CONTENTS

1.0 Stability Risk Assessment

The application for an IPPC Permit requires that an assessment is made of the stability/settlement characteristics of the waste and associated structures (e.g. lining and leachate management systems) and the underlying geological strata to prevent any damage to the barrier systems and ensure no unacceptable discharges. It also requires that the placement of waste be done in such a manner so as to avoid slippage within the waste itself.

This stability risk assessment was developed on the basis of detailed laboratory investigation of rockfill material to be deployed for the construction of Zwejra waste management installation and for geo-synthetic liners the previous experience in designing the interim landfills at Qrendi was utilised. On the basis of available data the model was developed for the evaluation of the strength and durability of construction materials on its own and in co-relation with each other. This document describes the manner in which the assessment has been carried out and presents the overall findings of the work. The enginering formation/foundation material used as non-cemented granular uniform limestone crushed and screened from local geological formation and geo-synthetic material are entirely imported.

The methodology adopted for this Stability Risk Assessment largely follows the principles already observed and implemented in designing the similar waste management facilities on the island like: rehabilitation of quarries at Ix-Xaghra tal-Maghlaq and at Il-Qasam il- Kbir, the Ghallis non-hazardous landfill etc. Other principles observed are outlined in the UK Environment Agency R&D Technical Report P-3852, volumes TR1 and TR2. Where additional analytical techniques have been used, these are described within the text.

2.0 Conceptual Stability Model

The conceptual stability model has been developed based upon the consultations with MEPA and the already developed technique which was designed for the construction of proposed Interim landfills at Qrendi and later utilized in construction of Zwejra – Cell 1 (drawing ZWR 1). The above principles can be summarized as follows:

- The landfill site is to be developed through the cleaning of already deposited mainly inert waste material which was identified as stable and having normal temperature in the site investigation performed by Messrs Scott Wilson. To improve the efficient use of this surface area and to achieve the necessary formation level, excavation of in-situ coralline limestone from Ta’ Zwejra site, in the vicinity of the Maghtab Landfill, shall be encouraged.

- Following the creation of the void which is in total for 2nd and 3rd phase 200,000 cub.m and construction of an access ramp, landfilling in the Zwejra waste installation will be divided into Phases 2 and 3. Phase 2 infilling will take place in 2-3m horizontal layers over the whole of the Phase. Phase 3 will be further subdivided into Phase 3a and Phase 3b with infilling taking place in line with that of Phase 2.

- Following the creation of the formation levels and construction of the new access ramp, infilling of the Zwejra 2 with non-hazardous waste would last for an estimated period of 3 months. Therefore the stability assessment is considered as a short term risk. Landfilling in the Zwejra 3 will continue upon the closure of Zwejra 2 and should last roughly 9 months.

- The base of the landfill is to be formed by cut and fill at existing site location of Ta Zwejra in closed proximity of the Maghtab Landfill. The general fill materials placed will comprise crushed limestone derived from excavation. The general fill will be placed upon in-situ rocks of the Lower Coralline Limestone Formation.

- The side slopes of the landfill are to be formed by placement of general fill against the existing Lower Coralline Limestone Rock to achieve formation levels. The lower side slopes (up to 3 m above the base of the landfill) will be constructed at an inclination of 1V:2H. The upper side slopes will be constructed in 3m lifts, in line with waste placement, at an inclination of 1V:1H to a maximum height of 20m above the base of the landfill.

- The underlying formation is formed in Lower Coralline Limestone. The excavation side slopes are to be formed in the solid limestone rock.

- The groundwater table elevations below the base of the proposed landfill site range between 0.5 and 1.25m above sea level. The lowest elevation of the landfill floor will lie at 18 m above sea level. As such, the unsaturated zone thickness below the floor of both phases is at least 15 m.

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

- Waste placement will occur in 3m lifts across the width of the individual phases.

- 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 and contain the generated gases within the landfill.

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 1(V):50(H) fall to the low point in each phase This is to ensure the natural flow by gravity of any contained liquids. As such, the basal levels are to be partially formed in the Lower Coralline Limestone Formation and partially in general fill materials.

The general fill materials placed will be crushed limestone of the Lower Coralline Limestone Formation, placed and compacted in a suitable manor to achieve stability.

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 where required to engineer the formation levels.

The lower side slopes (up to 3m above the base of the landfill) will be engineered at an inclination of 1V:2H. The upper side slopes will be engineered in 3m lifts with an inclination of 1V:1H to a maximum height of 20 m above the base of the landfill for the Zwejra 3 and maximum height of 5 m for Zwejra 2. On the engineered slopes the Geosynthetic Clay Liner (GCL) will be laid on the bed of lime stone sand of 0.5 m thickness and anchored as to achieve required permeability level of maximum 1x 10 – 9 m/s.

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 Lower Coralline Limestone Formation. The basal lining system will be placed upon the basal formation levels.

The proposed basal lining system from the top downwards comprises:

- 500mm thick, clean aggregate leachate drainage blanket
- Geotextile protector
- Double textured HDPE geomembrane
- Geosynthetic Clay Liner maximum permeability 1x10 -11 m/s
- 500mm thick screened/crushed lime stone sand (tal-franka), then compacted to achieve maximum permeability $1x10^{-7}$ m/s
- Minimum 300mm thick lining formation layer

The basal lining system will extend in general 2 to 3 m vertically above the base of the landfill, up the lower side slopes at 26.6 degrees (1: 1).

The basal lining system will extend for the full slope width and height for the attached slopes of all three landfill cells making it hydraulically identical. It is proposed that three cells will be constructed separate (in phases), but at the end they will present one hydraulically consistent leachate collection system.

2.4 Side Slope Lining System Model

The proposed side slope lining system from (inside the waste deposit area, towards the supporting body) comprises:

- 300mm thick, protector soils
- Geosynthetic Clay Liner max permeability 1x10 -11 m/s
- Minimum 1 m thick crushed / screened and compacted limestone fill

The side slope lining system will be installed in 3 m vertical lifts

2.5. Waste Mass Model

Waste will be placed in horizontal lifts across individual phases up to the proposed presettlement contours as indicated on Drawing No. ZWR 3. During infilling the waste slopes will be formed attached to the 45 degrees steep slopes as abutments for the long term stability. Waste slopes will be formed at an inclination of 1V:3H to a maximum height above the basal lining system of 30m. Temporary waste slopes above 30m high will be formed at an inclination of 1V:5H. There will be a general fall of the final layer to achieve final contour as to serve for minimization of infiltration of rainfall.

