Stability risk assessment (SRA)

Alteration to the internal lateral landfill profile to extend the Ghallis landfill capacity

STABILITY RISK ASSESSMENT
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1.0 INTRODUCTION

1.1 Report Context

This report supports the application by "Frisoli s.r.l." for an Environmental Permit to the alteration to the internal lateral landfill profile to extend the Ghallis landfill capacity, Malta.

The project involves the use of compacted waste, using engineered reinforcement to create retaining structures for waste. This wall would have a steeper profile than that currently implemented, and extend along the Ghallis landfill. The retaining wall would not involve any interventions in the old Maghtab landfill, and would provide the Ghallis landfill with a capping layer in the lateral part as required by the Landfill Directive 1999/31/EC. No increase in height beyond the permitted limits is being contemplated.

These engineering works would extend the Ghallis landfill lifetime, by increasing void space by circa 350,000 m3. Implementation of the project would be carried out by Frisoli s.r.l., an Italian company specialised in landfill engineering, and that owns the patents pertaining to the technologies that allow the construction of such retaining walls in landfills with an inclination of 70°.

The site has been historically used as an engineered landfill facility for the disposal of non-hazardous wastes, and is part of the Maghtab waste management complex. The latter is dedicated to the disposal needs of all non-hazardous waste streams generated in Malta, or to the diversion of waste streams to recovery or recycling processes in other permitted facilities.

This facility was designed as a disposal facility that implements the requirements of Directive 1999/31/EC on the landfill of waste as transposed by Legal Notice 168 of 2002 Waste Management (Landfill) Regulations. The landfill facility was originally approved for development by PA 04834/04 after an Environmental Impact Assessment process. Several development permits on site were required to allow various modifications and upgrades as part of a Master Plan for the Maghtab Environmental Complex, which was assessed via an update to the original EIS (GF 00121/06). The operations of this facility were originally permitted on the 6th April 2007 through the issue of the integrated pollution prevention and control permit IP001/06/A; this permit's renewal was decided on 31st January 2013 through the issue of IP001/06/B.

The construction of the landfill proceeded in phases consisting of independent cells, and certified via Construction Quality Assurance reports that were prepared during the construction of each cell. The engineering specifications were derived from the results of hydrogeological, landfill gas and stability risk assessments, to ensure that operations at the installation would not result in an adverse effect on the surrounding environment. Each cell has its own leachate collection/extraction system, as well as a gas extraction system connected to a central gas management facility. At the moment, the construction of the final cell is currently being completed, and the gas extraction system that was the subject of the IPPC permit renewal in 2013 is being implemented.

1.2 Conceptual Stability Site Model

The conceptual stability model (Appendix A) has been developed by “Frisoli s.r.l.” based upon the project proposal regarding the capacity extension of Ghallis landfill, Malta.

These engineering works would extend the Ghallis landfill lifetime by increasing void space by circa 350,000 m3 (see sections in Appendix C).

The proposal model consists of:

- A side-slope lining system constituted of (from bottom to up) a geological barrier, geocomposite bentonite layer (GCL), HDPE geomembrane sheet and a nonwoven geotextile;

- A “foundation” constituted of a layer of mixed stabilized material wrapped with a woven geocomposite

- A Retaining structure (Side-Cap) with an angle of 70° constructed according to the “Refuse dump containment structure®” of Frisoli EP 1661635 A1 (European patent).

The structure will be constructed around the eastern perimeter of the waste mass for about 667m.

1.2.1 Basal Sub-Grade Model

Not relevant.

1.2.2 Side Slopes Sub-Grade Model

Re-engineering of the new cell includes modification of the existing eastern cell by excavation of part of the existing perimeter road in order to guarantee the continuity of the liner of the existing cell with the new cell.

After guaranteeing the appropriate space for the realization of the works, in order to ensure the continuity of the existing side slope liner with the new one, a careful excavation will be realized with the aim to unearth the existing lining system. The depth of excavation depends on how deep is the HDPE into the ground. Field observations and trial pits indicate that the existing lining system is at a shallow depth (typically around 1m deep and anticipated to be no greater than 3m deep: see photo below).

Figure 1: Trial pit

In-situ limestone overlain by inert fill materials (crushed rock and fines) constitute the existing side slopes sub-grade of non-hazardous Ghallis landfill in the area proposed for increasing volume capacity.

The existing uneven side-slope will be 'regularised' by the addition of further inert fill material (crushed rock and fines) to achieve the final levels required for extension of the side-slope liner and construction of the new capping system.

Since there is a slope in the longitudinal profile of the side-slope area, a stepped solution is necessary to permit uniformity and continuity for the base of the new capping structure.

A prefabricated T-wall is to be installed on the upper surface of the inert fill. The T-wall has the main function to delimit Ghallis from Maghtab and serve as an anchor point for the various layers of the liner system.

A 3m wide access road to separate the proposed line of intervention from Maghtab landfill's boundary, fundamental for preserving the independence of Maghtab landfill from the project proposal, is planned. (Appendix B)

1.2.3 Basal Lining System Model

Not relevant.

1.2.4 Side Slope Lining System Model

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

- 500mm thick, protector soils
- Double textured HDPE geomembrane
- Minimum 2m thick crushed and screened limestone fill, maximum permeability $1x10^{-7}$m/s

According to the 1999/31/EC, the proposed new side slope lining system will be realized as below (from bottom to top):

- Crushed and screened material, maximum permeability $1x10^{-9}$m/s (the Geological Barrier) some of this is currently in situ and more may need to be placed where missing or levels are to be raised;

- Geosynthetic clay liner (GCL) (thickness = 6 mm);

- HDPE geomembrane (thickness = 2 mm);

- Nonwoven geotextile (400 g/m^2);

- Protector sand layer (0.5 m).

The connection to the existing side slope liner is guaranteed by welding the existent HDPE geomembrane, unearthed by careful excavation, with the new one. In order to protect the proposed lining system, on the side of the excavation, it is provided a layer of sand (thickness = 0,5 m) and a layer of inert materials are to be backfilled into the area excavated into the waste to reach the foundation level below the foundation capping structure (see Appendix A).

The geosynthetic elements of the lining system will be put over the side of the excavation, below the foundation layer and over the T-wall for anchorage.

1.2.5 Waste Mass Model

The waste mass comprises three elements:

- The existing in situ waste; mostly MSW with some inert waste;
- The waste to be utilised as part of the retaining structure construction (that is the selected material to be compacted between the geogrid layers); and
- The waste to be placed between the slope excavated into the existing profile and the rear face of the "structure of waste".

The space between the capping structures and the existent landfill body will be filled with the same type of waste material and at the same time as the construction of the reinforced waste structure.

The selected waste materials to be utilized are those generated from "Malta North" (Mechanical and Biological Waste Treatment Plant).

1.2.6 Capping System Model

The capping system for non-hazardous Ghallis landfill, as per PA 04834/04 and approved SRA (Stability Risk Assessment), is (from the top down):

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

This arrangement will be used for the upper surface of the landfill.

According to the “Refuse dump containment structure®” of Frisoli EP 1661635 A1 (European patent), materials of the capping system on the lateral side of the structure will be conform to the European legislation requirements and are equivalent to the approved capping system.

The permanent design of the proposed landfill capping system for the side slope of the structure is (from the outer surface inwards):

- Geo-net layer supported by an electrowelded mesh (see characteristics and volume in the following figure) with a HDPE monofilament nets;

- Restoration soil (thickness = 1000 mm);

- Geocomposite drainage layer for rainwater (thickness = 22 mm);

- Mineral layer (minimum permeability 1 x 10^{-9} m/s) (thickness = 500 mm);

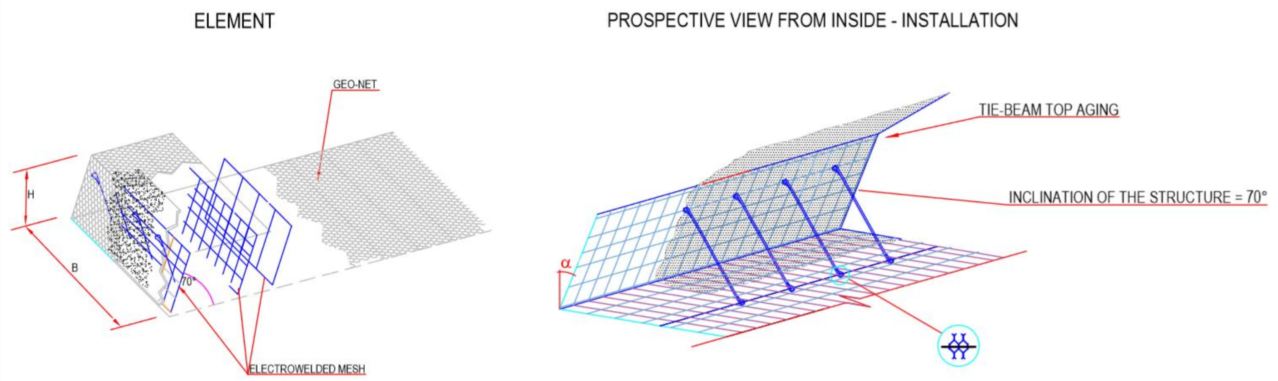
- Geocomposite drainage layer for landfill gas (thickness = 22 mm);

- Selected waste.

The capping system is part of the structure and represents the outer portion of the wall.

Figure 2: External capping over the retaining structure for waste

The outer layers of the capping structure are inclined with respect to ground level by 70°. This is structurally stable due to the reinforcement methodology used in the structure itself.



	α	H (m)	B (m)	TYPE OF MESH	DIAMETER	VOLUME (tot)
TERRAMESH external	70°	1	667	15 x 15	Ø 8mm	24,62 m3
TERRAMESH for drainage layers				20 x 20	Ø 6mm	

UNI EN 10223-3

Figure 3: Electrowelded mesh: characteristics and total volume

2.0 STABILITY RISK ASSESSMENT

2.1 Risk screening

Each of the six principal components of the conceptual stability site model has been considered and the various elements of that component have been assessed with regard to stability.

The principal components considered for this case are:

- Basal Sub-Grade;
- Side Slopes Sub-Grade;
- Basal Lining System;
- Side Slope Lining System;
- Waste mass;
- 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.

2.1.1 Basal Sub-Grade Screening

Not relevant.

2.1.2 Side Slopes Sub-Grade Screening

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

Considerations for Side Slope Sub-Grade (Unconfined)		
Excavation for unearth the existent liner system	Stability	Field observations and trial pits suggest that it will not be necessary for deep excavations due to the fact that for the majority of the proposed intervention area the existent lining system is at a shallow depth (probably less than 3m). The material is compacted inert material (crushed rock and fines) and is considered will be stable in the short term at slopes up to 2m high of up to 1v:1h subject to inspection and proof rolling.
	Groundwater	The influence of groundwater on side slope stability is not considered since the regional groundwater table is far below the base of the landfill.
Fill	Stability	Fill slopes are not expected to exceed 1m to 2m and will be formed of well compacted inert material (crushed rock and fines) and are expected to be stable in the short term at slopes of up to 40°.

Prefabricated concrete T-wall		Prefabricated concrete T-wall has the main function to delimit Ghallis boundary to Maghtab and to serve as an anchor point for lining system. These segments will be placed on compacted fill. No further assessment is required.

Where the combined total height of the excavated side slope and the filled placed to make up levels to regulate longitudinal slopes is 3m or greater an analysis is required.

Once the excavation to expose the existing liner (and the liner is extended) is backfilled (with compacted inert material) its long term stability will be increased; this requires no further assessment.

2.1.3 Basal Lining System Screening

Not relevant.

