Stability Risk Assessment

Salt Valley Road Landfill – Cells 3 & 4 Development

Prepared for Opalvale Pty Ltd

16 July 2021

Project Number: TW21027

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[redacted]	Senior Waste Engineer	TW21027_Opalvale_Cell 3&4_SRA_2.0
Signature	[redacted]	

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1 Introduction

Talis Consultants Pty Ltd (Talis) was engaged by Opalvale Pty Ltd (Opalvale) to undertake a Stability Risk Assessment (SRA) to support the design of the basal engineering to Cells 3 and 4 for their Class II Salt Valley Road Landfill facility (the Site).

Opalvale operates the Site, which is located 80km east of Perth, on Lot 11 Chitty Road, Hoddy's Well in the Shire of Toodyay. The Site, which has been operating since 2019 as a Class II facility, with two (2) current active Class II landfill cells (Cells 1 and 2).

1.1 Report Context

There is no set guidance requirement for undertaking SRA's for solid waste facilities, therefore this report has been prepared in general accordance with the UK Environment Agency's Environmental Permitting (England and Wales) Regulations Stability Risk Assessment template, and similar stability assessments of projects undertaken by Talis in Western Australia. Talis has previously carried out a Capping Stability Risk Assessment, dated 8 March 2021, for the Capping Layer and the template and some of the data used therein will be re-employed here.

This document describes the way the SRA was carried out for the design of Cells 3 and 4 at the Site and presents the overall findings of the work.

It should be noted that in December 2014 Golder Associates carried out a number of technical studies in support of the Works Approval for the Site (Appendix 5 - "Opalvale Landfill – Technical Studies to Support Design" dated 22 December 2014). This included an assessment of the stability of the then proposed Class II landfill and although it included some aspects of the basal engineering it primarily focussed on the entire facility. The Golders' studies were based on the engineering design prepared by IW Projects. In view of the context of this work and its overlap into the design of Cells 3 and 4, the work by Golders will be incorporated, in part, into this SRA and where done so will be referenced.

1.2 Location and Topography

The Site is located 80km east of Perth being some 1.25km east of Chitty Road and 13km south of Toodyay, as shown on Figure 1. It comprises part of Lot 11 Chitty Road on Deposited Plan 34937. The Site is located at the Williamsons Clay Pit and is situated in the south-eastern portion of Lot 11 and occupies an area of approximately 48 hectares (ha). The Site Boundary is shown in Licence L9089/2017/1 and depicted on Drawing TW21027 C-101_A. Access to the site is via Salt Valley Road, with an internal road providing access to the landfill cell.

The Site is surrounded by native vegetation and agricultural land. The Department of Agriculture and Water Resources (Catchment Scale Land Use 2018) defines agricultural land to the west and east of the Site as dryland and broad acre cropping, and grazing pastures. There are established irrigation areas (mainly for perennial horticultural purposes) located approximately 3 to 5 km north, east, and south of the Site.

Regarding the closest sensitive receptors, Jimperding Brook is located approximately 900m west and southwest from the Site. Jimperding Brook flows in a general northerly direction and ultimately ends in the Avon River Valley. The Avon River Valley is listed on the Directory of Important Wetlands in Australia (DIWA) and is situated approximately 12.5km northwest of the Site.

The Works Approval application document titled *Opalvale Salt Valley Road Class 2 landfill. Lot 11 Chitty Road, Toodyay. Works Approval Application Supporting Documentation by IW Projects dated 21 December 2014* (IWP, 2014) states the closest neighbouring residential property is a farmhouse located 400m to the southwest of the Site. A further two residential properties are located approximately 1,350m from the northeast boundary of the Site.

The Site's elevation ranges from approximately 270m Australian Height Datum (AHD) in the base of the pit to 300m AHD outside the Stage 1 footprint, in the eastern portion of the Site.

Stage 1 of the landfill comprises active Cells 1 and 2, and the proposed Cells 3 to 6. At the time of writing filling in Cell 1 is approximately level with the crest of its northern side slope (approximately 288m AHD). Waste levels in Cell 2 range between 280m to 285m AHD.

There is to be a temporary western waste slope forming the boundary between Cells 3 & 4 and Cells 5 & 6 and is graded at approximately 1V:3H.

A topographic survey was undertaken on 11 November 2020 using an unmanned aerial vehicle (UAV). The survey area includes the Stage 1 area and perimeter access road. The topography and layout of the existing and proposed cells in Stage 1 is shown on Drawing TW21027 C-101.

1.3 Geology

Geoscience Australia (1:2.5 million scale) classifies surface geology profiles occurring across the Site as "quartzite kyanite, sillimanite, muscovite/fuchsite, garnet, hornblende, clinopyroxene, epidote; psammitic and politic schists garnet, felsic gneiss and hornfels, quartz-mica-graphite schist, metaconglomerate, cordierite-bearing rock". The Department of Mines, Industry Regulation and Safety (DMIRS) Geological Survey of Western Australia (GSWA) 1:500,000 map series describes the underlying bedrock geology as "quartz – mica schist; includes sillimanite, andalusite, kyanite, graphite, and staurolite bearing varieties".

The regional geology has been detailed in a report by Stass Environmental in May 2015 ("Attachment L – Opal Vale Pty, Report on Groundwater Assessment"). As such, it will not be presented herein other than to highlight the near surface stratum which is of relevance to the SRA. Stass carried out an investigation at the site principally to assess the potential impacts on the groundwater. As part of their investigation, nine boreholes were drilled to a depth of up to 60m to determine the underlying geology. This information was employed in both the hydrogeological assessment, but also formed the basis of the aforementioned stability assessment undertaken Golders. As such, for consistency, the same data is employed within this this SRA.

The landfill is underlain at depth by granite and gneiss, but the boreholes show that the near surface soils to be a highly weathered zone comprising sandy clays and silty clays.

1.4 Hydrogeology

The Stass Hydrogeological Investigation included the installation of groundwater wells in the boreholes. Monitoring of the wells has been undertaken on a quarterly basis since 2014 recorded groundwater elevations levels ranging from 278.96mAUD (BH SE4) to 267.46 m AUD (BH SE1).

The generalised groundwater flow direction was inferred to be from the north-east to southwest of the landfill. Figure 1-1 presents the local groundwater regime beneath the site using the highest inferred levels recorded since 2014 (September 2018). As can be seen, the localised groundwater gradient is relatively flat under Cells 3 & 4 declining to the northwest and southeast in line with topography.

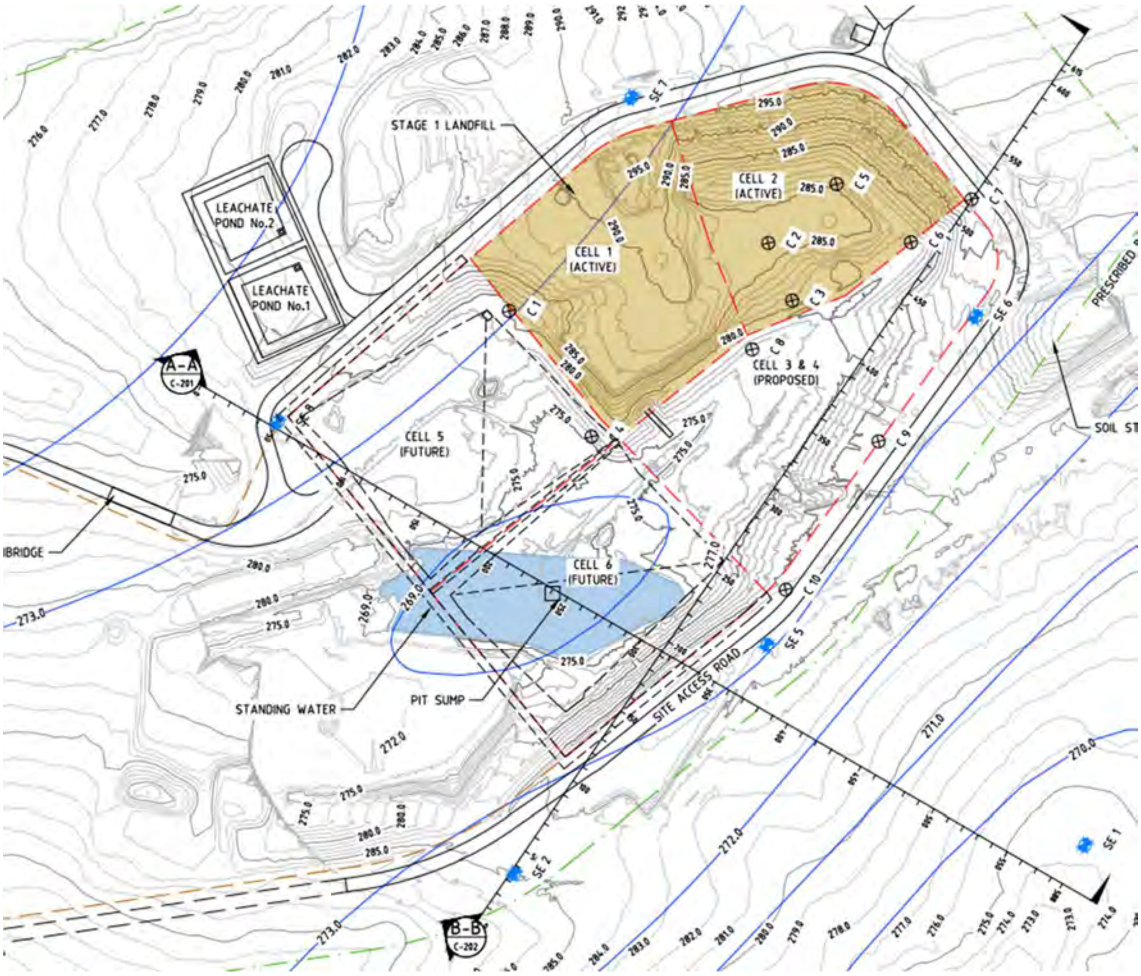


Figure 1-1: Maximum Inferred Groundwater Contours (m AHD)

The site groundwater monitoring well network comprises nine piezometers labelled BH SE1 – 9. A summary of the groundwater data is shown in Table 1-1.

Table 1-1: Groundwater Depth from Hydrogeological Monitoring

Bore ID	Min Static Water Level (m AHD)	Highest Historic Static Water Level (m AHD)
BH SE1	267.4	270.2
BH SE2	271.2	272.5
BH SE3	273.2	275.0
BH SE4	276.3	279.0
BH SE5	271.4	273.0
BH SE6	272.3	273.0
BH SE7	272.8	273.4
BH SE8	271.6	272.9
BH SE9	270.0	271.4

Note: m AHD stands for metres Australian Height Datum, and mbgl represents metres below ground level.

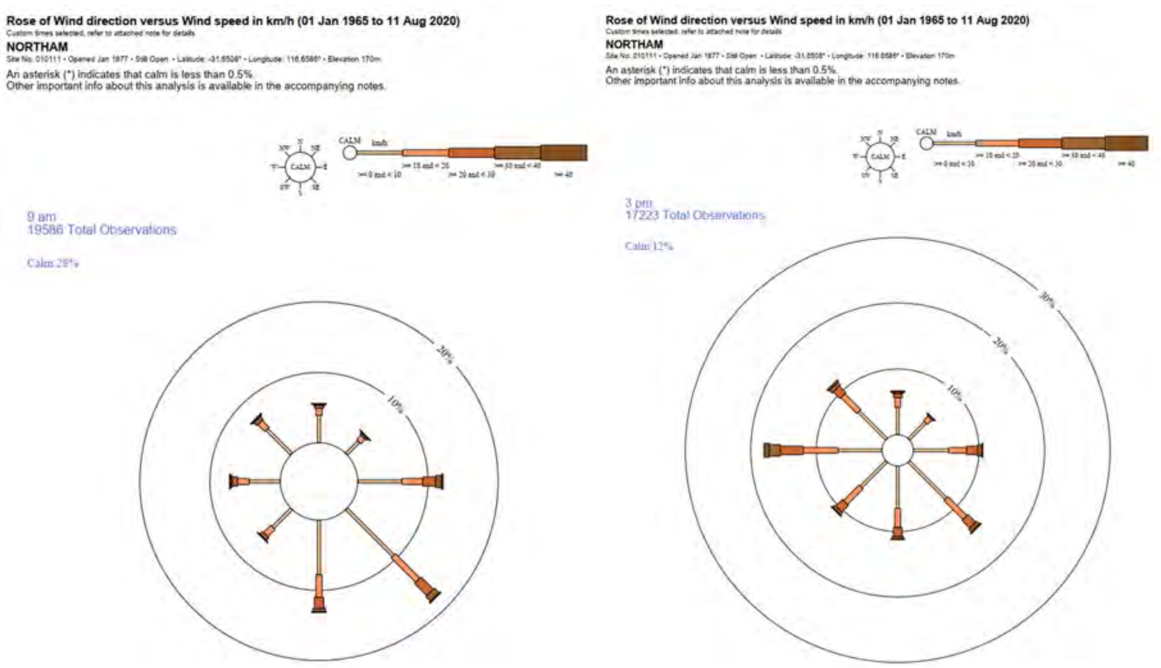
Over the majority of the base the groundwater achieves a highest elevation of between 273m and 274m AUD.

