APPENDIX I

SOILS

RPS

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1 INTRODUCTION

1.1 SCOPE AND OBJECTIVE OF REPORT

The purpose of this report is to present an assessment of the geotechnical findings of recent ground investigations undertaken in North County Dublin, in the townlands of Rowans Little, Rowans Big, Courtough, Nevitt, Hedgestown, Jordanstown, Ballystrane and Tooman. This report considers and assesses all of the available data with respect to the overburden conditions at the site and assesses its suitability for the siting of a proposed fully engineered Landfill site from a geotechnical perspective.

The report also considers the potential impacts / effects on soils during construction, operation and closure / aftercare phases. The report identifies remedial and reductive measures necessary to mitigate the potential impacts / effects.

1.2 DESCRIPTION OF PROPOSED DEVELOPMENT

The proposed development will comprise the construction of a new, fully engineered landfill facility in the townland of Tooman / Nevitt, north County Dublin. The entire site will cover an area of approximately 210 hectares. This includes approximately 153 hectares for landscaping, bunding, buildings, infrastructural elements and a landfill footprint area of approximately 57 hectares.

The landfill will be developed in discrete lined cells over a number of Phases. Infrastructural elements will include an administration building and associated facilities, leachate treatment plant, landfill gas utilisation area and a supervised public recycling facility for the public. The landfill will be capable of accepting up to 500,000 tonnes of non-hazardous waste annually up to a maximum of 9.5 million tonnes over the lifetime of the facility.

2 EXISTING INFORMATION

2.1 GEOLOGICAL MAPS

The Geological Survey of Ireland has produced maps detailing the bedrock underlying various regions in Ireland, and Sheet 13 'Geology of Meath' is the map that covers this area. Please refer to the Geology and Hydrogeology Report for further details (contained within Volume 5, Appendix H).

2.2 HYDROLOGY AND HYDROGEOLOGY

Separate reports have been carried out by the Water Services section of RPS in relation to the Hydrology and by the Environmental Section of RPS in relation to the Hydrogeology. These can be located in Volume 3, Appendix C and Volume 5, Appendix H respectively.

2.3 GROUND AND GEOPHYSICAL INVESTIGATIONS

2.3.1 Ground Investigation 2004

In 2004, Irish Geotechnical Services Ltd. carried out a ground investigation at four shortlisted sites in the North County Dublin Area and this was reported on in "Dublin Landfill Siting Scheme (Sites A - D) – Factual Ground Investigation Report (No. 9716)". This investigation was designed, procured and supervised by RPS Consulting Engineers.

The fieldwork comprised of cable percussive and rotary boreholes. In situ permeability tests and laboratory testing were carried out on the material encountered to aid classification. Standpipes were also installed to enable groundwater monitoring.

7 no. cable percussive boreholes and 8 no. rotary boreholes (incl. 3 no. Geobore S) were carried out at Site B, Nevitt / Tooman. Logs of these can be located in the Supporting Documents of this Appendix.

2.3.2 Geophysics 2004

BMA Geoservices undertook an initial geophysical survey to help identify a suitable landfill site and this was reported on in "Geophysical Survey on designated Sites A – D for Fingal Landfill Siting Study, Co. Dublin, April 2004".

A follow up survey "Extended Geophysical Survey on Designated Sites B & C for Fingal Landfill Siting Study, Co. Dublin" was carried out in July 2004.

Upon selection of Site B, and the completion of the Environmental Impact Assessment, the "Fingal Landfill Site B EIS, Geophysical Investigation" report was compiled and can be located in the Supporting Documents of this Appendix.

2.4 EXISTING DOCUMENTATION

In addition to the ground investigation and geophysical reports, the following sources of information were reviewed:

- Geological Survey of Ireland (GSI), 1999. Geology of Meath, Sheet 13. Scale 1:100,000 (1999);
- GSI and Fingal County Council 2005, Bog of the Ring Groundwater Source Protection Zones;
- Irish Geotechnical Services Ltd. (IGSL), 2004. Dublin Landfill Siting Scheme (Sites A - D) – Factual Ground Investigation Report (No. 9716) (Refer to Supporting Documents);
- Bernard Murphy and Associates, 2005 Fingal Landfill, Geophysical Investigation (An Interpretation of previous investigations and siting studies) (Refer to Supporting Documents);
- EPA, 2000, Landfill Manual on Landfill Site Design;
- Geological Survey of Ireland (GSI), 2003, GSI Guidelines for Assessment and Mapping of Groundwater Vulnerability to Contamination.
- William Lambe and Robert Whitman (MIT), 1979, Soil Mechanics, SI version.
- CIRIA report – Groundwater Control, Design and Practice, 2002

3 OVERVIEW OF THE RECENT INVESTIGATIVE WORK

3.1 GROUND INVESTIGATION 2005

Glover Site Investigations Ltd undertook a geotechnical ground investigation between May 2005 and August 2005. The geotechnical investigation consisted of the following:-

- 30 boreholes drilled by shell and auger methods;
- 29 boreholes drilled by air rotary methods;
- 26 boreholes drilled by Geobore S methods;
- Excavation of fifteen trial pits;
- In-situ testing including standard penetration tests, permeability tests and laboratory tests;
- Installation of groundwater monitoring network in seventy nine boreholes.

The investigation is detailed in Fingal Landfill Ground Investigation Factual Report (No. 05-271) Glover SI. (February 2006) (Refer to Supporting Documents).

A series of pumping tests, designed and supervised by RPS, was also carried out by Glovers Site Investigation in October 2005 and is reported separately.

3.1.1 Cable Percussion Boreholes

Cable percussive techniques were employed to examine the superficial deposits within and adjacent to the proposed landfill site. Thirty boreholes were drilled to depths ranging between 6.8mbgl and 21.2mbgl. Drilling was typically inhibited at relatively shallow depths by stiff to very stiff ground conditions or boulders.

The in-situ strength of the deposits was tested by means of a Standard Penetration Tests (SPT). Disturbed and undisturbed samples were obtained to aid classification and enable laboratory testing to determine the geotechnical properties of the material encountered.

In situ permeability tests (falling and rising head) were also carried out in a number of boreholes.

3.1.2 Rotary Boreholes

Rotary Boreholes were drilled to determine the depth to bedrock and the nature of the bedrock lithology. Twenty-nine rotary boreholes were drilled using symmetrex (open hole) techniques. The depth of the rotary boreholes ranged from 14m in HR07 to 59.5m in SHR01.

A further twenty-six rotary boreholes were drilled from surface using Geobore S with polymer mud flush in order to recover continuous core samples through the overburden.

3.1.3 Trial Pits

In order to obtain further information about an area known to contain made ground, a JCB was used to excavate 15 trial pits to depths ranging from 0.9mbgl to 3.5mbgl. Samples of the strata encountered were taken during the trial pitting.

3.1.4 Groundwater Installations

Groundwater levels were recorded during drilling and standpipes were installed in 79 locations (including siting study investigation installations) across the site to allow for long-term monitoring.

3.1.5 Laboratory Testing

Laboratory testing was carried out on selected samples recovered from the exploratory boreholes. The tests and their functions can be viewed in the following table:

Laboratory Test	Function
Moisture Content Particle Size Distribution Atterberg Limits Bulk Density	Classification
California Bearing Ratio (CBR) Moisture Condition Value (MCV) Dry Density / Moisture Content Triaxial Testing	Earthworks Design
pH / SO_3	Chemical
Triaxial Permeability Testing	Landfill Design

Table 3.1: Laboratory Test and Function

3.1.6 In situ Testing

Standard Penetration Tests (SPT's) were carried out in cable percussive boreholes at regular intervals to determine the in situ strength of the material.

In situ permeability (falling and rising head) tests were carried out in cable percussive boreholes at regular intervals within the overburden both during drilling and afterwards in the installations

Permeability tests were undertaken in the bedrock using single and double packer test techniques.

Pump tests were undertaken as part the Hydrogeological Assessment. These tests were designed and supervised by RPS and are discussed within Technical Appendices H – Hydrogeology.

3.1.7 Nomenclature Used for Exploratory Hole Locations

The scope of the investigation was to provide hydrogeological, geotechnical and environmental information to be used to assess the existing environment, to aid in the design of the landfill and to assess the impacts of such a development. As such a prefix was included prior to drilling to each exploratory hole location to aid reference, which were as follows:-

Hydrogeological Rotary Boreholes

HR series: located to assess groundwater flow adjacent to and down gradient of the site, particularly along structural geological features and to delineate a groundwater divide to the north of the site.

SHR series: located to confirm the direction of flow at depth and assess vertical hydraulic gradients in the bedrock.

Geotechnical and Environmental Boreholes

Shell and Auger (GS & ES series) boreholes located to assess geotechnical parameters (classification of material, stiffness, permeability, slope stability, etc) to be used for design.

Geobore S (GR & ER series) boreholes located to assess geotechnical parameters (classification of material, stiffness, permeability, slope stability, etc) to be used for design.

Additional prefixes

ASA: additional shell and auger borehole

AGB: additional geobore S borehole

PW: Pump test well location

TP: Trial Pit

3.2 GEOPHYSICS

As detailed in Section 2.3.2, a considerable amount of geophysical work was carried out during the site selection process. This work was supplemented by additional geophysics and a report was prepared to collate this and all previously gathered information. Geophysics was used to :-

- Investigate the suitability of the site as a potential landfill site;
- Determine variations in overburden thickness and type;
- Determine depth to bedrock;

- Examine variation in bedrock type and quality;
- Determine the presence of any faulting /change in lithology;

The geophysical profiles were confirmed and correlated with depths encountered during the ground investigation. Depth of overburden and depth to bedrock profiles are presented in the Geophysical Report entitled "Fingal Landfill, Geophysical Investigation – Final Report, November 2005", which is supplied in the Supporting Documents of this Appendix.