2.6. Capping System Model

The concept of capping system is to act as a landfill gas collection and surface water drainage system in the same time reducing infiltration of water. Mineral liner (bentonite – self sealing clay) should serve as an impermeable barrier for the escape of gases. The top 1 m thick layer of restoration soil (enriched compost from Sant Antnin Plant) with its irrigation system will be cultivated for the reason of erosion prevention and visual impact and with purpose to keep saturated upper layer of mineral barrier for enhanced impermeability.

The proposed capping system from the top downwards comprises:

- 1m thick restoration soils
- Water drainage layer – limestone aggregate
- Mineral liner (GCL + limestone fines)
- Protective bedding of shredded tyres
- Gas drainage layer – limestone aggregate
- Stabilization layer (inert waste, crushed/screened)

The capping system will be placed upon pre-settlement waste slopes with a maximum inclination of 1V:3H over a maximum vertical height of 25 m. The bed of shredding tyres will serve as a protective cushion beneath the mineral liner (GCL) as a part of gas drainage layer.

As recommended by the Landfill Directive and on the experience of many other similar facilities the geo-membrane was intentionally disregarded due to well known skid effect. It was found that the single but sufficient mineral liner is absolutely enough for effective containment of released gas and surface water/precipitation barrier.

3.0 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.

3.1 Stability Risk Analysis

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 principal components considered are:

- The basal sub-grade.
- The side slope sub-grade.
- The basal lining system.
- The sides 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.

4.0 Basal Sub-Grade

The key considerations for the basal sub-grade formed in a combination of Limestone of the Lower Coralline Limestone Formation and crushed limestone general fill, and the implications for stability/integrity, are presented below:

- ***Compressible sub-grade:***

The Limestone of the Lower Coralline Limestone Formation is not subjected to settlements upon loading. The crushed limestone general fill material will be placed in a suitable manner that would not give rise to any significant *differential* settlements across the site. Given this, this aspect is not considered to require further assessment.

- ***Basal heave***

There is a considerable thickness (at least 15 m) of Lower Coralline Limestone Formation 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 in the Lower Coralline Limestone Formation means there is potential for minor cavities in the sub-grade. Remediation of any voids detected after excavation will be required and may involve cutting back and filling with crushed limestone general fill. Laying the sub-grade fill and subsequent remedial works will eliminate the risk from presence of voids prior to lining system or waste. This aspect is therefore not considered to require further assessment. Since all aspects relating to the stability and integrity of the basal subgrade have been screened out, no further analyses are considered necessary.

4.1. Side Slope Sub-Grade

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

Stability:

Side slopes are to be formed through placement of crushed and screened limestone fill against the excavated void slopes. The 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 (less than 10 months). In the longer term, the side slopes will be buttressed by waste placement. The unconfined side slopes are considered to require further analysis as this is the worst case condition in terms of side slope sub-grade stability.

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 proposed landfill installation.

4.2. Geotechnical Parameters Used in Analyses

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 support of laboratory delivered data for this material, reference has been made to Maksimovic (5), who presents data on the shear strength of 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). The shearing strength of soil τ_f is described as the relationship between the effective normal stress σ_n' on the failure plane with and *the angle of the shearing resistance* ϕ' , or:

$$\tau_f = c' + \sigma_n' \tan\phi' \qquad (1)$$

For the non-linear failure criterion, the secant angle of the shearing resistance ϕ' depends on the normal stress on the failure plane:

$$\phi' = \phi_B' + \frac{\Delta\phi'}{1 + \sigma_n'/p_N} \qquad (2)$$

In general, with the cohesion term, though for sands *c'=0,* the shearing strength is:

$$\tau_f = c' + \sigma_n' \tan\left(\phi_B' + \frac{\Delta\phi'}{1+\sigma_n'/p_N}\right) \qquad (3)$$

The simple meaning of parameters and the curves is shown in Fig.1, left.

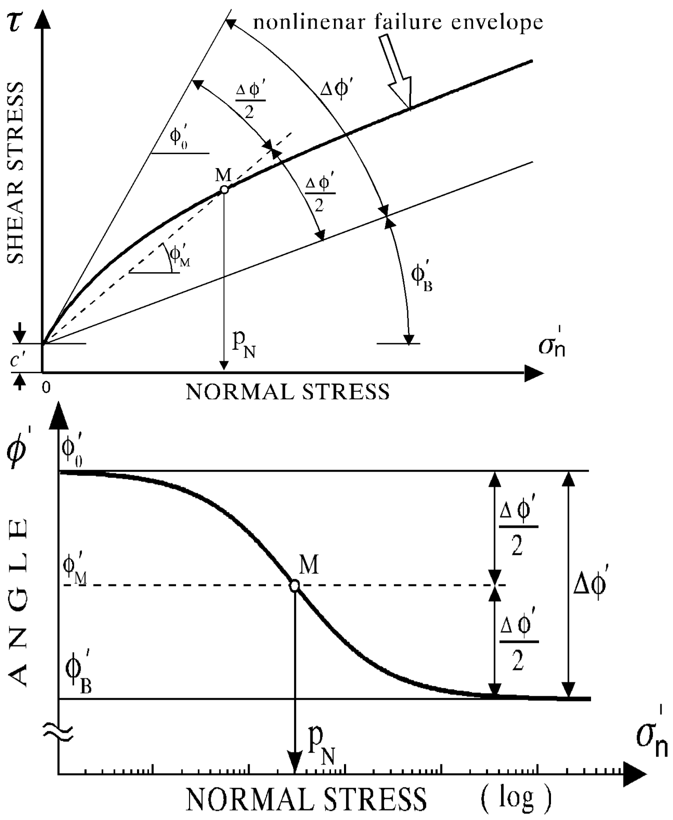


Fig. 1. Parameters of the non-linear failure envelope, in general. Note that for sands c'=0.

In a semi-logarithmic plot (Fig. 1 - right), the point M, corresponding to p_N, is a point of central symmetry and an inflection point on the curve. The initial angle ϕ_0' is an asymptote when the stress level tends to zero, and the basic angle ϕ_B' is an asymptote when the stress level tends to infinity.

In the expression (3) the parameters have the following meaning:

ϕ_B' denotes the basic angle of friction or the angle of friction at a constant volume $\phi'_B = \phi'_{cv}$.