1.4.1 Climate

The Site is located within a region that experiences a mild climate that is generally warm and temperate with moderate rainfall throughout the year. The Australian Bureau of Meteorology's (BOM's) closest weather station to the Site that has been recording long-term data is at Northam (Station 010111), approximately to the 18.2km north-east. The average annual maximum and minimum temperatures recorded over the last 118 years for this location are 34.2˚C and 5.4˚C, respectively (accessed October 2019). Additionally, the average annual rainfall at the location is 543mm with generally consistent rainfalls during the year although 75% falls within between May and September,

Figure 1-2 indicates that winds are predominately south-easterly in the morning (9am), switching to westerly in the afternoon (3pm), although for a significant amount of time the wind prevails from the southeast.

Figure 1-2: Annual Average Wind Rose Data for 9am (left) and 3pm (right) for Northam



1.5 Conceptual Site Model

The conceptual stability site model has been developed from information contained in the site's Closure Plan and the SRA for the Capping Layer, both prepared by Talis.

1.5.1 Rehabilitation Design

The Victorian Environment Protection Agency (EPA), Best Practice Environmental Management 'Siting, Design, Operation and Rehabilitation of Landfills', 2015 (BPEM) Landfill Guidelines have been adopted and supported by Opalvale for the operation and rehabilitation at the Site. The objectives of the proposed engineering design and rehabilitation measures include the following:

- A restoration profile which will incorporate a low permeability capping layer to restrict the infiltration of rainwater into the waste mass and minimise the production of leachate;
- A restoration profile which will optimise the landfill capacity within the existing landfill footprint, minimise aesthetic impact, stabilise the surface of the completed part of the landfill and minimise long-term maintenance requirements;
- A system of surface water management to positively deal with any accumulation of rainwater, and reduce suspended sediment and contaminated runoff; and
- A gas management regime to control the generation of landfill gases and reduce any significant risk of gas adversely impacting the surrounding environment.

1.5.2 Final Profile

During the preparation of conceptual final fill profiles, a number of factors were identified which affected the design including:

- The approved Works Approval prepared by IW Projects and the CPCMP prepared by Talis;
- The extent of existing waste at the Site;
- Constraints around the site boundary; and
- Maximising the void space over the proposed landfill footprint to maximise the remaining lifespan of the cell.

To address each of these factors, the final fill profile was developed to ensure that:

- The quantity of waste requiring excavation is minimised as much as practicably possible;
- Best practice slopes of not less than 1V:20H and no greater than 1V:5H will be achieved, except for the temporary 1V:3H slope forming the western edge of the proposed initial 6 cells. Suitable engineering controls will be adopted in order to:
  - Ensure the long-term stability and integrity of the capping material and containment layer;
  - Promote natural surface water run-off;
  - Minimise erosion as much as reasonably practicable;
  - Provide, as far as possible, an aesthetically acceptable landform;
  - Minimise long-term maintenance requirements; and
- The maximum post-settlement elevation will not exceed 312m AHD.

1.5.3 Capping System

The proposed final capping system is as follows, commencing from the top of the waste:

1. Gas collection and Regulation layer consisting of fine-grained sandy material at least 300mm thick;
2. A 1.5mm Linear Low-Density Polyethylene (LLDPE) textured geomembrane;
3. Sub-surface drainage layer comprising a drainage geocomposite (geonet); and
4. Revegetation layer at least 1,000mm thick, comprising an 800mm thick, clean sub-soil layer and a 200mm thick mulch/growth medium.

Due to the intention to extend the landfill to the west into the adjacent quarry void, the western side slope of the proposed disposal area will be temporarily capped. The temporary cap will include 500mm of compacted site-won soils.

1.5.4 Basal Lining System

As this is pertinent to this SRA a detailed presentation is given in Section 2.3 of this report and, consequently, will not be reiterated here.

1.5.5 Waste Mass Model

The existing cells (1 & 2) are basally lined with a leachate management system also being established at the beginning of landfilling operations. This will be replicated for the remaining cells (3 to 6). It consists of creating basal gradients overlain with a leachate collection layer designed to convey all the fluid to one of four low points (in the north-western corner Cell 1, south-western corner of Cells 3 and 4, north-eastern corner of Cell 5 and south-eastern corner of Cell 6, where extraction wells are located. A pump is then utilised to transfer the collected leachate to a leachate pond.

For the purpose of the waste mass model, the future temporary waste slopes adjacent to the quarry excavation on the western extent of the current landfill are proposed/modelled at a gradient of 1V:3.0H, to maximum 30.2m vertical height.

The waste shall ultimately be placed in line with the pre-settlement restoration levels at a maximum gradient of approximately 1V:5H slopes on the southern, northern and eastern extents with these three sides forming a west to east ridgeline at an elevation of 312mAUD. The proposed restoration profile is presented on Drawing TW20122 C-106.

There is no site wide leachate monitoring to determine the true leachate levels across the currently filled landfill. Notwithstanding this, for the purpose of this assessment, the pore-water pressure in the waste mass has been taken as a function of the overburden stress (R_u). A R_u value of 0.0, 0.1 and 0.2 have been utilised in all limit equilibrium assessments for the Waste Mass Model. It should be noted that the design, as a whole, minimises the risk of perched leachate within the waste, by having a basal collection system together with a granular regulating layer beneath the cap and sand being employed as daily cover. Additionally, the waste has a greater comparative percentage of construction material which would increase the overall porosity of the mass. As such, under normal conditions there is little opportunity for perched leachate to occur and, if they do, they are usually limited in extent. Nonetheless, the integrity of the waste mass has been tested for a theoretical presence of leachate. As it is not possible to predict if, and where, leachate would perch, the use of the R_u concept is the only practical approach to assess the potential presence of leachate.

Notwithstanding the aforementioned, Opalvale practices leachate recirculation. The potential adverse effect is to increase the pore pressures in the waste mass. Normally, it is unlikely for the R_u to rise above 0.1, but with recirculation in place it is feasible for the R_u to increase beyond this. The waste model will be split to divide the waste section into two, a near slope zone and the remainder set back 30m from the temporary slope face. This permits an assessment of varying the R_u in both zones and simulate the effect of restricting recirculation close to the slope.

1.5.6 Landfill Gas Management

A total of 21 vertical gas extraction wells will be installed in Stage 1 (cells 1 to 6) in a regular grid-like pattern at a spacing of 50m. The gas wells will be designed to actively extract the gas for transmission through a network of gas mains to a suitable gas destruction system. The conceptual design of the system is presented in detail within the CPCMP.

2 Screening

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

The principal components considered are:

- The basal subgrade;
- The side slope subgrade;
- The basal lining system;
- The side slope lining system;
- The waste; and
- The capping system.

The principal components relating to stability and integrity of the proposed development have been subject to review to determine the need to undertake further detailed geotechnical analyses.

2.1 Basal Subgrade Screening

2.1.1 Deformability

Some investigation data was made available beneath the landfill footprint to ascertain the geological sequence following the historic quarrying operations.

The subgrade consists of the superficial of weathered granitic rocks that have produced a fine to medium sand, and because of its granular nature any effective settlement in relation to the imposed stress from the waste mass will be nominal and not considered a risk to the integrity of the basal containment. The location of Cell 3 and 4 lies within a former quarry and, as a result, approximately 8 metres of sand has been removed. This means that the base of the site was previously loaded by approximately 150kPa. This, by itself, is equal to the mass imposed by 15m of waste. As such the formation has already been preloaded to between 40% and 50% of the maximum loading likely to be experienced within these cells. This minimises the risk of unacceptable strains within the subgrade.

Additionally, the basal containment has been assessed by Golders in carrying out their technical studies. Section 7 of their report identifies that the compression of the subgrade is unlikely to generate tensile strains in the lining system greater than 0.5% which is considered acceptable.

As such, this is not considered further in this assessment.

2.1.2 Basal Heave

The design carried out by IW Projects, supported by Stass, and more recently by Talis demonstrate the underlying maximum seasonal potentiometric head recorded is at an elevation of 273mAUD which means it lies a minimum of 3m below the basal lining system. As a result, if this buffer is maintained, basal heave via hydraulic uplift is not considered a viable failure mechanism.

This is not considered further in the assessment.

2.1.3 Cavities in the subgrade

No underground mining activity is known to have occurred in the vicinity of the landfill footprint. Being granitic in nature the underlying geology is not conducive to the formation of solution features.

Therefore, this is not considered further in the assessment.

2.2 Sideslope Subgrade Screening

The side slopes have been, with the remainder to follow, were excavated as part of the initial development works within the residual quarry excavation. All sideslopes are infilled and buttressed with waste deposits. In section 8 of their report, Golders has considered the stability of the side slopes within their technical studies and not deemed to be a concern.

Assessment of the side slopes has therefore been screened out from this report.

2.2.1 Deformability

As per the basal subgrade (Section 2.1.1) the sideslope subgrade comprises insitu and reformed Weathered Schist. It is therefore considered that effective settlement in relation to the imposed stress from the waste mass will be minimal.

This will not be taken forward for further consideration.

2.2.2 Groundwater

The highest groundwater levels revealed by the monitoring wells shows the phreatic surface to lie between 273m and 274mAUD over the majority of the base for Cells 1 to 6. The focus of this SRA is the combination of Cell 3 and 4's basal engineering, the waste mass and the restoration profile and, as such, a key issue will be the relative distance between the groundwater and the toe of the restoration profile. This is at a minimum along the temporary western edge where the ground level is around 278mAUD, giving a separation distance of approximately 5 metres from the groundwater. Along the southern, northern and eastern boundaries the separation distance increases appreciably and lies between 9m and 22m.

Although, the separation distance is reasonably large, its presence has been included as a potential factor of influence with the assessment and is included, principally, in the Slope Stability Analyses.

2.3 Basal & Sideslope Lining System Screening

2.3.1 Basal Lining Stability

The basal lining system comprises (in ascending order):

- Attenuation Subgrade Layer – A nominal 500mm thick engineered layer will be constructed by compacting in-situ soils on the base and side slopes of the landfill to form an engineered attenuation layer. The key purpose is to provide a level of natural attenuation and a suitable engineered surface for the placement of the geosynthetic lining system.
- Geosynthetic Clay Liner (GCL) – A low permeability GCL, consisting of a layer of bentonite sandwiched between two layers of needle punched geotextile, will be installed in direct contact with the Compacted Subgrade Layer as the lower (secondary) sealing layer in the composite lining system.
- 2.0mm High Density Polyethylene (HDPE) Geomembrane Layer – A HDPE geomembrane will overlie the GCL to form the upper (primary) sealing layer of the composite lining system. The HDPE liner is welded together to form a continuous artificial barrier to facilitate the direction of leachate towards the leachate extraction point.
- Protection Geotextile Layer – The lining system will be protected from the overlying materials by a non-woven protection geotextile. The protection geotextile will be specified to account for the grading of the gravel and long-term loading from waste disposal operations.

- Leachate Drainage Layer – A 300mm thick layer of non-calcareous aggregate designed to transmit leachate to the sumps for extraction and treatment by evaporation.
- Separation/Filter Geotextile Layer – A non-woven separation geotextile will separate the underlying leachate drainage aggregate from the overlying waste to minimise sedimentation of the Leachate Drainage Layer.

This was essentially designed by IW Projects and assessed by Golders. Moreover, the system has been approved for development. Therefore, further consideration is not considered necessary.

Assessment of the basal lining system has consequently been screened out.

2.3.2 Sideslope Lining Stability

Although the side slope lining system will be only temporarily exposed before waste is deposited against it and consequently buttressing the face, it is important to assess whether or not there is any temporary risk of instability, especially during construction.

Assessment of the side slopes has therefore been included.

2.3.3 Waste Mass Screening

For the purpose of the waste mass model, the temporary waste slope that will be formed on the western extent of Cell 3 and proposed/modelled at a gradient of 1V:3.0H.

The waste shall ultimately be placed in line with the pre-settlement restoration levels at a maximum gradient of approximately 1V:5H slopes on the southern side of the facility meeting at a west to east ridgeline at an elevation of 312mAUD.

In the case of unconfined (temporary) waste faces, the stability of the unconfined waste mass may be affected by leachate head within the waste that could increase pore fluid pressure.

The landfill is currently in its comparatively early stages of infilling and therefore the existence of significant perched leachate is unlikely. The engineering design and the use of granular daily cover further minimises the risk of perched systems. Additionally, the higher proportion of construction waste will promote a more porous waste mass. However, recirculation is practised at the site and therefore for the purpose of the assessment the pore-water pressure in the waste mass as a function of the overburden stress has been adopted to represent the potential effect that leachate and gas that could increase pore fluid pressure within the waste. R_u values of 0.0 and 0.1 have been modelled for the near surface, during further assessments of the temporary slope, (30m in from the face of the slope) but includes a value of R_u up to 0.2 for the main body of the waste deposited in the cells. This simulates the potential to isolate the near slope waste from the remainder of the waste to consider the potential restriction in the location of the recirculation operations.

2.3.4 Internal Slope Stability

The stability of the basal lining system placed on the internal side slopes is being assessed herein as confirmed by Section 2.3.2. There also remains the potential for the waste to fail as a whole with the sliding mechanism passing from the top of the restoration profile, or near to it, and then travelling through the quarry walls or upwards along the weaker basal lining system.

As such, the potential for this type of potential failure will be assessed as part of this SRA.

2.3.5 Construction

Construction vehicles shall not be allowed to operate directly on top of the geosynthetic basal lining system and conventional wheeled and tracked construction plant only be permitted to travel over the geosynthetics on haul roads that have a minimum thickness of 1m and constructed out of suitable soils material. It is proposed that the soils are placed and spread upslope as per good practice to prevent tension/damage within the lower geosynthetics.