2.1.4 Side Slope Lining System Screening

Considerations for Side Slope Lining System; Geological barrier and Geosynthetic components (including anchorage)		
Unconfined	Stability	The side slopes will be at an angle of max. 30°. Further analysis of geological barrier needed to be assessed .
	Geosynthetic integrity failure	Further analysis needs to be assessed.
	Stability and integrity in the excavation	Further analysis needs to be assessed.
	Anchorage	Further analysis needs to be assessed.
Confined	Stability	The buttressing effect of the backfilled inert material in front of the lining system will increase its stability. This aspect needs no further assessment.
	Integrity	In terms of side slope lining system integrity relating to the potential for waste mass movements to occur as a result of induced compression and settlement caused by the significantly increased loading (and strains induced) in both the existing down-slope and the newly installed

		lining system needs further assessment.

2.1.5 Waste Mass Screening

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

Considerations for Waste Mass		
Stability	Settlements	Due to the particular methodology used by Frisoli which permits to improve waste compaction and available space for waste disposal, it has considered the differential settlements related to the existent body landfill and the proposed extra-capacity waste as not relevant. Frisoli's methodology provides to re-compact the existent body landfill for the proposed area of intervention in order to create the necessary horizontal space for waste disposal following the patented technique. The re-compaction and the particular waste disposal methodology ensure to reduce the differential settlements and guarantee the same settlements during the years. This aspect is therefore not considered to require further assessment. Further investigation on post-settlement height of the structures is required.
	Backfill waste	The backfill waste necessary to infill the space between the structure and the existent landfill body will be emplaced at the same time and with the same methodology as the waste going into the structure (although without geo-grid). Provided no temporary longitudinal faces exceed 3m this aspect is not considered to require further assessment.
	Leachate	Due to the nature of the waste to be deposited, a significant volume of leachate will not be generated. This component does not require further consideration.
	Landfill gas	Due to the nature of the waste to be deposited, a significant volume of landfill gas will not be generated. This component does not require further consideration.

	Temporary excavation of the existent Ghallis landfill side to create the necessary space for the realization of the works	Re-engineering of the existing cell side slope by trimming a section of the existing Ghallis landfill, approximately triangular in section, to create the necessary space for the construction of the retaining structures - removed waste material will need to be deposited at the existing tip face currently being used at the Ghallis landfill. Engineering the side slopes of the excavated area to conform to a slope angle to ensure stability during works. Further investigation is therefore required for these slopes.
Failure involving liner and waste (Global Failure)	Long term stability	The combined stability of the waste mass and the capping structure requires analysis.

In order to ensure stability during works, the side slopes of the excavated area, more specifically the lateral side of Ghallis existent body landfill, need to be engineered to conform to a proper slope angle.

2.1.6 Capping System Screening

Stability components of Capping Lining System:

Stability	Reinforcement element – Geo-nets	The capping system is integrated, as per patented Frisoli's technique, into the structures of waste. Further assessment is required for this aspect.
	Restoration Soils	During the construction of the structure, hydroseeding and specialist planting will be undertaken to minimize erosion. Furthermore, there will be an HDPE monofilament nets between the electrowelded mesh and the restoration soil layer. The HDPE monofilament nets is anti-hail, ani-frost and run-resistant, fundamental to limit external erosion and small volatile particles dispersion. Further investigation is not considered to be required.

The Capping System structure comprises a number of integrated components. Analysis of each individual component is impractical or unnecessary so a 'global' stability assessment is to be undertaken.

2.2 Lifecycle Phases

The construction of the patented structures shall involve the following processes:

i. Process 1: re-engineering of the Ghallis eastern face, including removing excavated waste, in preparation for foundations;

ii. Process 2: extension of the existing side-slope liner by welding to extend containment, and laying of foundations; and

iii. Process 3: Building of retaining structures inclined at an angle of 70° with reinforced compacted waste; placing of geotextile and capping materials satisfying the requirements of the Landfill Directive 1999/31/CE, allowing for proper drainage and biogas collection.

Process 1: The area covered by foundations will be of approximately 10.000 m2 . Since the existing foundation profile has a significant slope, the profile will be stepped longitudinally to ensure uniformity and continuity.

The foundation will be constructed all along the perimeter of the project area, for a length of about 667 m and width of 15m, within the perimeter of the Ghallis facility, as shown in Appendix B.

This process will include the following steps:

• Re-engineering of the existing eastern cell side slope by trimming a section of the existing Ghallis landfill, approximately triangular in section, to create the necessary space for the construction of the retaining structures - removed waste material will need to be deposited at the existing tip face currently being used at the Ghallis landfill;

• Engineering the side slopes of the excavated area to conform to a grade to ensure stability during works;

• Installation of a prefabricated concrete T-wall, to serve as an anchor point for the various layers of the liner systems (after excavation); and

• In order to guarantee the continuity of the liner of the existing cell with the new construction, the existing HDPE will be unearthed by careful excavation.

Process 2: The new cell volume will be constructed above the base of the existing side-slope liner system of Ghallis landfill, as follows:

• A Geosynthetic Clay Liner (GCL) will be laid under the HDPE liner and above the existent screened/crushed material (max. permeability $10^{\wedge -7}$ m/s), where the continuity of the HDPE within the old and the new cell is maintained through double track fusion welding for long straight seams with free flaps on each side of the weld (where possible), or with HDPE extrusion welding;

• GCL and HDPE are covered with a non-woven geotextile, and all layers are draped over the prefabricated concrete T-wall;

• Sand and compacted inert material are then used to protect the underlaying liner system, to fill the excavated space and to create a basis for the retaining structure;

• A layer of mixed stabilized inert material of 0.5m thickness will be laid over a 0.3m thick gravel, and will be wrapped in geotextile. (Both upper and lower sides of the geological barrier and inert layer are given a gradient of 1%, to ensure that leachate and gas generated are directed into the landfill mass.)

Process 3: Preparation of foundations and laying of the side-slope liner is followed by the building of the waste retaining structures. The stability of the slope is facilitated by compacting masses of waste in layers, where each layer of the retaining wall is enveloped in a geonet, and integrated with other retaining structures, particularly the electro-welded steel mesh.

The construction steps are as follows:

• Placing of steel frames on the foundation bed, at the outer boundary of the retaining wall;

• Laying of the bottom geogrid layer;

• Inclusion of vertical reinforcements (electro-welded steel mesh) as support for the outer mineral liner;

• Draping of geocomposite materials used for gas drainage (on the inner side of the landfill layer) and water drainage (on the outer side of the landfill layer);

• Outer capping soil and mineral clay layers laid, and held in place with HDPE monofilament net;

• Waste laid in 0.5m layer, compacted, and covered with a geogrid after 1m (depends on the software calculation); and

• The above process repeated as necessary until the required heights are achieved.

In the area between the prefabricated concrete T-wall and the retaining structure for waste will be realized the external drainage system for rainwater with gravel and a ditch (ø250).

Works are shown in Appendix D.

The particular methodology to construct the patented retaining structures for waste laying horizontal layers of waste in length all along the perimeter of the proposed intervention area and not constructing the structure all at once per the proposed height, permits to improve the compaction and the stability of the structure. This methodology guarantees the integrity of the structure components, the long-term stability of the waste mass and the capping system and to allow the structure to adapt itself to the possible waste mass settlement concerning the existent body landfill.

2.3 Data Summary

The following data are required as input for the analyses undertaken for this Stability Risk

Assessment:

- material unit weight;
- engineering characteristics of the materials;
- engineering characteristics of the geo-nets;
- geometry of the proposed intervention;
- factors of safety.

2.4 Justification for Modelling Approach and Software

In order to perform a comprehensive Stability Risk Assessment, the components of the landfill development, as previously described in this document, have to be considered not only individually but also 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 geological barrier components. The analytical methods used in this Stability Risk Assessment include:

- Limit equilibrium stability analyses for the derivation of factors of safety for the unconfined subgrade, side slope liner and temporary waste slopes.

The limit equilibrium analyses have been undertaken using:

1. RESSA Version 3.0 (ADAMA Engineering, 2017). The Bishop Slip Circle and Spencer methods of analysis

2. SSAP version 4.9.2, Stability Slope Analysis Program (Borselli, 2017). The Bishop Slip Circle and Morgenstern-Price Non-Circular methods of analysis

The limit equilibrium analyses have been undertaken using the package Ressa, version 3.0, utilising the Bishop simplified (Moment equilibrium method of analysis has been used).
ReSSA (3.0) is an interactive program used to assess the rotational and translational stability of slopes. It was specially developed to allow for convenient inclusion of horizontally placed reinforcement, thus enabling the design and analysis of mechanically stabilized earth slopes. Reinforcement properties follow AASHTO guidelines. However, the user can override all default values. ADAMA Engineering had developed the copyrighted program ReSSA Version 1.0 for the US Federal Highway Administration (FHWA). Version 1.0 has been designated exclusively for use by US State Highway Agencies and by US Federal agencies.

The software includes an important enhancement allowing exploration of 3-part wedge mechanism, with and without reinforcement, using Spencer's method.

ReSSA (3.0) can be used as a generic slope stability program considering circular slip surfaces (Bishop Method) and 2- or 3-part wedge slip surfaces (Spencer method).

All modes of potential failure have been analysed for the case of the temporary slope related to the excavation for the part concerning the search of the existent side slope lining system and the temporary waste slope related to the re-engineering of the existent Ghallis landfill for the part concerning the proposed area of intervention. The analysis has considered the stability of the components in terms of circular and non-circular 2-D limit equilibrium using the computer program

SSAP version 4.9.2 (2017). The software developed by Dr. Geol. Lorenzo Borselli, uses the Bishop Slip Circle and Morgenstern-Price Non-Circular methods of analysis.

The integrity assessments of the unconfined side slope lining system was undertaken using the methods proposed by Fowmes et al. (2006b), Koerner, (1998) and Grisolia (1993). The integrity assessment for long time integrity of the side slope lining system was studied comparing two approaches: the first one considers a real case, “Passo Breccioso” landfill, in which there was applied the Frisoli’s technique and the second one using the computer program Adonis 2.3, a Finite Element Software for Geo-Engineers developed by the University of California at Berkeley. The first approach reproduces the failures in the geosynthetic elements as a function of tensile strain, with rapid increase in geomembrane stress following placement of waste at 60 m above the side slope lining system. The second approach estimates the waste body deformation, interface displacement and geosynthetic strains.

A settlement model was developed for predicting the compression of each layer of refuse in response to the weight of overlying refuse in the landfill. Settlements have been studied following two different approaches. The first one uses the computer program Adonis 2.3. The second one is based on Sowers (1973) and Grisolia (1993) studies on MSW landfill settlements during time. The first approach does not consider the differential weight of the overlying layers during disposal of waste allowing for the estimation of total settlements but only considers the total settlements related to the basal lining system in order to verify if the settlements are congruent with the possible deformations of the lining system and it is guaranteed the necessary slope of the bottom (slope necessary for leachate collection); the second approach considers the complexity of the waste mass during time allowing for results that are more accurate, as described in section 2.7.5.

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 2.5 of this report.

2.5 Justification of Geotechnical Parameters Selected for Analyses

2.5.1 Parameters Selected for Basal Sub-Grade Analyses

Not relevant.

2.5.2 Parameters Selected for Side Slopes Sub-Grade Analyses

From the information given by WasteServ Malta Ltd., it is assumed that the side slopes sub-grade material is essentially granular soils derived from limestone quarrying operations. In the absence of site-specific data for this material, reference has been made to Maksimovic, who presents data on the shear strength of a limestone sand, as reported in the initial SRA concerning Ghallis landfill (November, 2004). 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). Screened / crushed material, in-situ limestone and inert materials constitute the existing side slopes sub-grade of non-hazardous Ghallis landfill. Any additional material added to make up levels will be of a similar selected material.

