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STABILITY RISK ASSESSMENT

APPENDICES

1.0 INTRODUCTION

1.1 Report Context

SLR Consulting Limited (SLR) has been appointed by the WasteServ Malta Limited (WasteServ) to prepare a geotechnical Stability Risk Assessment in support of the development of the non-hazardous waste and hazardous waste landfills at Ghallis, Malta. From hereon the non-hazardous waste landfill will be referred to as the Non-Hazardous Landfill and the hazardous waste landfill will be referred to as the Hazardous Landfill.

This document describes the manner in which the assessment has been carried out and presents the overall findings of the work.

The relevant background information describing the site setting (including geological and engineering information, site monitoring data and development proposals) is detailed within the Environmental Statement[1] produced for the proposed development and is not repeated here.

The methodology adopted for this Stability Risk Assessment largely follows the principles outlined in the England and Wales Environment Agency (EA) R&D Technical Report P-385[2] (regarding the stability of landfill lining systems), volumes TR1 and TR2 (and, while not representing official Environment Agency Guidance, from hereon referred to as the Guidance). Where additional analytical techniques have been used, these are described within the text.

1.2 Conceptual Stability Model

The conceptual stability model (Drawing No. SRA 1) has been developed based upon the information contained within the Environmental Statement[1], which, in summary, indicates that:

- The Non-Hazardous and Hazardous Landfills are to be developed within the limestone deposits of the Globigerina Formation and Lower Coralline Formation.

- The landfill site is to be developed to the west of the existing Maghtab Landfill which was closed in May 2004. The proposed development will include the Non-Hazardous Landfill and the Hazardous Landfill. The Non-Hazardous Landfill is to be developed in 4 phases with Phase 1 sub-divided in to Phase 1A and Phase 1B. The Non-Hazardous Landfill will have a capacity of approximately 1.7Mm^3 and will be active for approximately 7 years. Due to the low inputs of hazardous waste expected, the Hazardous Landfill single cell will be worked in a phased manor with small areas constructed at any one time. The Hazardous Landfill will have a capacity of approximately 100,000m^3 and will be active for approximately 20 years.

- The formation levels of the proposed landfills will be achieved through a phased combination of cut and fill resulting in the extraction of approximately 1.2Mm^3 of limestone. Within the Non-Hazardous Landfill, the phases will be constructed sequentially in line with landfill development.

- The base of the landfill is to be formed within the limestone deposits. The excavation surface will be overlain by a minimum 300mm thick formation layer of crushed site won limestone. The formation layer will provide a surface on which to install the lining system and will be used to create the required basal inclinations (1V:50H).

- The side slopes of the landfills are to be largely formed by excavation of in-situ limestone and placement of a granular formation layer to achieve formation levels.

- The base and side-slopes of the Non Hazardous Landfill will be lined with a composite lining system comprising an artificially installed geological barrier and artificial sealing liner. The base and side-slopes of the Hazardous Landfill will be lined with a double composite lining system comprising an artificially installed geological barrier and artificial sealing liner.

- Waste placement will occur in lifts across the width of the individual phases. Within both the Non-Hazardous and Hazardous Landfills, temporary waste slopes are proposed at inclinations of 1V:3H. Within the Non-Hazardous Landfill the maximum depth of waste above the basal lining system will be between 45 and 50m. Within the Hazardous Landfill the maximum depth of waste above the basal lining system will be approximately 17m.

- The groundwater elevations are at least 15m below the base of the proposed development.

- Following the completion of infilling in each phase to pre-settlement levels, the waste slopes will be capped with a composite capping system that will inhibit the infiltration of rain water into the waste.

1.2.1 Basal Sub-Grade Model

The basal sub-grade is to be formed through a combination of cut and fill to achieve base levels with a 1V:50H fall to the low point in each phase.

The basal levels are to be formed in limestone deposits of both the Globigerina Limestone Formation and the Coralline Limestone Formation. The excavation surface is overlain by a minimum 300mm thick layer of crushed and screened limestone fill. The fill layer allows a suitable surface for lining system placement and enables the required basal gradients to be formed.

During 2003 and 2004 SLR undertook site investigations to determine the geological conditions in the area of the proposed development. The results of the ground investigations were summarised within the Ghallis Site Investigation Factual Report[3].

The ground investigations indicated that the Globigerina Limestone Formation (GLF) overlies deposits of the Lower Coralline Limestone Formation (CLF). The GLF is described as pale cream to yellow, moderately strong, fine to medium grained limestone. The CLF is more variable in composition and ranges from brown to light yellow in colour and is weak to moderately strong? and is fissured?. The grain size is typically very coarse.

Field observations of the basal sub-grade indicate the presence of both discontinuities (fissures, fractures and joints) and karstic solution features. A geophysical survey is proposed to investigate the presence or absence of karstic solution features close to the sub-grade surface.

The general fill materials placed will be crushed site derived limestone, placed and compacted in layers to a suitable specification.

1.2.2 Side Slope Sub-Grade Model

As with the basal sub-grade, the side slopes are to be formed through placement of fill against the excavated faces to engineer the formation levels.

The side slopes of the landfills are to be largely formed by excavation of in-situ limestone and placement of a formation layer to achieve formation levels. The eastern side slope of Phases 1A and 1B in the Non Hazardous Landfill will be formed at an inclination of 1V:2.5H to a maximum height of 16m. The upper 9m of the eastern side slopes of Phases 1A and 1B will be formed by a granular rockfill buttress placed against the toe of the Maghtab Landfill waste slope. Below the toe of the rockfill buttress slope, excavation of in situ limestone will be undertaken to achieve the formation levels. Within the Non-Hazardous Landfill the basal formation level of Phases 1A and 1B will be developed at approximately 30m aOD. The basal formation levels of Phases 2 to 4 will be developed at a lower level of approximately 18m aOD. The slope separating Phases 1A and 1B from Phases 2 to 4 will be formed within in-situ limestone at an inclination of 1V:2.5H to a maximum height of 14m. Within the Non-hazardous Landfill, the northern and western side slopes of Phases 2 to 4 are to be formed within in-situ limestone at an inclination of 1V:1H to a maximum height of 31m. Above the crest of the Phase 2 to 4 excavation, a 10m wide bench separates the 1V:1H excavated side slopes from the 5m high, 1V:2.5H inclination, western screening bund slope. The screening bund is to be formed by site derived granular limestone rockfill. The screening bund slope forms the upper northern and western Phase 2 to 4 side slope. The side slopes of the Hazardous Landfill will be formed within in-situ limestone at a maximum inclination of 1V:1.8H to a maximum height of 16m.

Within the Non-Hazardous Landfill, the toe of the excavated side slopes will be buttressed through placement of rockfill. The site derived limestone rockfill buttress will form the lower side slopes in all phases. The details of the lower side slope buttress are shown on Environmental Statement Drawing No. 5/4F[1]. The buttress side slope will be formed to a height of 3m above the base of the landfill and will be engineered at an inclination of 1V:2H.