$\Delta\phi'$ denotes the maximum angle difference

p_N denotes the mean angle stress is the value of the normal effective stress for which the secant angle of the shearing resistance equals to the mean value between the initial value $\phi_0' = \phi_B' + \Delta\phi'$ and basic ϕ_B' i.e., for the stress level at which $\phi' = \phi_B' + \Delta\phi' / 2$.

The form of equation (2 and 3) is convenient for interpretation of the conventional direct shear test and application in the limit equilibrium methods.

As indicated on the resultant chart (Chart SRA 1-1) within a normal stress of 57KPa (slope height of 3m), equates to an angle of shearing resistance of 54^o (53.8^o). Through detailed laboratory testing of deployed lime stone sand the results are found to be within expected range. Further more, the laboratory testing conducted at University of Malta - Building and Civil Engineering Department Laboratory exactly matched the theoretical findings giving high degree of confidence. From the produced calculations and Laboratory Report presented below, for 3m high slopes, the value of shearing resistance angle of 54^o has been used.

The results from laboratory testing are given overleaf.

Building and Civil Engineering Department
University of Malta

Laboratory Report

Commissioned by:	Waste Serv Malta Ltd
Test requested:	Determination of shear strength by direct shear (Small shear box)
Standard:	BS 1377-7:1990
Sample Preparation :	BS 1377-7:1990 Section 4.4.4.2
Date:	7/27/2004

Initial moisture content	Wo	
Initial Dry Density		
Initial Bulk Density		0.958
Normal Load on test specimen	kg	10
Normal Stress	k Pa	31.47

Time sec	Horizontal Disp mm	Vertical Disp. mm	Horizontal Load N	Shear Stress kPa
30	0.12	0	8	2.57
60	0.757	0	14	4.49
90	1.391	0	18	5.77
120	1.93	0	21	6.74
150	2.493	0	23	7.38
180	2.979	0	26	8.34
210	3.606	0	28	8.98
240	4.151	0	33	10.58
270	4.695	0	33	10.58
300	4.722	0	33	10.58
330	5.067	0	38	12.19
360	5.623	0	40	12.83
390	6.197	0	43	13.79
420	6.706	0	46	14.75
450	7.25	0	47	15.08
480	7.731	0	51	16.36

510	8.343	0	54	17.32
540	8.91	0	57	18.28
570	9.463	0	58	18.60
600	10.00	0	60	19.25
630	10.54	0	62	19.89
660	11.141	0	65	20.85
690	11.745	0	66	21.17
720	12.296	0	69	22.13
750	12.877	0	71	22.77
780	13.434	0	73	23.42
810	14.709	0	80	25.66
840	15.848	0	84	26.94
870	16.891	0	88	28.23
890	17.531	0	87	27.91
910	17.839	0	82	26.30
940	18.147	0	74	23.74
970	18.455	0	65	20.85

Building and Civil Engineering Department
University of Malta

Laboratory Report

Commissioned by:	Waste Serv Malta Ltd
Test requested:	Determination of shear strength by direct shear (Small shear box)
Standard:	BS 1377-7:1990
Sample Preparation :	BS 1377-7:1990 Section 4.4.4.2
Date:	7/27/2004

Initial moisture content	Wo	
Initial Dry Density		
Initial Bulk Density		0.958
Normal Load on test specimen	kg	20
Normal Stress	k Pa	62.93

Time sec	Horizontal Disp mm	Vertical Disp. mm	Horizontal Load N	Shear Stress kPa
30	0.34	0	27	8.66
60	1.599	0	43	13.79
90	2.744	0	56	17.96
120	3.988	0	67	21.49
150	5.092	0	79	25.34
180	6.224	0	91	29.19
210	7.421	0	104	33.36
240	8.591	0	115	36.89
270	9.728	0	124	39.77
300	10.827	0	133	42.66
330	12.023	0	143	45.87
360	13.086	0	152	48.75
390	14.678	0	164	52.60
420	15.541	0	172	55.17
450	16.595	0	182	58.38

480	17.811	0	190	60.94
510	19.198	0	202	64.79
540	20.762	0	213	68.32
570	22.739	0	222	71.21
600	24.26	0	219	70.25
630	23.12	0	216	69.28

Initial moisture content	Wo	
Initial Dry Density		
Initial Bulk Density		0.958
Normal Load on test specimen	kg	40
Normal Stress	k Pa	125.84

Time sec	Horizont al Disp mm	Vertical Disp. mm	Horizont al Load N	Shear Stress kPa
30	0	0	34	10.91
90	0	-0.002	107	34.32
150	0.9	-0.002	153	49.08
210	2.1	-0.004	196	62.87
270	3.9	-0.025	238	76.34
330	5	-0.045	262	84.04
390	6.7	-0.125	283	90.77
450	8.9	-0.134	305	97.83
510	11.1	-0.144	344	110.34
570	13.4	-0.158	374	119.96
630	15.4	-0.172	413	132.47
690	18.2	-0.177	455	145.94
750	20.3	-0.189	487	156.21
810	22.6	-0.198	515	165.19
870	24.9	-0.198	537	172.25
930	27.6	-0.198	530	170.00
990	29.6	-0.196	568	182.19
1050	12.104	-0.182	565	181.23

Building and Civil Engineering Department
University of Malta

Laboratory Report

Commissioned by: Waste Serv Malta Ltd
Test requested: Determination of shear strength by direct shear (Small shear box)
Standard: BS 1377-7:1990
Sample Preparation : BS 1377-7:1990 Section 4.4.4.2
Date: 7/27/2004

Initial moisture content	Wo	
Initial Dry Density		
Initial Bulk Density		
Normal Load on test specimen	kg	51
Normal Stress	k Pa	160.44

Time sec	Horizontal Disp mm	Vertical Disp. mm	Horizontal Load N	Shear Stress kPa
30	0	-0.004	1	0.32
90	0	-0.03	51	16.36
150	0	-0.025	355	113.87
210	0	-0.058	541	173.53
270	0.785	-0.094	691	221.64
330	1.637	-0.128	816	261.74
390	2.456	-0.153	894	286.75
450	3.339	-0.18	981	314.66
510	4.118	-0.2	1060	340.00
570	5.008	-0.219	1122	359.89
630	5.858	-0.233	1183	379.45
690	6.755	-0.246	1227	393.57