The potential effects of construction plant activity on the 1V:3H slope gradient during placement of leachate collection layer should be considered as geosynthetics are to be used in the basal engineering.

2.3.6 Wind Uplift

There is the possibility that the temporarily exposed geosynthetics on the side slopes could be uplifted by the wind. When the wind blows, the air pressure varies locally (i.e. increases or decreases), depending on the geometry of obstacles met by the air flow, and therefore has the potential to develop large negative air pressures (suctions). Surcharging of the temporary capping slope will therefore need to be considered.

2.4 Capping Screening

All aspects of the stability of the restoration and capping layer has been screened out of this report because this is the subject of a separate Stability Risk Assessment previously conducted by Talis.

3 Stability Risk Assessment Modelling

3.1 Modelling Approach & Software

A stability assessment undertaken represents the considered scenarios for the different modelled phases of Cell 3 and 4's 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 tension within liner components).

Methods used in this Stability Risk Assessment include:

- Limit equilibrium stability analyses for the derivation of factors of safety for the sideslope and outer quarry walls along the southern boundary of Cell 3 and 4.
- Limit equilibrium analyses for the derivation of factors of safety for the temporary waste slope at the western end of the Cell 3.
- Closed form analysis for the stability of the basal side slope liner on the temporarily exposed cell side walls.

The stability analysis program SLIDE2 (Version 9.008) from RocScience has been used to undertake the limit equilibrium using the Bishop simplified and Janbu for potential non-circular forms of analysis.

TR1[1] states 'circular surfaces are seldom appropriate in the study of landfills, with recorded failures for lined landfill sites defined by Koerner and Soong[2], 1998b, as translational. This is largely due to the inherent anisotropy formed by the layering created by the deposition of the individual waste layers, placement of daily cover and the potential presence of perched leachate. The limit equilibrium analyses for the waste mass modelling have therefore been undertaken using non-circular analysis for R_u up to 0.2, depending on the analytical model, both modelled with and without seismic loading scenarios.

The scenarios assessed are considered to be the critical worst case (highest) slopes, notably the 30m high western 1V:3H slope that will be temporary prior to the future phases being developed and the 25.5m high 1V:5H permanently capped southern slopes. Clearly, flatter slopes will generate greater (higher) factors of safety for the same conditions.

SLIDE2's Auto Refine Search was utilised as the search method to define the critical 'circular' slip surfaces within SRA. 'The Auto Refine Search method uses a simple but effective algorithm for iteratively refining the search area on the slope, until the critical surface is located'. (RocScience 2016).'

The inherent 'Cuckoo' Search approach was utilized for 'non-circular' slip surfaces. 'The 'Cuckoo' Search is a global optimization algorithm search method. The Cuckoo search has been found to be much faster than "Simulated Annealing" method within the software, and in many cases also finds a lower safety factor slip surface. For this reason, the Cuckoo Search is recommended as the initial and primary search method which should always be tried first for a slope model with non-circular failure modes. (RocScience 2016).'

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[1] Jones, D.R.V. & Dixon, N. (2003). 'Stability of landfill lining systems: Literature review, Environment Agency Research and Development Project P1-385', Report 1.
[2] Koerner, R.M. & Soong, T.-Y. (1998b). 'Analysis and critique of ten large solid waste landfill failures', GRI Report No. 22, Geosynthetic Research Institute, December 1998.

The minimum calculated FoS values presented within the SRA report (critical slip surfaces) are the lowest reported values for the scenarios assessed, are within the extents of the model and are not believed to be generally constrained by the slope limits or external boundaries.

3.2 Data Parameters

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

- Material unit weight in kN/m^3.
- Drained shear strength of the various soil strata, the interfaces between the geosynthetic components and the waste. Shear strength has largely been defined using the effective shear strength parameters of cohesion, (c'), and the angle of shearing resistance, (ø').
- Groundwater has been taken from the maximum elevations obtained the hydrogeological assessment.

The geotechnical soil parameters used in this SRA for the basal geology and waste mass will, for consistency, be those employed by Golders in their Report. Section 8.2.3.3 confirms that the shear strength parameters they adopted for the basal weathered soils was an angle of friction (Phi) of 24 degrees with zero cohesion (c). This is the lower bound of the strength of the assessment they performed. This value will be taken forward to the stability assessment as corroborated in Table 3-3.

The corresponding shear strength for waste has been taken as Phi=25 degrees and C=5kPa as reported in Section 8.3.1.1 of the Golder's Report. Their assessment of waste strength is verified by the conservative values of effective shear strength parameters as derived from a study of geotechnical properties of municipal waste by Van Impe and Bouazza[3] , these values being backed up in later work by Kavazanjian et al[4] and later confirmed in a research summary by Jotisankasa[5].

The values for c' and ø' adopted throughout the modelling were 5kPa and 25°, respectively.

It should be appreciated that the shear strength of waste will vary considerably depending on composition and strain. The landfill will accept a range of municipal, commercial. construction and demolition wastes. The aforementioned values relate to more municipal and putrescible wastes and, therefore, the additional construction waste is likely to increase the effective shear strength properties. However, because there are no site-specific shear values available, the more conservative set of data will be incorporated into the analysis. In reality, the shear strength of the waste mass is likely to be higher.

Laboratory testing was performed on a set of samples of the soil that would be used as the 500mm attenuation layer beneath the basal geosynthetics. This was undertaken by E-Precision Laboratory, in their National Association of Testing Authorities (NATA) accredited laboratory. The results of laboratory characterisation testing are presented in Appendix B.

Tables 3.1 and 3.2 present a summary of the principal geotechnical characteristics of this set of data.

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[3] Van Impe, W. F. and Bouazza, A., 'Geotechnical properties of MSW', draft version of keynote lecture, Osaka, 1996.
[4] Kavazanjian, E., Matasovic, N., Bonaparte, R. & Schmertmann, G.R. (1995), 'Evaluation of MSW properties for seismic analysis'. Proc. Geo-environment 2000, ASCE Special Geotechnical Publication, pp 1126-1141.
[5] Jotisankasa, A., 'Evaluating the Parameters that Control the Stability of Municipal Solid Waste Landfills', Master of Science Dissertation, University of London, September 2001.

Table 3-1: Summary of Consolidated Drained Shear Box Testing

Sample	Soil	Peak Angle of Shearing Resistance ø' (°)	Peak Effective Cohesion c′ (kPa)	Post Peak Angle of Shearing Resistance ø' (°)	Post Peak Effective Cohesion c′ (kPa)
LF_SOIL_TALIS2012_DDST3	Light Brown Fine to Medium Sand	28.4	34	21.8	21.2

Table 3-2: Summary of Basic Geotechnical Properties

Sample	Soil	Maximum Dry Density (Mg/m3	Optimum Moisture Content (%)	D_{10} (mm)
LF_SOIL_TALIS2012	Light Brown Silt and Fine to Medium Sand	1.72	17.5	0.0045

Apart from the attenuation layer and the underlying geology, the majority of the basal containment engineering comprises manufactured geosynthetics and a specified leachate drainage gravel. As these elements, at this time, are yet to be confirmed, because they will be obtained and deployed by the successful construction contractor, it is important that the SRA considers appropriate design parameters that can be achieved via the normal construction process.

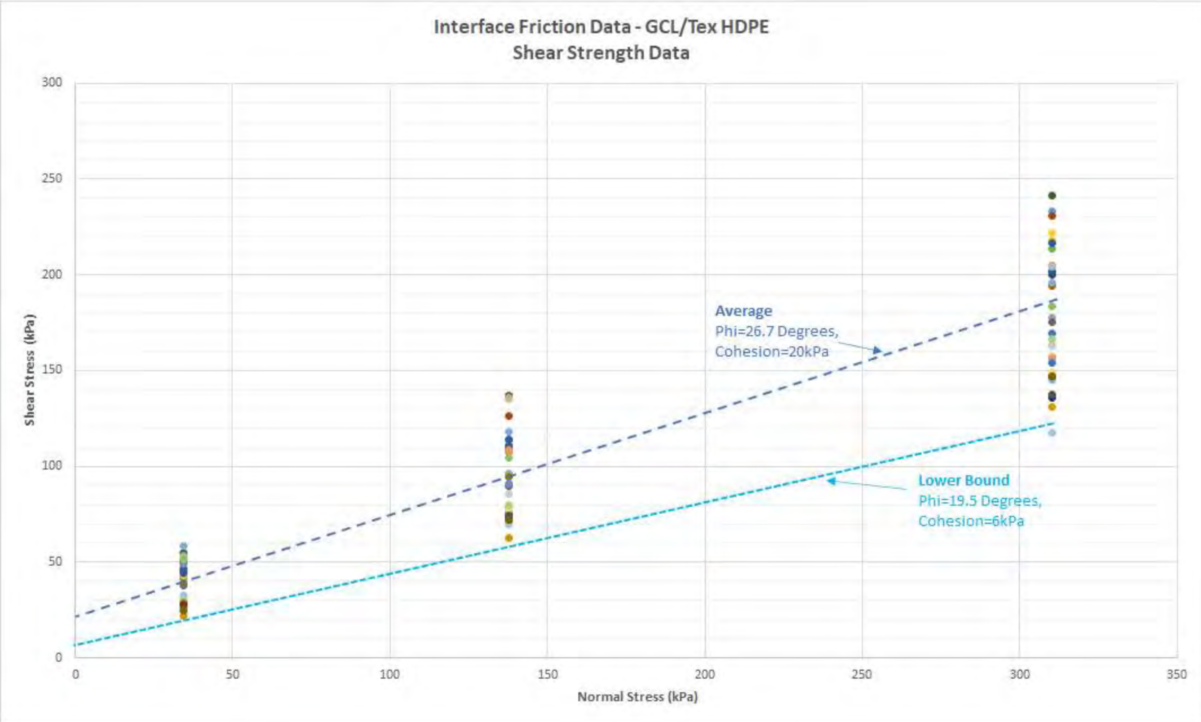
TR1[1] has conducted a literature study on interface friction for materials against a textured geomembrane such as a sand and gravel. For this SRA, the values recommended therein have been employed, as reproduced in Table 3-4. Unfortunately, TR1[1] does not cite appropriate design data for the interface friction of a non-woven Geosynthetic Clay Liner (GCL) placed in concert with a textured HDPE geomembrane.

A reasonably comprehensive study by McCartney, Zornberg and Swan[6], examined the interface shear strength from 34 tests between a non-woven GCL and textured HDPE geomembrane.

Talis has taken the data from the shear box tests and plotted them on a single chart which is shown in Figure 3-1.

6 Zornberg, J. G., McCartney, J. S. and Swan Jr, R. H., 'Analysis of a Large Database of GCL Internal Shear Strength Results', Journal of Geotechnical and Geoenvironmental Engineering, 2005.

Figure 3-1: Summary of GCL/Textured HDPE Interface Shear Box Data from McCartney, Zornberg and Swan



As can be seen there is a wide range of results and the average regression through the points leads to an angle of friction of 26.7 degrees with a correspondingly high cohesion of 20kPa. It would be unreasonable at this stage to adopt such design parameters because, based on the data, there would be a 50% risk that the actual strengths in the field would not match the design values.

A lower bound design line is considered more appropriate and if bettered in the field would yield a higher factor of safety against a potential failure. As viewed, the risk of not achieving the angle of friction of 19.5 degrees and a cohesion of 6kPa is minimal.

In any case it will a prerequisite in the contract documents that the aforementioned shear strength properties must be achieved and will be central to the CQA regime employed during the contract.

As part of the detailed design, or prior to tendering the construction contract, appropriate laboratory testing should be carried out for the following geosynthetic interfaces:

- Leachate drainage aggregate against the protection geotextile
- Protection geotextile against the HDPE Geomembrane
- The HDPE geomembrane against GCL
- The GCL against the Engineered Attenuation Layer

The above testing will be needed to confirm that the real-world shear strength achieves the minimum standard set by the design and as reproduced in Table 3-4. It will be essential that for each interface a statistically meaningful number of tests are performed. As can be seen from Figure 3-1 a potential erroneous value could be found from carrying out a single data point.

Elsewhere in the assessment, where no direct measurement of a particular property is available, reference has been made to relevant experience from the same or similar materials.

Considering the lab testing data, from Golder’s report and literature the following parameters are used.

Table 3-3: Material Parameters

Material	Bulk Unit Weight γ (kN/m^3)	Effective cohesion c′ (kPa)	Angle of Shearing Resistance ø' (°)	Typical Description
Leachate Collection Gravel	18	0	30	Fine Gravel
Waste	10	5	25	Mixed Putrescible Waste.
Basal Soils*	20	0	24	Weathered Schist

Nb: from Golders Report "Opal Vale – Technical Studies in Support of the Design – December 2014" and estimated

For the closed form analysis interface design parameters are presented in Appendix E, friction angles and cohesion are as published in R&D technical report TR1[1], McCartney, Zornberg and Swan[7] and Talis historic shear box data.

Table 3-4: Closed Form Interface Design Parameters

Interface	Peak	
	c' (kPa)	ø'
Attenuation Soil[1]	5	28
Attenuation Soil/GCL[2]	0	33
GCL/Textured HDPE Geomembrane[3]	3 (6)	19.5
Textured HDPE Geomembrane/Protection[4] Geotextile	6.9	25.8
Protection Geotextile/Leachate Collection Layer[5]	0	35

1. Site Specific Laboratory Test designated LF_SOIL_TALIS2012_DDST3
2. Estimated from data within TR1.
3. From data obtained from McCartney, Zornberg and Swan. Cohesion intercept on the graph is 6kPa, but has been reduced for additional conservatism
4. Taken from TR1.
5. Taken from TR1.