Above the top level of the rockfill buttress, the rockfill side slope formation layer will be engineered in 3m lifts, in line with waste placement, at an inclination of 1V:1H to a maximum height of 31m above the base of the landfill (western side slope of Phase 4).

1.2.3 Basal Lining System Model

The basal formation levels will be formed through placement of general fill (crushed Limestone) to at least 300mm thickness above the in-situ limestone. The basal lining system will be placed upon the basal formation levels and, within the Non-Hazardous Landfill, will extend 3m up the lower side slope buttress.

For the Non-Hazardous Landfill there are two proposed basal lining systems:

Non-Hazardous Landfill Basal Lining System 1

From the top down, the proposed lining system comprises:

- 500mm thick, clean aggregate leachate drainage blanket
- Geotextile protector
- Double textured HDPE geomembrane
- Geosynthetic clay liner (GCL)
- 500mm thick screened crushed material, maximum permeability $1x10^{-7}$m/s (limestone)
- Minimum 300mm lining formation layer

Non-Hazardous Landfill Basal Lining System 2

From the top down, the proposed lining system comprises:

- 500mm thick, clean aggregate leachate drainage blanket
- Geotextile protector
- Double textured HDPE geomembrane
- 100mm thick mineral liner, maximum permeability $5x10^{-11}$m/s (Trisoplast or similar bentonite enriched, approved material)
- 400mm thick screened crushed material, maximum permeability $1x10^{-7}$m/s (limestone)
- Minimum 300mm lining formation layer

Both of the proposed Non-Hazardous Landfill basal lining systems would extend 3m vertically above the base of the landfill, up the lower side slopes.

From the top down, the Hazardous Landfill basal lining system would comprise:

- 500mm thick, clean aggregate leachate drainage blanket
- Geotextile protector
- Double textured HDPE geomembrane
- Geosynthetic clay liner (GCL)
- Geocomposite drainage layer

- Double textured HDPE geomembrane

- 500mm thick screened crushed material, maximum permeability $1x10^{-7}m/s$ (limestone)

- Minimum 300mm lining formation layer

1.2.4 Side Slope Lining System Model

As described in Section 1.2.3, the Non-Hazardous Landfill basal lining system will extend up the 3m high, 1V:2H inclination, lower side slopes. The lower side slopes are formed by the rockfill buttress placed against the toe of the excavated side slopes. As such, the proposed lower side slope lining systems are equivalent to the two basal lining systems described within Section 1.2.3.

Above the level of the rockfill buttress, the Non- Hazardous Landfill upper side slope lining system, from the top down, will comprise:

- 500mm thick, protector soils

- Double textured HDPE geomembrane

- Minimum 2m thick crushed and screened limestone fill, maximum permeability $1x10^{-7}m/s$

The side slope lining system will be installed in 3m vertical lifts inline with waste infilling.

The Hazardous Landfill side slope lining system from the top down would comprise:

- 500mm thick, protector soils

- Double textured HDPE geomembrane

- Geosynthetic clay liner (GCL)

- Minimum 2m thick crushed and screened limestone fill, maximum permeability $1x10^{-7}m/s$

The Hazardous Landfill side slope lining system will be installed in 3m vertical lifts inline with waste infilling.

1.2.5 Waste Mass Model

Waste will be placed in horizontal lifts across individual phases up to the proposed pre-settlement contours as indicated on Drawing No. SRA 1. Temporary waste slopes will be formed during the future filling operations at an inclination of 1V:3H to a maximum height above the basal lining system of 45m.

1.2.6 Capping System Model

Three capping systems have been proposed for both the Hazardous Landfill and Non-Hazardous Landfill. Two of the capping designs are permanent and will be constructed on pre or partially settled waste of a suitable inclination determined within the relevant section of this Stability Risk Assessment. The third landfill capping design is the use of a temporary cap which will be used on all pre-settled waste slopes of an inclination steeper than acceptable, as defined within the relevant section of the Stability Risk Assessment.

The two permanent designs of the landfill capping system are detailed below:

Option 1:

- 1000mm thick restoration soils
- Geocomposite drainage layer
- 1.5mm textured VFPE geomembrane
- 300mm gravel stabilisation layer
- Waste

Option 2:

- 1000mm thick restoration soils
- 300mm sand drainage layer
- 1.5mm textured VFPE geomembrane
- 300mm gravel stabilisation layer
- Waste

A temporary cap will be used where the gradient of the pre-settled waste is too steep for the permanent capping designs to be utilised. The temporary cap will be constructed by placing the 300mm gravel stabilisation layer, followed by lapped but unwelded sheets of geomembrane weighted down with suitable locally sourced materials.

1.3 Selection of Appropriate Factors of Safety

The factor of safety is the numerical expression of the degree of confidence that exists, for a given set of conditions, against a particular failure mechanism occurring. It is commonly expressed as the ratio of the load or action which would cause failure against the actual load or actions likely to be applied during service. This is readily determined by limit equilibrium slope stability analyses.

Prior to determining appropriate factors of safety for the various components of the model, it is necessary to identify key ‘receptors’ and evaluate the consequences in the event of a failure (relating to both stability and integrity). Consideration of the following receptors is required:

- Groundwater

- Property - relating to site infrastructure, third party property

- Human beings (i.e. direct risk)

The Factor of Safety adopted for each component of the model would be related to the consequences of a failure. Factors of safety adopted for each component are discussed in the relevant sections of this assessment.

1.4 Modelling Approach and Software

In order to perform a comprehensive Stability Risk Assessment, the components of the landfill development, as previously described in Section 1.2 of this document, have to be considered not only individually but in conjunction with one another where relevant. Any analytical techniques adopted for such an assessment should adequately represent all of the considered scenarios (i.e. the different modelled phases of the lifecycle) for both confined and unconfined conditions (where appropriate). The methodology and the software should also achieve the desired output parameters for the assessment (e.g. determination of limit equilibrium factor of safety or calculation of strains within liner components).

The analytical methods used in this Stability Risk Assessment include:

1. Limit equilibrium stability analyses for the derivation of factors of safety for the mineral liner and temporary waste slopes.

2. Finite difference analyses for the determination of displacement along the critical interfaces within the side slope and basal lining systems and the determination of shear strains induced within the Trisoplast materials.

3. Closed-form analyses for side slope stability analyses of the side slope sub-grade, integrity of the upper side slope lining system, stability and integrity of the capping system and unconfined side slope lining system analyses.

