratio (CBR)
- 2 No. Permeability in a triaxial cell
- 7 No. Dry Density / Moisture Content Relationship
- 3 No. Slake Durability Tests.
- 10 No BRE SD1 suite

The testing was undertaken at the following locations:

Exploratory Hole	Target	Sample Depth	Moisture Content	Plasticity Index	PSD C/G/S/Fines	CBR	Perm triaxial cell	Dry Density / Moisture Relationship	Slake Durability Test	BRE SD1-SO4
Units		m	w%	%	%passing	%	m/s	Mg/m3/w%	%	g/l
TP1	Land raise	0.8	7.6		0/43/45/12			1.88/8		
TP1	Land raise	2.5	10		0/65/18/17					
TP2	Land raise	0.8	9.4		26/28/22/24					0.02
TP3	Land raise	0.8	8.1		0/47/41/12	17.7				
TP4	Land raise	0.8	14					1.96/8		
TP4	Land raise	3.5	16							0.03
TP8	Quarry base	0.2	6.9		0/85/7/8			2.12/10	96.7	
TP9	Quarry base	0.9	12		0/67/23/10				94.1	
TP9	Quarry base	2.0	13		13/51/17/19					0.02
TP10	North quarry face	0.8	7.9		0/87/9/4				97.1	
TP11	General	0.7	21			17.1				
TP12	Clay stockpile	1.0	13		4/83/7/6		9.25E-11			
TP14	Field	0.8	30	50	0/10/11/79		6.19E-11	1.71/13		
TP14	Field	1.8	16		0/49/16/35			1.91/14.9		
TP18	Stockpile	1.8	14		0/77/12/11					<0.01
TP18	Stockpile	2.0	16		0/31/20/49					
TP19	Land raise	2.5	23		X					
TP20	Land raise	3.0	12		22/41/15/22					
TP21	Land raise	2.0	9.3		27/51/15/7					
TP22	Land raise	1.8	10		0/72/14/14					<0.01
CP1	Land raise	1.7	13		0/43/25/32					<0.01
CP1	Land raise	3.7			0/39/37/24			1.95/11		
CP1	Land raise	5.7	18	24						
CP1	Land raise	6.8	12		0/13/66/21					
CP1	Land raise	9.7	16	11	0/43/35/22					
CP2	Land raise	0.2	22		0/19/22/59					
CP2	Land raise	4.7			0/41/20/39					
CP2i	Land raise	0.2	19	19	0/35/28/37					
CP2i	Land raise	2.7	16	13	0/27/38/35					
CP2i	Land raise	4.7	16		0/41/20/39					<0.01
CP2i	Land raise	6.8			51/5/23/21					
CP2i	Land raise	9.5	16							
CP3	Land raise	0.5	16							
CP3	Land raise	1.7	13		0/33/43/24			2.06/8.5		
CP3	Land raise	5.7	16							0.2
CP3	Land raise	6.8	14	33						
CP3	Land raise	8.0	15		0/15/60/25					
CP4	Land raise	0.3	21	17	0/28/38/34					

The geotechnical laboratory test results are presented in Report WE00298/R1 Appendix J.

The geotechnical test results indicate the following:

Moisture contents are variable ranging from 7 to 30%. The mean value was 14.6% and the second highest moisture content was 23%. The clay samples range from 12 to 30% with an average of 17.6%. The gravel samples ranged from 7 to 22% with an average of 12.3%. The sand samples ranged from 7.6 to 21% with an average of 14.8%.

Plasticity Index results are variable ranging from 11 to 50%. The mean value is 24%. The two highest results were 33 and 50%. The value of 50% was obtained from TP14 in the natural strata obtained from the field in the north west corner of the site. This is potentially significant in terms of re-use potential of this deposit and indicates that the material would be better re-used in landscape areas.

29 grading analyses were undertaken only nine of which contained <15% fines; this is the cut-off below which the Specification for Highway Works designates material as "granular" and above which the material is regarded as "cohesive". A majority of the soil samples would be Class 2C, with a lower proportion of Class 1C. The uniformity coefficient is typically >10 for the Class 1C materials.

Two remoulded CBR tests were undertaken. The results of both tests indicate CBR values of 17%. The moisture content was 8% in the sandy gravel sample and the moisture content was 21% in the slightly sandy slightly gravelly clay.