From an inspection of the data in Table 3-4 is it clear that the weakest set of interface data is that pertaining to the GCL/Geomembrane. This data set will be used as the critical set of shear strength data in the stability assessment.

3.3 Seismic Conditions

There is no set guidance requirement in Australia for assessing seismic conditions for solid waste facilities. ICOLD 'Selecting Seismic Parameters for Large Dams Guidelines'[7], calculates total risk factor based on capacity, height, evacuation requirements and potential downstream damage. Applying the ICOLD guidance, the risk factor ratings to the proposed closure plan design are: Capacity 1-120 hm^3

[7] ICOLD (International Commission on Large Dams), Selecting Seismic Parameters for Large Dams Guidelines, 2009.

(High [4]), Height 15m-35m (Moderate [2]), Evacuation Requirements - None (Low [0]), Potential downstream damage, (Low [4]).

Total Risk Factor = RF Capacity + RF Height + RF Evacuation Reqts + RF Potential Downstream Damage

Total Risk Factor = 4 + 2 + 0 + 4 = 10

Total Risk Factor between 7-18 = Risk Class (Risk Rating) II (Moderate)

For a moderate ('significant' Class II risk class) category dam, ANCOLD July 2019 'Guidelines for Design of Dams and Appurtenant Structures for Earthquake[8] Table 2.1' recommend deterministic analysis seismic design ground motions for Operating Base Earthquake (OBE), and Safety Evaluation Earthquake (SEE) [Maximum Credible Earthquake - MCE] return periods are 1:475 and 1:1000 Annual Exceedance Probability (AEP), respectively.

The recently published Global Industry Standard on Tailings Management[9] states 'the selection of the design ground motion should consider the seismic setting and the reliability and applicability of the probabilistic and deterministic methods for seismic hazard design'. For significant consequence classification a 1:1000 AEP is recommended for maximum credible earthquakes for operations and closure (active care).

The 2018 National Seismic Hazard Assessment for Australia[10] (NSHA18) seismic design values, GIS data[11] indicates that the Site is located on the 0.05 Peak Ground Acceleration (PGA) contour for an annual probability of exceedance (AEP) of 1:475 at a Spectral Acceleration (SA) period of 0.0s, as shown on Figure 3-2.

The NSHA18[10] seismic design values, GIS data[11] indicates that the Site is located midway between the 0.16 and 0.2 (=0.18g) Peak Ground Acceleration (PGA) contour intervals for an annual probability of exceedance (AEP) of 1:2475 at a Spectral Acceleration (SA) period of 0.0s, as shown on Figure 3-3.

[8] ANCOLD (Australian National Commission on Large Dams), Guidelines for Design of Dams and Appurtenant Structures for Earthquake, July 2019.
[9] Global Industry Tailings Standard on Tailings Management, ICMM, UN Environment Programme, PRI – Principles for Responsible Investment, GlobalTailingsReview.org, August 2020.
10 Allen, T. I. 2018. 'The 2018 National Seismic Hazard Assessment for Australia': data package, maps and grid values. Record 2018/33. Geoscience Australia, Canberra. http://dx.doi.org/10.11636/Record.2018.033
11 https://data.gov.au/dataset/earthquake-hazard-risk-contour-map-national-geoscience-dataset

Figure 3-2: NSHA18 - 10% probability of exceedance in 50 years (1:475 AEP) contours

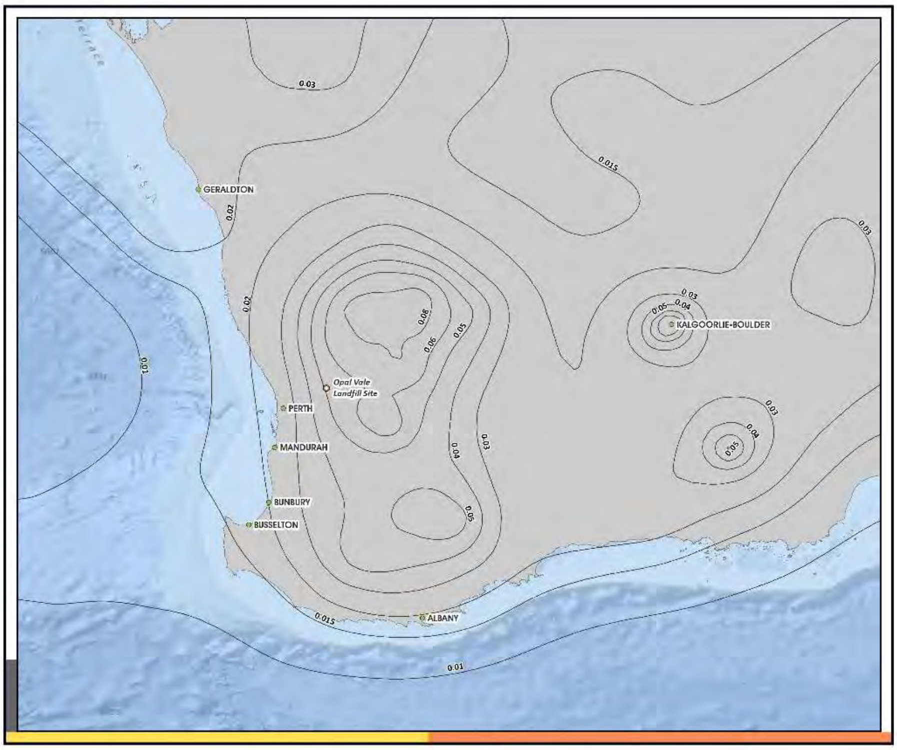
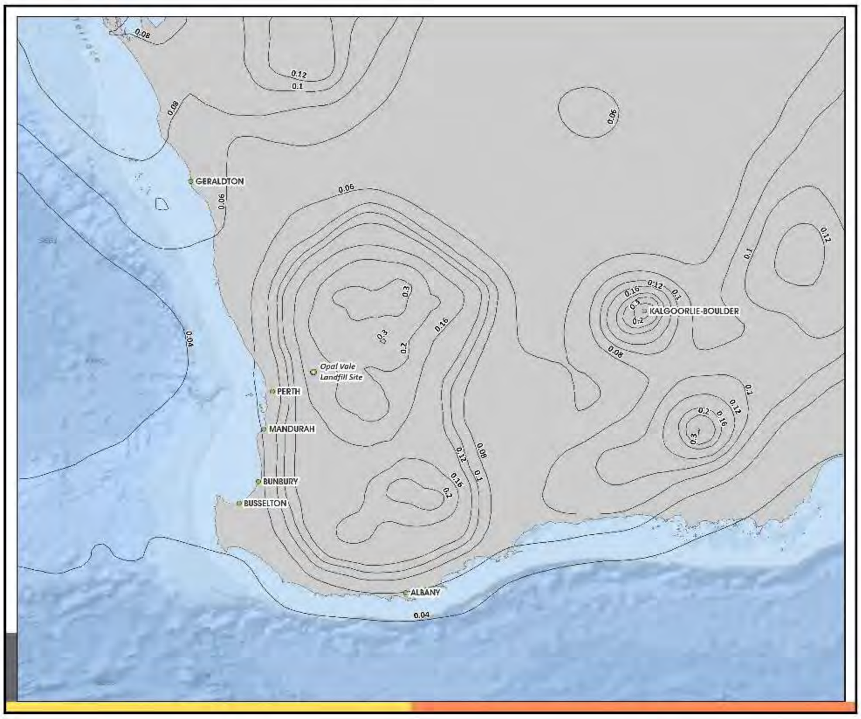


Figure 3-3: NSHA18 - 2% probability of exceedance in 50 years (1:2475 AEP) contours



Utilising a logarithmic interpolation between the conservative NHSA18 values of 0.05g and 0.18g for the 1:475 AEP and 1:2475 AEP respectively, a 1:1000 AEP equates to a PGA 0.109g.

AS1170.4[12] identifies the sub-soil class across the site as Class B_e – Rock and Class C_e – Shallow Soil. The normalised response spectra for the site sub-soil Class B_e indicates an amplification of 1.0 for a period of 0.0s, while for the site sub-soil Class C_e indicates an amplification of 1.3 for a period of 0.0s. The site sub-soil Class C_e amplification has been utilised within the assessment.

Horizontal seismic load coefficients for the pseudo-static seismic return periods based on the amplification factor of 1.3 are as follows:

- OBE. PGA 0.015g with an amplification of 1.3 relates to a horizontal seismic load coefficient of 0.065g
- SEE/MCE. PGA 0.109g with an amplification of 1.3 relates to a horizontal seismic load coefficient of 0.142g

Pseudo-static seismic return periods considered within the analysis were:

- 1:475 – Operating Base Earthquake (OBE)
- 1:1000 AEP – Safety Evaluation Earthquake (SEE) / Maximum Credible Earthquake (MCE)

3.4 Factors of Safety

There is no set guidance requirement in WA for minimum factors of safety for solid waste facilities, factors of safety have been established based on internationally accepted guidance and similar stability assessments of projects in NSW and interstate. The UK Environment Agency document TRI2[13] states "Slopes should be designed to obtain factors of safety in the region of 1.3 to 1.5".

ANCOLD Guidelines on Tailings Dams[14] indicates recommended minimum factors of safety for tailings dams as 1.0-1.2 for pseudo-static loading conditions.

For the limit state equilibrium analyses, a factor of safety of ≥1.5 is considered appropriate when using peak shear strength parameters under static loading. A factor of safety of ≥1.1 under earthquake loading for an operating base earthquake (OBE), and a factor of safety of ≥1.0 for a safety evaluation earthquake (SEE) / Maximum Credible Earthquake (MCE).

For the closed form interface analyses, construction plant and gas pressures, a factor of safety of 1.3 is considered appropriate when using conservative peak shear strength parameters, and a factor of safety greater than unity for reduced post peak shear strength parameters.

The risk of failure of the lining system will be assessed in terms of interface stability with acceptable tension induced in the lining system geosynthetics.

For temporary waste slopes where the slopes will be buttressed with the filling operations in the adjacent cell over a short period of time, a factor of safety of ≥1.3 is considered appropriate when using peak shear strength parameters under static loading.

[12] AS1170.4, Australian Standard – 'Structural design actions Part 4: Earthquake actions in Australia'. 2nd Edition 2007.
[13] N Dixon and D R V Jones, 'Stability of Landfill Lining Systems: Report No. 2 Guidance, R&D Technical Report P1-385/TR2', Environment Agency UK, 2002
[14] ANCOLD, 'Guidelines on Tailings Dams – Planning, Design, Construction, Operation and Closure', May 2012

3.5 Modelling Results

3.5.1 Basal & Sideslope Subgrade Analysis

The requirement for an analysis of the heave potential and deformability of the basal subgrade has been screened out.

3.5.2 Basal & Sideslope Lining System Analysis

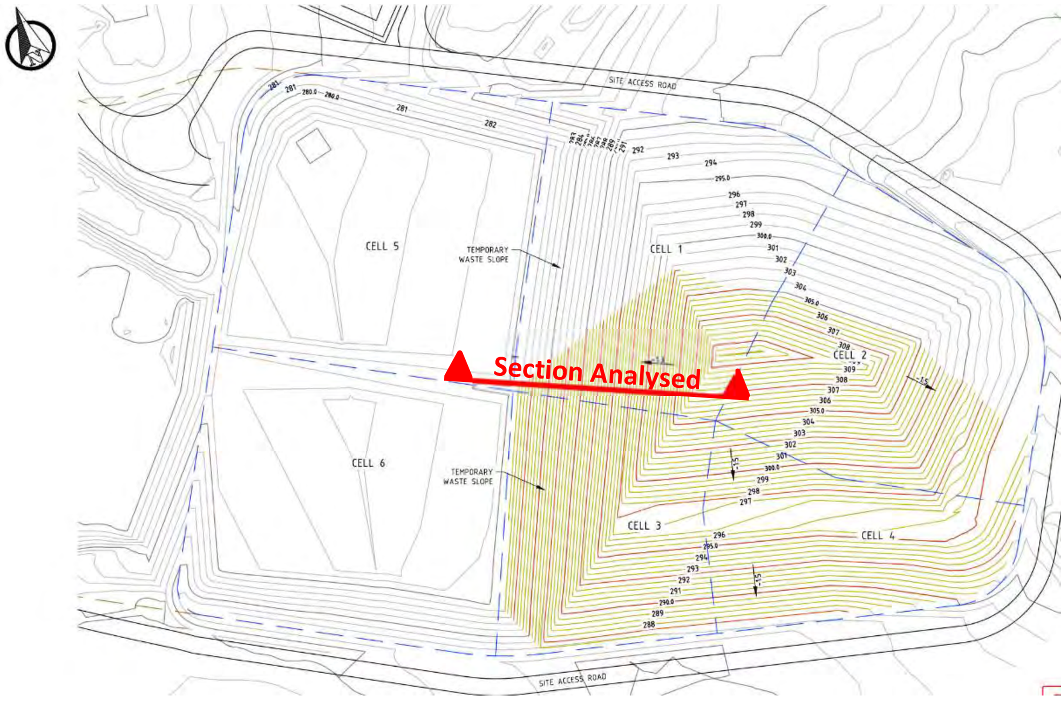
The requirement for an analysis of the basal and sideslope lining system has been screened out.