The limit equilibrium analyses have been undertaken using:

1. STABLE Version 7.5 (MZ Associates, 1995). The Bishop Slip Circle and Morgenstern-Price Non-Circular methods of analysis
2. SLOPE/W Version 6.1, Build 1320 (Geo-Slope International Limited, 2004). The Bishop Slip Circle and Morgenstern-Price Non-Circular methods of analysis

The proprietary software FLAC, Version 3.30 (Itasca Consulting Group Inc, 1995) has been used for the side slope and basal lining system integrity assessment. This is a two dimensional explicit finite difference programme which simulates the behaviour of structures built of soil, rock or other materials that may undergo plastic flow when their yield limits are reached. Materials are represented by elements, or zones, which form a grid that is adjusted by the user to conform to the shape/cross section of the object (in this case, slopes) being modelled. FLAC was originally developed for geotechnical and mining engineers undertaking studies of the behaviour of geological and similar materials and is therefore well suited for application to the lining system assessment.

The FLAC programme has been used to demonstrate a suitable analytical technique for side slopes in the Guidance. The authors of the Guidance specifically state that the finite difference approach is more suitable for such analyses than the finite element method.

The stability/integrity assessments of the capping system soils and unconfined side slope lining system were undertaken using the methods proposed by Jones and Dixon[4] and Kerkes[5]. The equations developed by these authors were input into Microsoft Excel spreadsheets for processing.

It should be noted that the geotechnical parameters for limit equilibrium and closed-form analyses include the shear strength and unit weight of each material within the model plus pore water or gas pressure assumptions. Shear strength has been defined using the effective shear strength parameters of cohesion, (c′), and the angle of shearing resistance, (ø′).

For the FLAC modelling, effective stress shear strength parameters are used, but with the addition of the elastic properties of the materials (bulk modulus, K, and shear modulus, S) or interfaces (shear stiffness, Ks, and normal stiffness, Kn). Unit weight is not applied for interfaces. Where structural elements have been used to model geosynthetics, the shearing resistance along the edge of the element, $ø_s$, has been assumed to be equal to the shear strength of the interface that it crosses such that the presence of the element does not influence the shear strains along the interface. The other material parameters input for the structural elements are the Young's modulus, E, the yield strength, Y, the cross sectional area, A, and the perimeter, P. These have been adopted from manufacturer's data which complies with EA Guidance[6].

Details of geotechnical parameters adopted for each component of the Stability Risk Assessment are discussed in the relevant sections of the report. For reference purposes, a summary of all geotechnical parameters for materials and interfaces used in the analyses are included in the tables presented in Section 3.0 of this report.

2.0 STABILITY RISK ASSESSMENT

Each of the six principal components of the conceptual stability model has been considered and the various elements of that component have been assessed with regard to stability and integrity. The considerations given to these components effectively address the geotechnical issues which could affect the performance of the low permeability elements of the landfill. Therefore, no further consideration

The principal components considered are:

- The basal sub-grade.
- The side slope sub-grade.
- The basal lining system.
- The side slope lining system.
- The waste mass.
- The capping system.

In each case the component is first considered as part of a risk screening process which essentially consists of a preliminary review to determine the need to undertake further detailed geotechnical analyses.

2.1 Basal Sub-Grade

2.1.1 Risk Screening

The key considerations for the Non-Hazardous and Hazardous Landfill basal sub-grade formed within in-situ limestone overlain by a crushed limestone formation layer, and the implications for stability/integrity, are presented below:

Table SRA 1 Considerations for Basal Sub-Grade

Excessive Deformation	Compressible sub-grade	The in-situ limestones are not subjected to settlements upon loading. The crushed limestone general fill material will be placed in a suitable manor that would not give rise to any significant *differential* settlements across the site. This aspect is not considered to require further assessment.
	Basal heave	There is a considerable thickness (at least 15m) of in-situ limestone bedrock between the minimum elevation of the site and the groundwater table. Therefore, this aspect is not considered to require further assessment.
	Cavities in sub-grade	The presence of karstic solution features within both the GLF and the CLF means there is potential for cavities in the sub-grade. The opening of a void may impact on the stability of the landfill lining systems and as such must be considered. An electromagnetic geophysical survey will be necessary to detect the presence of underlying voids and karstic solution features. Remediation of any voids detected will be required and may involve cutting back and filling with crushed limestone general fill or grouting. Since the electromagnetic survey and subsequent remedial works will eliminate the presence of voids prior to lining system placement, this aspect is therefore not considered to require further assessment.

Since all aspects relating to the stability and integrity of the basal sub grade have been screened out, no further consideration of this component is necessary.

2.2 Side Slope Sub-Grade

2.2.1 Risk Screening

The key considerations for the side slope sub-grade formed by placing general fill materials (crushed and screened limestone), against the excavated faces to engineer the Non-Hazardous and Hazardous Landfill formation levels, and the implications for stability/integrity, are presented below:

Table SRA 2 Considerations for Side Slope Sub-Grade

Fill Slopes	Granular soils	Stability	Side slopes are to be formed through: 1. Placement of crushed and screened limestone fill against the excavated slopes formed in the in-situ limestone. The maximum 1V:1H inclination side slopes are to be formed in 3m high lifts in line with waste placement and are to remain unconfined by waste in the short term (usually less than 12 months). 2. Placement of crushed and screened limestone fill to form the Hazardous Landfill lower side slopes, the Maghtab Landfill toe buttress and to form the western screening bund. In the longer term, the side slopes will be buttressed by waste placement. The unconfined limestone fill side slopes are considered to require further analysis as this is the worst case condition in terms of side slope sub-grade stability.
		Time dependency	See above
		Groundwater	The influence of groundwater on side slope stability is not considered since the regional groundwater table is far below the basal elevation of the quarries (see Section 2.1.1)

The side slopes in the in-situ limestone will be formed through blasting and excavation to produce the required slope profiles. It is envisaged that the methods of formation may result in blasting damage to the rock. As such, the slopes have the potential to be relatively unstable, with a high potential for i) sliding of blocks and wedges ii) toppling of blocks and iii) general ravelling to occur. The progressive placement of a general fill formation layer against the excavated slopes will stabilise the slopes, but it is pointed out that an appropriate Method Statement must be developed prior to buttressing works which will ensure safety during filling operations. Such a Method Statement would normally be provided by the construction works contactor.

2.2.2 Geotechnical Parameters Used in Analyses

Engineered fill side slope sub-grade stability

The only component of the limestone general fill side slope sub-grade requiring analysis is the unconfined stability. It is assumed that this material will comprise essentially granular soils derived from limestone quarrying operations. In the absence of site-specific data for this material, reference has been made to Maksimovic[7], who presents data on the shear strength of a limestone sand. The work indicates that the shear strength is strongly dependent upon the stress regime within the soil mass, with higher shear strength being exhibited for lower normal stresses. Conservative input parameters have been used in Maksimovic's relationship to derive values of the angle of shearing resistance of a granular limestone fill for various normal stresses (since normal stress is dictated by slope height). Appendix SRA 1 presents the resultant chart (Chart SRA 1-1) indicating that a normal stress of 57KPa (slope height of 3m), equates to an angle of shearing resistance of 54^o (53.8^o) and that a normal stress of 171KPa (slope height of 9m), equates to an angle of shearing resistance of 46^o. Within the calculations presented below, for 3m high slopes an angle of shearing resistance of 54^o has been used and for 9m high slopes an angle of shearing resistance of 46^o has been used.