The adopted peak angle of shearing resistance of the superficial sub-grade material, according with vthe parameters utilized and given by WasteServ Malta Ltd., is 46°, which can be considered appropriate provided the material is fully compacted and the underlying side slopes sub-grade material is in situ rock.

The analysis has considered the short-term stability of the proposed shallow excavation, necessary to identify and unearth the existent side slopes liner system.

2.5.3 Parameters Selected for Basal Liner Analyses

Not relevant.

2.5.4 Parameters Selected for Side Slope Liner Analyses

In terms of the unconfined protector soil stability, geotechnical parameters for the protector soils (assumed to be sand similar to the crushed and screened general fill material) have been adopted from Maksimovic, 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. In particular, the interface shear strengths between sand and geosynthetics, according with typical parameters (Coulomb), has been considered as 35,8°.

A summary of the engineering characteristics of the selected geomembranes, used in the design and analysis of the development are presented below in tabular form for each component.

GCL (Geocomposite clay liner)

Property	**Test method**	**Unit**	**Value**
Elongation at break	ENI ISO 10319 / ASTM D6768	%	10.0 / 6.0
Max tensile strength	ENI ISO 527	N/mm^2	36,0

2mm thick HDPE geomembrane

Property	**Test method**	**Unit**	**Value**
Elongation at break	ENI ISO 527	%	>800
Max tensile strength	ENI ISO 527	N/mm^2	36,0

Minimum 400 g/m2 Non-woven geotextile

Property	**Test method**	**Unit**	**Value**
Elongation at break	EN ISO 10319	%	50
Max tensile strength	EN ISO 10319	(kN/m)	4,00

Settlements on the side slope liner system

For the evaluation of the settlements on the side slope liner system, it has considered the following parameters:

1) Landfill Waste / Geometry

h_1	15	Height of waste above ground surface (m)
h_2	3	Depth of waste below ground surface (m)
s_1	0.7	Slope of waste above ground surface (-)
s_2	0.4	Slope of waste below ground surface (-)
L	2	Distance between the two selected points (m)

2) Landfill Waste / Material Properties

γ_{waste}	10	Unit weight of waste (kN/m^3)

3) Clay Layer / Geometry

H_o	0.5	Thickness of clay layer (m)
H_w	15	Groundwater table (m)

4) Clay Layer / Material Properties

γ_{SAT}	18.8	Saturated unit weight of clay (kN/m^3)
e_o	0.6	Initial void ratio (-)
C_r	0.008	Recompression Index (-)*
C_c	0.09	Compression Index (-)*
OCR	1	Overconsolidation Ratio*

* Determined from Consolidation Test. OCR shall be equal to 1.0 (for normally consolidated soils) or large than 1.0 (for overconsolidated soils) .

The adopted geotechnical parameters are assumed appropriate for this analysis and very conservative for this specific case.

2.5.5 Parameters Selected for Waste Analyses

According to the landfill elevation and age, the existing body landfill can be divided into different characteristic layers. In the absence of site test data, parameters were assumed by literature. According to the study “Analysis of Stability and Control in Landfill Sites Expansion” made by Fang Ronga, Guo Zhaoguib and Feng Tugen about the physical components and property indexes of the characteristics of all layers, there are the following tables:

Table 1: Physical Index of Layers of the Old Landfill [8]

Layer	Elevation (m)	Age (year)	Filling Year	Components (%)					Organic Content (%)
				Plastics	Inorganic waste	Fiber	Putrilage	Dregs	
LW4	>50	0~3.5	2002.12~ now	22.6	3.2	15.1	13.1	46.0	28.2
LW3	40~50	3.5~6	2000.11~2002.12	9.8	2.2	9.1	17.0	61.9	26.1
LW2	30~40	6~9.5	1997.01~2000.11	15.7	2.0	8.1	16.2	58.0	24.4
LW1	15~30	9.5~13	1993.07~1996.12	9.4	2.9	4.8	18.5	64.4	23.3

Table 2: Physical index of Layers of the Old Landfill [8]

Layer	Elevation (m)	Age (year)	Filling Year	Water Ratio (%)	Unit Weight (kN/m3)	Unit Dry Weight (kN/m3)	Initial Void Ratio eo
LW4	>50	0~3.5	2002.12~now	83.43	10.00	5.3	2.89
LW3	40~50	3.5~6	2000.11~2002.12	58.87	11.68	7.5	1.90
LW2	30~40	6~9.5	1997.01~2000.11	82.09	10.89	6.1	2.20
LW1	15~30	9.5~13	1993.07~1996.12	60.83	11.98	7.6	1.76

Table 1: Fang Ronga, Guo Zhaoguib and Feng Tugen

It has considered a medium unit weight for the existing body landfill of 11 kN/m^3, a very conservative friction angle of 25.0° and the very conservative situation of zero cohesion, as can be seen in the following table showing medium parameters [1]:

	Unit Weight (γ)kN/m^3	Cohesion (c) kPa	Friction angle (ϕ)
μ	9.17 – 13.09 (11.13)	6.63- 29.85 (18.23)	27.6° – 36.93° (32.27°)
σ	1.86 – 3.38 (2.62)	3.81 – 24.0 (13.91)	6.11° – 9.66° (7.89°)
CoV	20.24-25.81% (23.54%)	57.41 – 80.40% (68.9%)	22.14% – 26.17% (24.16%)

Table 2: Range of statistical parameters, i.e., mean, standard deviation and Coefficient of Variation (CoV%) of MSW obtained from pubblished data

A summary of the parameters used in the analysis of the temporary waste slope related to the re-engineering of the existing eastern cell side slope useful for the construction of Frisoli's retaining structures, are presented below:

Material Type	Unit Weight *(kN/m^3)*	Friction Angle *(degree)*	Cohesion *(kPa)*
Existing waste	11.0	25.0	0.0
Waste for retaining structures	11.0	25.0	0.0

Waste to gain the space between the retaining structures and the existent landfill body	11.0	25.0	0.0

A summary of the parameters used in the analysis of the settlements are presented below.

	Structure for waste	Foundation layer
Unit weight	11 kN/m3	19 kN/m3
Initial void ratio	1,3	0,96
Void ratio after primary compression	0,4	1,4
Primary compression index	0,195	0,3
Secondary compression index	0,036	0,126

In the way to have the most conservative analysis, it has considered a zero cohesion instead of the real situation (probably, in the proposed area of intervention, cohesion has values from 8.0 to 20.0 according to the above mentioned literature related to landfill expansion analysis).

2.5.6 Parameters Selected for Capping Analyses

Reinforcement element – Geo-net

Geo-nets are commonly made of polymer materials, such as polyester, polyvinyl alcohol, polyethylene or polypropylene. They may be woven or knitted from yarns, heat-welded from strips of material, or produced by punching a regular pattern of holes in sheets of material, then stretched into a grid.

The geo-nets have been provided with high-modulated continuous polyester woven fabric with PVC coating are among the most used for reinforcement. Polyester fiber is one of the polymer fibers that best combines high resistance values with low deformation values (CREEPs).

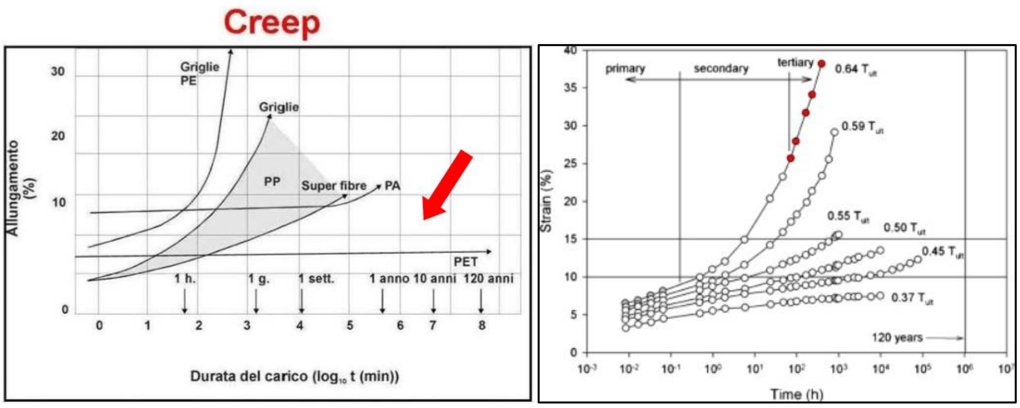


Figure 4: Creep for reinforcement elements

The PVC dipped is made of polyester filaments to protect them from mechanical and chemical attack, in order to give to the geo-net a great durability and important performance even in the long term (120 years).

The geo-net must be marked CE for the reinforcement function in accordance with the Directive 89/106 / EC on Building Materials, in order to obtain approval from an independent institute on the design and manufacturing conformity of the product.

Geo-nets must be produced under ISO 9001 quality assurance system to ensure the final customer that the product has been manufactured with quality control systems and approved by an independent institute, and that the supply corresponds to what is offered, also through the issuance of the ISO 17050 compliance certificate.

At the current state, for the project under consideration, they were considered the following geo-nets:

Type#	*Geo-nets*	*Ultimate Strength, T_{ult} [kN/m]*
1	Geo-net 55	55,00
2	Geo-net 80	80,00
3	Geo-net 110	110,00
4	Geo-net 150	150,00
5	Geo-net 200	200,00

Geological data

The patented structures require, for this project, a supplementary foundation constituted by 500 mm of inert materials as per project design description. Ideally, in the analysis must be also considered the different parameters regarding the existing waste and the waste that will be used for the structures. Furthermore, because of the excavation required for searching the existing High Density Polyethylene (HDPE) geomembrane below the perimeter road, at the current state is not possible to estimate if there will be the same subsurface conditions in the entire area of intervention. Moreover, the excavation will be filled with sand and inert material as per project design description.

The estimated soil/waste parameters to be used in the slope design are summarized below, and should be verified at the beginning of construction.

Material Type	Unit Weight *(kN/m^3)*	Friction Angle *(degree)*	Cohesion *(kPa)*
On-site Coralline Limestone	19.0	46.0	0.0
Sand	14.0	32.0	0.0
Inert material	20.0	36.0	0.0
Existing waste	11.0	25.0	0.0
Waste for structures	11.0	25.0	0.0

Capping: vegetal soil	17.0	26.0	0.0
Capping: mineral layer	18.0	26.0	0.0

It should be noted that the cohesion values shown above were used in the combined internal/external stability and in the global stability analysis. For the project under discussion, will be used the worse condition for the cohesion of soils (wet soils).

The adopted geotechnical parameters are assumed appropriate for this analysis and very conservative for this specific case. Furthermore, Frisoli' experience in this field guarantee their appropriateness in relation to expected results.

Effect of water

Water is not present.

Surcharge

A surcharge load consisting of 30 kN/m2 (simulating the presence of any vehicular traffic for maintenance works) was applied above the slope in the calculations.

Seismicity

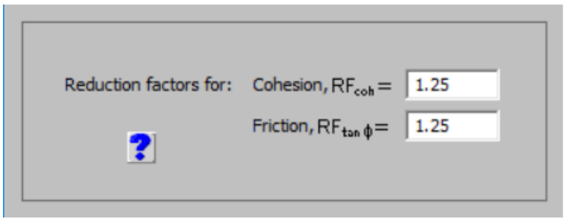
Seismic events are analyzed under Extreme Event I limit state as per AASHTO (2007). Seismic events tend to affect both external and internal stability of walls. Parameters to be used for the analysis must be also coherent with the geographic area, using the 1,000- yr return period seismic hazard maps in AASHTO (2007).