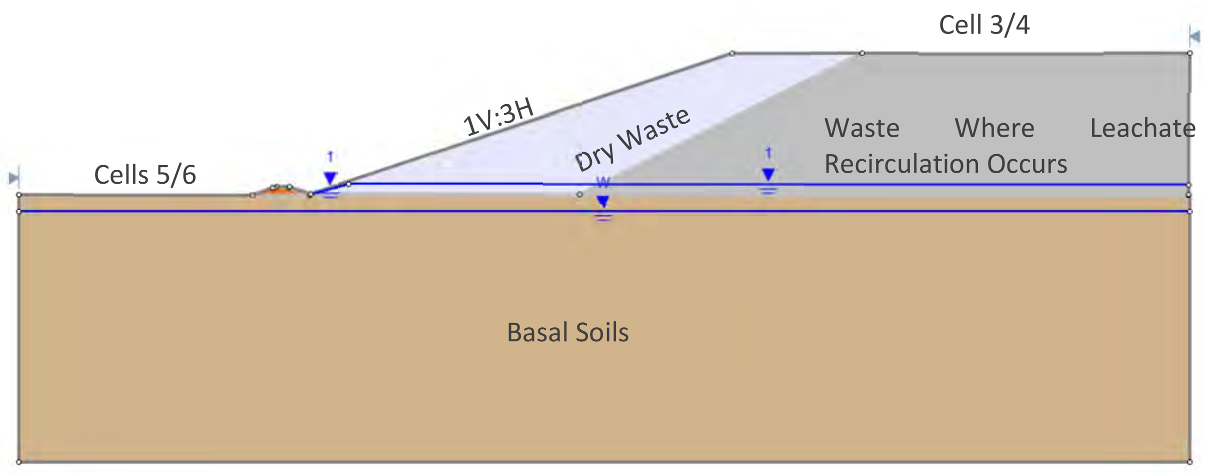
3.5.3 Waste Mass Analysis

The limit equilibrium analyses for the waste mass modelling have been undertaken using the Bishop and Janbu non-circular forms of analysis, for the 1V:3.0H temporary waste slopes adjacent to the yet to be filled Cells 5 and 6. In the case of unconfined (temporary) waste faces, the stability of the unconfined waste mass may be affected by leachate head within the waste that could increase pore fluid pressure. As described in Sections 1.5.5 and 2.3.3, for the purpose of the assessment, a R_u value of 0.0 and 0.1 has been utilised to represent the potential effect that leachate in the that could increase pore fluid pressure within the waste located within 30m of the slope face. However, in view of the recirculation practised at the site the R_u will be increased to 0.2 for the main body of the waste mass.

The typical section analysed is illustrated by the extract taken from the drawings and SLIDE2 in Figure 3-4.

Figure 3-4:: Generic Cross Section Through Edge of Filling Operations Cell 3 and Cell 5/6





The Waste Mass Stability Result summary is presented in Table 3.5.

Table 3-5: Waste Mass Analysis Results

Scenario	Method	Factor of Safety	Comments
Temporary Capping Slope Outer Slope Waste Dry (Ru=0.0) No Seismic Loading	Drained Non-Circular	1.497	1V:3.0H temporary waste slope. R_u value of 0.0 applied to bulk of waste. Acceptable (FoS > 1.3)
Temporary Capping Slope Outer Slope Waste Dry (Ru=0.0) No Seismic Loading	Drained Non-Circular	1.501	1V:3.0H temporary waste slope. R_u value of 0.1 applied to bulk of waste. Acceptable (FoS > 1.3)
Temporary Capping Slope Outer Slope Waste Dry (Ru=0.0) No Seismic Loading	Drained Non-Circular	1.390	1V:3.0H temporary waste slope. R_u value of 0.2 applied to bulk of waste. Acceptable (FoS > 1.3)
Temporary Capping Slope Outer Slope Waste Dry (Ru=0.1) No Seismic Loading	Drained Non-Circular	1.369	1V:3.0H temporary waste slope. R_u value of 0.0 applied to bulk of waste. Acceptable (FoS > 1.3)
Temporary Capping Slope Outer Slope Waste Dry (Ru=0.1) No Seismic Loading	Drained Non-Circular	1.367	1V:3.0H temporary waste slope. R_u value of 0.1 applied to bulk of waste. Acceptable (FoS > 1.3)
Temporary Capping Slope Outer Slope Waste Dry (Ru=0.1) No Seismic Loading	Drained Non-Circular	1.378	1V:3.0H temporary waste slope. R_u value of 0.2 applied to bulk of waste. Acceptable (FoS > 1.3)

Temporary Capping Slope Outer Slope Waste Dry (Ru=0.0) with Seismic Loading (OBE - 1:475)	Drained Non-Circular	1.122	1V:3.0H temporary waste slope. R_u value of 0.0 applied to bulk of waste. Acceptable (FoS > 1.0)
Temporary Capping Slope Outer Slope Waste Dry (Ru=0.0) with Seismic Loading (OBE - 1:475)	Drained Non-Circular	1.123	1V:3.0H temporary waste slope. R_u value of 0.1 applied to bulk of waste. Acceptable (FoS > 1.0)
Temporary Capping Slope Outer Slope Waste Dry (Ru=0.0) with Seismic Loading (OBE - 1:475)	Drained Non-Circular	1.119	1V:3.0H temporary waste slope. R_u value of 0.2 applied to bulk of waste. Acceptable (FoS > 1.0)
Temporary Capping Slope Outer Slope Waste Dry (Ru=0.1) with Seismic Loading (OBE - 1:475)	Drained Non-Circular	1.037	1V:3.0H temporary waste slope. R_u value of 0.0 applied to bulk of waste. Acceptable (FoS > 1.0)
Temporary Capping Slope Outer Slope Waste Dry (Ru=0.1) with Seismic Loading (OBE - 1:475)	Drained Non-Circular	1.037	1V:3.0H temporary waste slope. R_u value of 0.1 applied to bulk of waste. Acceptable (FoS > 1.0)
Temporary Capping Slope Outer Slope Waste Dry (Ru=0.1) with Seismic Loading (OBE - 1:475)	Drained Non-Circular	1.037	1V:3.0H temporary waste slope. R_u value of 0.2 applied to bulk of waste. Acceptable (FoS > 1.0)

The lowest calculated factor of safety for the temporary waste slopes geometric scenario are greater than 1.3, and 1.0 for the static and OBE AEP scenarios, and are therefore considered acceptable. It should be noted that it is anticipated this face will not be exposed permanently and, as such, it is justified not to consider the 1 in 1000 years seismic event. With an exposure time measured in terms of a decade then it is unlikely that the 1:475 year event will manifest itself and with the eventual support provided by the waste from Cells 5 & 6 it is deemed appropriate for the slope to have a factor of safety that prevents failure.

Of greater importance is the nature of any potential leachate that accumulates within the waste mass, even over a relatively short period of time. Opalvale currently operates a system of recirculation which theoretically is designed to maximise the absorptive capacity of the waste. However, it may hold up leachate within the waste which produces a level of pore pressure which can influence the overall Factor of Safety.

As noted in Table 3-5 the Factor of Safety is marginal for the situation where the near slope waste becomes wetter (R_u=0.1) and there is, in the unlikely event, a medium level seismic occurrence. This applies to the condition where the main body of the waste is dry or even slightly wetter than that close to the slope. Although not shown in the Table if the waste in the slope (i.e. within the 30m zone) is subject to recirculation of leachate then there is a strong risk that the pore pressure would rise and the FoS would become less than 1.0. It is therefore essential that any recirculation of leachate is confined to the waste mass at least 30m from the top of the waste slope.

The factors of safety are similar for some scenarios modelled because the bulk of the waste mass has a comparatively higher effective stress then potential failure planes will be forced through the weaker near surface waste. If the Ru in the near surface waste remains the same in these scenarios, then the failure plane will be more or less the same and hence the FoS will also be more of less the same.

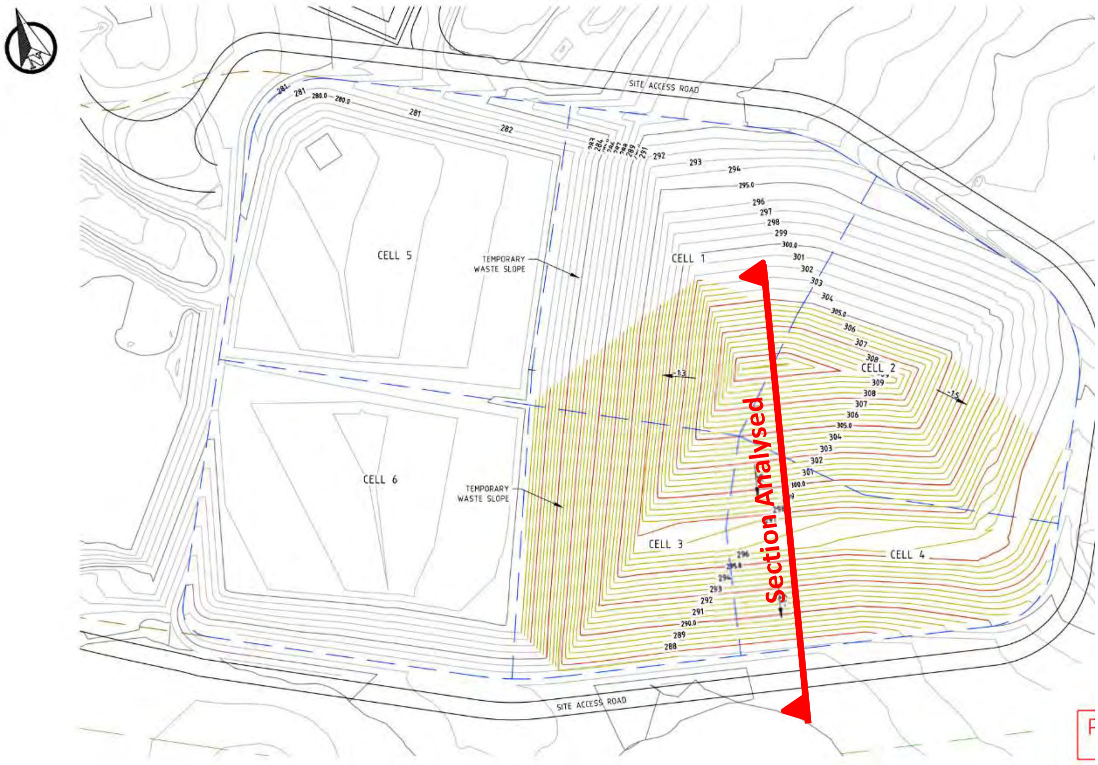
The model results for temporary waste slopes are presented in Appendix C.

3.6 Capping Stability Analysis

As there are essentially only two faces to Cells 3 and 4, apart from the temporary waste face presented in Section 3.5, there is just one other topographic scenario which needs to be considered, namely, a North to South Section perpendicular through the crown and part way down the other side of the profile achieving the maximum height difference of 25.5m from the toe. This is shown in Figure 3-5.

The critical pre-settlement slope gradient assessed is 1V:5H (~11.31°) over a maximum vertical height of approximately 25.5m. There is no as-built survey of the existing basal footprint, therefore for the purpose of the stability assessment an inferred excavation profile has been utilised. The limit equilibrium for the capping using the Bishop Simplified, Janbu and Morgenstern Price and Spencer non-circular forms of analysis. A summary of the results for the North to South Section is presented in Appendix D.

Figure 3-5: North to South Section through Southern Face of the Landfill



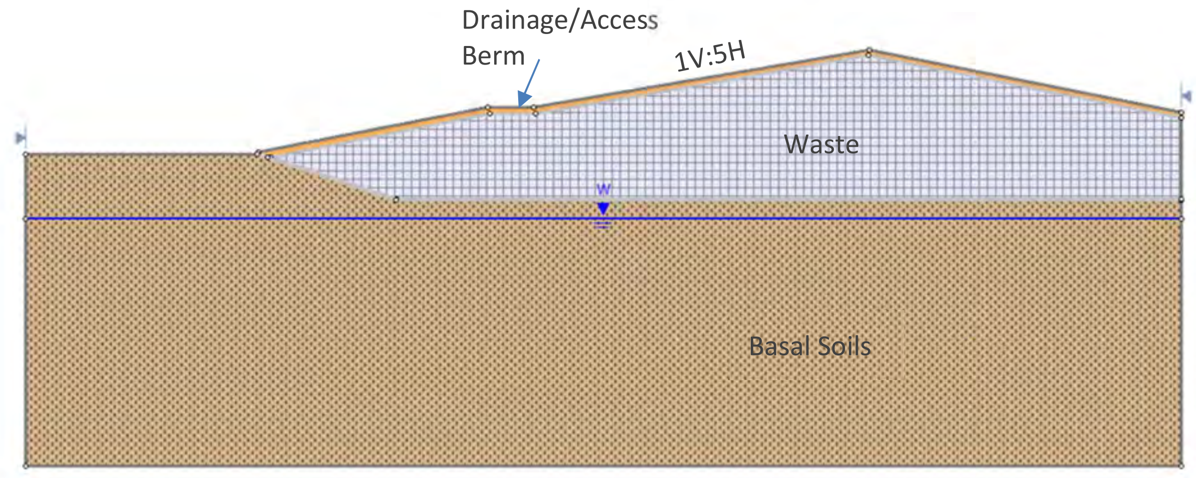


Table 3-6: Summary of Stability Analysis for Southern Face

Scenario	Method	Factor of Safety	Comments
Permanent Capping Slope General North to South Section No Seismic Loading	Drained Non-Circular	2.918	1V:5H Capping Profile – 25.5m vertical height. Ru Value of 0.0 applied to waste Acceptable (FoS > 1.5)
Permanent Capping Slope General North to South Section No Seismic Loading	Drained Non-Circular	2.705	1V:5H Capping Profile – 25.5m vertical height. Ru Value of 0.1 applied to waste Acceptable (FoS > 1.5)
Permanent Capping Slope General North to South Section No Seismic Loading	Drained Non-Circular	2.500	1V:5H Capping Profile – 25.5m vertical height. Ru Value of 0.2 applied to waste Acceptable (FoS > 1.3)
Permanent Capping Slope General North to South Section With Seismic Loading (OBE - 1:475 AEP)	Drained Non-Circular	2.141	1V:5H Capping Profile – 25.5m vertical height. Ru Value of 0.0 applied to waste Acceptable (FoS > 1.1)
Permanent Capping Slope General North to South Section With Seismic Loading (OBE - 1:475 AEP)	Drained Non-Circular	1.989	1V:5H Capping Profile – 25.5m vertical height. Ru Value of 0.1 applied to waste Acceptable (FoS > 1.1)
Permanent Capping Slope General North to South Section With Seismic Loading (OBE - 1:475 AEP)	Drained Non-Circular	1.839	1V:5H Capping Profile – 25.5m vertical height. Ru Value of 0.2 applied to waste Acceptable (FoS > 1.1)
Permanent Capping Slope General North to South Section With Seismic Loading (SEE/MCE - 1:1000 AEP)	Drained Non-Circular	1.633	1V:5H Capping Profile – 25.5m vertical height. Ru Value of 0.0 applied to waste Acceptable (FoS > 1.0)