The stability of the limestone fill buttress placed at the toe of the existing Maghtab Landfill will be assessed with regards the potential for instability within the Maghtab waste slope to affect the stability of the Ghallis Landfill side slope formed by the outer slope of the buttress.

The analysis will consider the critical Maghtab Landfill waste slope stability and the potential for a critical slip surface to affect the limestone fill toe buttress stability. In terms of waste strength, SLR adopts conservative values of effective shear strength parameters as derived from a study of geotechnical properties of municipal waste by Van Impe and Bouazza[8], these values being backed up in later work by Kavazanjian et al[9] and later confirmed in a research summary by Jotisankasa[10]. The values for c' and ø' adopted throughout the modelling were 5kPa and 25°, respectively. The unit weight of the waste was taken as $11kN/m^3$, a value slightly higher than that generally adopted ($10kN/m^3$). This is based upon experience gained from some of SLR's most recent modelling and stability work, which indicates that the long term unit weight of waste can reach $13kN/m^3$.

2.2.3 Analyses

Engineered fill side slope sub-grade stability

In the simplest terms, the factor of safety for a slope formed in purely frictional soils is equal to the tangent of the angle of shearing resistance of the soil divided by the tangent of the angle of the slope.

The analysis undertaken considers the unconfined stability of:

1. A 3m high lift of fill placed to form a 45° inclination slope against which the side slope lining system is to be placed.
2. A 9m high 21.8° (1V:2.5H) inclination slope formed by the placement of limestone fill to buttress the toe of the Maghtab Landfill waste slope.

For the 3m high slope, given that the angle of shearing resistance of the soils is at least 54° and at a side slope angle of 45°, the simplified factor of safety reported is 1.38 (tan 54 / tan 45).

For the 9m high slope, given that the angle of shearing resistance of the soils is at least 46° and at a side slope angle of 21.8°, the simplified factor of safety reported is 2.6 (tan 46 / tan 21.8).

Further analyses were undertaken to consider the stability of the Maghtab Landfill waste slope and the potential for a critical slip surface to affect the stability of the unconfined limestone fill toe buttress that partially forms the Ghallis Landfill side slope sub-grade (see Section 2.2.2 above).

An initial sensitivity analysis was undertaken to determine the critical cross section through the Maghtab Landfill waste slope since the waste slope varies in both height and inclination. The sensitivity analyses were undertaken assuming a circular critical slip surface and effective shear strength parameters using computer program STABLE. For all sections considered, the critical slip surfaces were located within the Maghtab Landfill waste slope and would not pass through the Ghallis Landfill side slope sub-grade. Figure A1-1 within Appendix SRA 1 presents the critical cross section analysed.

2.2.4 Assessment

Engineered fill side slope sub-grade stability

Since the slopes will only remain unsupported in the short term (usually less than 12 months) a factor of safety of 1.3 is considered appropriate.

In analysis, a simplified factor of safety of 1.38 was calculated for a 3m high, 45^{o} inclination unsupported fill slope utilising data from Maksimovic[7]. In addition, a simplified factor of safety of 2.6 was calculated for a 9m high, 21.8^{o} inclination unsupported fill slope. The geotechnical parameters used are considered conservative for the unconfined condition representing worst case conditions for side slope sub-grade stability. As such, the analysis undertaken has demonstrated an acceptable factor of safety for worst case conditions.

The analysis of the Maghtab waste slope and toe buttress stability demonstrated that critical slip surfaces would be entirely located within the Maghtab Landfill waste mass and would not pass through the toe buttress that partially forms the Ghallis Landfill side slope.

2.3 Basal Lining System

2.3.1 Risk Screening

The key considerations for the basal lining system and the implications for stability/integrity, are presented below:

Table SRA 3 Stability Components for Basal Lining System

Geosynthetic / mineral	Stability and Integrity	In terms of potential for movements along the basal lining system, the development of the Non-Hazardous and Hazardous Landfill voids in a series of distinct phases will result in the generation of a number of temporary waste slopes. The presence of temporary slopes may result in excessive straining or instability in the basal lining system. Since this issue is dependant upon the geometry of the waste mass and the lining system, this aspect of the stability review is covered under Section 2.5, Waste Mass. In terms of basal lining system integrity relating to the potential for waste mass movements to occur, provided that satisfactory factors of safety exist, the integrity of the lining system will not be compromised.
	Compressible sub-grade	Not applicable (See Section 2.1)
	Cavities	Not applicable (See Section 2.1)
	Basal heave	Basal heave will not be an issue as the base of the proposed Non-Hazardous and Hazardous Landfill development will be constructed well above the regional groundwater elevation. The integrity of the basal lining system will not, therefore, be affected and this issue is not assessed further.

Basal lining system stability and integrity require further analysis and are discussed further in Section 2.5 Waste Mass.

2.4 Side Slope Lining System

2.4.1 Risk Screening

The proposed side slope lining systems include:

1. The two proposed Non-Hazardous Landfill lower side slope lining systems.
2. The Non-Hazardous Landfill upper side slope lining system.
3. The Hazardous Landfill side slope lining system

The key considerations for the side slope lining systems, and their implications for stability/integrity, are presented below:

Table SRA 4 Geotechnical Considerations for Side Slope Lining System

Unconfined	Stability	For the Non-Hazardous Landfill lower side slope lining system 2 (see Section 1.2.4), the 100mm thick Trisoplast or other such suitable mineral liner placed against the crushed and screened limestone formation layer may remain unsupported in the short term. The mineral liner will be placed to a maximum height of 3m against slopes at a maximum inclination of 1V:2H. The stability of the unconfined mineral liner requires further assessment. The stability of the 500mm layer of protector soil which is placed against the side slope lining system ahead of the waste requires further assessment. The analyses undertaken will be used to determine the stability of the protector soils and the degree of tension induced within the geosynthetics.
	Integrity	The integrity of the side slope lining system geosynthetics is considered as part of the protector soil stability assessment (see above).
Confined	Stability	The presence of temporary slopes may result in excessive straining or instability within the proposed lining systems. Since this issue is dependant upon the geometry of the waste mass and the lining system, this aspect of the stability review is covered under Section 2.5, Waste Mass.
	Integrity	In terms of side slope lining system integrity relating to the potential for waste mass movements to occur, provided that satisfactory factors of safety exist, the integrity of the lining system will not be compromised.

2.4.2 Geotechnical Parameters Used in Analyses

Geotechnical parameters for the Trisoplast material proposed for the mineral liner have been adopted from data published by the manufacturer[11]. The lowest residual effective stress parameters reported by the manufacturer are adopted and are considered conservative for short term unconfined conditions. The shear strength parameters for the Trisoplast are a peak angle of shearing resistance of 34^{o} and a cohesion of 12 kPa.