At the current state, for the project under consideration, parameters are:

Horizontal peak ground acceleration coefficient, Ao = 0.035

Design horizontal seismic coefficient, kh = Am = 1.00 x Ao = 0.035 & design vertical seismic coefficient, kv (down) = 0.500 x kh = 0.018

Reduction factors to Soils – Cohesion and Friction



2.6 Selection of Appropriate Factors of Safety

2.6.1 Factor of Safety for Basal Sub-Grade

Not relevant.

2.6.2 Factor of Safety for Side Slopes Sub-Grade

The short term unconfined stability analysis of the side slope sub-grade model (the excavation), it has appropriate to consider the stability using a factor of safety of 1.2. This factor of safety can be considered appropriate and conservative because the side slopes sub-grade will only remain unsupported in the very short term and it has considered seismic conditions.

The adopted factor of safety is based upon the “Eurocode 7” and can be considered appropriate for this analysis. The same factors of safety have been utilized in the Stability Risk Assessment, SLR (Nov. 2004; Ref. 4C-585-001/SRA).

2.6.3 Factor of Safety for Basal Lining System

Not relevant.

2.6.4 Factor of Safety for Side Slope Lining System

When considering the short term unconfined stability of the proposed side slope lining system into the excavation it has appropriate to consider only the stability of the protective sand layer because anchorage provides for stability of the lining system; using conservative effective stress parameters, a factor of safety of 1.1 is considered appropriate since the protective sand layer will only remain unsupported in the very short term.

The adopted factor of safety is based upon the “Eurocode 7” and can be considered appropriate for this analysis. The same factors of safety have been utilized in the Stability Risk Assessment, SLR (Nov. 2004; Ref. 4C-585-001/SRA).

2.6.5 Factor of Safety for Waste Mass

The factors of safety for the temporary excavated waste slope can be considered as below:

	Factor of safety
Rotational (Circular Arc; Bishop) Stability Analysis	1.3
Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis	1.3

The adopted factors of safety are considered appropriate for this analysis and conservative. Waste can be considered a heterogeneous soil with different characteristics. In particular, waste presents generally a lower unit weight respect to the soil and a more flexibility due to its nature.

Factors of safety used for this application are based on an elaboration of AASHTO design parameters. Furthermore, it has been adopted conservative factors of safety despite the analysis was carried out considering seismic conditions.

All factors of safety are based upon the Eurocode 7 and can be considered appropriate for this analysis. The same factors of safety have been utilized in the Stability Risk Assessment, SLR (Nov. 2004; Ref. 4C-585-001/SRA).

2.6.6 Factor of Safety for Capping System

Regarding the analysis of the stability of the structure, the factor of safety (FOS) for slopes must be selected based on the supporting structure type, impact of slope failure, uncertainty of waste parameters and temporary or permanent, conditions such as rapid drawdown, seismic etc.

In the calculation, four safety coefficients were introduced on the characteristics of durability of geo-nets and four general safety coefficients. In summary:

• Reduction factor for installation damage: it represents the loss of strength that the specific geogrid is due to the installation damage caused during compaction of the waste. The factor depends on the type of waste used and was determined by laboratory tests. In the case under consideration, a factor of 1.11 was applied.

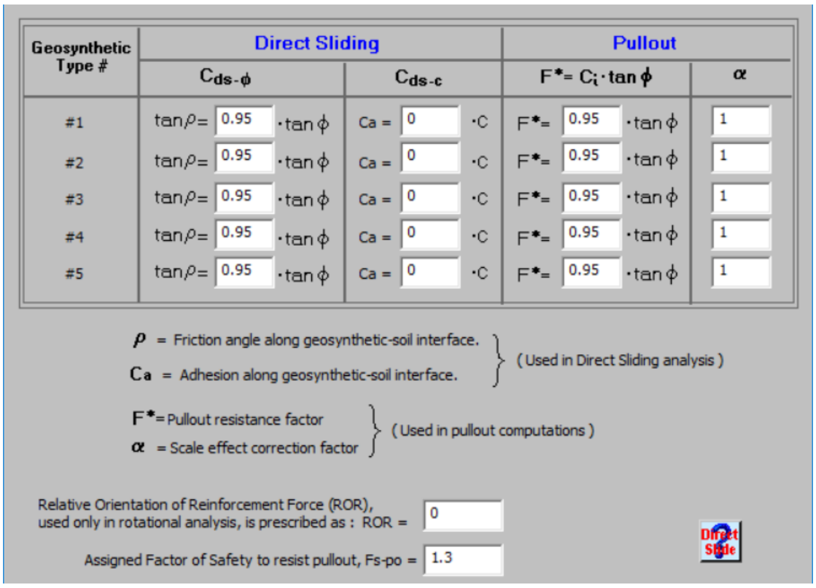
• Reduction factor for chemical degradation: represents the loss of strength that the geogrid specification has as a result of the chemical attack on the waste on which it is deposited. The parameter depends on the chemical properties of the waste and is determined by laboratory tests. In the case under consideration, a factor of 1.22 was applied.

• Reduction factor for creep deformation: represents the loss of strength that the geo-net specification has due to the creep. This is an intrinsic feature of the geogrid and is always determined through laboratory tests. In this case, a factor of 1.20 was applied.

• Reduction factor for joints (seams and connections): represents the relationship between the friction angle to the waste-geosynthetic interface and the waste friction angle, and is also determined by laboratory tests. Depending on the physical characteristics of the geogrid, a coefficient of 1.00 was applied in this case.

The reduction factors, adopted for this analysis, are based upon supplier test data, product specific data and literature data extrapolated from “Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes” developed by AASHTO LRFD Bridge Design and AASHTO LRFD Bridge Construction Specifications, and can be considered appropriate for this specific case and very conservative.

Parameters regarding direct sliding and pullout are presented below:



Below are presented in tabular form the factors of safety considered for the analysis:

	Factor of safety
Rotational (Circular Arc; Bishop) Stability Analysis	1.1
Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis	1.1

The adopted factors of safety are considered appropriate for this analysis and conservative regarding this specific case. These factors of safety are based upon considerations related to MSE (Mechanically Stabilized Earth) walls and Reinforced Soil Slopes (RSS), similar to Frisoli's technique. Waste can be considered a heterogeneous soil with different characteristics. In particular, waste presents generally a lower unit weight respect to the soil and a more flexibility due to its nature. Because of these considerations, Frisoli's patented structures could be considered as an application of the MSE walls to the landfills. Factors of safety used for this application are based on an elaboration of AASHTO and FHWA design parameters and on Frisoli's experience because of the uniqueness of this technique, which does not permit to have official technical standards. Furthermore, it has been adopted conservative factors of safety despite the analysis was carried out considering seismic conditions.

The recent ASD guideline (AASHTO, 2002, 17th edition Standard Specification for Highway Bridges) discusses global stability (Section 5 "Retaining Walls", 5.2.1.4 Mechanically Stabilized Earth Walls) and highlights that the overall stability of the retaining wall and/or retained slope shall be evaluated for all walls using limiting equilibrium methods of analysis such as the Modified Bishop, simplified Janbu or Spencer methods of analysis, as used in this analysis. A minimum factor of safety of 1.3 shall be used for walls designed for static loads. A minimum factor of safety of 1.1 shall be used when designing walls for seismic loads as made for this case.

As used in this analysis, seismic forces applied to the mass of the slope were based on a horizontal seismic coefficient kh equal to onehalf the ground acceleration coefficient A, with the vertical seismic coefficient kv equal to zero.

2.7 Analyses

2.7.1 Basal Sub-Grade Analyses

Not relevant.

2.7.2 Side Slopes Sub-Grade Analyses

According to the project sections, the side slope of the excavation will be realized with a maximum height of 2m and with an angle of 30° in order to guarantee the temporary slope stability.

When considering the short term unconfined stability of the proposed side slope sub-grade model (the excavation) it has appropriate to consider the stability using conservative effective stress parameters, a factor of safety of 1.2 is considered appropriate.

The peak angle of shearing resistance of the sub-grade material, according with typical parameters utilized and given by WasteServ Ltd., can be considered as 46°.

All modes of potential failure have been analysed for the case of the temporary slope related to the excavation for the part concerning the search of the existent side slope lining system. The analysis has considered the stability of the components in terms of circular and non-circular 2-D limit equilibrium using the computer program SSAP version 4.9.2 - 2017.

The results presented in Appendix E show a conservative minimum factor of safety of 1.25. Stability in the excavation does not require further consideration. Following, there is an analysis on the possible failure/collapse occurring to the excavation area and Frisoli's proposed methodology to ensure security and stability during works because at the current state it is not possible to know more about how deep is the HDPE into the ground and how big the bank of the excavated area needs to be.

Surveyors will conduct the site survey to mark the lines and levels of the objective excavation for foundations as per indicated in the approved drawing. The area to be excavated will be properly marked and clear before starting the works. The depth of excavation will be periodically checked to avoid over excavation. The bank of excavated area will be sloped 30 degrees and will be maintained to avoid a collapse of the bank into the excavated area. The bank depends on how deep is the HDPE in order to create the connection between the old lining system and the new one.

2.7.3 Basal Liner Analyses

Not relevant.

2.7.4 Side Slopes Liner Analyses

Stability of the lining system in the excavation

According to the project sections, in order to allow the continuity between the existent lining system and the new one, there will be an excavation for finding the existent HDPE geomembrane sheet and welding it with the new one.

Therefore, also on the side slope of the excavation will be realized the new lining system. The side slope of the excavation will be realized with an angle of 30 degrees, in order to guarantee the waterproofness of the system, putting a geocomposite bentonite (above the existent screened/crushed material [the existing Geological Barrier] max. permeability 10^{-7} m/s) with a thickness of 4 mm, having a coefficient of permeability equal to 10 E-9 m / sec.

Subsequently, throughout the new side-slope lining system will be put an artificial waterproofing mantle made of a rough HDPE (see above) with a thickness of 2mm.

In addition, to protect the HDPE geomembrane sheet from the actions of mechanical loads or atmospheric agents and ultraviolet rays, will be put over the abovementioned non-woven geotextile and a layer of sand (circa 1 m) (see drawings).

When considering the short term unconfined stability of the proposed side slope lining system into the excavation it has appropriate to consider only the stability of the protective sand layer because anchorage provides for stability of the lining system.

The interface shear strengths between sand and geosynthetics, according with typical parameters (Coulomb), can be considered as 35,8°.

A slope that extends for a relatively long distance and has a consistent subsurface profile may be analyzed as an infinite slope. The failure plane for this case is parallel to the surface of the slope and the limit equilibrium method can be applied readily.

A typical section or “slice” through the potential failure zone of a slope in a dry cohesionless soil, e.g., dry sand, is shown in the following figure, along with its free body diagram.

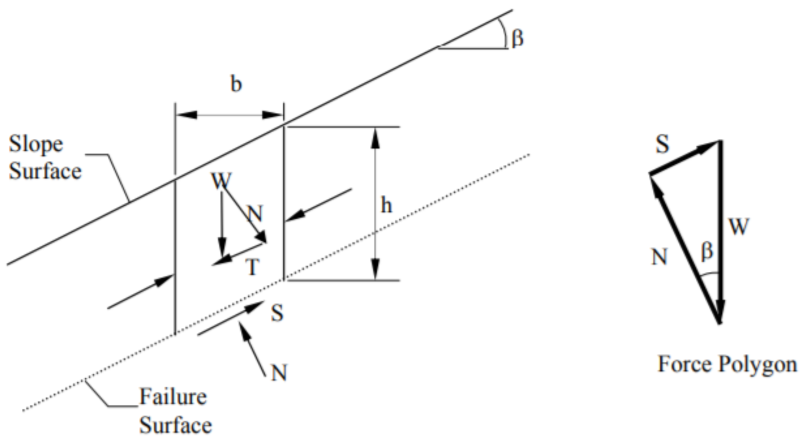


Figure 5: Infinite slope failure in dry sand

The factor of safety (FS) is defined as the ratio of available shear strength to strength required to maintain stability. For an infinite slope analysis, the FS is independent of the slope depth, h, and depends only on the angle of internal friction, φ, and the angle of the slope, β. The slope is said to have reached limit equilibrium when FS=1.0. Also, at a FS = 1.0, the maximum slope angle will be limited to the angle of internal friction, φ.