3.3 ADDITIONAL GROUND INVESTIGATION 2006

An additional investigation was conducted in February 2006 primarily to provide information for use in design of the proposed access road. This investigation consisted of 12 no trial pits, pavement coring, dynamic probing and laboratory testing. 2 no. Geobore S, including 2 no. groundwater installations, were drilled within the footprint of the landfill in order to supplement the information gathered for the landfill design process. This additional work is detailed in Fingal Landfill Additional Ground Investigation Factual Report (No. 06-074) Glover SI. (2006) contained in the Supporting Documentation of this Appendix).

4 GROUND CONDITIONS WITHIN LANDFILL FOOTPRINT

This section presents the findings of the recent ground investigations. The proposed landfill footprint was chosen as detailed in Figure 4.1 and incorporated a number of constraints, including visual impact and buffer from adjacent properties in line with guidelines set out within the Draft EPA Landfill Siting Manual (1997) and as described in Chapter 2 of the main EIS report.

In addition the landfill footprint has been located so that a matrix response of R1 can be achieved in the Response Matrix for Landfills, (DoEHLG/EPA/GSI, 1999). The landfill is underlain by an Lm aquifer, which is described by the GSI as a locally important, moderately productive aquifer and in order to achieve an R1 response, the landfill footprint must be underlain by greater than 10m of low permeability subsoil material.

As a considerable quantity of investigation was undertaken across the study area, this section will deal primarily with the superficial deposits encountered within the proposed landfill footprint as illustrated in Figure 4.1).

4.1 GENERAL

The ground conditions identified within the footprint during the ground investigations typically comprise clay deposits overlying gravel and bedrock. Geobore S drilling techniques enabled retrieval of continuous cores through the overburden that enabled logging and provision of samples for laboratory testing. Geobore S drilling extended to depths up to 27.25m in clays.

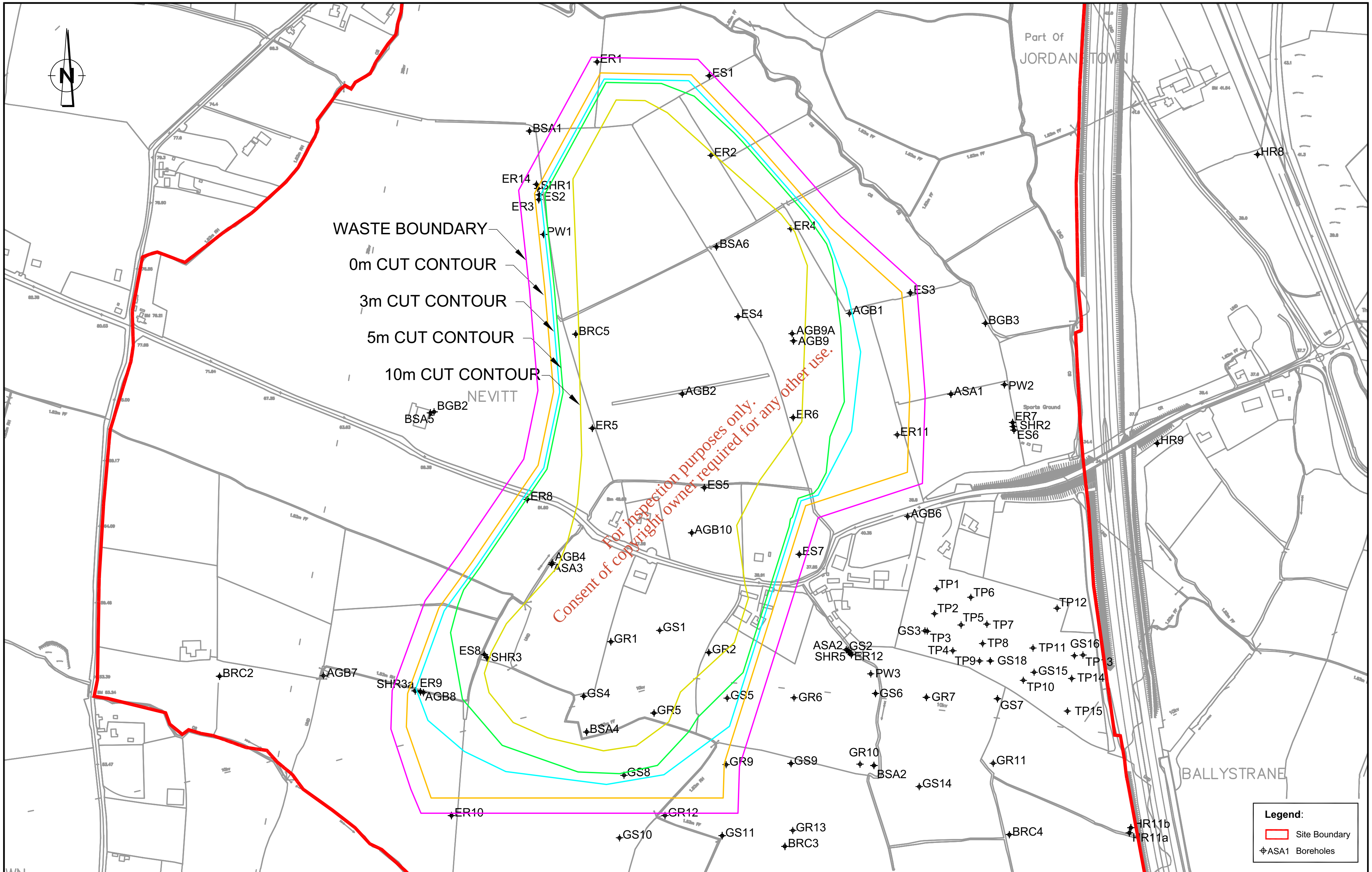
Cable percussive boreholes were also constructed and enabled in situ testing (permeability and SPT) at regular intervals. Whilst this method was unable to extend to depths achieved by the Geobore S method owing to the stiff to very stiff nature of the overburden, large diameter (300mm) cable percussive boreholes was mobilised at certain locations and (ASA1 – ASA3, ER5) were able to penetrate the stiff material and retrieve samples up to 21.2m depth.

The landfill footprint was confined to the west of the site during the Landfill Siting Study (2004) due to the discovery of shallow bedrock located in BGB2 at 6.7m and during the 2005 investigation at AGB7 at 7.3m.

The depth to rockhead across the landfill footprint is typically greater than 24m, as shown in the Geophysical Report (see Supporting Documents to this appendix). The rock encountered was typically Limestone. However, Mudstones and Siltstones were also identified in some boreholes during the ground investigation.

The footprint outlined in Figure 4.1 illustrates the envisaged excavation contours associated with the landfill, i.e. 3m contour represents a maximum excavation depth of 3m bgl, the 5m contour represents a maximum excavation depth of 5m bgl and the 10m contour represents a maximum excavation depth of 10m bgl. From the evidence obtained from the ground investigations these areas are all underlain by a minimum of 10m of low permeability material (i.e. CLAY).

In order to satisfy the Groundwater Protection Response Matrix for landfills of R1, the vulnerability of the underlying natural material must be ‘Low’ since the landfill is underlain by an Lm aquifer (locally important, moderately productive). Therefore, under the footprint of the landfill a minimum of 10m of low permeability material must be present below the cutting.



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NOTES

1. This drawing is the property of RPS-MCOS Ltd., it is a confidential document and must not be copied, used, or its content divulged without prior written consent.
2. All Levels refer to Ordnance Survey Datum, Malin Head.
3. DO NOT SCALE, use figured dimensions only, if in doubt ask.

No.	Date	Dr.	Ch.	Amendment / Issue	App.
F01	Apr'06			Final Issue	KW

Project: FINGAL LANDFILL PROJECT

Title: BOREHOLE LOCATIONS WITH PROPOSED BASE CUTS

Drawn by:	EB	Job No:	MDR0303
Checked by:	EB	File No:	MDR0303FG4.1F01
Approved by:	LOT	Drg. No:	Rev:
Scale:	1:5000@A3	Fig. 4.1	F01
Date:	Apr. '06		

It should be noted that in the event of the underlying low permeability material being between 5 and 10m, or the underlying material being of moderate permeability and a vulnerability rating of 'moderate' being achieved the Response Matrix would class the site as R2[2] which is still acceptable, subject to guidance by the EPA and GSI for the development of landfill. Maintaining a low permeability clay depth of 10m and a Response Class of R1 provides an additional level of protection and security to the surrounding groundwater.

4.2 SUPERFICIAL DEPOSITS

In the proposed landfill footprint, exploratory boreholes indicate deep clay running from north to south typically extending to depths ranging from 20mbgl to 27.25mbgl. However, geophysics indicated clay to greater depths in places within the landfill footprint (BMA, 2005, Fingal Landfill – Geophysical Investigation).