In terms of the unconfined protector soil stability, geotechnical parameters for the protector soils (assumed to be limestone sand similar to the crushed and screened general fill material) have been adopted from Maksimovic[7], who presents data on the shear strength of limestone sand. Residual side slope lining system interface shear strength parameters have been adopted from those published in the Guidance.

2.4.3 Analyses

The Non-Hazardous Landfill basal lining system will extend up the 3m high 1V:2H lower side slope formed by rockfill. Two basal lining systems are proposed for the Non-Hazardous Landfill (see Section 1.2.3). Basal lining system 2 would incorporate a 100mm thick mineral liner of Trisoplast or other similar bentonite enriched soil which would be placed in a suitable manor and remain unconfined in the short term.

Figure A2-1 in Appendix SRA 2 presents the STABLE output plot results for unconfined mineral liner stability analysis. The factor of safety is reported as 1.712 for conservative effective stress parameters (see Tables SRA 10 and 11).

The analysis of unconfined protector soil stability has been undertaken using the method of analysis proposed by Jones and Dixon[4]. All analyses were undertaken assuming conservative peak interface shear strength parameters which are considered appropriate since the protector soils will only remain unconfined in the short term before being buttressed by waste placement.

For the Non-Hazardous Landfill, it is considered that the worst case for unconfined protector soil stability would be a 3m high lift of soil placed against the geosynthetics of the upper side slope lining system. Full input and output details are presented in Appendix SRA 2 Table SRA 2-1.

The analysis demonstrates factors of safety in excess of 1.9 for all interfaces within the Non-Hazardous Landfill upper side slope lining system. The analysis also demonstrates that no tensions will be induced within the lining system geosynthetics.

For the Hazardous Landfill, it is considered that the worst case for unconfined protector soil stability would be a 3m high lift of soil placed against the geosynthetics of the side slope lining system. Full input and output details are presented in Appendix SRA 2 Table SRA 2-2.

The analysis demonstrates factors of safety in excess of 1.5 for all interfaces within Hazardous Landfill side slope lining system. The analysis also demonstrates that no tensions will be induced within the lining system geosynthetics.

2.4.4 Assessment

When considering the short term unconfined stability of the proposed Non-Hazardous Landfill mineral liner, using conservative effective stress parameters, a factor of safety of 1.3 is considered appropriate. The analysis undertaken demonstrates an acceptable factor of safety of 1.712.

When considering the short term unconfined stability of the Non-Hazardous and Hazardous Landfill side slope lining system protector soil when adopting conservative interface shear strength material parameters, a factor of safety of 1.3 is considered appropriate. The analysis undertaken demonstrates acceptable factors of safety for all side slope lining system interfaces considered and also demonstrates that no tensions will be induced within the lining system geosynthetics.

2.5 Waste Mass

2.5.1 Risk Screening

The Hazardous Landfill will accept a range of hazardous wastes that will largely comprise contaminated soils. Waste will be placed across the basal footprint of the developed area of the landfill in even layers with only small temporary waste slopes being formed.

It is considered that the critical areas of the site in terms of waste mass stability and its potential affects upon lining system integrity will be within the Non-Hazardous Landfill.

The waste mass assessment will focus upon the Non-Hazardous Landfill because;

1. relatively high temporary waste slopes will be formed, and

2. the anticipated non-hazardous waste mass settlement of 25% will potentially affect the stability and integrity of the landfill lining systems.

The controlling factors that influence the stability of the Non-Hazardous Landfill waste mass are presented in Table SRA 5 below:

Table SRA 5 Stability/Integrity Considerations for Waste Mass

Failure wholly in waste	Stability		It is considered that the temporary waste slopes represent the greatest risk in respect of a failure occurring solely within the waste mass. Further investigation is therefore required for these slopes.
Failure involving liner and waste	Geocomposite	Stability	The development of the void in a series of distinct phases will result in the generation of a number of temporary waste slopes. The presence of temporary slopes may result in instability of the waste and the underlying lining system. The stability of the waste mass and the underlying side-slope and basal lining systems are therefore considered further within this report.
		Integrity	If the waste mass demonstrates acceptable factors of safety, the integrity of the lining systems will not be compromised.

Leachate and Gas Collection Systems

In terms of waste settlement and its potential effects on leachate and gas collection/control systems, there is no specific discussion within TR1 or TR2 on methods of analysis. This issue is considered largely to be an operational consideration and can be addressed by conservative design or development of mitigation plans at detailed design stage. For example, it is possible to employ telescopic leachate risers, which are specifically designed to overcome waste settlement affects on these installations. This is the case for the Ghallis Landfill. Furthermore, a contingency has been made for any problems that might arise with the leachate risers in that target pads are installed to allow retro-drilling of back-up wells to be undertaken if necessary. Also, it is standard practice for all basal pipework to be designed for a maximum 5% deflection to resist the static forces of the waste.

Leachate recirculation will be undertaken in completed phases, far away from active waste placement and unsupported temporary waste slopes. This measure prevents leachate recirculation from affecting the stability of the unsupported waste mass, considered to represent worst case conditions.

In terms of landfill gas management installations, gas extraction will be provided by combined gas/leachate extraction wells installed within the waste mass and connected to gas collection pipework. The effectiveness of the extraction system will be affected by differential settlement of the waste leading to low spots along the gas carrier mains across previously filled areas. These low spots can lead to collection of condensate which in turn will lead to blockages in the collection system. To minimise the effect of waste settlement on the effectiveness of the gas collection system, gas extraction mains will be installed to suitable gradients across filled areas and condensate sumps will installed at strategic locations. These measures will ensure that the effectiveness of the collection system will not be affected by settlement of the waste mass.

2.5.2 Geotechnical Parameters Used in Analyses

The analyses are undertaken adopting the waste mass shear strength parameters discussed in Section 2.2.2.

The shear strengths of the interfaces present within the basal lining systems have been adopted from those reported within the Guidance. Side slope lining system interface shear strength parameters have been adopted from those published in the Guidance and data published by the Trisoplast manufacturer[11]. A full summary of interface shear strength parameters used in the analyses is presented within Table SRA 11.

The waste shear strength parameters presented within the Guidance are considered conservative and can be considered to already include an element of partial factoring. Therefore, it is considered appropriate to adopt a factor of safety of 1.2 if adopting these shear strength parameters in combination with the Traditional Approach (Section 2.2.4 of the Guidance). A factor of safety of 1.0 is considered appropriate where residual interface shear strengths are applied.

2.5.3 Analyses

In considering the stability of the waste mass, the stability and integrity must also be addressed, as they are intrinsically linked.

Waste Mass Stability

In order to undertake the stability aspect of the waste analysis, three potential modes of failure have been considered, namely:

- Mode 1 critical slip surfaces passing solely through the waste
- Mode 2 critical slip surfaces passing through the waste and along the basal lining system
- Mode 3 critical slip surfaces passing through the side slope lining system and basal lining system

All modes of potential failure described above, have been analysed for the case of a temporary waste slope. The analysis has considered the stability of the components in terms of circular and non-circular 2-D limit equilibrium using the computer program SLOPE/W.