The analysis undertaken demonstrates an acceptable factor of safety of 1.24.

Integrity of the lining system in the excavation

The assessment of the stresses in the side slope liner system is one of the key design aspects that govern the selection of appropriate geosynthetic materials. Stresses on the liner are assessed pre-waste placement, during operation and post-waste deposition.

Prior to waste placement, the only stresses acting on the lining system components are self-weight of individual components. During operation and post-waste placement, the stresses on the liner are due to the settlement and movement of the waste that results in the development of drag and mobilising forces.

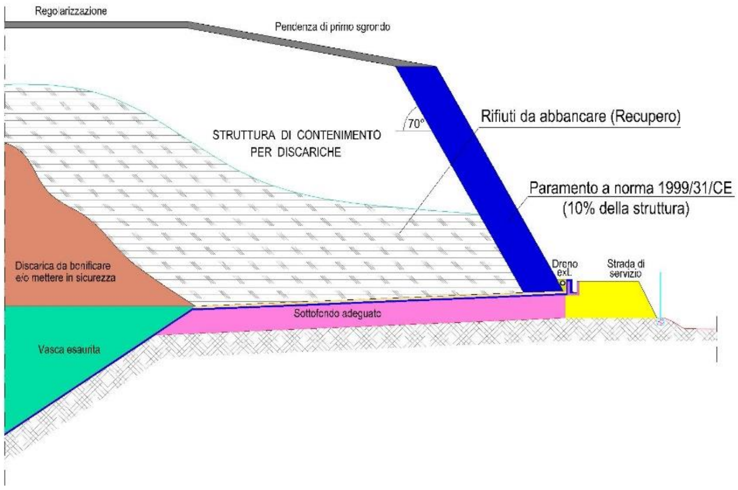
Stresses developed during the placement of the waste have to be managed by a liner system capable of resisting the development of tensile stresses beyond an allowable stress that is a function of the mechanical properties of the selected material. This can be achieved by selecting a material that (if required) can withstand tensile stresses; or by selecting geosynthetic materials with interface shear strength capable of minimising the stresses of the liner and maximising the transference of the stresses from the waste mass to the sub-base soil.

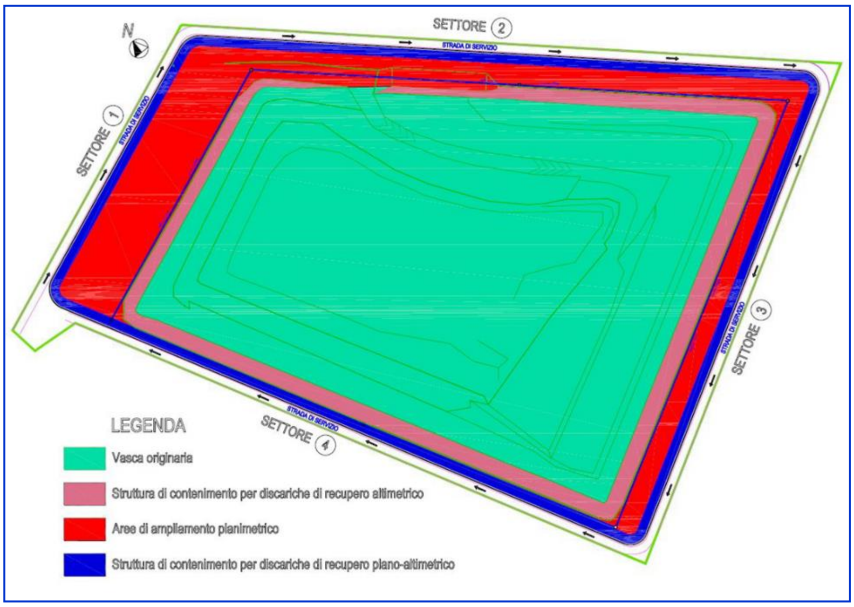
Furthermore, it has analysed the integrity of the side slope lining system studying the anchorage and the integrity after any kind of settlements occurring to the side slope sub-grade.

Geosynthetic integrity failure

In order to assess the performance of the side slopes liner model for confined conditions related to the lateral volumetric expansion impact, back analysis of a Frisoli's extension area realized in Foggia landfill and results from computer software Adonis 2.3 were carried out.

Frisoli's lateral extension in Foggia ("Passo Breccioso" landfill) can be considered similar to the proposed alteration to the internal lateral Ghallis profile for extend landfill capacity. The geometry of Frisoli's intervention in Foggia can be seen in the image below.


Regolarizzazione
Pendenza di primo sgrondo
STRUTTURA DI CONTENIMENTO PER DISCARICHE
70°
Rifiuti da abbancare (Recupero)
Paramento a norma 1999/31/CE
(10% della struttura)
Discarica da bonificare e/o mettere in sicurezza
Dreno ext.
Strada di servizio
Sottofondo adeguato
Vasca esaurita


N
SETTORE 1
SETTORE 2
SETTORE 3
SETTORE 4
STRADA DI SERVIZIO
LEGENDA
Vasca originaria
Struttura di contenimento per discariche di recupero altimetrico
Aree di ampliamento planimetrico
Struttura di contenimento per discariche di recupero plano-altimetrico

The modelling and results are discussed by Fowmes et al. (2006b). The integrity failure usually involves the tensile failure of the geomembrane liner due to forces induced by waste loading, from weight and compression induced downdrag. For “Passo Breccioso” landfill, the lining system comprised, from the bottom up, a geological barrier, a GCL, a 2mm smooth HDPE geomembrane, a non-woven protection geotextile, and a 300/500mm fine protection layer.

Two models were used in the analysis; the first modelled a full height section of side slope to assess the waste and lining system. The second model looked in more detail at a single section of the side slope in order to assess the behaviour of the lining system in more detail (Fowmes et al., 2006b).

Individual beam elements were used to model each of the three geosynthetics, with strain dependent interfaces controlling interactions. Staged construction was considered with 12m waste lifts to represent waste placement against subsequent steps, then loading increments of 30 kPa added to the upper waste surface structure to represent the upper capping system. Waste was modelled with a volumetric hardening criterion, so that further compression, per unit stress increase, reduced as volumetric strain occurs.

The model was able to reproduce the failures in the geosynthetic elements as a function of tensile strain, with rapid increase in geomembrane stress following placement of waste at 60 m above the reference level. The tensile strength of the geomembrane is approximately 36 kN/m, and the model predicts that this value is exceeded shortly following the waste loading increasing to the equivalent of 60 m above bench height. The rapid increase coincided with post peak strength reduction on the interface underlying the geomembrane as smooth geomembrane - geocomposite drainage interface has a peak interface friction an 13° and a large displacement friction angle of approximately 8°.

Waste height above bench.	**Vertical pressure (kPa) at waste ref level**	**Maximum axial strain in geomembrane (%)**	**Maximum tensile stress in geomembrane (kN/m)**	**Location of max stress (m below top of bench)**
0	0	0.14	0.42	3.2
10	140	0.17	0.51	1.2
20	280	0.20	0.60	1.2
30	420	0.20	0.59	1.2
40	560	0.37	1.32	2.4
50	700	0.40	1.43	4.8
60	840	8.37	25.1	3.6
70	980	14.7	44.9	1.2

Table 3: Axial strains and tensile forces in the geomembrane related to waste height. (after Fowmes et al. 2006b)

This study demonstrates that failure on the geosynthetics does not occur because of the load of the waste above the lining system. Furthermore, since the max height of the structures for waste is 15 m it is excluded any failure on the side slope lining system due to the stress caused by the overlying waste mass.

The second approach uses the Finite Element Software for Geo-Engineers developed by the University of California at Berkeley, Adonis 2.3. The geosynthetics were modelled as elastic beam elements anchored at the top of the slope, as below described in the section regarding the anchorage. There are three interfaces between lining components: mineral layer/geomembrane,

geomembrane/geotextile, geotextile/sand, and additionally the sand/waste interface (above considered for the excavation). Information on geosynthetic tensile behaviour was provided by the suppliers of the materials and are shown in Section 2.5.4.

Geosynthetics were not expected to fail through excessive tensile deformations. Soil and waste materials where represented by Mohr –Coulomb failure criterion and the properties assigned to the materials are given in Section 2.5.6. Waste properties are based on data available from the literature and are shown in Section 2.5.4.

The importance of interface strength parameters has been emphasised previously by various authors (e.g. Filz et al. 2001, Jones & Dixon 2005). For this analysis were used the same criterions used for “Passo Breccioso” side slope lining integrity assessment. The tensile strength of the geomembrane was considered 36 kN/m, and it has considered the worst situation that is the interface geomembrane - geocomposite with a friction angle of approximately 8°. Results are shown in Appendix M.

Analysis demonstrates that failure does not occur to geosynthetics because the geomembrane tensile strength exceeds the max. calculated stress occurring to the analysed interface.

Settlements on the side slope lining system

Depending on the technical characteristics of the geomembrane, it can be traced back some behavior of the waterproofing sheet on the occurrence of cracks located in the underlying materials.

In fact, if occurs a collapse in a restricted area of the basin, the mantle it would be stressed in that area by traction effort. Such efforts would tend to move to neighbouring areas. However, this is immediately mitigated for friction effect between the mantle and the layers of material in contact with it.

The friction is much bigger the greater the mass of the above cumulus. Therefore, in the most unfavorable conditions, namely in the hypothesis that the basin is completely filled and in conditions of the occurrence of a failure such as to cause an admissible traction effort, in the following experimental formula is obtained:

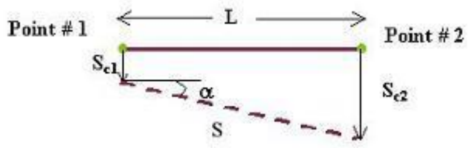
$$X = \sigma * d/2Q$$

From the abovementioned material characteristics, it could be estimate an elongation at break of 10%.

This collapse is not conceivable given the nature and type of treatment of the ground beneath the mantle and the very low settlement measured.

The uneven distribution of the stresses (different waste thickness), creates a differential settlement between different points along the landfill cross section. The potential impact of these settlements on the performance of the leachate collection system and liner system has been evaluated.

The stresses need to be evaluated at the worst case loading scenario in the way to estimate the vertical side slope stresses due to the waste weight at two points of consideration, point 1 and point 2.



The worst scenario is at the basis of the excavation in which they can be considered the two extremes of the underside of the excavation.