750	7.66	-0.256	1283	411.53
810	8.561	-0.266	1340	429.81
870	9.429	-0.275	1380	442.64
930	10.294	-0.285	1422	456.11
990	11.204	-0.291	1473	472.47
1050	12.104	-0.296	1522	488.19
1110	13.022	-0.301	1567	502.62
1170	13.963	-0.305	1610	516.41
1230	14.487	-0.313	1655	530.85
1290	15.732	-0.319	1693	543.04
1350	16.627	-0.349	1805	578.96
1410	17.431	-0.361	1867	598.85
1470	18.345	-0.373	1912	613.28
1530	19.307	-0.386	1959	628.36
1590	20.188	-0.385	1985	636.70
1650	21.112	-0.392	1989	637.98
1710	21.255	-0.395	1892	606.87
1770	21.268	-0.394	1868	599.17

norm. stress (kPa)	shear stress (kPa)	f (deg)
31.47	28.23	41.8
62.93	71.21	48.5
125.84	182.19	55.3
160.44	637.98	75.8

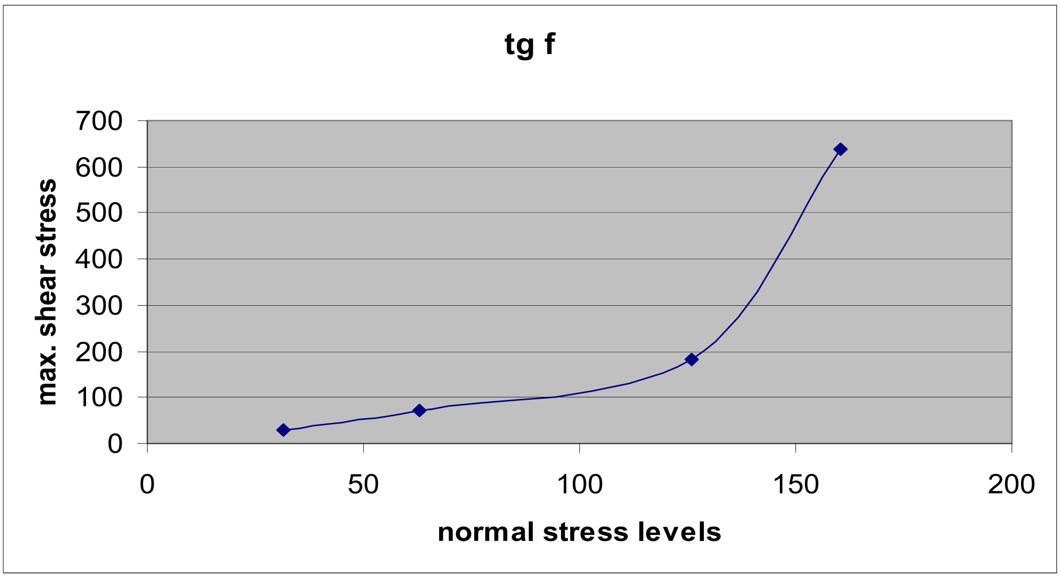
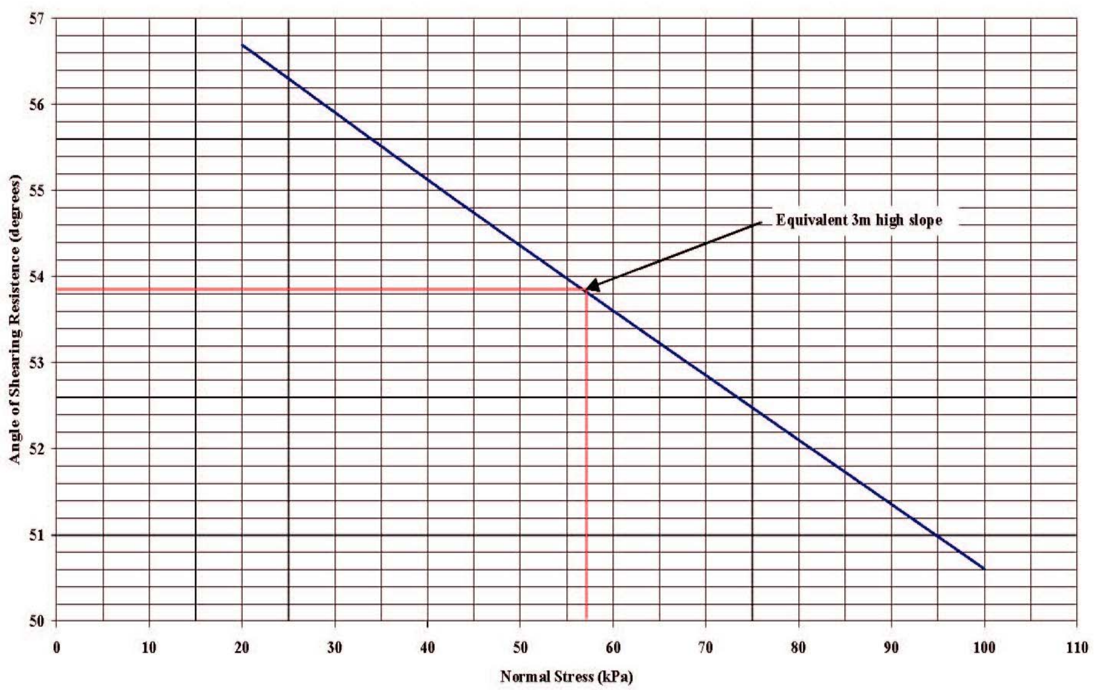


Chart SRA 1-1: Plot of Normal Stress Against Angle of Shearing Resistence Limestone Sand (after Maksimovic 2002)



4.2.1. Analyses

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. Given that the angle of shearing resistance of the soils is at least 54o and at a side slope angle of 45o, the simplified factor of safety reported is 1.38 (tan 54 / tan 45).

4.2.2. Risk Assessment

Since the slopes will only remain unsupported in the short term (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 the unsupported side slope utilizing data from Maksimovic. 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.

5.0 Basal Lining System

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

Stability and Integrity

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.

Geosynthetic / mineral basal heave - Basal heave will not be an issue as the base of the proposed 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.

6.0 Side Slope Lining System

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

Stability

The crushed and screened soft (ta franka) limestone fill placed to at least 2m height to form the base of the lining system for leachate collection system, forming the 1(V):2(H) slope gradient. As such the unconfined stability of this component of the lining system requires no further assessment. The Geosynthetic Clay Liner will be placed against the crushed and screened limestone formation layer and consequently covered with Geo-membrane and Geo-textile.