The sections analysed are based upon worst case scenarios for the proposed development. For Modes 1 and 2 the conceptual section incorporates a 43m high, 1V:3H inclination temporary waste slope overlying the basal lining system.

For Mode 3 the critical section is considered to be a north to south cross section across Phase 2 and incorporates:

- A 33m high, 1V:3H inclination temporary waste slope.
- A basal width between the toe of the lower side slope and the toe bund of 66m.
- A 3m high, 1V:2H inclination lower side slope.
- A 19m high, 1V:1H inclination upper side slope.
- The landfill screening bund above the upper side slope.

Whilst it has been assumed that the leachate head on the base of the landfill is controlled to a maximum level of 1m, within the body of the waste pore fluid pressures may exist. Pore fluid pressure is the combined effect of water and gas pressures. The distribution of pore fluid pressure varies within the waste mass due to a number of factors, including; nature of the waste, presence of perched water tables and the presence of a gas extraction system. In order to model the pore fluid pressures in the waste mass, the analysis has assumed that the pore water pressures within the waste will either;

i) simply reflect the basal leachate level or

ii) be represented by a pore water pressure ratio (r_u) of 0.1 to allow for pore fluid pressures to build up within the waste mass above the basal leachate level.

It is pointed out that, in all analyses, the pore pressures acting upon the critical basal lining system interface will reflect the maximum leachate levels i.e. no r_u is used.

The results presented in Table SRA 6 below, represent the calculated factors of safety for a critical slip surface that passes solely through the waste (Mode 1), assuming a circular slip plane and effective stress parameters. Output plots derived from the analyses are included in Appendix SRA 3.

Table SRA 6 Summary of Waste Stability Analysis for Mode 1

Case	Method	Pore Pressure Ratio (r_u)	Factor of Safety	Comments	Figure
1	Drained Circular	0	1.664	Acceptable (Peak FoS>1.2)	A3-1
2	Drained Circular	0.1	1.501	Acceptable (Post peak FoS>1)	A3-2

Cases 1 and 2 (Table SRA 6) were used to assess the reduction in the factor of safety between the anticipated effective stresses for varying pore fluid pressure conditions within the waste mass. In both cases, the factor of safety reported is considered acceptable.

Mode 2 considers critical slip surfaces that pass through the waste and along the basal lining system. The critical interface within the two proposed basal lining systems is considered to be that between geotextile protector and double rough HDPE geomembrane. Both peak and residual shear strength conditions for the basal interface were examined.

The variation of pore fluid pressures in the waste previously used for the investigation of Mode 1 (critical slip surfaces occurring solely within the waste) has been applied to the investigation of Mode 2.

The results presented below in Table SRA 7, represent the calculated factors of safety for Mode 2 analyses, assuming a non-circular slip plane and effective stress parameters. Output plots derived from the analyses are presented in Appendix SRA 3.

Table SRA 7 Summary of Waste Stability Analysis for Mode 2

Case	Method	Pore Pressure Ratio (r_u)	Angle of Shearing Resistance (^{o})	Cohesion (kPa)	Factor of Safety	Comments	Figure
3	Drained Non-circular	0	24	0	1.563	Acceptable (Peak FOS >1.2)	A3-3
4	Drained Non-circular	0.1	24	0	1.236	Acceptable (Peak FOS >1.2)	A3-4
5	Drained Non-circular	0	13	0	1.432	Acceptable (Post Peak FOS >1.00)	A3-5
6	Drained Non-circular	0.1	13	0	1.146	Acceptable (Post Peak FOS >1.00)	A3-6

Cases 3 and 4 assume the interface shear strength parameters along the critical interface in the base of the landfill are at peak values. The stability analysis demonstrated that the factor of safety for this scenario decreases from 1.563 to 1.236 as the r_u value rises, which are both considered to be acceptable.

Cases 5 and 6 assume that the interface shear strength parameters along the critical interface in the base of the landfill are at residual values. Since residual values have been assumed for the critical interface, the allowable factor of safety has been reduced to 1, in line with the recommendations made in the Guidance. The analysis has demonstrated acceptable factors of safety for all cases considered.

Mode 3 considers a critical slip surface that passes through the waste, down the critical side slope lining system interface and along the critical basal lining system interface.

The critical lower side slope and basal interface for the two proposed lining systems is that between geotextile protector and rough HDPE geomembrane. The critical interface within the upper side slope lining system is that between the granular protector soils and the waste mass.

The analyses were undertaken using the peak and residual shear strength parameters for the critical interfaces. The results presented in Table SRA 8 below, represent the calculated factors of safety for Mode 3 analyses, assuming a non-circular slip plane and effective stress parameters. Output plots derived from the analyses are presented in Appendix SRA 3.

Table SRA 8 Summary of Waste Stability Analysis for Mode 3

Case	Method	Pore Pressure Ratio (r_u)	Factor of Safety	Comments	Figure
7	Drained Non-circular	0	1.894	Acceptable (Peak FOS >1.2)	A3-7
8	Drained Non-circular	0	1.138	Acceptable (Post Peak FOS >1.00)	A3-8

Cases 7 and 8 assume peak and residual interface shear strength parameters respectively. The analyses undertaken demonstrated that the critical failure surfaces would not occur within the upper side slope lining system, assuming residual lining system interface shear strength parameters, but would occur within the waste mass and along the critical lower side slope and basal lining system interface.

Acceptable factors of safety are demonstrated for both peak and residual interface shear strengths.

Confined Upper Side Slope Lining System Integrity

It is considered that the integrity of the lower side slope and basal lining system is the critical area requiring analysis. As demonstrated by the limit equilibrium analysis above, critical slip surfaces passing through the waste and along the critical basal and lower side slope lining system interface demonstrate the lower factors of safety and represent the critical case. The mode 3 limit equilibrium analysis demonstrates that, under residual shear strength conditions, critical slip surfaces would not pass along the upper side slope lining system critical interface but would pass through the waste mass and along the lower side slope and basal lining system.

Not withstanding the preceding, a closed form tension calculation has been undertaken for the upper side slope lining system. The calculation is undertaken to demonstrate that the presence of a low shear strength interface above the geosynthtic means that no tensions could be induced within the geomembrane and as such the confined integrity of the upper side slope lining system could not be affected by shearing of the waste mass.

The theoretical approach to the calculation of side slope geosynthetics tension presented within the Guidance has been adopted to calculate tensions induced within the geomembrane of the steep side slope lining system. In simplistic terms, this approach considers the relative shear strengths of the interfaces within the side slope lining system to determine whether or not tension can be induced into the geomembrane. The interface along which sliding between the waste and lining system occurs (protector soils to waste mass) dictates the maximum shear stress that can be transmitted into the underlying geosynthetics.