Maximum dry densities range from 1.71 to 2.06Mg/m^3. Therefore 95% maximum dry density values range from 1.62 to 1.96Mg/m^3.

Optimum moisture contents range from 8 to 15%. The preliminary testing indicates an upper moisture limit of 16 to 22% for the clay samples to achieve 95% maximum dry density, which correlates with the CBR value of 17% for a moisture content of 21%. The upper limits for the gravel samples ranged from 13 to 22%. The upper limit for the sand was 10 to 11%. This indicates that some of the clay and sand deposits will require drying prior to placement to achieve 95% maximum dry density.

Oxidisable sulphides range from 0.13 to 1.87% with a mean value of 0.75%. This indicates the potential for pyrite rich shale. The Total Potential Sulphate (TPS) values range from 0.15 to 2.22% with a mean value of 0.87% and the upper 20% value of 1.77%. This indicates a design sulphate class of DS4-Ac4.

7.0 GEOTECHNICS OF PROPOSED DEVELOPMENT PLATFORM.

7.1 Introduction

The most probable development platform will require the ground levels within the quarry to be raised to a level of approximately 72mAOD on the north side. The ground levels would typically drop towards the south boundary. A central ridge trending from north to south forms a watershed, where the ground surface drops down towards surface water attenuation ponds in the south west and south east corners. The pond level in the south west corner is shown as 51mAOD and the pond level in the south east corner is 56mAOD. A copy of the plan is presented in Figure 4.

The base level of the quarry varies. The central area ranges from 45-48mAOD. The east area drops from 45-37mAOD in the area of the lower pond. The levels in the south west corner vary, where quarry arisings have been placed at the base of the quarry resulting in stockpiles and land raises.

In order to create the development platform infilling of the quarry will be required resulting in 16m of fill in the central zone and 23m to 25m in the south east corner. Ground levels will be lowered in the south east corner of the site.

The placement of thick layers of fill presents potential hazards requiring mitigation prior to eventual construction. The following issues require consideration:

- Self weight settlement, including immediate, primary consolidation and long-term creep settlement.
- Collapse settlement resulting from initial inundation of the fill.
- Applied stresses due to foundation loading.
- Negative skin friction on pile foundations.
- Differential settlement of fill at site boundaries and over 'knife edges'.

The development platform needs to be formed in a manner that will allow construction of buildings, associated roads and drainage. The following methods of construction and ground improvement are likely to be applicable to this particular development:

- Placement of Class 1 engineering fill in the upper 6m in areas of proposed roads and buildings.
- Pre-loading surcharge of main road corridors.
- Pre-inundation of the fill to prevent collapse settlement.

The filling needs to be carried out in such a way that future development can be formed on shallow spread foundations or driven pile foundations. In addition, the filling needs to be carried in such a way that material excavated from foundation and drainage trenches is chemically and physically suitable for re-use or disposal at the lower rate of landfill tax.

7.2 Preparation of Quarry Base

The potential for 'knife edge' effects should be considered prior to backfilling the quarry. It may therefore be necessary to reduce the angle of both the lower parts of the side slopes and any internal slopes within the present quarry void to minimise the deflection ratio generated by the settlement of the fill after filling. This is essential to minimise differential settlements and tilt ratios, which would be detrimental to any future development on the site.

Consideration also needs to be given to the potential for down-drag effects between the existing quarry slopes and backfill.

7.3 Self Weight and Collapse Settlement

The placement of the fill material will result in high initial total stresses applied to the fill materials. At a depth of 6m this is likely to be in the order of 120kPa due to the self weight of the overlying material. Above this level the self weight stress reduces significantly. This zone is therefore likely to settle significantly under structural loads, unless it is adequately compacted and improved.

The immediate settlement of the fill under self weight is of little relevance; however the primary consolidation within the clay layers, collapse settlement on first inundation and the long-term creep are of significance.

It is therefore necessary to estimate both the magnitude of compression and the time scales for settlement. The magnitude of settlement is primarily controlled by the moisture content and porosity resulting from the compaction during placement of the fill material. The relative density may be used to describe the compactness of coarse fills, which is related to porosity. The moisture content dictates the degree of compaction that may be achieved for a fine grained soil.

It is therefore essential to place and compact the engineered fill in a controlled manner in order to provide a reliable predictions of future performance.