Generally borehole and trial pit logs indicate approximately 0.2m of topsoil overlying clay. The clay can generally be divided into two layers. Immediately below the topsoil, a clay layer, typically 2.5m thick, was identified and consisted predominantly of a firm light brown sandy gravelly (angular to sub-rounded) CLAY, with occasional cobbles. This was underlain by a second clay layer described as a stiff to very stiff grey to black sandy gravelly CLAY, containing occasional cobbles and boulders. The clay generally becomes stiffer with depth.

The depth of cohesive overburden decreases to the east and south east where shallower granular deposits were encountered, e.g. ASA1 at 11m bgl, ASA2 at 12.2m bgl, GS10 at 4.5m bgl. The gravel was generally described as medium-dense to dense brown sandy fine to coarse sub-rounded GRAVEL containing occasional cobbles and boulders.

For the purposes of the report the following tables present the depths of clay overburden within the landfill footprint for each of the cut contours, see Tables 4.2 & 4.3, dividing the site into North and South of the Nevitt Road. A number of cable percussive boreholes were omitted from the tables owing to refusal on boulders at shallow depths within cohesive overburden.

An anomaly was encountered in AGB4 where sandy GRAVEL to a depth of 4.5m was encountered. A secondary borehole, ASA3, was constructed adjacent to AGB4 and encountered CLAY to a depth of 19m.

Made Ground was encountered southwest of the M1 - Nevitt Overbridge. 15 Trial Pits were excavated to obtain further information on the nature and extent of the material. Trial pit depths ranged from 0.9mbgl to 3.5mbgl. Material encountered was predominately found to contain: brick, wood, ash, plastic, concrete, metal, occasional organics, intermixed with cohesive material.

Hole	Depth of Clay	Underlain by	Location within Footprint			
			Adjacent to Waste Boundary	Waste Boundary to 3m Cut Contour	Within 5m Cut Contour	Within 10m Cut Contour
ER1	Clay to 21m	Rock		x		
ES1	Clay to 12.3m	*		x		
BSA1	Clay to 16.6m	*	x			
ER2	Clay to 25m	Gravel				x
ER3	Clay to 21.45m	Gravel		x		
ES2	Clay to 9.4m	*		x		
ER4	Clay to 25.75m	Gravel				x
BSA6	Clay to 14m	*				x
ES3	Clay to 13.8m	*		x		
AGB1	Clay to 20.65m	Gravel			x	
ES4	Clay to 13m	*		x		
BRC5	Clay to 20.2m **	Gravel			x	
AGB2	Clay to 27.25	Gravel				x
ASA1	Clay to 11m	Gravel	x			
ER6	Clay to 21m	Gravel				x
ER5	Clay to 21.2m	*				x
ER11	Clay to 13.5m	Gravel		x		
ES5	Clay to 20.5m	*				x
AGB9	Clay to 24.9m	Rock				x
AGB10	Clay to 21.7m	Gravel				x
ES7	Clay to 14.7m	*		x		

* refusal on boulder / obstruction ** drilled using open-hole techniques *** scheduled depth

Table 4.2: Depth of Overburden within Footprint (North of Nevitt Road)

Hole	Depth of Clay	Underlain by	Location within Footprint			
			Adjacent to Waste Boundary	Waste Boundary to 3m Cut Contour	Within 5m Cut Contour	Within 10m Cut Contour
ER8	Clay to 15m	Rock			x	
ASA3	Clay to 19m				x	x
GS1	Clay to 10.9m	*				x
GR1	Clay to 19.75m	***				x
GR2	Clay to 22.9m	Gravel				x
ES8	Clay to 10.1m	*			x	
SHR3	Clay to 20m**	Gravel			x	
GR6	Clay to 13.9m	Gravel	x			
GS4	Clay to 9.6m	*				x
GR5	Clay to 25.95m	***				x
ER9	Clay to 25.4m**	Rock		x		
AGB8	Clay to 21.5m **	*** Rock		x		
BSA4	Clay to 12m	*				x
GR9	Clay to 14.1m	Rock		x		
GS9	Clay to 10.2m	*	x			
GS8	Clay to 20m	***		x		
GR12	Clay to 19.3m	Sand	x			
ER10	Clay to 14.9m	Gravel	x			
GS11	Clay to 11.5m	*	x			
BRC3	Clay to 10.5m	Rock	x			

* refusal on boulder / obstruction ** drilled using open-hole techniques *** scheduled depth

Table 4.3: Depth of Overburden within Footprint (South of Nevitt Road)

4.3 CLASSIFICATION OF CLAY DEPOSITS WITHIN LANDFILL FOOTPRINT

4.3.1 Particle Size Distribution

Figure 4.2 presents a number of particle size distribution test results at various depths and it is evident that the material is consistently uniform with depth. The plot indicates that the material generally consists of the following constituents – 25% GRAVEL; 30% SAND; 30% SILT and 15% CLAY.

BS5930 states that all soils should be described in terms of their likely engineering behaviour and as such it is necessary to look at the plasticity test results (section 4.3.2) in conjunction with the particle size distribution results, these indicate that the material is classified as a low to intermediate plasticity CLAY.

Therefore, analysis of the particle size distribution test results confirms the description of "sandy gravelly CLAY" as detailed on the borehole logs.

As indicated in Section 4.2, two clay types were evident during the logging of samples namely the upper light brown CLAY and the deeper dark grey to black CLAY. However, analysis of the test results did not indicate any noticeable difference in their constituents as both exhibited a sandy gravelly matrix.

It should be noted that occasional lenses of more granular material were encountered within the clay and this was noticeable on a small number of particle size distribution curves, e.g. GS08 @ 17m. The particle size distribution curves have been amended to exclude material greater than 20mm in accordance with guidelines presented by the GSI (Guidelines for Assessment and Mapping of Groundwater Vulnerability to Contamination, 2003).

Table 4.4 presents the percentage clay and fine material derived from the particle size distribution tests.