The results of the analysis are presented within Appendix SRA 3 and demonstrate that due to the presence of a low shear strength interface above, no tensions will be induced within the geomembrane liner.

The crushed and screened limestone formation layer, against which the upper side slope geomembrane liner is to be installed, will be placed to achieve a maximum permeability of 1×10^{-7}m/s. In terms of the confined integrity of this layer, previous numerical modelling undertaken for a similar slope configuration (but greater overall slope height) at the Qrendi Landfill[12] indicated very low levels of deformation (shear strain) within the formation material. At such shear strains there would be an insignificant effect on formation material integrity. This issue is therefore not considered to require further analysis.

Confined Basal and Lower Side Slope Lining System Integrity

The two key areas requiring analysis are;

i) the affect of waste deformations, either as a result of long term settlement or movement towards a temporary waste face in the short term, upon basal and lower side slope lining system integrity, and

ii) the shear strains induced within the Trisoplast lower side slope mineral liner element as it has potential to soften in the long term and move out towards the waste which provides a limited support capacity.

Two finite difference FLAC models were set up to analyse the two key areas described above in the short term and the long term. These models are briefly described below:

a) Model 1: Analysis of deformation within the temporary waste slope and assessment of the displacements along the critical basal and lower side slope lining system interfaces.

b) Model 2: Analysis of the confined lower side slope lining system integrity when overlain by the maximum depth of waste.

Model 1

A finite difference FLAC model has been used for the determination of the basal and lower side slope lining system geosynthetic integrity. Model 1 covers both short and long term performance of these design elements. This model essentially reflects the critical cross section analysed in the mode 3 limit equilibrium analysis above. This effectively represents the short term condition where the toe of the waste is retained by a 2m high intercell bund. The critical cross section was combined with maximum waste displacement. In reality, the maximum waste displacement condition will occur in the long term when the landfill is complete and the waste configuration is more favourable. By combining a worst-case geometry with worst-case lining system interface shear strength conditions, the conclusions reached can be assumed to be conservative.

The key elements of the modelling exercise undertaken are summarised below:

- The FLAC grid incorporates a left hand edge and base which were fixed in the x and y directions as they represent non-moveable boundaries.

- The lower side slope is modelled at an inclination of 1V:2H to a height of 3m above the basal lining system.

- The waste mass was modelled against the lower side slope protector soil, overlying the lower side slope and on top of the basal lining system. The waste mass is modelled to a maximum depth above the lower side slope of 30m.

- The right hand side of the model represents the outer waste slope at an inclination of 1V:3H to a maximum height of 33m above the basal lining system. A 2m high intercell bund, with an internal 1V:2H side slope has been modelled at the toe of the waste slope.

- An interface was placed between the lower side slope lining system protector soil and the waste in order to allow shearing to take place between the low stiffness waste and the higher stiffness protector soil.

- A second interface was modelled between geotextile and double rough HDPE geomembrane, which is considered to be the most critical in terms of stability within the two proposed lower side slope and basal lining systems.

- The stiffness properties of the waste have been selected such that the maximum waste settlement is 25% of the maximum depth.

- All interfaces modelled are assigned residual interface shear strength parameters and as such the modelling is considered conservative.

- It is considered important to model a realistic sequence of events when examining the behaviour of the side slope lining system. Therefore, the waste and soil protector elements have been modelled as being placed in discrete lifts.

The drawings relating to the FLAC analysis are presented in Appendix SRA 3. Figure A3-9 presents the FLAC model used for the analyses, while Figure A3-10 presents the calculated waste displacements.

The critical aspect of geosynthetic integrity for the landfill is whether tensions could be induced into the geosynthetics of the proposed basal and lower side slope lining systems. The geosynthetics will be anchored within a yielding anchor trench that will be designed to give additional material if a tensile force is applied, thus eliminating strains induced by the force. The FLAC analysis undertaken indicates that the maximum displacement along the critical basal and lower side slope lining system interface would be approximately 0.5m (see figure A3-11 within Appendix SRA 3). As long as the anchor trench is able to give sufficient materials to incorporate the anticipated level of displacement, no tension could be induced within the lining system geosynthetics. It is recommended that the anchor trench should be designed to be able to give at least 1m of additional material to incorporate the calculated displacement.

Model 2

Basal and lower side slope lining system 2 (see Sections 1.2.3 and 1.2.4) would incorporate a 100mm thick layer of Trisoplast or similar bentonite enriched soil placed against the lower side slope formation layer. A finite difference FLAC model has been used for the determination of Trisoplast mineral liner integrity. Model 2 addresses the long term performance of this design element. This model adopts the maximum possible depth of waste above the lower side slope lining system of approximately 43m.

This effectively represents the long term condition where waste displacements cause development of shear forces in the mineral liner. The model combines the worst-case waste mass geometry with worst-case leachate and residual lining system interface shear strength conditions, and as such the conclusions reached can be assumed to be conservative.

The key elements of the modelling exercise undertaken are summarised below:

- The FLAC grid incorporates a left hand edge, base and right hand edge which were fixed in the x and y directions to represent non-moveable boundaries.

- A 0.1m thick Trisoplast mineral liner has been modelled up the lower side slope and along the base of the model.

- The waste mass was modelled against the protector soil up the side slope and on top of the basal mineral liner.

- A 0.5m diameter soil protector layer was modelled against the side slope Trisoplast mineral liner element of the model. An interface was modelled between the mineral liner and the soil protector. This interface is present to represent the conditions of the critical interface within the lower side slope lining system considered to be that between geotextile protector and rough HDPE geomembrane. The critical interface is modelled with residual shear strength parameters (see Table SRA 11 for a full summary of interface shear strength parameters used in the assessment).

- A second interface was placed between the lower side slope liner protector soil and the waste in order to allow shearing to take place between the low stiffness waste and the higher stiffness protector soil.

- The maximum depth of waste above the lower side slope lining system for the configuration described is approximately 45m.

- The stiffness properties of the waste have been selected such that the maximum waste settlement is 25% of the maximum depth.

- The initial shear strength of the mineral liner was represented by peak shear strength parameters until the full side slope height had been reached. Following this, the mineral liner was modelled as being fully-softened to represent very long term conditions.

- The shear strength of the critical lower side slope lining system interface was taken as being very low in order to examine the potential for this condition to actually be present as a result of shear displacements along the interface. This is undertaken to avoid the need to model strain-softening interfaces.

The drawings relating to the FLAC analysis are presented in Appendix SRA 3. Figure A3-12 presents the FLAC model used for the analyses, while Figure A3-13 presents the deformed grid and the waste displacements, respectively.

For the purposes of assessing the integrity of the Trisoplast mineral liner (effectively a stiff compacted clay) it is necessary to select a suitable criterion which can relate the model's reported output to permeability. This design criterion requires some understanding of the permeability-strain relationship of a stiff clay. While no material specific data exist with respect to this, research by Arch *el al*[13] has shown that permeability of compacted clays tends to decrease for strains up to the yield point of the material (typically 6%) after which increases in permeability are exhibited. However, values above the original permeability of the compacted clay are only indicated after much larger strains (around 11%).