The constrained modulus is the ratio of vertical stress to vertical strain produced under drained conditions. This value is the inverse of the coefficient of volume compressibility. This is a useful parameter to for assessing the lower and upper bound magnitudes of the primary settlement.

The self weight of the Fill will result in primary settlement prior to application of any construction loads. The hydraulic conductivity of the fill will dictate the rate of settlement. The coarse grained material will exhibit high hydraulic conductivity and hence primary settlement will be immediate. The fine-grained materials will exhibit a hydrodynamic lag in response to applied stress, where the undrained response will result in the stress being distributed between the soil particles and the porewater. This will result in an excess porewater pressure and reduction in effective stress on the soils particles. The dissipation of the excess porewater pressure will result in consolidation settlement.

The following table presents likely magnitudes of settlement and time-scales as a function of layer thickness.

Fill thickness	Layer thickness	Coarse grained	Fine grained	Total Settlement	Time	Residual settlement
m	m	D=Mpa k=m/s	D=MPa k=m/s	mm	years	mm
20m	1	D=6 k=1E-07	D=3 k=1E-11	1030	2.59	50
20	0.5	D-6, k=1E-07	D=3, k=1E-11	1030	0.65	50

The above table demonstrates the importance of controlling Fill layer thickness. Layers should be limited to 500mm and alternate fine-grained and coarse-grained fill layers placed to reduce drainage path-lengths and hence minimise the time required for self weight consolidation settlement.

The long-term creep settlement follows an approximately linear relationship based on the long-term settlement versus the logarithm of time. The following table provides an indication of the magnitude of the long-term creep settlement based on published values for compacted and non-compacted fill material:

Fill	Compaction	Alpha %	Creep Settlement Fill Height 20m
Sandy gravel fill	Heavy vibrating roller	0.04sigma'v	0.0035m
Mudstone fill	Heavy vibrating roller	0.12sigma'v	0.01m
Sandstone/mudstone rockfill	Heavy vibrating roller	0.13sima'v	0.011m
Sandstone/mudstone rockfill	No systematic compaction	0.9-1.5	0.0786m-0.13m
Stiff clay	Heavy dynamic compaction	0.5	0.043m

The above table demonstrates the importance of appropriately controlled and compacted placement of engineering fill to minimise long-term creep settlements.

Collapse settlements to due groundwater level rise have previously been reported to range from 1-5% of fill thickness. Therefore in a 20m thick layer of fill vertical settlements could range from 0.2m to 1m. The lower bound magnitude of 200mm is significant, the upper bound is catastrophic. This again indicates the requirement for appropriately controlled and compacted placement of engineering fill to minimise potential collapse settlement.

In order to mitigate these potential hazards he following options require further consideration:

- Acceptability testing of engineered fill to determine lower and upper limits of moisture content based on optimum moisture content.
- Controlled placement and compaction of engineered fill placed in specified layers of defined thickness and number of passes based on mass of vibratory roller.
- Placement of monitoring instruments including vibrating wire piezometers and magnetic extensometers to measure self weight settlement during construction and control rate of filling based on measured excess porewater pressures induced in the fill and the time for these to dissipate to acceptable levels.
- Pre-inundation of the fill material should be considered to build in the settlement prior to placement of roads and structures.
- Pre-loading surcharge may be considered for main road corridors.
- Dynamic compaction and/or vibro-compaction techniques may be considered for specific development proposals.

The potential for ground related heave should also be considered. Pyritic shale has the potential to oxidise sulphides to form sulphates, which may result in heave and corrosion of buried concrete. The analysis indicates the presence of oxidisable sulphides within the Crackington Formation. The Total Potential Sulphate values indicate the requirement for buried concrete to be mixed in accordance with BRE Special Digest 1 Design Sulphate Class DS4-Ac4.

7.4 Earthworks

The potential for long-term creep settlement and collapse settlement following the initial inundation of the fill requires special consideration in order to provide a valuable working platform. The use of an appropriately specified and compacted engineering fill will mitigate many of these long-term settlement risks.

The Specification for Highways Works (SHW) provides a basis for classification of acceptable earthworks materials and methods of compaction.