Permanent Capping Slope General North to South Section With Seismic Loading (SEE/MCE - 1:1000 AEP)	Drained Non-Circular	1.503	1V:5H Capping Profile – 25.5m vertical height. Ru Value of 0.1 applied to waste Acceptable (FoS > 1.0)
Permanent Capping Slope General North to South Section With Seismic Loading (SEE/MCE - 1:1000 AEP)	Drained Non-Circular	1.375	1V:5H Capping Profile – 25.5m vertical height. Ru Value of 0.2 applied to waste Acceptable (FoS > 1.0)

NB: A Ru=0.2 pore pressure condition has been included in these results because these are permanent slopes.

Consultation of the table will reveal that the minimum factors of safety for all conditions are above the recommended minimum factor of safety. The visual output from SLIDE2 is presented in Appendix D.

3.6.1 Closed Form Analysis

A closed form analysis for the basal lining assessment has been undertaken to assess the stability of the unsupported, albeit temporarily, leachate collection material deployed on the cell's sideslope, and this is presented in Appendix E.

From Drawing TW21027 C-302_A, the leachate collection layer is to be deployed over a vertical height of 4.5m on the 1V:3H gradient, basal side slope. An attenuation layer which will provide a suitable foundation on which to place the geosynthetics comprising a Geosynthetic Clay Liner overlain by a textured 2 mm HDPE geomembrane and a protection geotextile. Overlying the geosynthetics is the 300mm thick leachate collection layer formed from a porous fine to medium gravel. This collects any percolating leachate filtering through the waste mass. In turn the leachate collection layer channels leachate to suitable extraction points. While exposed, prior to being covered with waste, the leachate collection layer will be exposed to rainfall. As such, in view of its porous nature, it is unlikely to amount to a great deal, but for the purpose of the closed formed analysis the presence of the possible fluid within the leachate collection layer will be modelled using the low parallel submerged ratio (PSR), as defined by Jones and Dixon[13].

The assessment was undertaken for both peak and post peak conditions although in reality the side slope with the leachate collection will not be exposed for particular long for the concept of post peak conditions to develop. The reason for the short exposure time is that the side slope drainage/protection layer is deployed in a number of stages in advance of the filling, so only 4.5m vertical height will be placed at any one time.

The results of the capping stability analysis are presented in Appendix D with a range of PSR's from 0 to 0.3. A PSR of 0.3 represents a saturated layer above the interface equivalent to 30% of the overlying thickness. It should be noted that this is not the same as R_u. When adopting the peak shear strength for the various liner interfaces for the 1V:3H capping profile, and a PSR of 0.3, the minimum reported factor of safety is 1.17, along the restoration HDPE/ GCL interface. For assumed post peak strength conditions with a PSR of 0.3 the minimum factor of safety reported is 0.97, along also the HDPE/GCL interface. As part of the analysis, no induced tensions are reported in the geosynthetics, although there is a theoretical failure. However, as stated previously it is very unlikely that post peak conditions would develop in the short time the slope would be exposed and therefore it is not considered a viable failure mechanism.

All FoS's calculated are in excess of the minimum values for the peak scenarios, and therefore deemed acceptable considering the restricted exposure time.

3.6.2 Plant Operations on Geosynthetics

Analysis has been carried out to determine the effects from construction plant on the placement of restoration soils on the 1V:3H geosynthetic basal side slopes. The stability of a 1V:3H basal side slope under the influence of construction plant operations has been assessed using the procedure proposed by Kerkes[15] and is presented in Appendix F.

It is assumed that the leachate collection gravel is spread upslope as per normal good practice to prevent tension/damage within the lower geosynthetics.

The analysis shows that based on a 0.3m thickness of leachate gravel, a factor of safety of 0.85 against rupture of the geomembrane assuming the lowest peak shear strength conditions (19.5° & 3 kPa) at the GCL to HDPE geomembrane interface at time of placement. The analysis has been undertaken assuming no limiting tension in the geomembrane. This also assumes that a specialist low ground pressure compact tracked loader/bulldozer is employed. This is not conventional earthmoving equipment used in large scale earthmoving operations and would have to be specifically mobilised to site. As can be seen the sub 1.0 FoS constitutes a theoretical failure. However, as previously stated this assumes there is no tensile stress transferred to the geomembrane. If a small amount, say 5kPa, is permitted then the FoS increases to a still unacceptable 0.97. The main cause of the unacceptable FoS's is the comparatively thin amount of gravel above the geosynthetics so there is little distance for the stresses from the bulldozers tracks to dissipate. When compared with what is deemed an acceptable factor of safety of 1.3, conventional mechanical construction practice is not recommended. In order to achieve an acceptable FoS using a specialist compact low ground pressure bulldozer the thickness of the leachate gravel would have to be increased to at least 600mm.

Alternatively, the material could be initially stockpiled at the base of the slope and then backcast with a backhoe excavator over the slope without having to traffic over the gravel. However, this is a much slower means of construction. Once the first layer or two of waste is deposited the next 300mm layer can be constructed. However, it is unlikely that a vertical height of 4.5m could be deployed this way in one go.

3.6.3 Wind Uplift

In order to assess the possibility of wind uplift on the temporarily exposed geosynthetics on the side slope an assessment has been undertaken utilising the method proposed by Giroud[16] and the adoption of the AS 1170.0[17] AEP design events. Salt Valley is located in Wind Regions A1 as indicated by AS1170.0. The design working life for construction equipment and temporary works for an importance level 2 indicates a 1:100 AEP wind design event.

AS1170.0 documents the peak wind gust data for Region A1 (non-cyclonic) for a 1:100 AEP as 41m/s, ≈ 148km/hr.

Assuming a wind suction factor of 0.7, density of air 1.2kg/m^3 as per AS1170, density of 2.0mm HDPE 940kg/m^3 and wind gusts 41m/s, the surcharging of the temporarily exposed slope to resist uplift are indicated to be 75 kg/m^2.

The calculations with the AS1170 AEP peak wind velocities indicate a significant surcharge is

[15] Kerkes, D.J., (1999),'Analysis of equipment loads on geocomposite liner systems', Proc. Geosynthetics 1999
[16] Giroud J.P. Pelte T. & Bathurst R.J. 1995, 'Uplift of Geomembranes by Wind', Geosynthetics International, Vol. 2, No. 6, pp. 897-952.
[17] AS/NZS 1170.0:2002 Structural Design Actions – Design Principles. Standards Australia.

required on the temporary exposed side slopes to resist wind uplift. Where the 300mm leachate gravel covers the geosynthetics then this is more than sufficient. However, for the upper exposed slopes, Opalvale is recommended to consider the exposure time, i.e. when buttressing with adjacent waste deposits are likely to be undertaken, to determine the exact requirements of surcharge during detailed design.

3.7 Assessment Summary

3.7.1 Seismic Conditions

ANCOLD states if a pseudo-static analysis is undertaken, a factor of safety greater than 1.0 may be taken as indicative of limited deformation being caused by the design earthquake. The US EPA 'Seismic Design Guidance for Municipal Solid Waste Landfill Facilities'[18] states "If the minimum factor of safety, FoS_{min}, exceeds 1.0 and 0.3m (1 ft) of deformation is acceptable, the seismic stability analysis is completed." All analysed scenarios with regards to the OBE and SEE/MCE have a FoS >1 therefore, no deformation analysis is deemed to be required.

All FoS's calculated during seismic conditions assessed are in excess of the minimum values.

3.7.2 Basal & Sideslope Assessment

No global basal and side slope assessments were undertaken as they were included as part of the Golder's Technical Studies and this showed the strains to fall within acceptable tolerances. As such, this was screened out from this SRA.

An analysis was carried out to determine any risk of instability of the leachate collection/protection layer on the basal side slope. This showed that even with Specialist Low Ground Pressure construction equipment then unacceptable FoS's are achieved. Consideration should be given to either increasing the thickness of the layer to 600mm or backcasting the material with a backhoe onto the slope without trafficking it.

3.7.3 Waste Mass Stability

Temporary capped waste slopes are proposed/modelled at a gradient of 1V:3.0H, to maximum restoration height adjacent to the quarry. The waste shall ultimately be placed in line with the pre-settlement restoration levels at a maximum gradient of approximately 1V:5H grading to a west to east ridgeline at 312mAUD.

In the case of unconfined (temporary) waste faces, the stability of the unconfined waste mass may be affected by leachate head within the waste that could increase pore fluid pressure. The waste mass was therefore modelled with a R_u value of up to 0.1 to represent the potential effects that leachate and gas could have on increasing pore fluid pressure within the near surface 30m of waste and up to R_u=0.2 for the bulk of the waste mass. This is to reflect the recirculation practices occurring on site.

The hydraulic head of leachate over the landfill liner surface should continue be managed during the landfill operation and closure phases in accordance with best practice standards through extraction of leachate from the sumps. Leachate levels on the landfill base should be maintained as low as reasonably practicable through regular extraction.

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[18] RCRA Subtitle D (258) 'Seismic Design Guidance for Municipal Solid Waste Landfill Facilities'. US EPA, EPA600/R-95/051, April 1995.

The calculated factor of safety for the temporary waste slopes are greater than 1.3 and 1.0 for the static and OBE AEP scenarios, and are therefore considered acceptable.

3.7.4 Basal Cell Assessment

For Cells 3 and 4 the main and important section lies on the southern edge of the landfill. An assessment of this Southern Section is based on a limit equilibrium analysis using SLIDE2 software and this has demonstrated that a satisfactory factor of safety will be achieved for all long-term conditions, including those under potential seismic events.

A closed form analysis was performed on the stability of the leachate collection/protection system on the Cells' internal side slope. With conservative values of interface friction angle and PSR, the analysis has demonstrated that a satisfactory factor of safety will be achieved for all scenarios assessed.

However, an analysis of even specialist construction plant deploying the system has shown that unacceptable Factors of Safety (less than 1.0) will be arise. As stated in section 3.7.2, an alternative means of construction or change to the design must be considered.

The wind uplift assessment has indicated the required surcharging of the temporarily exposed cells' basal side slopes to resist uplift was 75 kg/m^2 for 1:100 AEP Region A1 wind speeds. Opalvale is recommended to consider the exact requirements of surcharge during detailed design.

4 Monitoring & Risk Management

As part of the future development and ongoing landfilling operations a monitoring scheme should be conducted as part of normal operations, to confirm assumptions made in the stability risk assessment remain valid.

4.1 Groundwater

To ensure compliance with the assumed screening and calculations within the report, groundwater monitoring should continue and be compared to current inferred levels to ensure all future development and basal offsets above the seasonal high groundwater table are maintained.

4.2 Construction Quality Assurance

Monitoring during construction will comprise construction quality assurance to ensure earthworks and geosynthetic material compliance with the construction specification.

Construction quality assurance during earthworks operations is also required to confirm the absence of near surface voids, monitor for any perched seepages/groundwater and to ensure minimum compaction requirements are met.

4.3 Material Balance & Parameters

The stability assessment assumes that volumes of materials of suitable quality are available throughout the closure and restoration works. Limited laboratory testing has been undertaken from current onsite stockpiles. If the specific type and quantity of material is not available on site, then alternative capping designs will need to be assessed or the stability assumptions reviewed. The most critical aspect of the restoration profile is the interface friction properties between the geosynthetic layers. It is crucial that a comprehensive range of tests are conducted prior to construction in order to corroborate the shear strength properties adopted in this Stability Risk Assessment

4.4 Waste Mass Monitoring

Monitoring required for the waste mass shall entail waste elevation and temporary waste slope gradients across each cell. Leachate level monitoring should also be undertaken to assist in defining potential pore water pressures within the waste mass.

4.5 Basal Side Slope Engineering

Monitoring during construction will comprise construction quality assurance to ensure compliance with the construction specification.