The mineral liner will be placed in 3m lifts against slopes at a maximum inclination of 1V:1H. The stability of the 500mm layer of protector soil which is placed against the side slope lining system ahead of the waste was discussed in previous chapter: 'Side slope Stability'. The analyses will be undertaken to determine the stability of the protector soils and the degree of tension induced within the geosynthetics (GCL).

The integrity of the side slope lining system geosynthetics is considered as part of the protector soil stability assessment. 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.

6.1 Geotechnical Parameters Used in Analyses

Geotechnical parameters for the materials proposed for the mineral liner have been adopted from data published by the manufacture of GCL: 'Naue Fasertechnik GmbH'. Bentofix® GCLs with a non-woven on both sides typically allows steeper designs than with woven components, due to an increase of the interface friction angle. 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 peak angle of shearing resistance of 34o and a cohesion of 30 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 Maksimovic5, 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 and data published by the manufacturer.

6.2 Risk Assessment

The placement of Foundation layer, the low permeable mineral liner GCL and protective inert, soil layer above the GCL will be done in line with waste placement. The non-woven GCL will be anchored and sealed within the anchor trench formed in solid lower coralline lime stone. When considering the short term unconfined stability of the mineral liner at residual effective stress parameters a factor of safety of 1,2 is considered appropriate. The analysis undertaken demonstrates an acceptable factor of safety of 1.35.

Where: $\tau_f = c' + \sigma'_n \tan \phi'$

131 kPa = 30 kPa + 57tgΦ
tg Φ= 60.7
tg Φ/tg β= 1.35

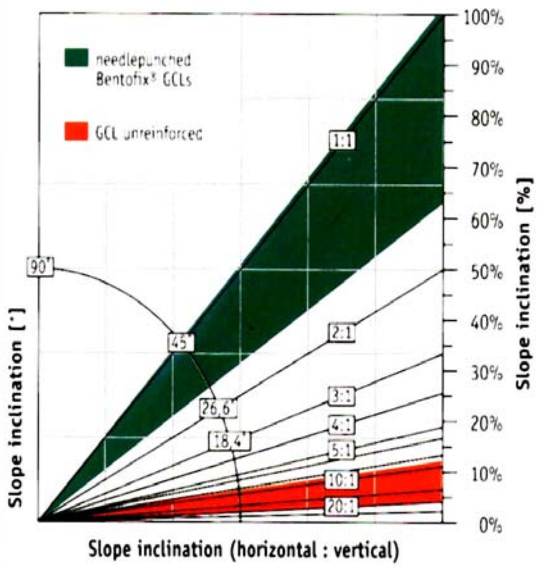
When considering the short term unconfined stability of the side slope lining system protector soil at residual interface shear strength and effective stress material parameters a factor of safety of 1,3 is considered appropriate. The analysis undertaken demonstrates acceptable factors of safety for all side slope lining system interfaces considered and also demonstrates that no tensions will be induced within the lining system geosynthetics.

Examples of interface shear values between different geosynthetics and soil. The indicated approximate values result from over 15 years of project experience. The specific design values must be determined on a project by project basis and follow as close as possible on-site conditions.

	needle-punched nonwoven, e.g. Secutex®	thermally fused nonwoven from special Secudran®	Carbofol® smooth	Carbofol® Orgakron	Carbofol® Karo Noppe	Carbofol® Megakron	Sand 0/2 mm	Gravel 8/16 mm	Mixed grained top soil
Bentofix® cover nonwoven	19°	25°	11°	18°	25°	30°	29°	32°	26°
Bentofix® nonwoven impregnated with bentonite	18°	22°	10°	15°	22°	25°	28°	30°	25°
Bentofix® Thermal Lock carrier nonwoven	27°	-	11°	17°	22°	27°	27°	30°	-
Bentofix® Thermal Lock carrier woven	28°	-	11°	17°	20°	25°	26°	28°	-

Bentofix® with a peel strength of 60 N/10 cm achives at a confining stress of 80 kN/m² (24 h prehydrated under 80 kN/m²) an internal shear stress of approx. 70 kN/m², with a peel strength of 100 N/10 cm even approx. 80 kN/m².

Internal shear angle of needlepunched Bentofix® GCLs for a confining stress up to 100 kN/m².



7.0 Waste Mass

The controlling factors that influence the stability of the waste mass are presented below:

Integrity Considerations for Waste Mass

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.

Stability

The development of the void in a series of distinct phases will result in the generation of a number of temporary waste slopes, in the short term. 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.

Geocomposite Integrity

If the waste mass demonstrates acceptable factors of safety, the integrity of the lining systems will not be compromised.

8.0 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 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. Also, it is standard practice for all basal pipe work 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 horizontal grid of perforated HDPE pipes leading to Maghtab Gas Treatment Compound for further gas stripping and flaring. Gas extraction pipes installed within the last, final layer of waste mass will be embedded in protective surroundings of shredded tyres and connected to active gas collection pipe work. Above this a stabilization layer of inert crushed material will be providing the amortization and balancing for settlement movements. 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 minimize 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.

- ***Geotechnical Parameters Used in Analyses***

In terms of waste strength, conservative values of effective shear strength parameters as derived from a study of geotechnical properties of municipal waste by Van Impe and Bouazza (7), these values being backed up in later work by Kavazanjian et al8 and later confirmed in a research summary by Jotisankasa9. The values for c' and ø' adopted throughout the modeling were 5kPa and 25o, respectively. The unit weight of the waste was taken as 11kN/m3, a value slightly higher than that generally adopted (10kN/m3). 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 manufacturers.

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.

- ***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 liner
- Mode 3 critical slip surfaces passing through the side slope liner and basal liners

All modes of potential failure described above, have been analyzed 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 STABLE.

The section analyzed is based upon a conceptual worst case scenario for the proposed development of the landfill. For Modes 1, 2 and 3 this section is composed of:

- 45m high, 1V:2H inclination temporary waste slope
- basal diameter between the toe of the side slope and the toe bund of 125m
- 40m high, 1V:1H inclination side slope

It is acknowledged that this scenario is unlikely to exist at any stage in the development of Zwejra waste management facility but is analyzed as a conservative case.

The distribution of pore fluid pressure varies within the waste mass, due to a number of factors, including; under drainage, nature of the waste, presence of perched water tables and the presence of a gas extraction system. Two pore fluid pressure conditions have been assigned to the waste mass in the analyses. The first condition assumes that the pore water pressure within the waste is represented by the phreatic surface of the leachate. The phreatic surface is modeled 2m above the basal liner and is considered a conservative approach. The second condition adopts a ru value of 0.1, which essentially represents a more conservative condition that describes the pore fluid pressure regime as a function of the slope height at any given point. This can be used to reflect perched leachate and internal gas pressures within the waste mass.