The primary consolidation settlement is evaluated using the following equation:

$$S_c = \sum_{i=1}^{n} \frac{H_i}{1+e_0}\left[C_r*\log(OCR)+C_c*\log\left(\frac{\sigma'_{v0i}+\Delta\sigma_{vi}}{\sigma'_{v0i}*OCR}\right)\right]$$

where:

Symbol	Description	Unit
S_C	primary consolidation settlement	m
n	number of clay sublayers	m
C_r	recompression index (as shown in Figure 2)	-
H_i	thickness of clay sublayer # i	m
e_o	initial void ratio	-
OCR	overconsolidation ratio	-
σ'_{voi}	effective normal stress (evaluated at the middle of the sublayer # i)	kPa
C_c	compression index (as shown in Figure 2)	-
$\Delta\sigma_{vi}$	additional normal stress (at the middle of the sublayer # i)	kPa

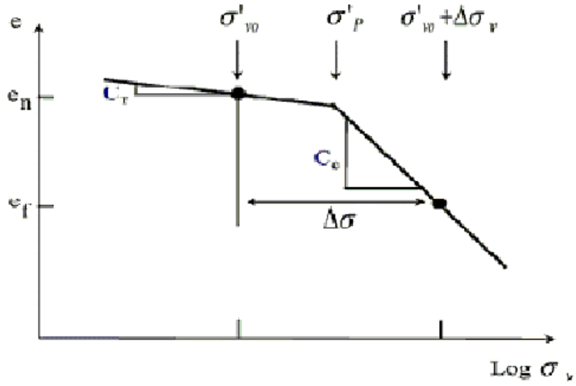


Figure 6: Settlement Calculations for Overconsolidated Soils

The calculated consolidation settlement is the sum of settlement of five sublayers. Based on the original slope and design tolerance, the resulted slope from the calculated differential settlement can be considered accepted.

	Point 1	Point 2
Settlement S_C (m)	0.003	0.006
Differential Settlement Slope S (%)	0.111	

Settlements of equal size in larger areas would result in inferior extensions; settlements of the same magnitude in smaller areas would also be of interest for the closer areas in the sense that the deformations of the mantle beyond the collapse area would not be negligible and therefore the relative deformations would be contained.

Anchorage

As part of the assessment of geosynthetic interface sliding, and geosynthetic barrier layer integrity, the anchorage of a geosynthetic system must be considered. Geosynthetics will be anchored prior to placement of waste, in order to prevent uncontrolled sliding. If one end of a geosynthetic is fixed then stresses transferred into the lining system will result in tensile stresses and associated strains within the geosynthetics.

The role of the anchor is to withstand the tensile force generated by friction along the slope. Since the side slope of the proposed project, albeit for a very short part (depends on how deep is the HDPE into the ground), it will be realized at an angle of about 30°, greater than the interface friction angle between the lining layers which would have allowed the equilibrium, an anchorage system is fundamental.

For this specific design, the use of the prefabricated concrete T-wall and the drainage area between the T-wall and the structure, the 500m thick fine protection layer above the lining system and the external road besides the T-wall, exclude any possible failure due to the geosynthetics interface sliding. It is recommended to realized an external road besides the T-wall and above the lining system with at least 1m thick inert material. As per calculations made, considering a thickness of 1m of cover soil, an embedment length of 1 m with a trench anchorage 0,7m x 0,7m is sufficient to avoid any kind of failure related to the non-use of an anchorage, as shown in Appendix H.

ANCHORAGE	
L_anc	1 m
b	0,7 m
h	0,7 m
Cover soil	1 m

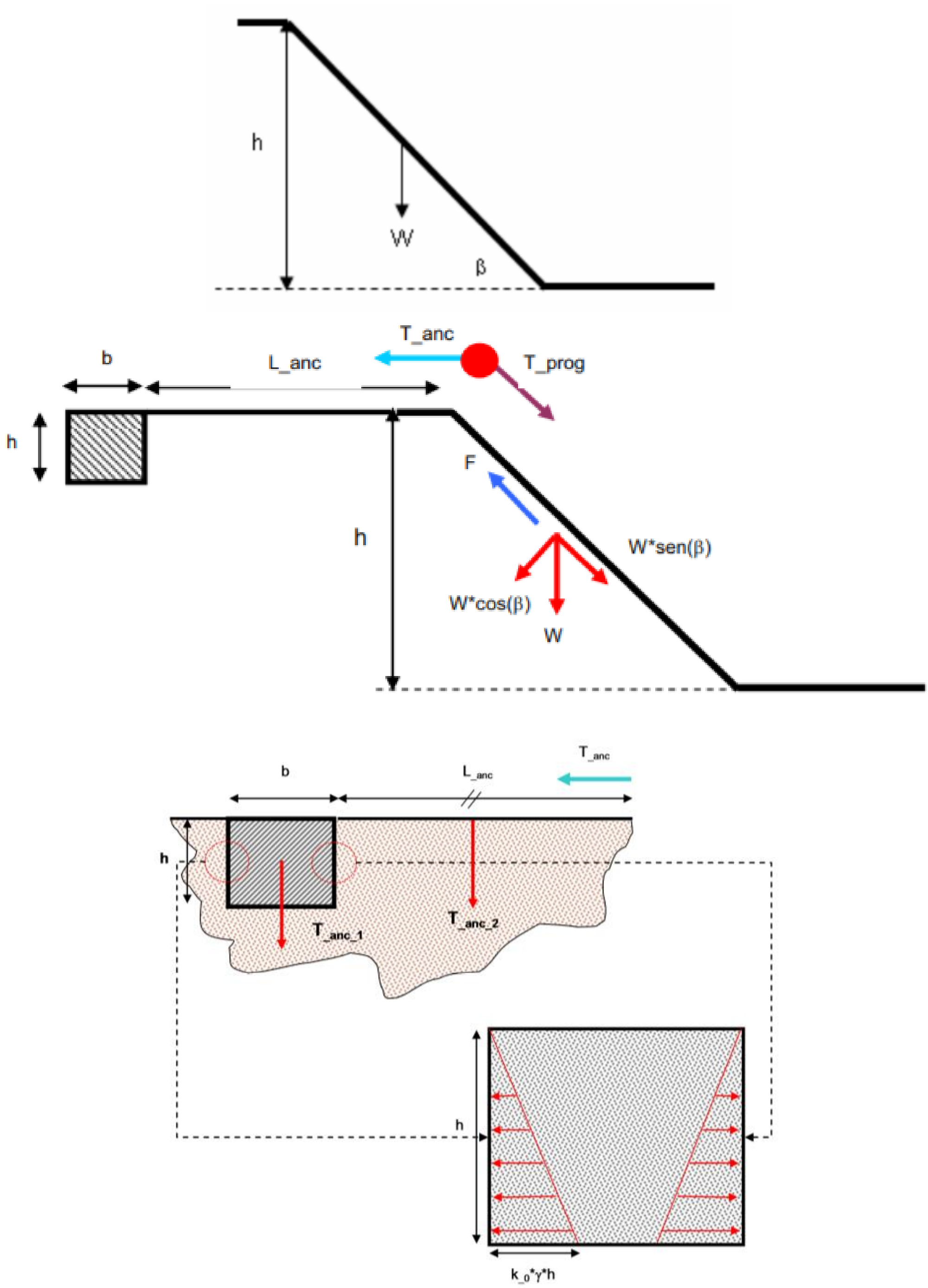


Figure 7: Anchorage scheme

2.7.5 Waste Analyses

In order to undertake the stability aspect of the waste analysis related to the re-engineering of the existing cell side slope by trimming a section of the existing Ghallis landfill, approximately triangular in section, to create the necessary space for the construction of the retaining structures, it has considered the shear strength parameters provided by WasteServ Ltd..

The 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. A factor of safety of 1.0 is considered appropriate where residual interface shear strengths are applied. All factors of safety are based upon the Eurocode 7 and can be considered appropriate for this analysis. The same factors of safety have been utilized in the Stability Risk Assessment, SLR (Nov. 2004; Ref. 4C-585-001/SRA).

All modes of potential failure have been analysed for the case of the temporary waste slope related to the re-engineering of the existent Ghallis landfill for the part concerning the proposed area of intervention. The analysis has considered the stability of the components in terms of circular and non-circular 2-D limit equilibrium using the computer program SSAP version 4.9.2 - 2017.

The results presented in Appendix F show a conservative minimum factor of safety of 1.37. The re-engineering of the existent cell side slope for the proposed area of intervention will be realized trimming a triangular section with an angle of approximately 20°, enough to guarantee the temporary stability of the mass.

The backfill waste necessary to gain the space between the structures and the existent landfill body will be disposed at the same time and with the same methodology of the waste going into the structures.

The tracked dozers, prior to direct compaction by the waste compactors, will spread out waste in thin layers, less than 50 cm thick.

Landfill settlements

In order to demonstrate the settlements during the years following Frisoli's technique it has undertaken an analysis showing the very low settlements associated with it.

Landfill settlements can be studied with independent models separating the contribution of the settlement of the foundation by the settlement of the waste body.

The settlement of the foundation will be minimum and neglectable considering the fact that the foundation of the proposed project will have a width of about 15 m and a thickness of 0,5 m. That represents an important advantage for the side slope lining system because stresses due to settlements or any loads can be considered neglectable.

The foundation will have settlements from a minimum of zero at the foot of the walls and a maximum in the inner part. Under these conditions, the residual slope of the foundation is sufficient to ensure the percolation of leachate inside the old cell (minimum slope 1%).

A waste settlement study is of fundamental importance for both stability and durability hydraulic sealing systems, as well as drainage and superficial fluid disposal systems. In fact, excessive settlement can lead stresses to the top cover higher than the material resistance, resulting in breakage and loss of hydraulic seal.

The mechanisms that govern the settlements of MSW are many and complex to be schemed because of the extreme heterogeneity of the materials, the deformability of the particles themselves, the high presence of empty and, finally, the degradation they undergo over time. However, the phenomenon is not yet well known and despite the many efforts to try to understanding the laws (Sowers, 1973), there are no reference models to be adopted with a substantial reliability. Another crucial aspect is the estimate of the time when such settlements will happen. Sowers (1973) was one of the first to propose a transposition of mechanical behaviour relationships of the compressible soils to the waste. This transposition was limited to the oedometric conditions, which corresponded to the conditions of deposit in column (with negligible lateral strain) of a waste sufficiently far from the edges of the cell.

Solid urban waste is characterized by a high degree of deformability, depending on a series of factors such as (Grisolia et al, 1993):

• the initial composition;

• the content of organic matter;

• the age of refusal;

• initial density or index of initial voids;

• the laying method;

• the height of the cumulation of waste;

• the amount of material used for daily coverage;

• the production of percolate, its level on the waterproofing barrier and its bottom permeability within the mass of the waste;

• environmental factors such as humidity, temperature, biogas production and its degree of disposal.

All of these factors are not independent of each other while affecting each other and at the state of the proposal is not possible to define with a certain accuracy due to the fact that for this purpose it would be necessary a more in-depth study.

The Sowers model for the estimation of the settlements is still the most widely used today. This model provides for the use of two separate expressions: the first provides the settlement yield to primary compression, the second being caused by secondary compression.

Primary compression is mainly related to the expulsion of water and gases from the within the waste structure. The settlement due to primary compression is usually done quickly, usually within the first 30 days that follow the application of overload. That duration, however, considering the complexity of the phenomenon and the numerous contour conditions affecting, it may result extremely variable (even 5 years after the landfill closure).

Regarding, instead, secondary compression, it is generally due to viscous behaviour of the solid waste skeleton and biodegradation of the organic substance. Secondary compression may last for many years after the end of landfill (even 50 years).

Since the solid waste is a highly heterogeneous material and can settle either due to biodegradation of waste, or by its own weight or by overlying pressure applied above the barrier, development of differential settlement within the landfill area is common but not relevant for this case, as following explained. The excessive differential settlements can result in the development of tension cracks in the soil barrier or tearing of geomembrane or displacement of bentonite from GCL, near the zone of sharp curvatures there by resulting in loss of integrity of the whole cover system. In the case of this proposed project, in the light of the “Frisoli s.r.l.” s work methodology and the observed settlements in other landfills when was applied the same technique, there will not be important settlements.

A good compaction, the realization of the foundation and the methodology of construction of the structures for waste ensure uniform settlements all along the perimeter of the working area.