For the purposes of this report, a design criterion value of 10% strain has been adopted, since this represents a point at which permeability still remains within the as-compacted specification.

When examining the strains within the mineral liner element of the model, zones within the model representing materials around the mineral liner are removed for clarity. Figure A3-14 in Appendix SRA 3 presents the mineral liner shear strains, and indicates that the maximum continuous shear strain across the liner is less than 1%, occurring towards the toe of the slope. This effectively indicates a high factor of safety for mineral liner integrity.

2.5.4 Assessment

Acceptable factors of safety are demonstrated for unconfined waste mass stability for all potential failure considered.

The assessment of confined lining system integrity indicates that the basal and lower side slope lining system geosynthetic integrity would not be affected by waste mass displacements if the yielding anchor trench is designed to give at least 1m of additional geosynthetics (see Section 2.5.3). The analyses also indicate that no significant shear strains will be induced within the 0.1m thick side slope mineral liner.

2.6 Capping System

2.6.1 Risk Screening

The capping system will be designed to control the stresses and ensure compatibility of strains within the various components. The controlling factors that influence the stresses in the capping system are given in Table SRA 9 below.

Table SRA 9 Stability Components of Capping Lining System

Mineral Cap	Stability	Pre-settlement slope inclination	Not applicable.
	Integrity	Compressible waste	Not applicable.
		Slope deformation	Not applicable.
		Construction	Not applicable.
		Cavities in waste	Not applicable.
Geosynthetic /mineral	Stability	Pre-settlement slope inclination	Stability of the lining system requires assessment with regard to interface shear strengths. In terms of the potential influence of gas pressures on the capping stability, gas extraction will be undertaken at the site. This effectively controls gas pressures under the cap and eliminates the potential for any significant pressure to build up beneath the capping system. It is therefore considered that the issue of gas pressure beneath the cap does not require further assessment.
	Integrity	Compressible waste	No external factors will be present to cause anything other than deformations normally associated with waste settlement. Further investigation is not considered to be required.
		Slope deformation	No external factors will be present to cause anything other than deformations normally associated with waste settlement. This aspect is therefore not considered to require further assessment.
		Construction	The potential affects of construction plant activity on the cap during placement of restoration soils need to be considered.
		Cavities in waste	It is proposed that the final waste surface be graded and inspected prior to placement of the regulation layer. This practice will eliminate the potential for near-surface cavities to be present, and this issue does not therefore require further assessment.

2.6.2 Geotechnical Parameters Used in Analyses

The shear strength of the interfaces present within the capping system have been adopted from the values reported within the Guidance and are considered conservative.

In considering the stability of the restoration soils overlying the geosynthetics of the capping system, the influence of possible partial saturation of the soil has been investigated. The analysis models the saturation of the soils overlying the lining system by using the Parallel

Submerged Ratio (PSR). In the analysis the soils are assumed to be placed in a uniform layer over the slope and the phreatic surface of the water within the soil is assumed to be parallel to the slope. The PSR is the ratio of the saturated depth of soils versus the full depth of the soils. Since a drainage layer is included in both proposed designs, drainage from the restoration soil layer will be efficient, consequently a PSR ratio of 0.25 has been assumed.

A factor of safety of 1.3 is considered appropriate for stability calculations which adopt peak shear strength parameters, while a factor of safety of 1.0 is appropriate where residual shear strengths are adopted, in accordance with the Guidance.

2.6.3 Analyses

For each of the permanent capping system options, a sensitivity analysis was conducted to assess the steepest gradient that each capping system could be constructed on and achieve the factors of safety stated in Section 2.6.2 above. The analysis has been undertaken following the methods presented within the Guidance, with the results being presented in Appendix SRA 4 Table SRA 4-1.

The analyses show that the option 1 capping system design is suitable for all gradients at a shallower angle than 15 degrees (1V:3.73H). For option 2 the capping system design is suitable for all gradients at a shallower angle than 20 degrees (1V:2.75H).

Any pre-settlement waste slopes at a steeper angle than 20 degrees will require temporary capping (see Section 1.2.6) until, due to settlement, the waste slope inclination is less than 20 degrees when one of the permanent capping designs can be utilised.

Using the method proposed by Kerkes[5], an assessment of the construction induced loads on the factor of safety has been made. In the analysis it has been assumed that a Cat D6 Low Ground Pressure Dozer has been used to spread the restoration soils up the pre-settlement waste slopes. Analyses were undertaken adopting the shear strength parameters for the critical capping system interfaces as detailed in Table SRA 11 for each of the two permanent capping designs.

The results are presented in Appendix SRA 4, Tables SRA 4-2 and 4-3. For the analysis undertaken a factor of safety of 1.03 can be demonstrated when assuming softened conditions for option 1 capping system design. Whereas, for option 2 a factor of safety of 1.01 can be demonstrated when assuming softened conditions.

2.6.4 Assessment

The analysis undertaken has demonstrated that the option 1 capping system should not be placed on slopes in excess of 15 degrees inclination and that the option 2 capping system should not be placed on slopes in excess of 20 degrees inclination. Any pre-settlement slopes in excess of those detailed above should be temporarily capped with permanent capping undertaken after waste settlement has reduced the slope inclination.

3.0 GEOTECHNICAL PARAMETER VALUES SUMMARY

Table SRA 10 Materials Geotechnical Design Parameters

Material	Bulk Unit Weight γ (kN/m^3)	Effective cohesion c′ (kPa)	Angle of Shearing Resistance ø′(°)	Bulk Modulus K (MPa)	Shear Modulus S (MPa)
Limestone Fill	19	0	46	-	-
Protector soil	19	0	54 - 46	20	10
Waste	11	5	25	0.3 – 0.32	0.15 – 0.16
Trisoplast	20	12	34	12.5	5.8
Restoration soils	18	1	28	-	-

Table SRA 11 Interface Design Parameters

Interface	Peak		Post Peak		Kn	Ks
	c'	ø'	c'	ø'	MPa	MPa
Trisoplast / formation layer	0	30	-	-	-	-
Protector soils / geomembrane	0	30	-	-	-	-
Geomembrane / formation layer	0	30	-	-	-	-
Textured HDPE geomembrane / GCL	0	24	0	14	13	3
GCL / formation layer	0	30	-	-	-	-
Waste mass / protector soils	0	26	0	23	13	3
Restoration soil / geocomposite drainage layer	0	24	0	16	-	-
Restoration soil / sand	0	28	0	25	-	-
Geocomposite drainage layer / Textured VFPE geomembrane	7	25	13	3.6	-	-

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Appendix SRA 1
Side Slope Sub-Grade Analyses Results

Appendix SRA 2
Side Slope Lining System Analyses Results

Appendix SRA 3
Waste Mass Analyses Results

Appendix SRA 4
Capping System Analyses Results

Drawing