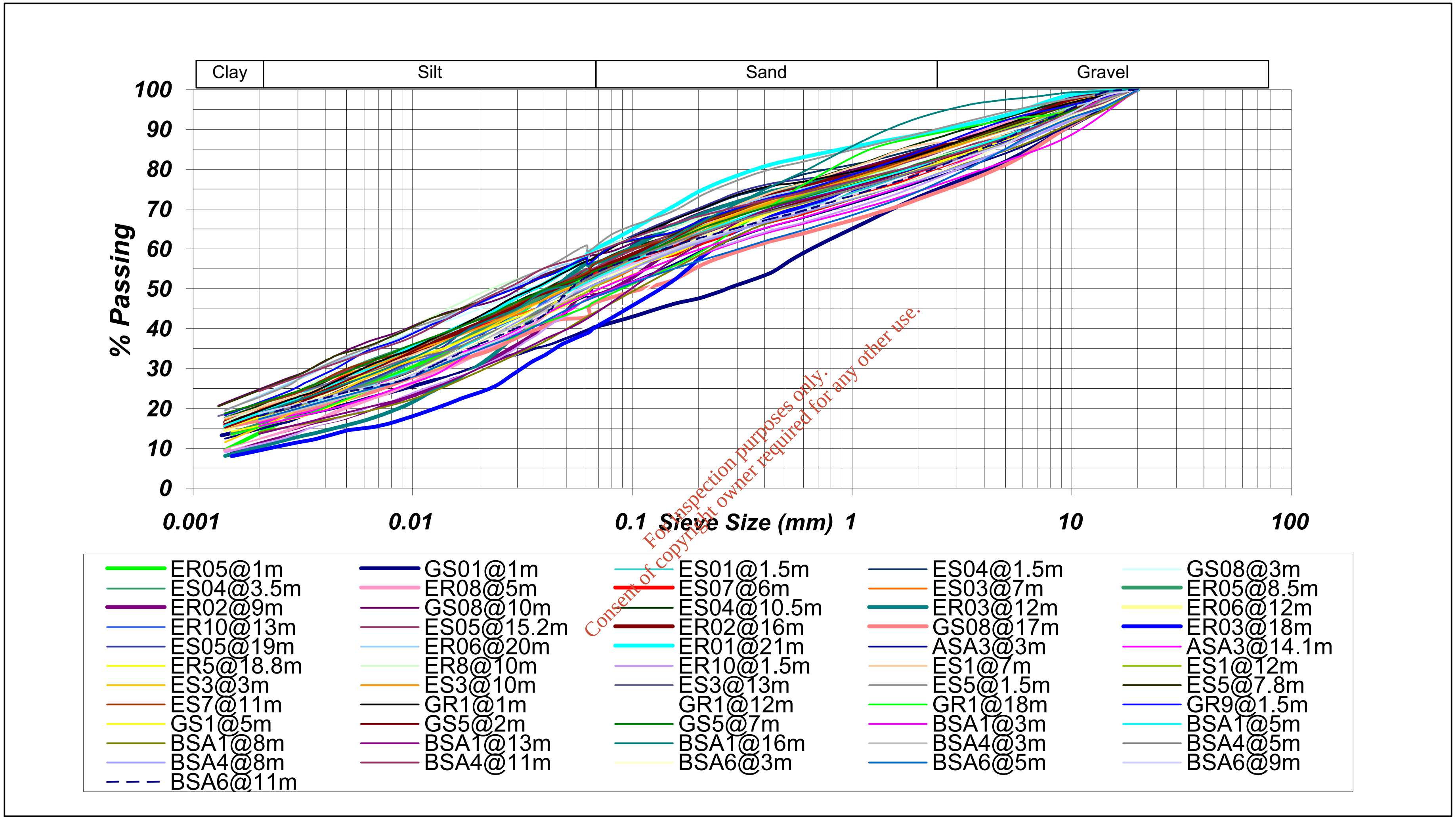

Figure 4.2: PSD curves within the CLAY material

Borehole	Depth	Fines (%)	CLAY (%)		Borehole	Depth	Fines (%)	CLAY (%)
ER01	21	59	18		ES04	10.5	54	20
ER02	9	51	11		ES05	1.5	59	22
ER02	16	54	18		ES05	7.8	57	24
ER03	12	56	10		ES05	15.2	55	20
ER03	18	39	9		ES05	19	57	18
ER05	18.8	49	15		ES07	6	51	19
ER05	1	53	13		ES07	11	54	21
ER05	8.5	54	15		GR01	1	57	19
ER06	12	50	17		GR01	12	40	12
ER06	20	57	22		GR01	18	46	15
ER08	5	50	11		GR09	1.5	56	21
ER08	10	59	19		GS01	1	40	15
ER10	1.5	46	11		GS01	5	51	17
ER10	13	53	18		GS05	2	53	18
ES01	1.5	51	18		GS08	3	55	23
ES01	7	52	20		GS08	10	58	24
ES01	12	49	17		GS08	17	43	17
ES03	3	52	15		ASA3	3	48	16
ES03	7	50	20		ASA3	14.1	48	14
ES03	10	52	21		BSA4	3	51	17
ES03	13	50	21		BSA4	5	53	18
BSA1	3	49	16		BSA4	8	50	17
BSA1	5	52	19		BSA4	11	59	24
BSA1	8	43	14		BSA6	3	51	17
BSA1	13	43	14		BSA6	5	47	17
BSA1	16	52	19		BSA6	9	51	19
ES04	1.5	55	17		BSA6	11	53	18
ES04	3.5	54	20					

Table 4.4: Percentage Clay and Fine values within Landfill Footprint

The GSI Guidelines for Assessment and Mapping of Groundwater Vulnerability to Contamination (2003) refers to use of the clay % and fine fraction % derived from the particle size distribution curves as a means to classify permeability in conjunction with other indicators.

Low permeability material is described as having >14% CLAY or >50% FINES by weight (corrected to remove particles over 20mm diameter). Table 4.4, presents the corrected Clay and Fine content percentages within the landfill footprint.

It is evident that the majority of the curves satisfy the 14% CLAY fraction criteria with only 6 no. plots failing to satisfy the 14% boundary. Of these plots, 3 plots, ER3 at 18m, ER10 at 1.5m and GR1 at 12m fail to satisfy either the clay > 14% or fines > 50% criteria. However, results at ER3 and ER10 can be ignored as they lie within the 0-3m cutting contour and outside the waste boundary respectively.

It is acceptable to classify the material within the landfill footprint as low permeability based on the GSI DRAFT Guidelines for Assessment and Mapping of Groundwater Vulnerability to Contamination (2003), since the final requirement is that for all of the samples 95% of the results must meet the overall result criteria of CLAY > 13% and Fines > 37%. Therefore, 100% of the PSD samples in the landfill footprint meet the criteria.

4.3.2 ATTERBERG LIMITS

Figure 4.3 represents the Casagrande Plasticity Chart for samples tested within the proposed landfill footprint. All samples tested are classified as a low to intermediate plasticity clay falling above the A line. The low plasticity of this clay and its sandy gravely nature would indicate that under loading expected settlements are likely to be small and occur relatively quickly.

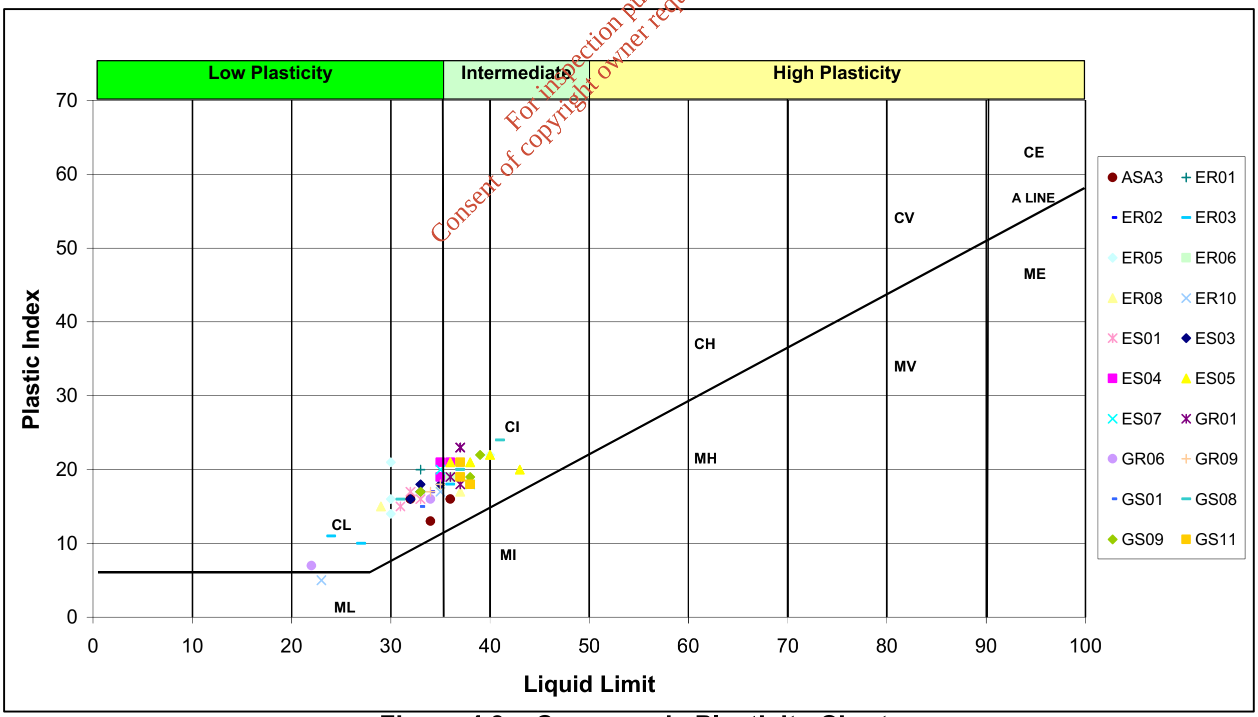

Figure 4.3 – Casagrande Plasticity Chart

4.3.3 MOISTURE CONTENT

Figure 4.4, illustrates the moisture content with depth profile for the CLAY deposits encountered during the investigation. It is evident that the brown sandy gravelly CLAY (up to 2.5m deep) exhibits a slightly higher moisture content (15% - 22%) than the black sandy gravelly CLAY (10% - 19%).

The plot also indicates that the moisture content within the black sandy gravelly CLAY appears to reduce with depth, which is reflected in the material being classified as becoming stiffer with depth.

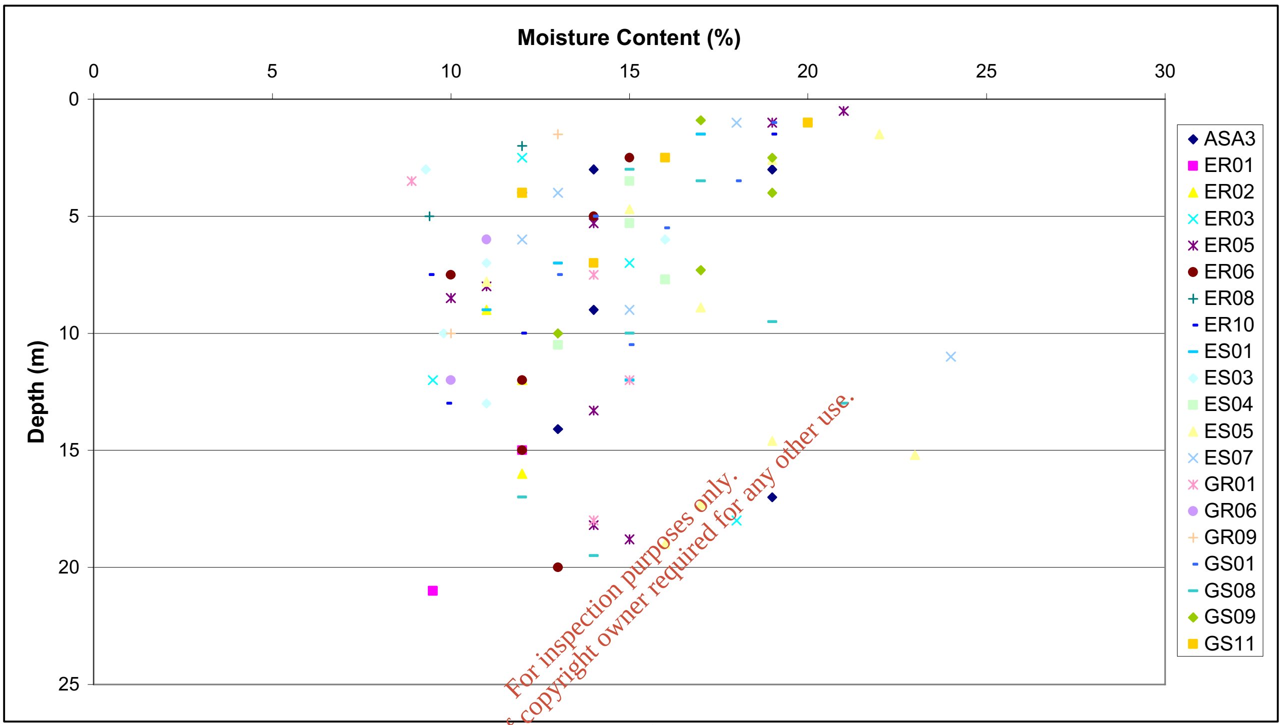

Figure 4.4: Moisture Content with Depth

Review of the dry density / moisture content results indicates that the brown sandy gravelly CLAY has an optimum moisture content of approximately 17% (?d = 1.8Mg/m^3). The dark grey to black sandy gravelly CLAY has optimum moisture contents between approximately 12.5% and 15% (?d = 1.9Mg/m^3).