Surface washout of the leachate collection layer is not considered as part of this report, and erosion protection may be required to prevent scouring of the gravels until the waste is placed on top. It is recommended that if erosion/gullying is identified, it is remediated as soon as practicable to prevent damage to the capping and exposure of the waste mass.

The sufficiency, spacing and design of the surcharge on the temporarily exposed side slopes will need to be considered during the detailed design stage to ensure wind uplift potential is minimised.

5 Limitations

5.1 Limitations

Talis have performed assessment and consulting services for this project in general accordance with accepted regulatory standards.

The assessment was limited to the Cells 3 and 4 of the landfill site (Stage 1 – Cells 1 to 6). These conditions cannot be extrapolated across any other portion of the Site without recourse back to Talis.

There is no leachate monitoring installed throughout the disposal area therefore true leachate levels and degree of saturation of the waste deposits are unknown.

Assessments of this nature are not capable of locating all soil and waste conditions (which can vary even over short distances). The advice given in this report is based on the assumption that the laboratory and in situ test results, and inferred conditions are representative of the overall soil conditions. However, it should be noted that actual conditions in some parts of the site might differ from those found. If further works reveal soil conditions, slope gradients and pore pressures significantly different from those assumed, the assessment should be reviewed.

APPENDIX A
Drawings

TW21027 C-101_A: Site Layout and Existing Topography

TW20122 C-106_A: Proposed Landfill Restoration Profile & Surface Water Management

TW21027 C-302_A: Standard Details Sheet 2 of 3

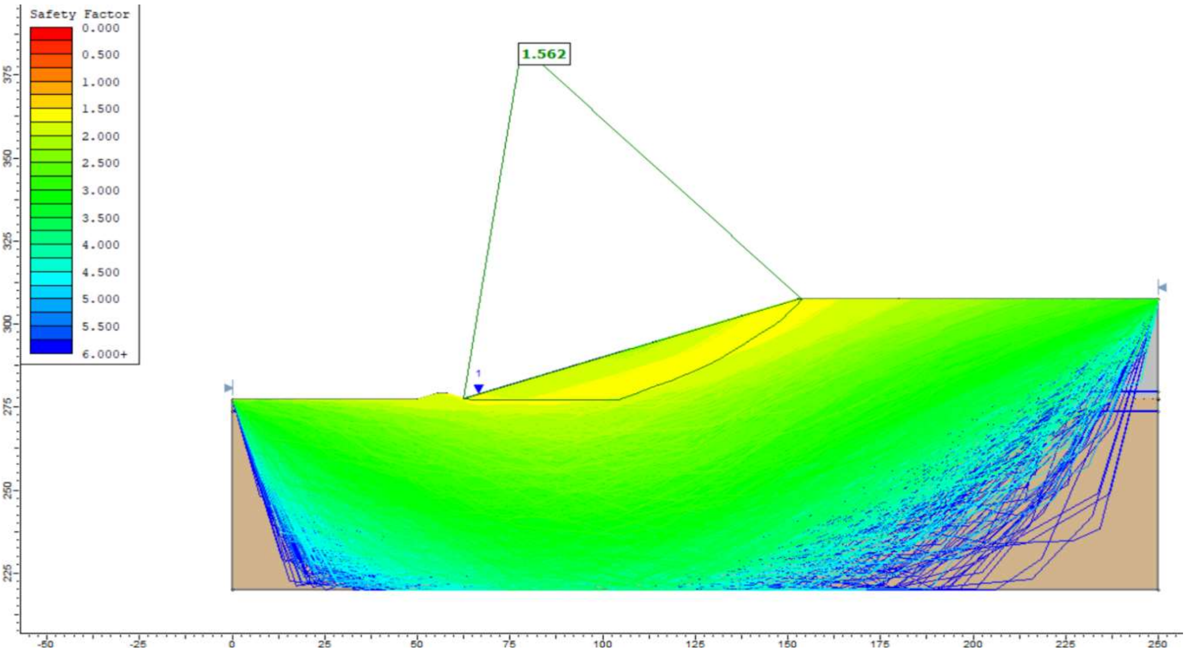
APPENDIX B
Laboratory Test Results

APPENDIX C
Waste Mass Stability Analysis

The following analyses are shown as screen grabs from the Slide2 output.

Temporary Western Slope at Edge of Cell 3

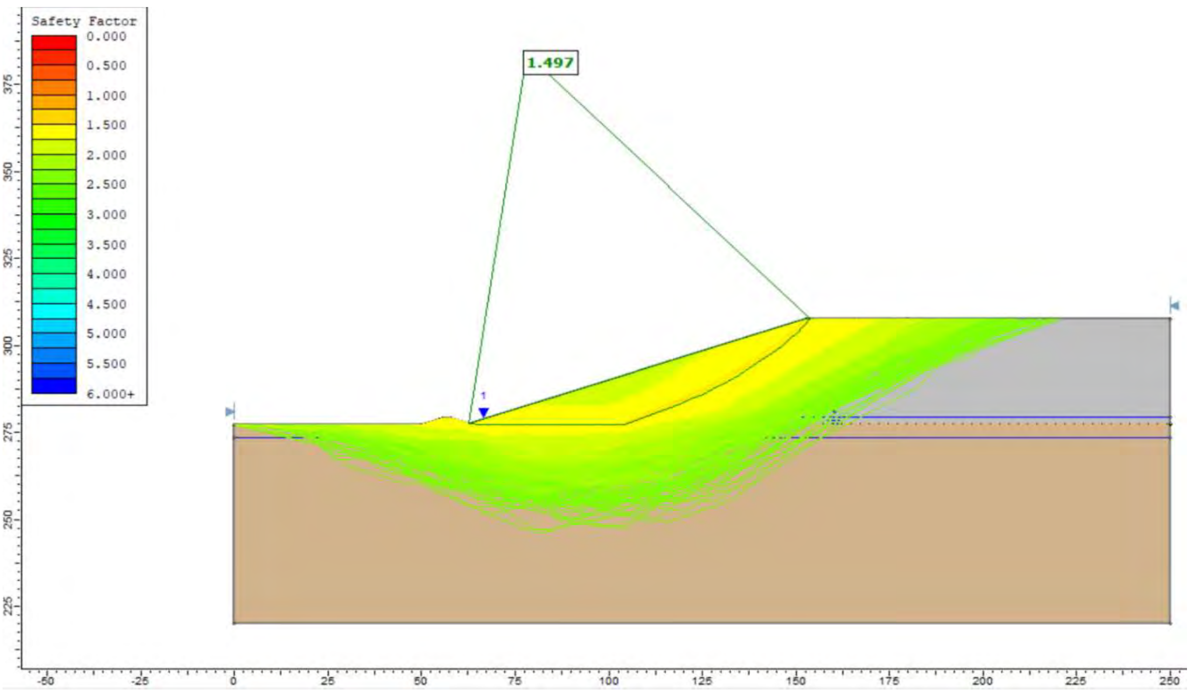
No Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.0 in Near Waste, Ru=0.0 in Remaining Waste



Numerous analyses are carried out for each scenario, as shown above, but because of the visual confusion this would cause, only the lowest are presented, as follows.

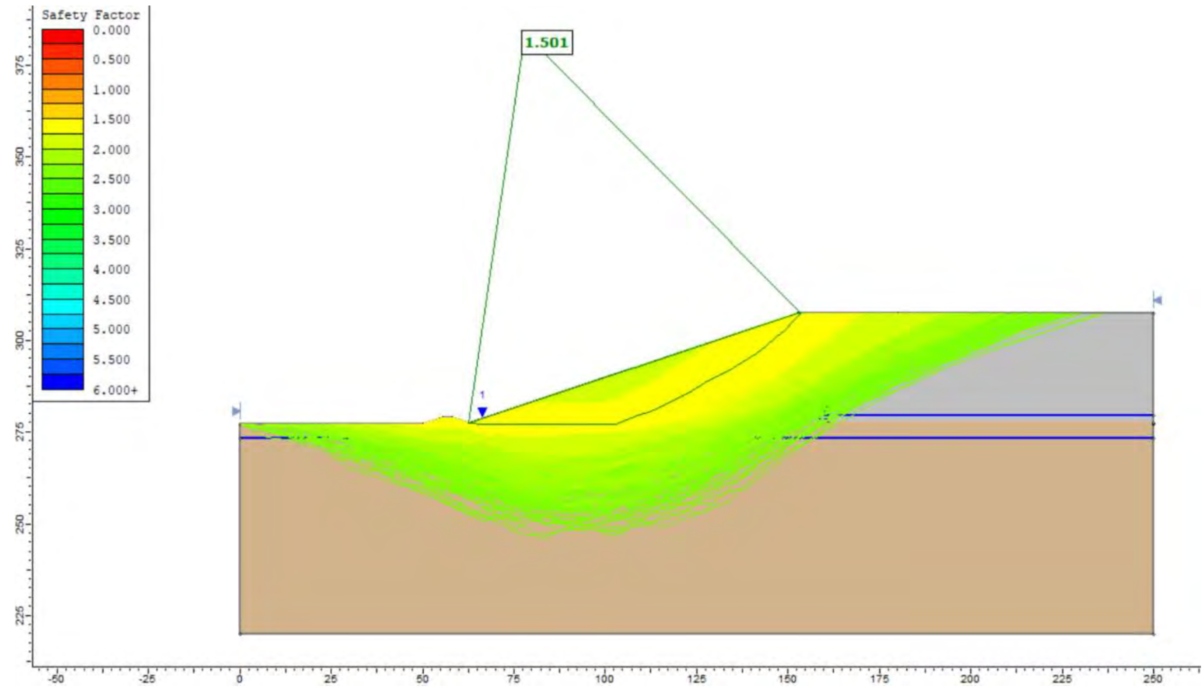
No Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.0 in Near Waste, Ru=0.0 in Remaining Waste

Displaying Failure Mechanisms up to FOS =2.5



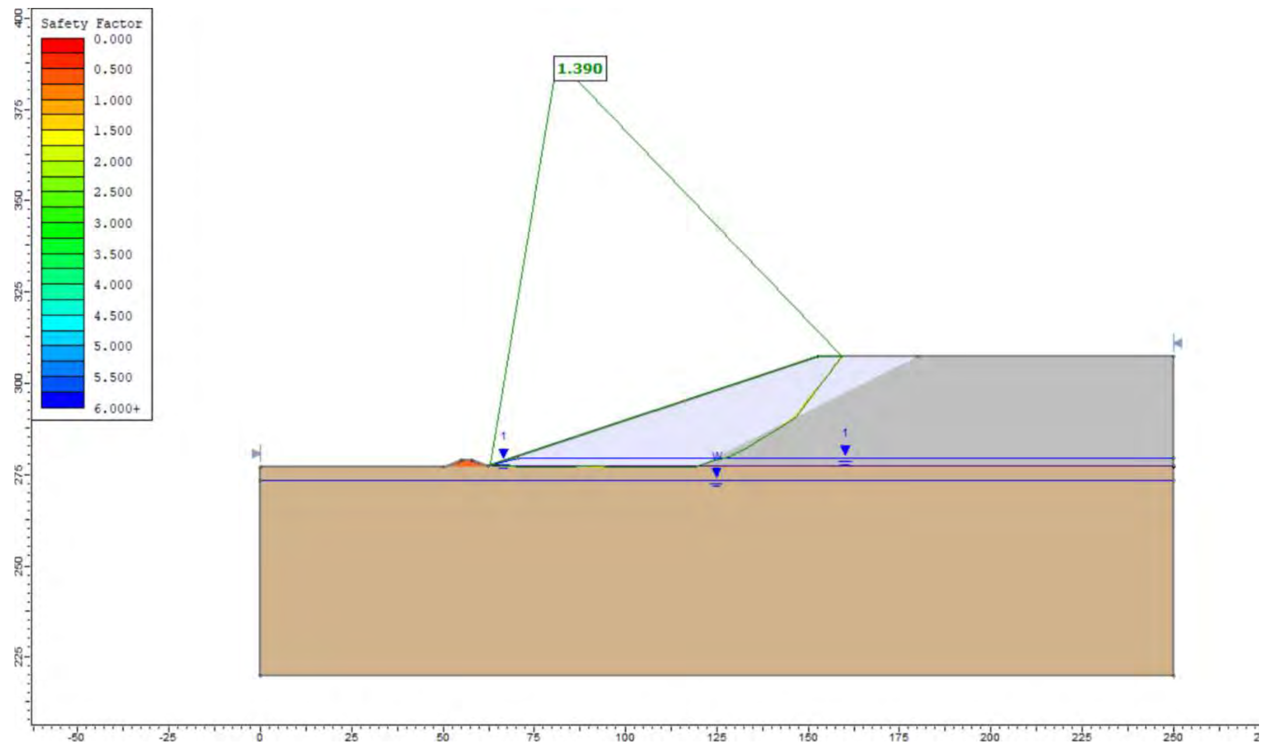
No Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.0 in Near Slope Waste, Ru=0.1 in Remaining Waste