The results presented 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.

Summary of Waste Stability Analysis for Mode 1
Case Method
Pore Pressure
Ratio (ru) Factor of Safety Comments Figure
1 Drained Circular 1.204 Acceptable (Peak FoS>1.2) A3-1
2 Drained Circular 1.078
Acceptable (Post peak FoS>1) A3-2

Cases 1 and 2 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 basal lining system is considered to be that between the geotextile protector and the 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, represent the calculated factors of safety for Mode 2 analyses, assuming a non-circular slip plane and effective stress parameters.

Summary of Waste Stability Analysis for Mode 2
Case Method
Pore
Pressure Ratio (ru)
Angle of Shearing Resistance (o)
Cohesion (kPa)
3 Drained Non-circular 0 25 6.9 1.377 Acceptable (Peak FOS >1.2) A3-3
4 Drained Non-circular 0.1 25 6.9 1.291 Acceptable (Peak FOS >1.2) A3-4
5 Drained Non-circular 0 13 3.6 1.03 Acceptable (Post Peak FOS >1.00) A3-5
6 Drained Non-circular 0.1 13 3.6 0.944 Unacceptable (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 1.377 to 1.291 as the ru 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 that the calculated factor of safety exceeds the minimum value of 1 for case 5 but does not exceed the minimum value for case 6.

Further analyses were undertaken to determine the maximum allowable height for a 1V:2H inclination waste slope when adopting residual interface shear strength parameters and an ru of 0.1. The analyses demonstrated an acceptable factor of safety of 1.015 for a 30m high temporary waste slope.

Further analyses were undertaken to determine the maximum allowable temporary waste slope inclination when adopting residual interface shear strength parameters and an ru of 0.1.

The analyses demonstrated an acceptable factor of safety of 1.086 for a 45m high, 1V:2.5H inclination temporary waste slope.

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

The critical side slope liner interface is that between the protector soils and the double rough HDPE geomembrane. The critical interface within the basal lining system is that between the geotextile protector and the double rough HDPE geomembrane.

Mode 3 analyses were undertaken on the 45m high, 1V:2H inclination temporary waste slope model adopted for Mode 1 analysis.

The analyses were undertaken using the peak and residual shear strength parameters for the critical interfaces. The results presented below, represent the calculated factors of safety for Mode 3 analyses, assuming a non-circular slip plane and effective stress parameters.

Summary of Waste Stability Analysis for Mode 3
Case Method
Pore Pressure Ratio (ru)
Factor of Safety
Comments Figure
7 Drained Non-circular 0 3.385 Acceptable (Peak FOS >1.2) A3-9
8 Drained Non-circular 0 1.987 Acceptable (Post Peak FOS >1.00) A3-10

Cases 7 and 8 assume peak and residual interface shear strength parameters respectively.

Acceptable levels of stability are maintained for both peak and residual interface shear strengths. At the factors of safety reflected, shear movements of the waste mass will be insignificant and therefore the integrity of the basal and lower side-slope lining systems will not be compromised.

Summary of Unconfined Waste Mass Analyses

The cases of Modes 1 and 3 for a 45m high 1V:2H slope, proved to be stable. However in residual conditions with an ru value of 0.1, failure through the waste and along the basal lining system was shown to be feasible (Mode 2, Case 6). Additional analyses showed that two options are available:

1) Limit the temporary waste slope at 1V:2H to a height of 30m (Case A3-7), or,
2) Limit the inclination of the 45m temporary waste slope to 1V:2.5H (Case A3-8).

Confined Basal and Side Slope Lining System Integrity

The two key areas requiring analysis are;

i) the strains and tensions induced in the lining system geosynthetics as a result of waste deformations, either as a result of long term settlement or movement towards a temporary waste face in the short term and
ii) the shear strains induced within the GCL mineral liner element of the side slope liner as it has potential to hydrate in the long term and reduce interfacial shear strength which provides a limited support capacity.

Three finite difference FLAC models were set up to analyze 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 tensions and linear strains induced in the basal lining system close to the inter cell bund toe.
b) Model 2: Analysis of the confined side slope lining system integrity when placed against the 1V:1H upper side slope.
c) Model 3: Analysis of the confined side slope lining system integrity within the 1V:2H lower side slope.

Model 1

Model 1 addresses the integrity of the basal liner. A finite difference FLAC model has been used for the determination of the basal lining system geosynthetic integrity. Model 1 covers both short and long term performance of these design elements. This model adopted an elongated basal width so that the shear strains induced around the basal liner and intercell bund would be unaffected by waste displacements near the side slope. The waste was modelled with a temporary slope formed at a gradient of 1V:2.5H to a height of 45m (as determined through Mode 2 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. This worst case free standing temporary waste slope condition 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 favorable. By combining a worst-case geometry with worst-case leachate and lining system interface shear strength conditions, the conclusions reached can be assumed to be conservative.

The key elements of the modeling exercise undertaken are summarized 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 side slope components were modeled with stiff material and strong interface parameters as the area of interest was in the basal lining system around the intercell bund.
- The waste mass was modeled against the protector soil up the side slope and on top of the basal clay layer.
- An interface was placed between the 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 right hand side of the model represents the outer waste slope at an inclination of 1V:2.5H. A 2m high intercell bund, with an internal 1V:2H side slope has been modeled at the toe of the waste.
- A second interface was modeled between the geotextile protector and the double rough HDPE geomembrane, which is considered to be the most critical in terms of stability within the basal lining system.
- The maximum depth of waste above the basal lining system for the configuration described is 25m.
- The stiffness properties of the waste have been selected such that the maximum waste settlement is 20% of the maximum depth.
- It is considered important to model a realistic sequence of events when examining the behavior of the side slope lining system. Therefore, the waste and soil protector elements have been modelled as being placed in discrete lifts.

The critical aspect of geosynthetic integrity for the landfill is tension induced into the geosynthetics of the basal lining system. The geosynthetic that exists (rough HDPE geomembrane), have been modelled using residual interface shear strength parameters (ø'=13o, c'= 3.6).

Figure A3-14 indicates that the tension induced in the basal geosynthetics is less than 1kN. This level of tension is insufficient to allow the yield strength of the geosynthetic (generally between 20 and 30kN, depending upon the geomembrane specification) to be exceeded.