4.3.4 BULK DENSITY

Figure 4.5 presents bulk density results for the CLAY material encountered during the site investigation. It is evident that the bulk densities within the brown sandy gravelly CLAY vary between 1.8 Mg/ m^3 and 2.2 Mg/ m^3 in comparison to the underlying black sandy gravelly CLAY with bulk densities ranging from 2.0 Mg/ m^3 and 2.3 Mg/ $m^{3.}$

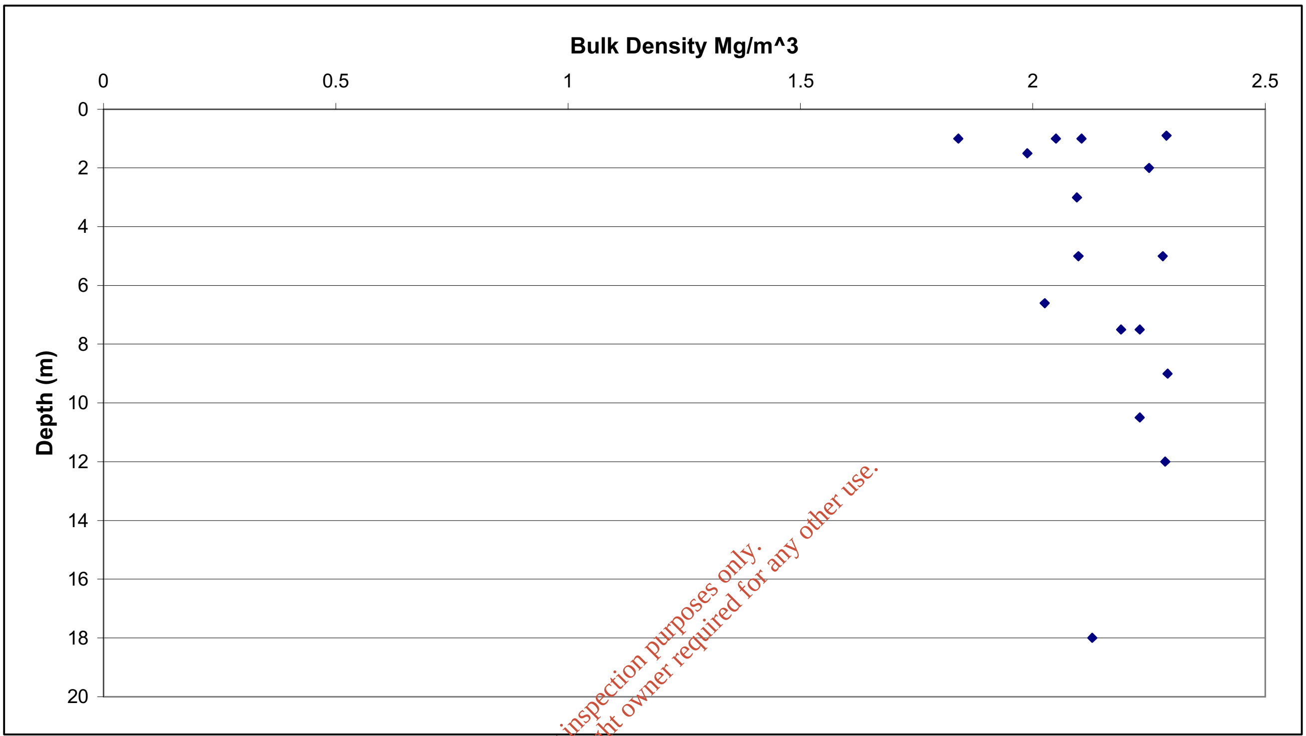

Figure 4.5: Bulk Density with Depth

4.3.5 MOISTURE CONDITION VALUE

Figure 4.6 presents MCV values for soils within the proposed landfill footprint which will have clay cuttings up to 10m deep. MCV's range from 0.5 to 17 but the critical value for re-use is 8. It is found that a moisture content of 15% is the upper moisture level, above which material is likely to require processing to enable re-use as a Class 2 General Fill (used in construction of embankments etc.). The highlighted area indicates the optimum conditions for re-use.

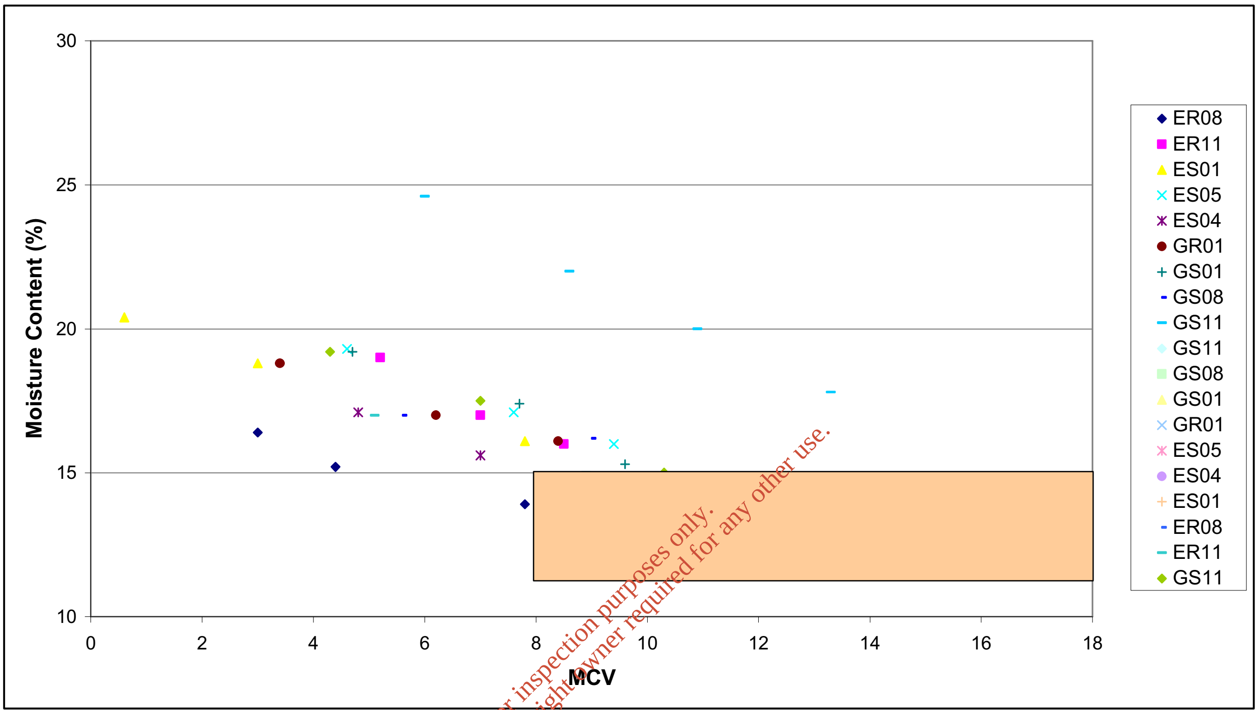

Figure 4.6 – MCV plot with moisture content

4.3.6 STANDARD PENETRATION TEST (SPT)

SPT (N) values were recorded during the drilling of the Cable Percussion boreholes (ES, GS and ASA series). Figure 4.7 presents the SPT results from boreholes drilled within the proposed landfill footprint.

It shows that the strength of the material typically increases with depth, with a significant number of boreholes encountering SPT refusal (>50 blows/300mm) between 3.2m and 10m depth. The SPT values reaffirm the descriptions detailed on the borehole logs, which indicate that the brown CLAY (up to 2.5m deep) is firm and that the underlying dark grey to black CLAY is stiff becoming very stiff.