Acceptability testing is required to ensure the engineering fill material does not suffer significant volume changes following placement. This is best achieved by determining upper and lower limits of acceptability from the optimum moisture content for granular fill. It should also be recognised that initially granular fill may be broken down during the compaction process to produce a fine-grained material. It is likely that the shale rock will be rapidly broken down to produce a fine grained fill material. The acceptability of cohesive fill (fine-grained) may be based on the following:

- Maximum moisture content.
- Moisture content related to Plasticity Index.
- Undrained shear strength.
- Moisture Condition Value.

The majority of the fill material is likely to fall into Class 1 or Class 2 (SHW) general fill, which will be divided by percentage fines content. This material may be used at depths greater than 6m below the final platform level.

- The uniformity coefficient of the Class 1 general granular fill determines the method of compaction. The uniformity coefficient is typically >5. The size of the coarse particles is likely to require Class 1C material to be compacted in accordance with Method 5. This would require a minimum vibratory roller of mass per metre width of 2900kg to allow placement in 500mm thick layers with five passes.
- The Class 2 general cohesive fill will require classification by particle size distribution testing and plasticity index testing. The material at the site is likely to be Class 2C thus requiring compaction in accordance with SHW Method 2. Using a vibratory roller of mass per metre width of 2900kg this would allow placement in 200mm thick layers with four passes.

It is suggested that layers of Class 2C are sandwiched between Layers of 1C. This will provide a short drainage path for dissipation of excess porewater pressures induced in the 2C material as a result of compaction and subsequent placement of layers above.

The upper 6m of the site should ideally be filled using Class 1 granular fill. The placement of fill below water within the existing ponds will require the use of Class 6A hydraulic fill. This grading requirement should be modified to define a minimum particle size as fine sand may take a long time to compact under self weight. Dewatering of ponds before filling will remove the need for the use of 6A material.

If shallow strip foundations are to be considered for future development the placement of 6N material beneath dwellings would be required. However, Section 7.5 following indicates that strip foundations are unlikely to be viable. If such filling is carried out the material should be placed and compacted to an end product specification based of 95% maximum dry density determined from vibrating hammer compaction.

The performance of the engineering fill will dictate the value of the development platform. It is therefore prudent to undertake pre-loading surcharge and monitoring to demonstrate that vertical settlements induced by applied stress of future construction are within normally tolerable limits.

7.5 Foundation Options

The purpose of the earthworks re-profiling is to create a stable platform for residential development. It is therefore essential to provide conditions suitable for developers to plan and design their specific schemes and allow regulators and assurance providers to accept these schemes with minimal uncertainty in order to maximise the value of the land.

There is likely to be a compromise between the cost of the compaction and stabilisation measures required to minimise actual risk compared to the degree of perceived uncertainty that future developers may have regarding the stability and hence value of the land. There is little point in providing a good working platform if it is not possible to prove to prospective purchasers, regulators and assurance providers that future significant risks have been mitigated.

It is therefore possible to identify two ends of the possible spectrum:

Worst-case scenario:
Fill material end-tipped without control of layer thickness and compaction. All future development on the site will be subject to large potential settlements. Therefore foundations will be formed by end bearing piles attracting high negative skin friction loads; piles must penetrate the underlying rock mass. This would require 20m to 30m long piles. Fragments of sandstone within the fill are likely to create an obstruction to certain lower cost methods of piling. This is likely to result in a very low value development platform.

Best-case scenario:
The best-case scenario would include prior agreement of earthworks methodology with Highways Authority, Building Control and assurance providers, such as NHBC, prior to filling. The prior agreement would take into account PPG14-Development of unstable land and PPS23-Development on potentially contaminated land. Selected engineering fill would be placed and compacted within appropriate moisture content limits. The earthworks would be monitored and validated throughout the filling process. Pre-loading surcharge and pre-inundation settlement measures would be undertaken and recorded and allowable bearing capacity values would be provided for specific foundation types, with warranties provided to developers and assurance providers. In addition specific measures required for particular development options including dynamic compaction and/or vibro compaction could be identified.

This best-case approach is likely to maximise the value of the land, but may not remove inherent and perceived uncertainty of developers purchasing the land and regulators agreeing to the final construction.

It is therefore prudent to review the master plan and assess cost benefits associated with standards of earthworks specification. The safest approach would be to apply very high standards to the compaction and improvement process thus mitigating long-term settlement hazards associated with creep settlement and collapse settlement.

Highway construction across the site would benefit from pre-loading surcharge of the main road corridors and this is suggested to provide an appropriate foundation for roads. The engineering fill used in the upper 6m of the earthworks should be granular and compacted to a higher standard, to assist in the adoption of these roads.