Furthermore, the tension is low enough to confirm that any strain within the material will be insignificant. It can be deduced that a minimum factor of safety of 12 against rupture will apply.

Model 2

A finite difference FLAC model has been used for the determination of:

i) side slope geosynthetic integrity and
ii) side slope mineral liner integrity. Model 2 addresses the long term performance of these design elements. This model adopted a 50m basal width for the waste, with the side slope modelled at a height of 20m and at a gradient of 1V:1H.

The right hand edge of the model was fixed.

This effectively represents the long term condition where waste displacements cause development of shear forces in the mineral liner. By modeling the 1V:1H slope over a 3m vertical height, the model is considered conservative as this scenario is unlikely to exist in reality. By combining a worst-case geometry with worst-case leachate and lining system interface shear strength conditions, the conclusions reached can be assumed to be conservative.

The key elements of the modeling exercise undertaken are summarized 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 0.5m diameter soil protector layer was modeled against the side slope mineral liner element of the model. An interface was modeled between the mineral liner and the soil protector. This interface is present to represent the conditions of the critical interface in the side slope lining system considered to be that between the protector soils and the rough HDPE geomembrane. The critical interface is modeled with residual shear strength parameters (ø'=28o, c'= 0).
- The waste mass was modeled against the protector soil up the side slope and on top of the basal mineral liner.
- A second interface was placed between the 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.
- A third interface was modeled between the geotextile protector and rough HDPE geomembrane, which is considered to be the weakest within the basal lining system.
- The maximum depth of waste above the basal lining system for the configuration described is approximately 25m.
- The stiffness properties of the waste have been selected such that the maximum waste settlement is 20% of the maximum depth.
- It is considered important to model a realistic sequence of events when examining the behavior of the side slope lining system. Therefore, the waste, mineral liner and soil protector elements have been modeled as being placed in discrete lifts. Furthermore, 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 modeled as being fully-softened to represent very long term conditions.
- The shear strength of the side slope 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 below.

The result indicates the degree of tension induced within the side slope and basal geosynthetics represented by structural elements within the FLAC model. The plot indicates that the tension induced in the basal geosynthetics is negligible, while the tension induced within the lower side slope geosynthetics is reported as being less than 1kN. This level of tension is insufficient to allow the yield strength of the geosynthetic (generally between 20 and 30kN, depending upon the geomembrane specification) to be exceeded. Furthermore, the tension is low enough to confirm that any strain within the material will be insignificant. It can be deduced that a minimum factor of safety of 12 against rupture will apply. For the purposes of assessing the integrity of the mineral liner 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 clay. While no material specific data exist with respect to this, research by Arch *el al*10 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. The result 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 factor of safety greater than 10 for mineral liner integrity.

Model 3

The geometry of the lower side slope is different to that of the upper side slope, in that a 7m high, 1V:2H slope is considered to be the worst case slope configuration. Waste displacements could affect the integrity of the side slope lining system. A finite difference FLAC model was set up to represent these conditions, and assess the tension induced in the geomembrane liner caused by waste displacements.
The modeling exercise undertaken is comparable to that for Model 3 except that the side slope modeled at a height of 7m at a gradient of 1V:2H, above which waste is too placed to pre-settlement contours.

The critical aspect of geosynthetics integrity for the landfill is tension induced into the geosynthetics of the lining systems. The result indicates the degree of tension induced within the side slope and basal geosynthetics represented by structural elements within the FLAC model. The plot indicates that the tension induced in the basal geosynthetics is negligible, while the tension induced within the lower side slope geosynthetics is reported as being less than 1kN. This level of tension is insufficient to allow the yield strength of the geosynthetic (generally between 20 and 30kN, depending upon the geomembrane specification) to be exceeded. Furthermore, the tension is low enough to confirm that any strain within the material will be insignificant. It can be deduced that a minimum factor of safety of 12 against rupture will apply.

Assessment

Acceptable factors of safety are demonstrated for unconfined waste mass stability for all potential failure modes assuming temporary waste slopes do not exceed inclinations of 1V:2.5H up to pre-settlement contours or do not exceed 30m in vertical height at an inclination of 1V:2H.
The assessment of confined lining system integrity indicates that no significant tensions will be induced in the lining system geosynthetics due to waste displacements. The analyses also indicate that no significant shear strains will be induced within the 0.1m thick side slope mineral liner.

9.0 Capping System

The capping system will be designed to control the stresses and ensure compatibility of strains within the various components. The main function of proposed lining design is to ensure no infiltration of surface water and to efficiently prevent escape of LFG in the atmosphere. Additionally it is to give a solid stable support for planting and re-cultivation of the area. The controlling factors that influence the stresses in the capping system are given below.

- ***Stability Components of Capping Lining System***

Stability Pre-restoration slope inclination
The relatively steep pre-settlement slopes present at the site dictate the need to undertake further assessment of this aspect of the design. It is pointed out that gas pressures underlying the capping system are not considered as part of this Stability Risk Assessment since active gas extraction will take place.

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.

Tyre shreds

When used as leachate / gas drainage layer material, should not significantly impact stability of the landfill. Available published data on shear strength of tire shreds indicates a wide range of shear strength properties for tire shreds and tire shred/soil mixtures. The data are from varying test types and test conditions [Bressette, 1984; Edil and Bosscher, 1992; Humphrey et al., 1993; Ahmed, 1993; Humphrey and Sandford, 1993; Benda, 1995; Benson and Khire, 1995; Cosgrove, 1995; Andrews and Guay, 1996]. The range of values indicates that tire shreds and tire shreds/granular soil mixtures have shear strengths at least comparable to typical values of MSW. Since landfill liner systems generally have critical layer interfaces weaker than MSW the use of tire shreds should not have a detrimental effect on landfill stability.

Geosynthetic Integrity 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 geosynthetics during placement of restoration soils has been 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.

- ***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 mineral drainage layer is included below the restoration soils for the proposed capping system, drainage from the restoration soil layer will be efficient, consequently a PSR ratio of 0.25 has been assumed.

- ***Analyses***

The results of the capping stability analysis for the proposed capping system. On slope inclinations lower than 1V:4H (15^{o}), the results demonstrate factors of safety in excess of 1.3 and 1.0 for peak and residual drained interface shear strength parameters respectively. For the proposed capping system design, no tensions are induced in the capping system geosynthetics in any of the cases considered.