Yen and Scanlon model (1975) permits to estimate the time (and the entity) in which these settlements will take place. The estimated time is 215 months (18 years), considering the extreme scenario of a pre-settlement height of the structure of 15m (Appendix I).

Considering Sowers (1973), it can be estimated a total settlements during a period of 18 years (operational and post-operational phase), with an error regarding the multitude of variables, a 15 % of settlements in total, which corresponds to a total of about 2,3 m for the structures of waste with a pre-settlement height of 15 m (1,9 m for those of 13 m; 1,4 m for those of 10 m).

It should be considered that this datum relates to the secondary compression phase, which occurs on average after a first initial phase of subsidence due to the so-called primary compression. The first phase usually corresponds to the storage of waste (waste disposal); the settlement of waste, on the other hand, is fully included in the second and third phases, as can be seen in the following diagram which highlights the evolution of settlements during these phases.

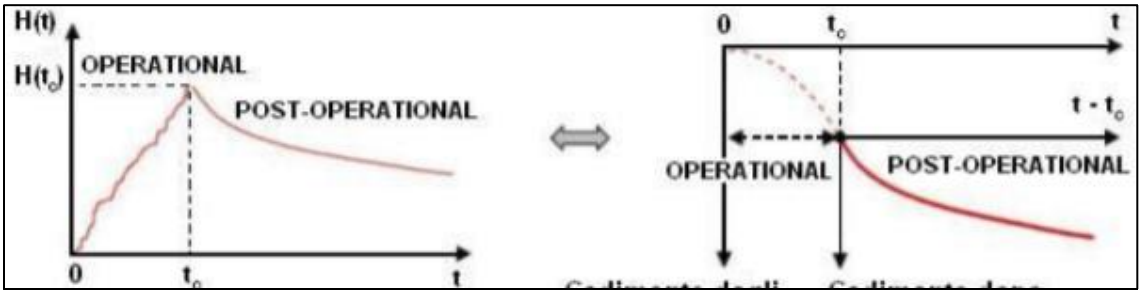


Figure 8: Height of waste body (left) and Settlements of the waste body (right)

However, every layer of the structure is independent and this is fundamental for the homogeneity of the settlements and for the flexibility of the structure, which responds equally to every stress. Furthermore, the structure will be constructed per parallel layers reducing the settlement of each layer mainly due to the weight of the overlying layers and the existing Ghallis body landfill will be re-compacted for the part related to Frisoli’s intervention in the way to reduce any possible differential settlements occurring to the new body landfill.

Results and calculations are presented in Appendix I.

A different approach was carried out with the use of the computer program Adonis 2.3, a Finite Element Software for Geo-Engineers developed by the University of California at Berkeley for the settlements occurring to the bottom of the landfill due to the proposed vertical expansion, which considers the entire mass of waste and the mutual interaction between each “soil”. It is important to highlight that this approach cannot be considered for the estimation of total settlements because does not permit to study the particular case of this proposal and the secondary compression associated with the biodegradation of waste. Results are presented in Appendix L.

2.7.6 Capping Analyses

Stability

Stability in this case, requires an accurate analysis and a more detailed demonstration. Capping system is integrated into the structure as a unique mass with different engineering characteristics. The analysis highlights, starting with the sequence of works, the stability criteria utilized for sizing the structure and reinforcements.

Sequence of works

Preparation of foundation is followed by the building of retaining structures for waste using “Refuse dump containment structure®” of Frisoli EP 1661635 A1 (European patent). The stability of the slope (70°) is facilitated by compacting masses of waste in layers, where each layer of the retaining wall is enveloped in a geo-net, and integrated with other retaining structures. Once the construction of the retaining wall is in place and additional void space becomes available, any wastes removed to allow preparation of foundations as well as fresh MSW – will be deposited in the new void space created.

The dimensions of the various structures, and the precise angle of slope, are derived from calculations that factor in the stability of the foundations and underlying substratum, as wells as the properties of the electrowelded geomesh and geofabric layers that envelope the compacted waste mass.

The first phase consists of the laying of the electrowelded mesh boxes, projected with a 70° slope angle, along the alignment that identifies perimeter of the project. The laying of the following layers will follow the alignment of the underlying layers.

The boxes will be placed on a parallel double row, spaced by 1 m. The welded mesh of the outer boxes will have the following dimensional characteristics 15x15 - ø8, while the second row, the inner one, will be 20x20 - ø 6.

After that, it will be stretched the strips of geotextiles by matching them to each other and overlap them by about 5-10% of the width of the rolls, leaving a part outside the outer casing for the next facing. The “casseroles” are pulled with the appropriate anchoring elements, so you have the draining geogrid on the second row of “casseroles”. The function of the geogrid will be to drain the meteoric waters resulting from any infiltration from the outside, which is further avoided by the slope of the outer shell.

Figure 9: External capping over the retaining structure for waste

Once the system is installed, it will be completed the filling, between the two boxes, with vegetable soil and manually will be filled the wedge at the base of the chassis so as to avoid the formation of gaps in this critical area where the excavators and rollers can't work.

After filling the area between the two boxes, the clay is laid, always handly for the aforementioned difficulties, and into the second slab for a thickness of 0.5m. Clay compaction is performed by a vibratory plate.

A second draining geocomposite is then installed, whose function is that of drainage of biogas.

Afterwards, it will be filled the structure with waste through the use of a compactor, for a depth, as said, of about 1.0 m. The filling must take place in at least two steps, taking care to compact suitably by making at least two layers of approximately 50 cm compacted, to obtain the final thickness of the bank, about 100 cm. When the layer is completed, the geogrid rim is moved.

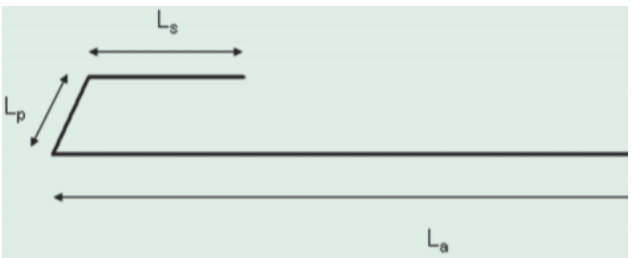


Figure 10: Schematic configuration of the i-th layer

Stability of the structures

The need to profile a side of a landfill beyond its natural geophysic or natural angle and the need to limit the footprint of a landfill and at the same time maximize the available space makes it necessary the realization of works of support. Such artificial works must therefore not only satisfy exclusive structural technical requirements, but also hydraulic and landscaping data at the same time. Furthermore, it is possible to limit the alterations of the filtration regime and to reduce the environmental impact associated to it.

Among the technologies for improving the capacity of a landfill used in modern environmental engineering, the use of horizontal reinforcement layers applied to waste of the ""Frisoli s.r.l." guarantees high tensile strength and ability to mobilize the friction of the surrounding layers. Works built according to this technology are in fact, structures where the layers of waste, resistant as known exclusively to the efforts of compression and cutting, are reinforced with specific types that can withstand even important traction efforts by creating a highly performing system from the point of view mechanic.

Retaining structures in landfills are therefore an effective technical / structural alternative to the typical landfill design models, with a lesser environmental impact, even greater competitiveness from the point of view of deformation capacity.

In fact, the layers are semi-independent one of each other in order to guarantee more flexibility to the structure because of the typical landfill settlements.

For steepened reinforced slopes (as the retaining structures for waste of Frisoli, with a face inclination of 70 degrees), design is based on modified versions of the classical limit equilibrium slope stability methods as shown in Figure 1:

- Circular or wedge-type potential failure surface is assumed.
- The relationship between driving and resisting forces or moments determines the slope factor of safety.
- Reinforcement layers intersecting the potential failure surface are assumed to increase the resisting force or moment based on their tensile capacity and orientation.
- The tensile capacity of a reinforcement layer is taken as the minimum of its allowable pullout resistance behind (or in front of) the potential failure surface and its long-term allowable design strength, T_{al}.

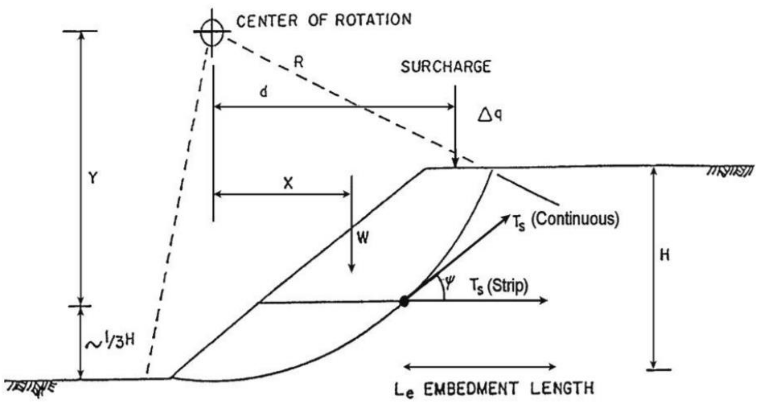


Figure 11: Modified limit equilibrium analysis for reinforced slope design

As shown in Figure 12, a wide variety of potential failure surfaces must be considered, including deep-seated surfaces through or behind the reinforced zone. For the internal analysis, the critical slope stability factor of safety is taken from the internal unreinforced failure surface requiring the maximum reinforcement. This is the failure surface with the largest unbalanced driving moment to resisting moment and not the surface with the minimum calculated unreinforced factor of safety. This failure surface is equivalent to the critical reinforced failure surface with the lowest factor of safety. Detailed design of reinforced zone is performed by determining the factor of safety with successively modified reinforcement layouts until the target factor of safety is achieved. External and compound stability of the reinforced zone are then evaluated.

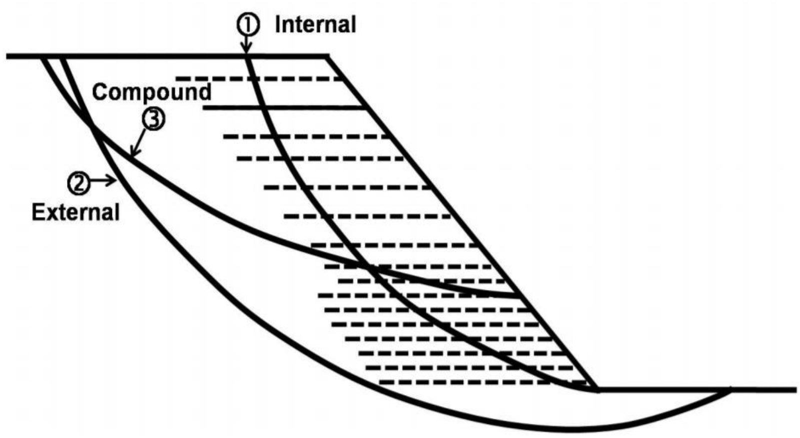


Figure 12: Failure modes for reinforced waste slopes including internal failure within the reinforced waste zone, external failure entirely outside the reinforced waste zone, and compound failure starting behind and passing through the reinforced waste zone.

The method presented uses any conventional slope stability computer program and the steps necessary to manually calculate the reinforcement requirements for almost any condition. Figure 2 shows the conventional rotational slip surface method used in the analysis. Fairly complex conditions can be accommodated depending on the analytical method used (e.g., Modified Bishop, Spencer). The computer program ReSSA (ADAMA, 2001), which will be used in this case, was developed by the FHWA to specifically perform this analysis.