Ground improvement would be required prior to foundation construction, due to the stress history of the upper 6m of the Fill material. Literature values of constrained modulus indicate a significant increase in the constrained modulus as a function of the pre-loading process. The use of wide pre-loading embankments improves the upper 6m of Fill and reduces the underlying self weight settlement issues. Constrained Modulus values may be increased by factors of 4 to 8. It is therefore recommended that a monitored pre-load surcharge is applied to specific areas prior to development in order to provide a platform for shallow spread foundations. This should include measurement of magnetic extensometers and vibrating wire piezometers.

The development is likely to comprise low rise dwellings. The following shallow foundation options may be considered:

- Raft foundations with timber frame construction (peak vertical stress 35kPa for a 20m by 20m raft)
- Box foundations (peak vertical stress of 75kPa)
- Shallow strip foundations for two storey dwellings (peak vertical stress 150kPa at 1m depth and 1m width)

Consideration of self-weight short-term total stress and long-term effective stress provides a basis for assessing the pre-consolidation history of the engineering fill placed and compacted in a controlled manner, with measures to ‘build in’ self-weight settlement and collapse compression. A plot of vertical stresses is presented in Appendix C.

Raft and Box Foundation
The raft would result in an applied stress exceeding the short-term total stress on the fill to a depth of 2m and exceeding the long-term effective stress of the fill material to a depth of 4m. The applied stress below 4m would be insignificant compared to the original in-situ total stresses. Therefore measurable settlements may be expected to depths of 4m. In order to ‘build in’ the settlement, pre-loading surcharge with monitoring instruments may be used as a simple expedient to allow construction. The height of the fill material would be reflected by the required allowable bearing capacity. An embankment height of 2m would be required to provide an adequate pre-loading surcharge.

The box foundation would require a surcharge embankment height of 4.5m to pre-load the ground and achieve an allow bearing capacity of 75kPa.

Strip Foundation
The strip foundation example above would result in an applied stress exceeding the short-term total stress of the fill to a depth of nearly 3m and exceeding the long-term effective stress of the fill material to a depth of 4m. No appreciable applied stress would occur below a depth of 6m. A preloading surcharge embankment of 9m height would be required to achieve the allowable bearing capacity of 150kPa; this would be required over the footprint of the dwelling. This is likely to preclude the use of shallow strip foundations, where localised load effects may result in an applied stress of 150kPa. In addition a 6m embankment would be required to provide an allowable bearing capacity of 100kPa following consolidation settlement. These surcharge heights are not believed to be practical and consequently the use of strip foundations may not be viable.

Driven (Displacement) Pile Foundations
The cost of a raft construction should therefore be compared with a driven (displacement) piling system with ground beams and beam and block floors and ventilated sub-floor void space. The following options for driven piles may be considered:

- Pre-cast driven piles
- Cast in-situ driven piles
- Vibro concrete columns
- Vibro stone columns

The use of such techniques may be hindered by the presence of overly large rock boulders in the compacted engineering fill. It would therefore be advisable to specify fill material within the upper 6m appropriate to the use of such a system, this is likely to include a maximum particle size constraint, possibly in the order of 150mm.

Non-displacement Piles
The use of non-displacement piles will not improve the ground surrounding the pile. These piles would therefore need to extend to depth to ensure the end-bearing capacity is sufficient to withstand both the vertical action of the structure and the negative skin friction induced by long-term settlement of the fill. This type of pile is likely to require a rock socket extending into the underlying bedrock. This may prove problematic for conventional CFA techniques, which may therefore require more expensive methods of pile construction such as rotary drilled ODEX piles with rock-sockets.

Summary
The 'middle ground' between the best and worst case scenarios will require a method compaction process that reduces long-term creep settlement, but requires ground improvement measures such as pre-loading surcharge and pre-inundation to 'build in' settlements to mitigate long-term differential settlements. The upper 6m of fill would still require ground improvement measures and foundations are likely to consist of driven pre-cast piles with ground beams and suspended floor slabs. These may be prefabricated to overcome the oxidisable sulphides issue and potential for ground heave. Raft foundations would be a technically acceptable option but experience on other similar sites suggests that the volume housebuilders do not favour raft foundations.