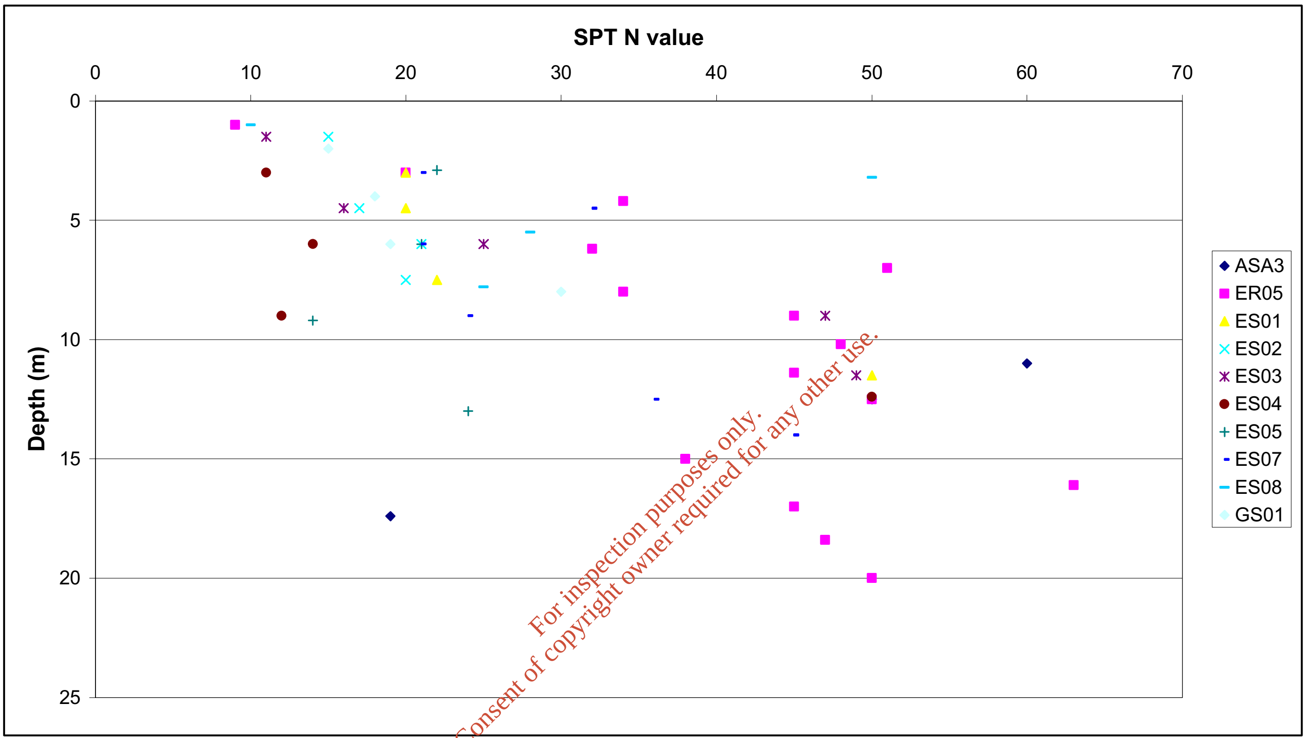

Figure 4.7 – SPT vs Depth

4.3.7 pH TESTS

A number of pH tests were carried out within the Clay material. Results between 7.5 and 8 were recorded, which is considered normal.

4.3.8 PERMEABILITY TESTS

Extensive permeability testing was carried out across the site in order to establish the suitability of the Clay material to be classified as low permeability, the results of which are detailed in Table 4.5. A number of different methods were used to calculate the permeability of the material which are as follows:-

Variable Head Permeability

Variable head tests were used in cable percussive boreholes to estimate the permeability of the clay material. However, give that refusal was typically encountered at approximately 10m, in situ permeability results were rarely taken beyond that depth.

The permeability is calculated by monitoring the change in water level over the test period, typically one hour. However, given the low permeability of the clay material, it was necessary to extrapolate results over significantly longer time periods.

It should be noted that BS5930 states that execution of the test requires expertise, and small faults in technique can lead to errors up to 100 times the actual value. Even with significant care, an individual test result is often accurate to one significant figure only.

In March 2006, RPS repeated a number of in situ permeability tests in borehole standpipes that exhibited higher than anticipated permeability values for clays. Continuous water level monitors (DIVERS) were used to calculate the permeability by accurately recording changing water levels over longer time periods (up to 10 days). As such, extrapolation of results was not required and the conclusion is that a more representative value of permeability was attained.

Triaxial Permeability Tests

A significant number of triaxial permeability tests were carried out at various depths using undisturbed samples extracted by Geobore S and cable percussive methods.

Hazen Formula

The Hazen formula utilises the particle size distribution curve to calculate the permeability of a material. The D_{10} value (the size that 10% of the particles are smaller than) is taken and inserted into the following formula to derive the permeability value:-

$$k = 10^{-2}D_{10}^{2}\ (m/s)$$

It should be noted that this formula is specifically used for sands but is commonly used to provide an estimate for permeability of clays and gravels. In some cases, the PSD curve did not extend fully to the D_{10} line and it was necessary to extrapolate to this point.

Hole ID	Depth (m)	Permeability (m/s)			
		Variable Head Test (Glover 2005)	Variable Head Test (RPS 2006)	Triaxial Permeability Test (Glover 2005)	Hazen Formula
ER8	5.0				2.3E-08
	10				4.9E-09
ASA3	3.0				8.1E-09
	9.4	5.60E-09			
	14.5	1.80E-06			
	14.1			8.80E-11	2.5E-09
	17.0	4.60E-06			
GS1	1.0				4.9E-09
	4.0	2.20E-08			
	5.0				6.4E-09
	9.0	1.80E-08			
GR1	1.0				4.9E-09
	12				2.6E-08
	18				8.1E-09
GR2	12.0			3.98E-10	
	16.0			6.38E-10	
GR6	6.0			3.44E-10	
GS4	4.0	2.20E-08			
GR5	18.0			2.09E-09	
GR9	1.5				3.6E-09
	6.0			5.14E-10	
GS8	3.0				3.6E-09
	10.0				1.6E-09
	17.0				2.5E-09
GR12	12.0			2.56E-10	
ER10	1.5				2.9E-08
	7.5			1.80E-09	
	13				1.6E-09
GS11	4.0	3.10E-08	1.60E-09		
	9.0	3.10E-08			
ER1	15.0			1.05E-08	
	21.0				6.4E-09
ES1	1.5				7.2E-09
	5.0	6.70E-07	1.60E-08		
	7.0				3.0E-09
	9.0			3.80E-11	
	9.8	5.30E-06			
	12				3.6E-09
ER2	9.0				2.9E-08
	12.2			1.99E-10	
	16				1.2E-09
	20.0			1.80E-10	
ER3	12.0				3.2E-08
	18.0				4.8E-08
AGB2	12.0			1.44E-10	

Table 4.5: Permeability test results within proposed landfill footprint

Hole ID	Depth (m)	Permeability (m/s)			
		Variable Head Test (Glover 2005)	Variable Head Test (RPS 2006)	Triaxial Permeability Test (Glover 2005)	Hazen Formula
ES3	3				1.7E-08
	4.8	1.70E-07	4.00E-08		
	7.0				3.6E-09
	7.5			5.30E-11	
	10				3.6E-09
	13				3.6E-09
ES4	1.5				2.5E-09
	3.5				3.0E-09
	5.0	8.60E-08			
	10.0	8.50E-08			
	10.5			8.60E-11	3.0E-09
ER6	12				7.2E-09
	17.0			9.30E-11	
	20.0				3.60E-09
ER5	1.0				2.25E-08
	8.5				4.9E-09
	13.3			1.00E-09	
	18.8				3.6E-09
ES5	1.5				2.0E-09
	7.8				1.6E-09
	11.0			4.90E-11	
	15.2				4.9E-09
	19				3.6E-09
ES7	4.8	3.40E-07	2.14E-07		
	11.0			8.00E-11	3.6E-09
ES8	5	3.60E-09	7.35E-09		
	10	3.90E-07			
GS5	2				6.4E-09
	7				3.6E-09
BSA4	1.0		4.70E-09	2.66E-10	
	3.0				1E-08
	5.0				4.9E-09
	8.0				1.96E-10
	11.0				2.5E-09
BSA6	3.0				6.4E-09
	5.0				6.4E-09
	9.0				2E-09
	11.0				4.9E-09
	12.5		2.14E-08	1.90E-10	
	13.0				9E-10
BSA1	3.0				8.1E-09
	5.0				3.6E-09
	8.0				1.21E-08
	13.0				3.2E-08
	14.0			7.36E-11	
	16.0				6.4E-09

Table 4.5: Permeability test results within proposed landfill footprint (contd.)

Lambe and Whitman, (Soil Mechanics, SI version, MIT, 1979), present the following table, Table 4.6, relating to the classification of Soils according to coefficients of permeability. A similar degree of permeability is listed in the CIRIA report – Groundwater Control, Design and Practice (2002).

Degree of Permeability	Permeability, k, (m/s)
High	Over 10^{-3}
Medium	10^{-3} to 10^{-5}
Low	10^{-5} to 10^{-7}
Very Low	10^{-7} to 10^{-9}
Practically Impermeable	Less than 10^{-9}

Table 4.6: Classification of Soils according to their Coefficients of Permeability

Based on the range of permeabilities measured and estimated from the various techniques the material is predominately low to very low permeability clay. This supports the existing low vulnerability classification of the aquifer determined by the GSI in the Bog of the Ring Groundwater Source Protection Zones Report (2005).

4.4 CLASSIFICATION OF BEDROCK

As mentioned in section 2.2, the predominant rock types that underlie the study area are Limestone, Shale and Sandstone. No strength tests (UCS or Point Loads) were carried out on cores but the recovery was typically excellent (>90%) and rock quality designation values were very good, typically greater than 60%.

The classification of bedrock is discussed in greater detail within the Geology and Hydrogeology Report (Volume 5. Appendix H).

5 ENGINEERING SIGNIFICANCE OF GROUND CONDITIONS

In this section geotechnical interpretations will be made in relation to key design and construction aspects associated with the proposed Landfill.