The method of analysis (Baker e Leshchinsky, Bishop method) in ReSSa assume the reinforcement force as contributing to the resisting moment, i.e.:

$$FS_R = \frac{M_R + T_S R}{M_D}$$

where,

FS_R = the required stability factor of safety

M_R = resisting moment provided by the strength of the waste

M_D = driving moment about the center of the failure circle

T_S = sum of tensile force per unit width of reinforcement (considering rupture and pullout) in all reinforcement layers intersecting the failure surface

R = the moment arm of T_S about the center of failure circle as shown in Figure 2

With this assumption, FS_R is applied to both the waste and the reinforcement as part of the analysis. As a result, the stability with respect to breakage of the reinforcement requires that the allowable reinforcement strength T_{al} must be greater than or equal to the required maximum design tension Tmax for each reinforcement layer.

The available long-term strength, T_{al}, is calculated as follows:

$$\mathbf{T_{al}} = \frac{\mathbf{T_{ult}}}{\mathbf{RF}} = \frac{\mathbf{T_{ult}}}{\mathbf{RF_{ID} \times RF_{CR} \times RF_{D}}} \text{ (in strength per unit reinforcement width)}$$

where,

T_{ult} = Ultimate Tensile Strength (strength per unit width). The tensile strength of the reinforcement is determined from wide strip tests per ASTM D4595 (geotextiles) or D6637 (geogrids) based on the minimum average roll value (MARV) for the product.

R_F = Reduction Factor. The product of all applicable reduction factors.

RF_{ID} = Installation Damage Reduction Factor. A reduction factor that accounts for the damaging effects of placement and compaction of soil or aggregate over the geosynthetic during installation. A minimum reduction factor of 1.1 must be used to account for testing uncertainties.

RF_{CR} = Creep Reduction Factor. A reduction factor that accounts for the effect of creep resulting from long-term sustained tensile load applied to the geosynthetic.

RF_D = Durability Reduction Factor. A reduction factor that accounts for the strength loss caused by chemical degradation (aging) of the polymer used in the geosynthetic reinforcement (e.g., oxidation of polyolefins, hydrolysis of polyesters, etc.).

To sum up, the stability risk analysis calculations evaluate the external stability (including base sliding and bearing capacity) and internal stability (including geo-net length, overstress, pullout resistance, and internal sliding) to determine the required geo-net strengths, number of layers, and reinforcement lengths. The capping is included into the analysis as part of the structure.

Considering the worst scenario of pre-settlement height of 15 m, geo-nets, as deeply shown in Appendix G, are disposed into the structure as follow (from bottom to up):

Reinforcement Layer #	*Geo-nets*	Height Relative to Toe [m]	L [m]
1	Geo-net 200	0,00	15,00
2	Geo-net 200	1,00	15,00
3	Geo-net 200	2,00	15,00
4	Geo-net 200	3,00	15,00
5	Geo-net 150	4,00	15,00
6	Geo-net 150	5,00	15,00
7	Geo-net 150	6,00	15,00
8	Geo-net 110	7,00	15,00
9	Geo-net 110	8,00	15,00
10	Geo-net 110	9,00	15,00
11	Geo-net 80	10,00	15,00
12	Geo-net 80	11,00	15,00
13	Geo-net 80	12,00	15,00
14	Geo-net 55	13,00	15,00
15	Geo-net 55	14,00	15,00

Stability analysis and results are presented in Appendix G. The results presented in Appendix G show conservative minimum factors of safety both Rotational (Circular Arc; Bishop) Stability Analysis and Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis, which guarantee the long-term stability of the entire patented structures. Regarding the case of pre-settlement height of 15 m (the worst scenario), minimum factor of safety for Rotational stability analysis is 1.13 (1.16 for a pre-settlement height of 13 m; 1.19 for a pre-settlement height of 10 m) and minimum factor of safety for Translational stability analysis is 1.15 (1.15 for a pre-settlement height of 13 m; 1.17 for a pre-settlement height of 10 m). Both can be considered appropriate and very conservative. Appendix G represents the above-mentioned results.

Erosion

During the construction of the structures, it will be applied an important hydroseeding in order to minimize the erosion.

Regarding the final plantation, Frisoli has experimented with planting, above the landfill's cover of Foggia landfill, many solutions potentially applied to Ghallis landfill.

Furthermore, it will put an HDPE monofilament nets between the electrowelded mesh and the restoration soil layer. The HDPE monofilament nets is anti-hail, ani-frost and run-resistant, fundamental to limit external erosion and small volatile particles dispersion.

2.8 Assessment

2.8.1 Basal Sub-Grade Assessment

Not relevant.

2.8.2 Side Slopes Sub-Grade Assessment

Assessment of the side slopes subgrade is not required since it has been eliminated from consideration by the screening process and the analysis process within sections 2.1.2. and 2.7.2..

2.8.3 Basal Liner Assessment

Not relevant.

2.8.4 Side Slopes Liner Assessment

Considering the short term unconfined stability of the side slope lining system protector sand when adopting conservative interface shear strength material parameters, it has studied the case of an infinite slope failure. The analysis undertaken demonstrates acceptable factors of safety of 1.24.

Regarding the long-term integrity of the side slope lining system, the analysis shows that no failure occurs to the geosynthetics. Furthermore, considering the fact that Frisoli's intervention does not exceeds the maximum projected height of Ghallis landfill, the integrity of the existing side slope lining system cannot be doubted due to approved SRA of the PA 04834/04.

2.8.5 Waste Assessment

In the case of the preparation for the necessary space for Frisoli's intervention, acceptable factors of safety are demonstrated for temporary waste mass stability for all potential failure considered. The side slope lining system geosynthetic integrity would not be affected by waste mass displacements because of the anchorage and the low additional weight of the waste related to Frisoli's intervention.

Waste has been considered as a soil with different characteristics. Although the deformability of this material is not regular, it manifests many similarities with that of organic soil, especially peat. The analysis carried out about settlements for this application has considered the following hypothesis.

Figure shows an ideal consolidation curve of a mass of MSW, obtained from the interpolation of experimental laboratory data: it presents notable similarities with those obtained with some natural earths containing organic matter, such as peat. The following phases can be extrapolated (Grisolia et al, 1995b):

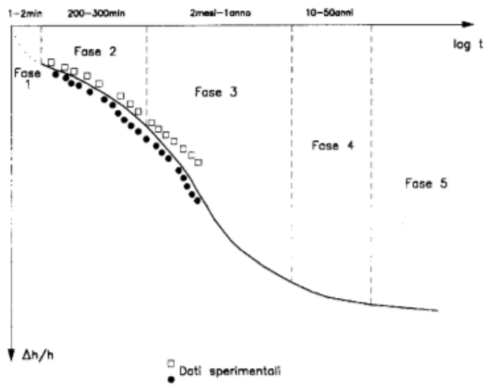


Figure 13: MSW Landfill settlements during time (Grisolia et al, 1995b)

• Phase 1: rapid initial deformation, with reduction of the macroporosity due to the settling of very deformable materials;

• Phase 2: settling of highly deformable elements;

• Phase 3: slow deformation (creep) and start of the decomposition of the organic component;

• Phase 4: completion of the decomposition of the organic substance;

• Phase 5: residual deformations.

Settlements, albeit they are only a prediction or estimation, can be considered in line with the literature and the provisions. Frisoli's construction methodology will reduce any differential settlements with an important compaction of the new waste mass to be disposed and a re-compaction of the area concerning its intervention on the existent Ghallis landfill body. Results/estimations show, as expected, uniform settlements on the landfill body. Attention needs to be paid, albeit not really relevant, in the upper intersection between Frisoli's waste mass and the existent landfill body with a careful compaction and monitoring.

Since the altitude of the basis of the proposed retaining structures along the area of intervention is variable, the height of the structures will have a max. height of 15 m, when it is possible, ensuring that the post-settlement height is in line with the approved post-settlement height of Ghallis landfill. The area of intervention is divided into three parts corresponding to different pre-settlement heights of the structure (15 m, 13 m, 10 m), as shown in Appendix C. Pre and post-settlement height of the structures are shown in Appendix C. No increase in height beyond the permitted limits is being contemplated.

The results show that the predicted settlement can vary significantly depending on the model selected and the specific values of model parameters used. The estimated total settlements during operational and post-operational phase, corresponds to a total of about 2,3 m for the structures of waste with a pre-settlement height of 15 m (1,9 m for those of 13 m; 1,4 m for those of 10 m), with a post-settlements height in line with the approved post-settlement height (54 m), as shown in the results and Appendix C.

Secondary compression in Yen and Scanlon (1975) is higher (10 cm more) respect to Sowers methodology (1973) because they refer to landfills in California (USA) where waste has very different characteristics, in particular it has a much higher organic fraction which significantly influences the entity and speed of settlements. In the case of European landfills, with a compaction of waste reaching about 11 kN/m^3 of unit weight, the expected yields will be on average lower and will develop more quickly. Furthermore, according to WasteServ Ltd., waste to be used for Frisoli's structures will be mostly the reject of the MBT process in Maghtab complex, which guarantees, following literature, a lower total settlement, but also digestate, lights and heavies fractions, street cleaning residues, bulky waste, in line with the current landfilled waste composition in Ghallis.

Therefore, this study has therefore highlighted how the proposed solution is not only technically feasible, but also provides all the necessary environmental guarantees.

2.8.6 Capping Assessment

The analysis undertaken has demonstrated that a conservative minimum factor of safety both Rotational (Circular Arc; Bishop) Stability Analysis and Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis guarantees the long-term stability of the entire structure. Waste is not part of the capping system but the analysis has considered the entire structure in order to demonstrate the stability of the capping system and the effectiveness of the function of the geo-nets.

Waste has been considered as a soil with different characteristics, which can be disposed per layers as MSE (Mechanically Stabilized Earth) walls and/or Reinforced Soil Slopes (RSS). Furthermore, waste has a lower unit weight respect to the soil, which guarantees to obtain a lighter structure.

Factors of safety used for this application are based on an elaboration of AASHTO and FHWA design parameters and on Frisoli's experience because of the uniqueness of this technique, which does not permit to have official technical standards. Furthermore, it has been adopted conservative factors of safety despite the analysis was carried out considering seismic conditions.

Because of the application of an important hydroseeding during construction phases and the installation of an HDPE monofilament nets between the electrowelded mesh and the restoration soil layer, erosion can be considered minimized.

3.0 MONITORING

3.1 The Risk Based Monitoring Scheme

Based upon the foregoing Stability Risk Assessment, a simple risk-based monitoring scheme is considered appropriate for the future development of the landfill. The monitoring is limited to ensuring compliance with the tipping rules and monitoring of groundwater levels.

3.1.1 Basal Sub-Grade Monitoring

Not relevant.

3.1.2 Side Slopes Sub-Grade Monitoring

Monitoring during construction will comprise construction quality assurance to ensure compliance with the construction specification. No additional instrumentation is deemed as being required during construction or post final landscape restoration.

3.1.3 Basal Lining System Monitoring

Not relevant.

3.1.4 Side Slope Lining System Monitoring

Monitoring during construction will comprise construction quality assurance to ensure compliance with the construction specification. No additional instrumentation is deemed as being required during construction or post final landscape restoration.

3.1.5 Waste Mass Monitoring

Monitoring during construction will comprise construction quality assurance to ensure compliance with the construction specification. No additional instrumentation is deemed as being required during construction or post final landscape restoration.

Given the potential heterogeneity of landfill settlements, monitoring of settlement is required during construction phases and/or after the completion of the structures, to determine whether any interventions are required. The most appropriate form of monitoring is being evaluated; one possible solution would be a horizontal inclinometer system, supported by specialized software, to obtain high-resolution profiles of settlement or heave.

3.1.6 Capping System Monitoring

Monitoring during construction will comprise construction quality assurance to ensure compliance with the construction specification. No additional instrumentation is deemed as being required during construction or post final landscape restoration.

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