7.6 Buried Services

The excavation and placement of buried services will result in the production of arisings; these should ideally be re-used on the site.

The buried services are likely to be subject to differential settlements. These are likely to be significant for drainage pipes, water supply pipes and gas supply pipes.

The locations where differential settlements are likely to be of significance is in the area of 'knife edge' effects. For example the west side of the lower pond will provide a potential knife edge, where settlement to the east will be greater than the settlement to the west.

It would therefore be beneficial in determining the tolerable settlements for such infrastructure works; such settlements can then be mitigated by controlled earthworks with monitoring to confirm rates and magnitudes. A constraint to construction is likely to be the time for self-weight settlements to be reduced to tolerable limits.

8.0 GROUNDWATER AND GROUND GASES

8.1 Groundwater

The preliminary monitoring of the groundwater standpipes and vibrating wire piezometers provides reasonable correlation. The peak measured piezometric surface on the north boundary is at 81mAOD. The levels on the north boundary are approximately 55mAOD. It is therefore reasonable to assume that in the long-term groundwater levels will rise within the engineering fill to form a piezometric surface approximately indicated by these preliminary levels. Fluctuation and local variation should be anticipated. It is therefore inevitable that groundwater levels will rise within the fill and this has the potential to result in first stage inundation settlement, unless measures are undertaken to eliminate this risk.

The groundwater will therefore come into direct contact with the engineered fill material. The water will inevitably act as a solvent and liberate any water soluble substances. It is therefore essential that the fill material is assessed to ensure that the construction does not result in the pollution of Controlled Waters. This will be a requirement of PPS23 "Development on potentially contaminated land".

The initial groundwater quality analysis indicates high chloride concentrations in RC2 207mg/l and MW2 199mg/l. The published values of chloride for the Crackington Formation is 30mg/l. These results are therefore anomalous and require further consideration. In addition high Chemical Oxygen Demand (COD) readings were obtained in RC3 at 23700mg/l.

8.2 Ground Gases

The potential for ground gas generation requires consideration. The made ground in the south west corner of the site has been identified as a source of ground gas where locally elevated concentrations of methane have been detected.

Prior to construction the base of the quarry should be checked to ensure material with a potential to generate ground gases is identified and removed prior to placement of fill. An example of this may include the area beneath the clay stockpile in the central area of the quarry base.

Removal of such material will reduce uncertainty regarding localised long-term settlement and the potential for long-term gas generation. The presence of such gases may blight the land and reduce its perceived value due to the cost of installing appropriate measures and time required for agreement with assurance providers.

Provided that the filling process is subjected to rigorous quality control procedures it is reasonable to create a fill mass that should not have a gas generation potential. Lack of control during the selection and placement of fill material could result in a perceived gas risk with the resultant requirement for a ventilated sub-floor void and provision for a methane and carbon dioxide barrier within the ground floor slab with all penetrations sealed. This would indicate the requirement for a driven pile with suspended floor slab construction.

The site is not within an area requiring radon protection measures. A copy of the BGS report is presented in Appendix D.

9.0 CONCLUSIONS & RECOMMENDATIONS

9.1 Slopes

The existing slopes provide a working example of the potential long-term stable slope angles within the former quarry. The Crackington Formation observed within the quarry exposure may be sub-divided on the basis of ratio of shale to sandstone. Poor strength characteristics are associated with ratios of 9:1-3:1 and good strength characteristics are associated with ratios of 7:3 to 3:2.

Back analysis of the slope failure on the north west boundary of the quarry indicates effective stress strength parameters where the shale to sandstone ratio is 9:1 to 3:1 of c'=0kPa and phi'=29 degrees. Porewater pressures measured in VWP1 indicate a mean piezometric level of 77mAOD, with a peak of 81mAOD.

The steeper slopes are characterised by shale to sandstone ratios of 7:3 to 3:2. The sandstone bands provide reinforcement to the shale layers, this provides a more resistant rock mass that is less susceptible to slope failure, but more prone to differential weathering, where the 'softer' shale is eroded and the more resistant sandstone forms a stepped slope profile.

Back analysis of the slopes consisting of shale to sandstone ratios of 7:3 to 3:2 suggests long-term effective stress strength parameters of c'=20kPa and phi'=36 degrees. Effective stress testing is required to confirm these values.