Using the method proposed by Kerkes4, 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 maximum allowable pre-settlement slope of 1V:2.5H. The analysis demonstrated that a factor of safety of 1.33 against rupture of the capping system geosynthetics is achieved when using the peak parameters and zero pore water pressure, which would be applicable as the materials are just placed and no time has passed for the materials to soften or saturate. The result demonstrates that the factor of safety drops to 1.02 when adopting residual shear strength parameters. In all cases considered, acceptable factors of safety are demonstrated.

- ***Assessment***

A minimum factor of safety of 1.3 is considered appropriate and has been adopted where peak interface shear strength conditions are applied for the pre-settlement slopes. A factor of safety of 1.0 is considered appropriate where residual interface shear strengths are applied. All cases modeled demonstrate acceptable factors of safety. It is acknowledged that the proposed pre-settlement contours have slopes that exceed the inclinations modeled. In order to give a satisfactory maintaining level of stability for the top soil cover and an efficient hydraulical separation between surface water drainage and gas drainage system there will be installed a controlled irrigation system. The main objective of control watering is to support development of rooting system for the planted vegetation allowing high degree of cohesion and in the same time keeping an acceptable level of hydratation for mineral liner.

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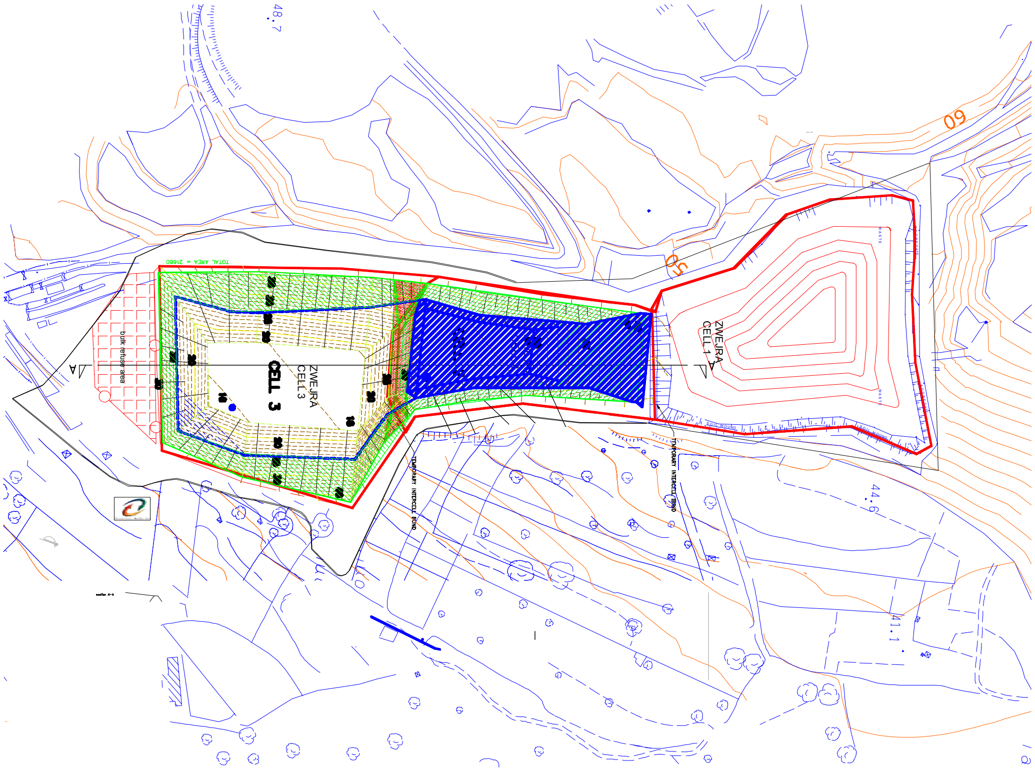
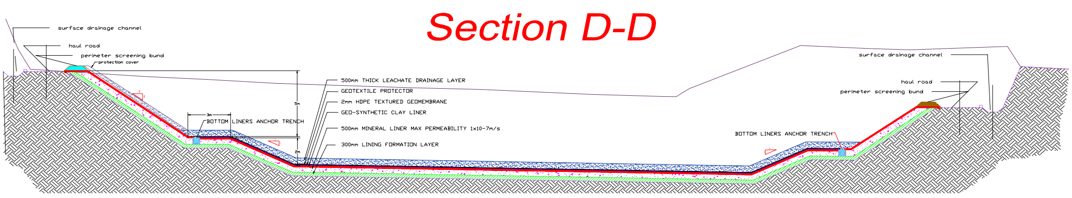
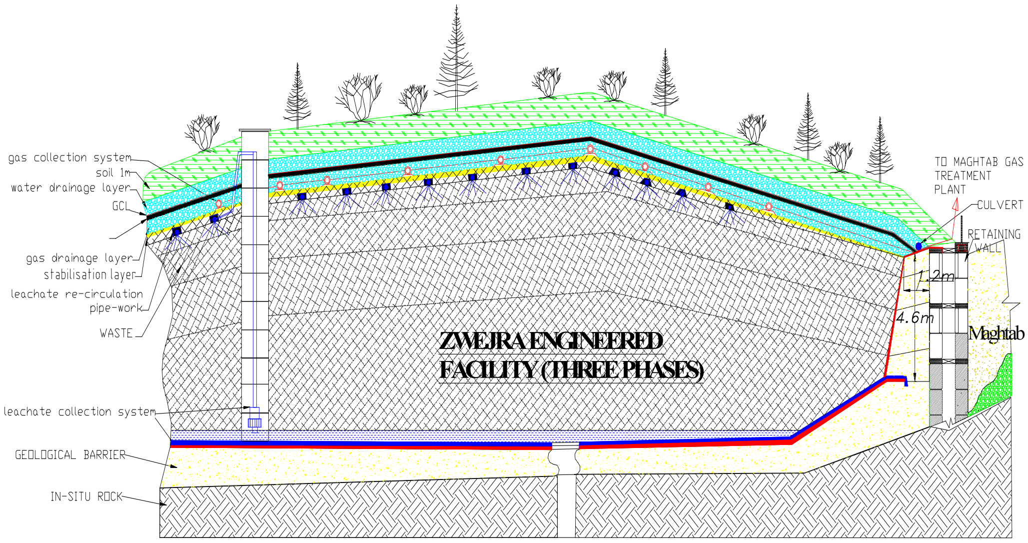
Drawing ZWR 1 (Site Plan)



Drawing ZWR 2 (Cross Section)



Drawing ZWR 3 (Capping)



Drawing 4 (Infilling Phases)