5.1 GROUNDWATER VULNERABILITY

The Draft Bedrock Aquifer Map for North Country Dublin indicates that the site is underlain by geogical formations and lithologies, which have a range of aquifer classifications. The Loughshinny, Lucan and Naul Formations have been classified as locally important bedrock aquifers which are 'generally moderately productive' (Lm) by the GSI. The Lm aquifers make up the majority of the underlying bedrock at the site. According to the GSI classification, such aquifers are capable of yielding enough water to springs or boreholes to supply villages, small towns of factories.

The Walshestown and Balrickard Formations have been classified as poor bedrock aquifers, which are 'generally unproductive except for local zones' (Pl) by the GSI. According to the GSI classification such aquifers are normally capable of yielding only sufficient water from wells or springs to supply single houses, small farms or small group water schemes. These Pl aquifers are located to the north of the site.

Exploratory boreholes indicate deep clay running from north to south typically reaching depths ranging from 20mbgl to 27.25mbgl and geophysics indicated clay to greater depths in places. The overburden thickness reduces to the west and shallow gravels are present to the east and south east. As such the landfill footprint has been tailored to avoid the areas where the depth of cohesive overburden cover decreases and cannot offer the 10m low permeability clay cover required to achieve the vulnerability classification of 'Low' which together with the Lm aquifer results in an R1 classification (Response Matrix for Landfills (DoEHLG/EPA/GSI, 1999).

The reducing clay buffer occurs to the east and south east of the proposed landfill footprint, where shallower granular deposits were encountered, e.g. ASA1 at 11m bgl, ASA2 at 12.2m bgl, GS10 at 4.5m bgl. The gravel in these areas was generally described as medium-dense to dense brown sandy fine to coarse sub-rounded GRAVEL containing occasional cobbles and boulders.

Depth to bedrock ranged from approximately 5m to 34m below ground level (mbGL) and was shallowest in the Hedgestown area and in the west at BRC2. Both of these areas are outside the proposed landfill footprint. To the northeast of the study area, depth to bedrock ranges from 9mbGL in the higher ground at HR3 to 17mbGL at HR1 in the lower lying ground.

The typical vulnerabilities in the areas surrounding the exploratory hole locations within the proposed landfill footprint have been mapped in accordance with the GSI Vulnerability Mapping Guidelines contained within the GSI Groundwater Protection Scheme (1999) and are summarised in Table 5.1.

As detailed in Section 4.3, it is established that the CLAY material underlying the landfill footprint can be classified as a low permeability material under the GSI guidelines, Mapping of Groundwater Vulnerability to Contamination 2003.

Hole	Type	Principal Soil type	CLAY Thickness proven	Location within Footprint	Permeability Indicator: Variable Head Test (Glover, 2005)	Permeability Indicator: Variable Head Test (RPS, 2006)	Permeability Indicator: Triaxial	Permeability Indicator: Hazen	Permeability Indicator: >50% Fine or >14% CLAY	Permeability Indicator: Subsoil Description	Permeability Rating (See Note 1)	Groundwater Vulnerability (See Note 2)	Comments
ER1	Geobore S	CLAY	21	< 3m Cut	See Note 3	See Note 3	Low	Low	Low	Low	Low	Low	
ES1	CP	CLAY	12.3	< 3m Cut	Low to Mod ^	Low	Low	Low	Low	Low	Low	Low (see ER1)	^ Variable Head Test (VHT) at 9.8m (200, superseeded by further VHT (2006)
BSA1	CP	CLAY	16.6	< 3m Cut	-	-	Low	Low	Low	Low	Low	Low	
ER2	Geobore S	CLAY	25	10m Cut	See Note 3	See Note 3	Low	Low	-	Low	Low	Low	
ER3	Geobore S	CLAY	21.45	< 3m Cut	See Note 3	See Note 3	Low	Low	Low to Mod +	Low	Low	Low	+ PSD at 18m: Fines 39%, CLAY 9%, 3m cutting therefore result not relevant
ER14	Rotary	CLAY	21.8 *	< 3m Cut	-	-	See Note 4	See Note 4	See Note 4	Low	Low	Low	permeability based on subsoil description
ES2	CP	CLAY	9.4 **	< 3m Cut	See Note 3	See Note 3	-	-	-	Low	Low	Low (see ER3)	permeability based on subsoil description, vulnerability based on nearby borehole extending to greater CLAY depth
ER4	Geobore S	CLAY	25.75	10m Cut	See Note 3	See Note 3	-	-	-	Low	Low	Low	permeability based on subsoil description
BSA6	CP	CLAY	14	10m Cut	-	Low	Low	Low	Low	Low	Low	Low (see ER4)	vulnerability based on nearby borehole
ES3	CP	CLAY	13.8	< 3m Cut	Low	Low	Low	Low	Low	Low	Low	Low	
AGB1	Geobore S	CLAY	20.65	< 3m Cut	See Note 3	See Note 3	-	-	-	Low	Low	Low	permeability based on subsoil description
ES4	CP	CLAY	13	10m Cut	Low	-	Low	Low	Low	Low	Low	Low (see AGB9a)	vulnerability based on nearby borehole extending to greater CLAY depth
BRC5	Rotary	CLAY	20.2 *	5m Cut	-	-	See Note 4	See Note 4	See Note 4	Low	Low	Low	permeability based on subsoil description
AGB9a	Geobore S	CLAY	24.9	10m Cut	See Note 3	See Note 3	-	-	-	Low	Low	Low	permeability based on subsoil description
AGB2	Geobore S	CLAY	27.25	10m Cut	See Note 3	See Note 3	Low	-	-	Low	Low	Low	
ER6	Geobore S	CLAY	21	10m Cut	See Note 3	See Note 3	Low	Low	Low	Low	Low	Low	
ER5	CP	CLAY	21.2	10m Cut	-	-	Low	Low	Low	Low	Low	Low	
ER11	Geobore S	CLAY	13.5	< 3m Cut	See Note 3	See Note 3	-	-	-	Low	Low	Low	permeability based on subsoil description
ES5	CP	CLAY	20.5	10m Cut	-	-	Low	Low	Low	Low	Low	Low	
ER8	Geobore S	CLAY	15	5m Cut	See Note 3	See Note 3	-	Low	Low	Low	Low	Low	
AGB10	Geobore S	CLAY	21.7	10m Cut	See Note 3	See Note 3	-	-	-	Low	Low	Low	permeability based on subsoil description
ES7	CP	CLAY	14.7	< 3m Cut	Low	Low	Low	Low	Low	Low	Low	Low	
ASA3	CP	CLAY	19	5m Cut	Low to Mod ^	-	Low	Low	Low	Low	Low	Low	^ Variable Head Test (VHT) at 14.5m and 17m
GS1	CP	CLAY	10.9	10m Cut	Low	-	-	Low	Low	Low	Low	Low (see GR2)	vulnerability based on nearby borehole extending to greater CLAY depth
GR1	Geobore S	CLAY	19.75**	10m Cut	See Note 3	See Note 3	-	Low	Low to Mod +	Low	Low	Low (see GR2)	+ at 12m Fines 40%, CLAY 12%, hazen at same depth indicates low k. vulnerability based on nearby borehole extending to greater CLAY depth
GR2	Geobore S	CLAY	22.9	10m Cut	See Note 3	See Note 3	Low	-	-	Low	Low	Low	
ES8	CP	CLAY	10.1	5m Cut	Low	Low	-	-	-	Low	Low	Low (see SHR3)	vulnerability based on nearby borehole extending to greater CLAY depth
SHR3	Rotary	CLAY	20 *	5m Cut	-	-	See Note 4	See Note 4	See Note 4	Low	Low	Low	permeability based on subsoil description
GS4	CP	CLAY	9.6	10m Cut	Low	-	-	-	-	Low	Low	Low (see GR5)	vulnerability based on nearby borehole extending to greater CLAY depth
GS5	CP	CLAY	10.5**	<3m Cut	-	-	-	Low	Low	Low	Low	Low (see GR5)	vulnerability based on nearby borehole extending to greater CLAY depth
GR5	Geobore S	CLAY	25.95	10m Cut	See Note 3	See Note 3	Low	-	-	Low	Low	Low	
ER9	Rotary	CLAY	25.4*	< 3m Cut	-	-	See Note 4	See Note 4	See Note 4	Low	Low	Low	permeability based on subsoil description
AGB8	Rotary	CLAY	21.5*	< 3m Cut	-	-	See Note 4	See Note 4	See Note 4	Low	Low	Low	permeability based on subsoil description
BSA4	CP	CLAY	12	10m Cut	-	Low	Low	Low	Low	Low	Low	Low (see GR5)	vulnerability based on nearby borehole extending to greater CLAY depth
GR9	Geobore S	CLAY	14.1	< 3m Cut	See Note 3	See Note 3	Low	Low	Low	Low	Low	Low	
GS8	CP	CLAY	20	5m Cut	-	-	-	Low	Low	Low	Low	Low	
GR12	Geobore S	CLAY	19.3	< 3m Cut	See Note 3	See Note 3	Low	-	-	Low	Low	Low	
ER10	Geobore S	CLAY	14.9	< 3m Cut	See Note 3	See Note 3	Low	Low	Low to Mod +	Low	Low	Low	+ PSD at 1.5m: Fines 46%, CLAY 11%, oustide waste boundary, result not relevant

Note 1: Permeability Rating was assigned using criteria detailed in the GSI Guidelines for Assessment and Mapping of Groundwater Vulnerability to Contamination, (2003), "The boundary between moderate and low permeability materials is practically measured in the range between 10-8 m/s and 10-7 m/s, if measured using field based falling head tests". Table 4.6 on Page 24 (Lambe and Whitman, Soil Mechanics, SI version, MIT, 1979), was also used.