The recommended slope on the north and north east boundaries will be dictated by the shale to sandstone ratio. The poor rock characteristics indicate slope of 1:2.5 with localised drains to intercept specific seepages. Observational assessment of the slope is therefore required.

Further investigation is recommended in advance of major excavations to adequately characterise the rock mass and hence the stability of the final slopes.

The slope on the south boundary should also be cut at an angle of 1:2.5, however provision for drainage measures is less onerous.

9.2 Earthworks

The likely site re-profiling will result in significant earthworks in-filling. The thickness could range from 16m to 25m of fill. The following hazards are anticipated:

- Self weight settlement consisting of immediate, primary (consolidation of fine-grained) and creep settlement.
- Collapse settlement due to first time inundation.
- Heave potential due to oxidation of sulphides in pyritic shale.
- Applied stress of roads and buildings resulting in additional vertical settlements.
- Percolating surface and groundwater will leach out soluble chemicals, which may result in aggressive ground conditions of pollution of Controlled Waters.
- Degradation of organic materials may result in the production of ground gases.

Strict control on earthworks acceptability is therefore essential to provide a low risk area of development land. Specification of the earthworks compaction and materials requires consideration of the likely foundation construction options.

Ground improvement measures should be considered to reduce the long-term settlement issues. Options include:

- Pre-loading surcharge of settlement sensitive areas.
- Pre-inundation of fill to induce controlled collapse settlement prior to development.
- Development specific dynamic compaction and vibro compaction.

It is recommended that large-scale oedometers are used to measure the coefficient of volume compressibility and hence determine the constrained modulus for the appropriate stress ranges applicable to the fill material as a function of self weight compression, pre-loading surcharge and post construction applied stresses.

9.3 Foundations

The placed and compacted engineering fill will initially be located in the unsaturated zone. The groundwater levels will however equalise with time. This will result in a lower effective stress condition with time. This will ultimately reduce the long-term strength characteristics. It is therefore essential that foundations are designed to be tolerant of predicted long-term settlements.

The likely self weight settlement of the Fill and applied stress settlement due to foundation loads will dictate the allowable bearing capacity of the ground in relation to the proposed foundation. The potential differential settlement will be critical in order to prevent unacceptable tilt and deflection. Therefore special consideration of the uneven Fill profiles and knife edges requires consideration.

The development platform is likely to be suitable for the following foundations options, subject to specific ground improvement methods:

- Raft foundation (subject to pre-loading surcharge)
- Box foundation (subject to pre-loading surcharge)
- Driven pile foundation (subject to suitability of fill for driven piles)

The following two options have been considered but ruled out due to practical challenges:

- Strip foundations (9m high embankment surcharge)
- Displacement piles end bearing in the underlying rock (obstructions)

Buried concrete mix specification will be subject to the quality of the engineering fill proximal to the foundations. The Total Potential Sulphate indicates a Design Sulphate Class DS4-Ac4. Therefore Fill material placed in the upper 6m should be carefully controlled to screen out pyritic shale if possible.

Floor slab design will be subject to the Gas Screening Value for the specific areas of the site. Provision for Characteristic Situation 2 should be made until such time that monitoring data confirms otherwise.

Road corridors should be pre-loaded with an embankment surcharge and monitored to confirm their long-term viability to adopting authorities.

Buried services will be located in the upper levels of the engineering fill. These will have been subjected to compaction stresses, but not subject to any significant pre-consolidation stress. These materials will therefore be subject to differential settlement. It is therefore necessary to identify mitigation measures for settlement intolerant services.

The 'middle ground' between the best and worst case scenarios may require a method compaction process applied to the earthworks that reduces long-term creep settlement, but requires ground improvement measures such as pre-loading surcharge and pre-inundation to 'build in' settlements to mitigate long-term differential settlements. The upper 6m of fill would still require ground improvement measures and foundations are likely to consist of either raft foundations, box foundations or driven pre-cast piles with ground beams and suspended floor slabs. The piles may be prefabricated to over-come the oxidisable sulphides issue and potential for ground heave or the upper 6m of Fill screened to reduce the potential of oxidisable sulphides to tolerable levels.

FIGURES

APPENDICES

A - Stereographic Projections
B - Slope Stability Analysis
C - Vertical Stress Plot
D - Radon Geological Assessment Report

A - Stereographic Projections

B - Slope Stability Analysis

C - Vertical Stress Plot

D - Radon Geological Assessment Report