Note 2: Vulnerability Rating based on Groundwater Protection Schemes Guidance Manual - DOELG, EPA, GSI 1999

Note 3: In-situ permeability test not possible in Geobore S hole owing to use of polymer mud flush.

Note 4: No sample recovery in rotary holes for laboratory testing

* drilled using rotary open-hole techniques ** borehole completed in clay CP Cable Percussive

Table 5.1: Permeability and Groundwater Vulnerability across proposed landfill footprint

Table 5.1 indicates that the landfill footprint can offer the required 10m low permeability clay buffer, below the proposed cuttings, required to achieve the R1 classification (Response Matrix for Landfills (DoEHLG/EPA/GSI, 1999). A number of boreholes within the proposed 10m cutting, namely ES2, GS1, ES8, BSA6, GS4, BSA4, ES4, GS5 were completed at depths less than 14m but were located in vicinity to boreholes where clay extended beyond 20m (ER3, GR2, SHR3, ER4, GR5, AGB9a). [Note: Geophysics information was also used to confirm depth of clay deposits in these areas].

It should be noted that in the event of the underlying low permeability material being between 5 and 10m, or the underlying material being of moderate permeability and a vulnerability rating of 'moderate' being achieved the Response Matrix would class the site as R2[2] which is still acceptable, subject to guidance by the EPA and GSI for the development of landfill. Maintaining a low permeability clay depth of 10m and a Response Class of R1 provides an additional level of protection and security to the surrounding groundwater.

5.2 CONSTRUCTION ASPECTS

5.2.1 Excavatability

As detailed in Section 4.3.6, the material is classified as firm becoming stiff to very stiff with depth. This should not represent a significant problem for conventional earthworks plant. However, given the low permeability nature of the material, it is recommended that seasonal effects should be considered when scheduling bulk earthworks.

5.2.2 Slope Stability

Based on provisional slope profiles through a possible cross section, cutting slope stability is unlikely to be problematic as shown in Figures 5.1 & 5.2 where calculated factor of safety (FOS) values > 1.6 for gradients of 2.5H:1V for drained conditions as shown. Embankments can be constructed to 2H:1V

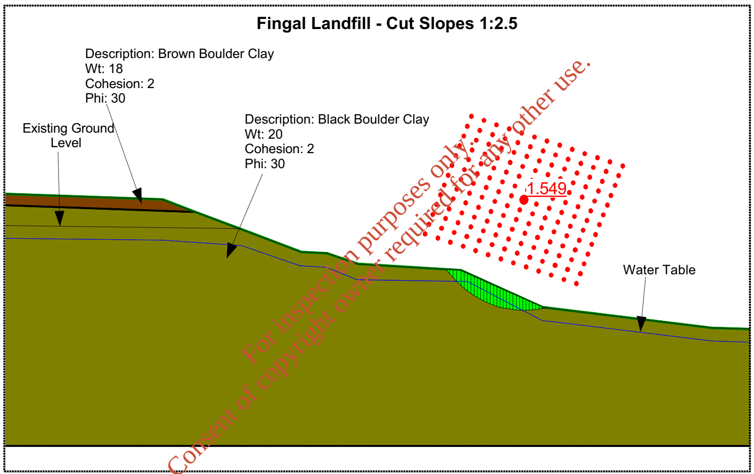

Figure 5.1: Slope Stability Analysis for Typical Cross Section of 2.5H:1V cut slope

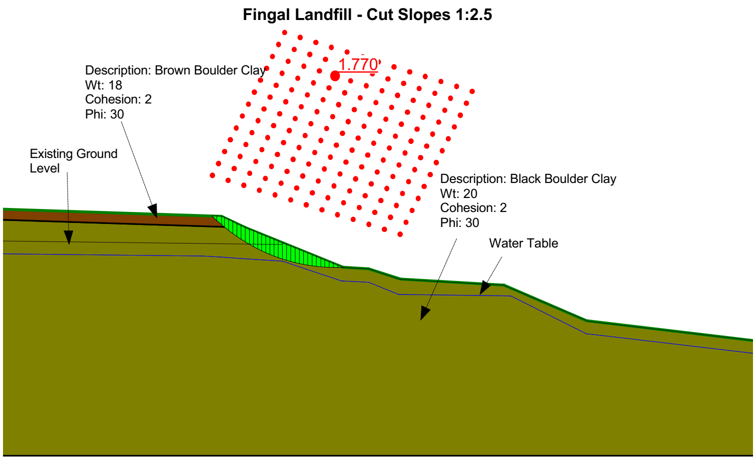

Figure 5.2: Slope Stability Analysis for Typical Cross Section of 2.5H:1V cut slope

5.2.3 Groundwater Control

Groundwater Control measures may need to be implemented and proceed concurrently with advancement of cuttings. This is likely to consist of immediate placement of drainage blankets along cut slopes. Temporary sumps and pumping is likely to be needed until the drainage system is fully implemented.

Attenuation / settlement ponds are likely to be required to settle out suspended solids prior to discharge to watercourses.

Necessary precautions shall be taken to avoid fuel spillages and infiltration to watercourses.

Special drainage measures may need to be implemented to deal with potential perched waters within coarser material lenses within the clays.

5.2.4 Suitability of Excavated Material for re-use

The suitability of the excavated material for re-use as a Class 2 General Fill is generally determined by the following criteria:-

- Fines > 15%
- MCV > 8
- SPT > 10, which equates to CBR >2%

Based on the material properties detailed in Section 4, it is likely that the majority of material will satisfy the Class 2 General Fill criteria. It is apparent that 15% is the critical moisture that the material must be below in order to satisfy the MCV criteria. Although a number of samples tested did not meet this moisture content (with MC's up to 18%), this will reduce sufficiently during the bulk earthworks and placement operation.

A certain amount of processing to remove the cobbles, boulders and coarse gravels may be required to meet potential basal lining and capping material standards.

6 POTENTIAL IMPACTS OF THE DEVELOPMENT

6.1 CONSTRUCTION PHASE

- No excavations or blasting into bedrock is planned; therefore there is no impact on the bedrock geology as a result of the landfill construction;
- Removal of subsoil will decrease the thickness of the material overlying the bedrock which has the potential to increase groundwater vulnerability;
- The removal of established vegetative cover could lead to the loss of large quantities of soil particles to watercourses, which can cause significant pollution of water through the generation of suspended solids;
- Compaction of soils will occur during the construction period as a result of construction traffic;
- It is envisaged that an Earthworks balance will be achieved on site with all excavated material (approximately 3 million m^3) reused in embankment construction or as capping / landscaping material thus negating the potential impact of importing material;
- Cut and Fill slopes represent a potential construction impact in that they could fail;
- Settlement of embankments is a potential impact should mitigation not occur during construction;

6.2 OPERATIONAL PHASE

- Cut and Fill slopes represent a potential operational phase impact in that they could fail;
- Settlement of embankments is a potential impact during operation should mitigation not occur during construction;
- Erosion control and maintenance of cut and fill slopes.

6.3 CLOSURE AND AFTERCARE PHASE

- Stability and Settlement of slopes and embankments represent a Closure and Aftercare Impact;
- The potential for failure of cut and fill slopes represent a potential impact in the closure and aftercare phase should appropriate mitigation not occur during construction.

7 REMEDIAL OR REDUCTIVE MEASURES

7.1 CONSTRUCTION PHASE

- A minimum of 10m of low permeability clay will be retained in situ to maintain low vulnerability classification thus mitigating the impact on groundwater vulnerability;

- Attenuation measures will be implemented to protect watercourses from soil particles mobilised as suspended solids during erosion of exposed (unvegetated) cut / fill slopes;

- The areas likely to be disturbed during construction will be minimised with temporary access roads being constructed for the delivery and removal of materials to the site. Topsoil will be removed and stored in advance of construction of temporary access roads. On completion the ground shall be scarified to restore the subsoil structure before reinstating the topsoil;

- Construction activities will be scheduled such as to minimise the area and period of time that soil will be exposed. In the case of sensitive operations, account of the weather forecast will be taken;

- The migration of fines will be mitigated by appropriate design of drainage systems including appropriate selection of separator geotextiles;

- To mitigate against surface instability, topsoiled slopes will be designed to incorporate a surface water drainage system. Cut slopes shall not exceed 2.5h:1v. Fill slopes shall not exceed 2h:1v;

- Embankment slopes will be topsoiled and seeded as appropriate to alleviate erosion of placed materials;

- Temporary bunds for potentially polluting materials will be used on the site and safe materials handling of all potentially polluting materials will be emphasised to all construction personnel employed during construction;

- Compaction of embankment fill material in accordance with relevant design codes will ensure that post construction settlements are minimised;

- Any unsuitable material excavated, such as the body of made ground, will be disposed of in accordance with relevant legislation;

- Compaction of embankment fill material in accordance with relevant design codes shall ensure that post construction settlements are minimised.

7.2 OPERATIONAL PHASE

Monitoring of settlement and slope stability will be undertaken by regular geotechnical site inspection in accordance with EPA requirements;

7.3 CLOSURE AND AFTERCARE PHASE

Regular geotechnical site inspection will be conducted to examine settlement and slope analyses at the site in accordance with EPA requirements;

8 RESIDUAL IMPACTS

No significant residual impact on the soils and geology is anticipated as a result of development of this scheme.