Displaying Failure Mechanisms up to FOS =2.5



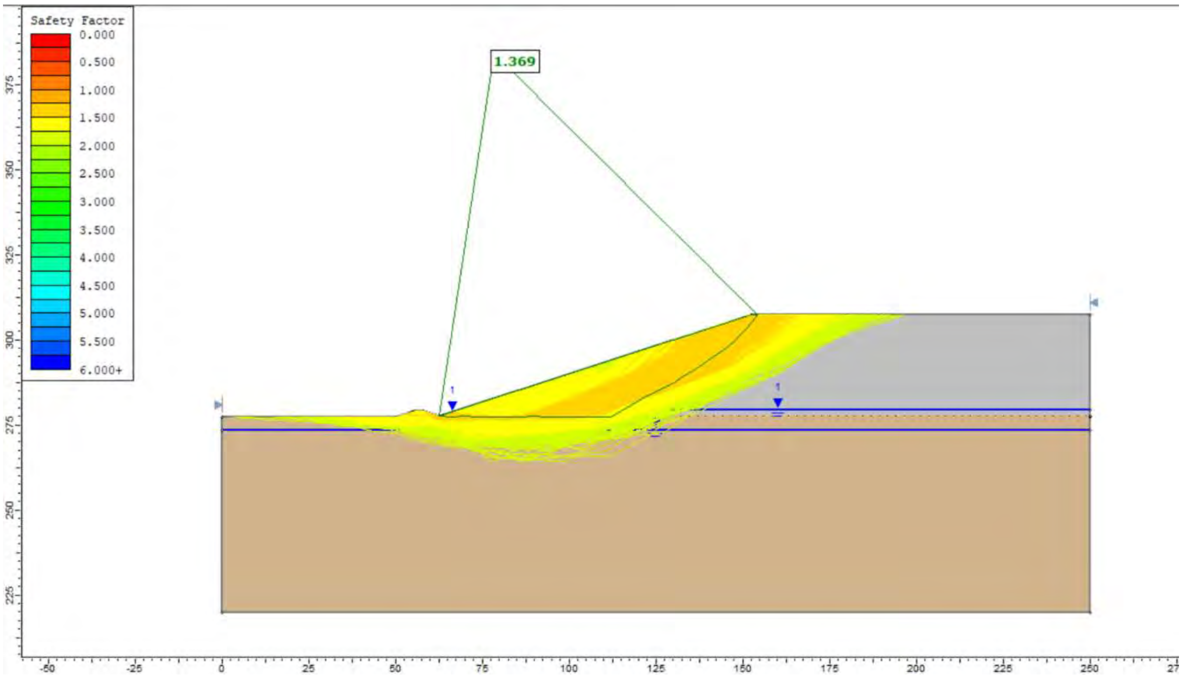
No Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.0 in Near Slope Waste, Ru=0.2 in Remaining Waste

Displaying Mechanism with Lowest FOS



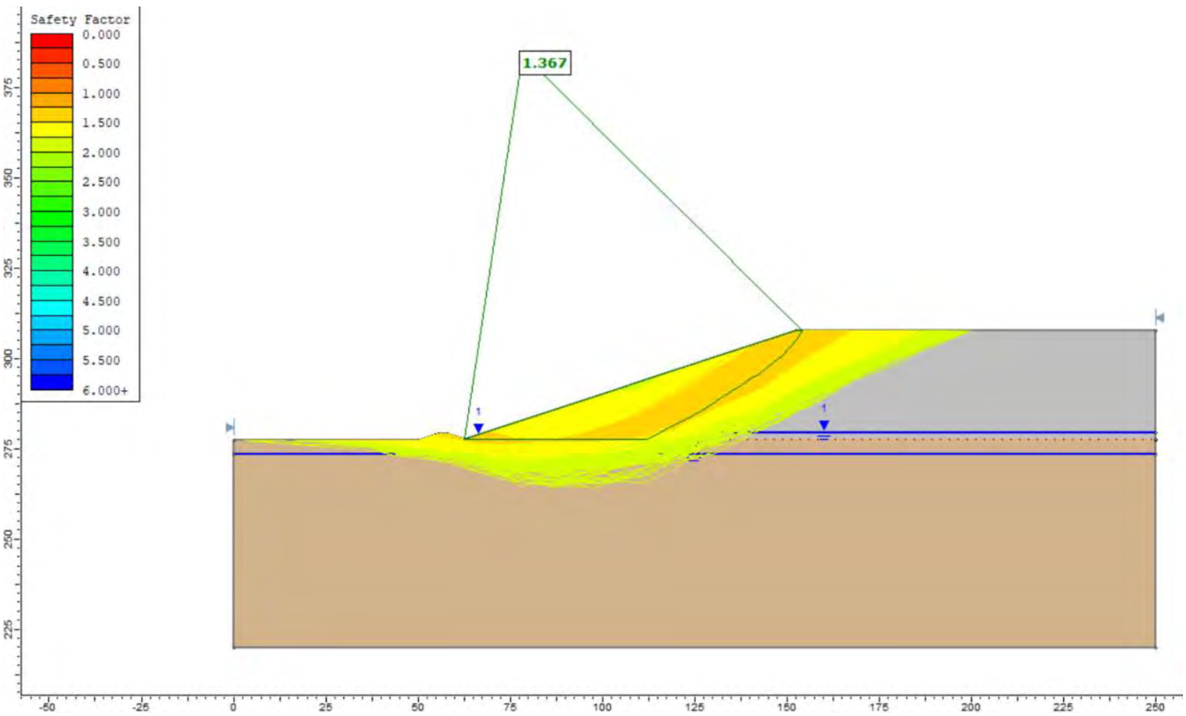
No Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.1 in Near Slope Waste, Ru=0.0 in Remaining Waste

Displaying Failure Mechanisms up to FOS =2.0



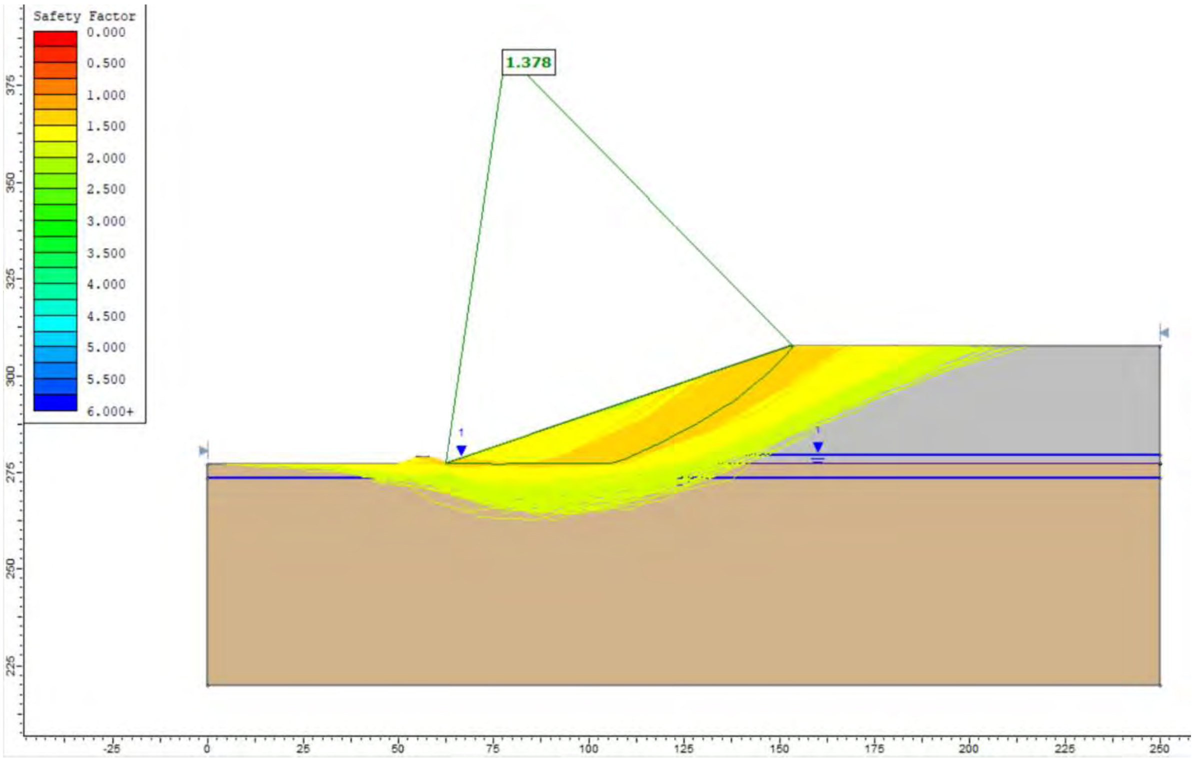
No Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.1 in Near Slope Waste, Ru=0.1 in Remaining Waste

Displaying Failure Mechanisms up to FOS =2.0



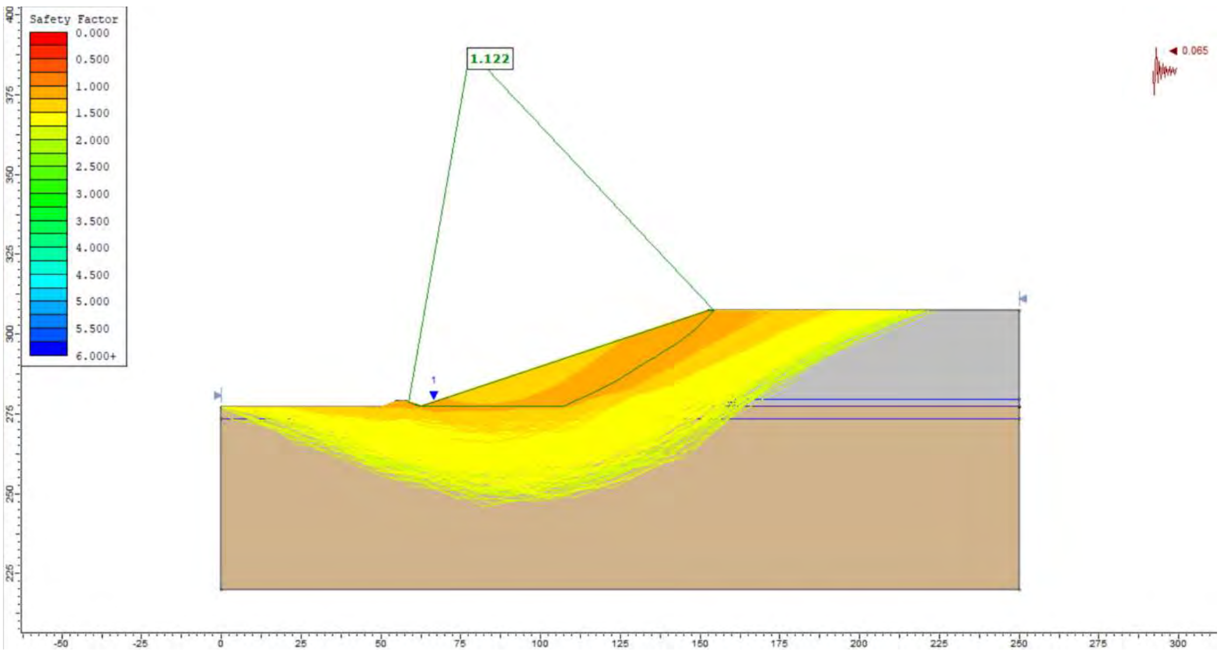
No Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.1 in Near Slope Waste, Ru=0.2 in Remaining Waste

Displaying Failure Mechanisms up to FOS =2.0



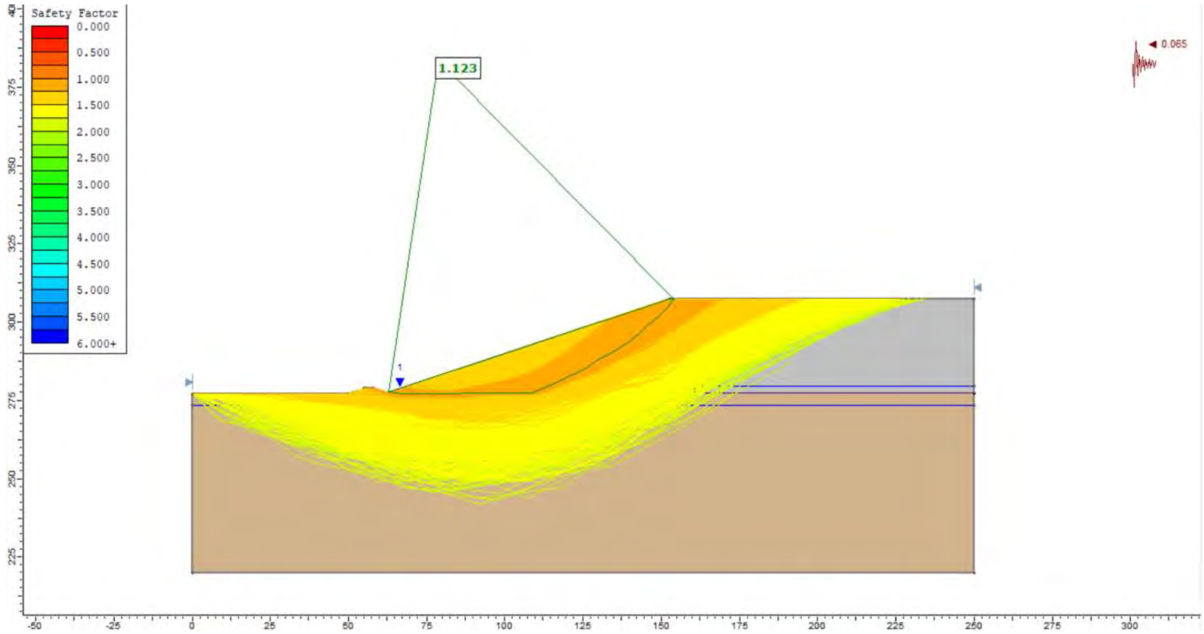
OBE Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.1 in Near Slope Waste, Ru=0.0 in Remaining Waste

Displaying Failure Mechanisms up to FOS =1.8



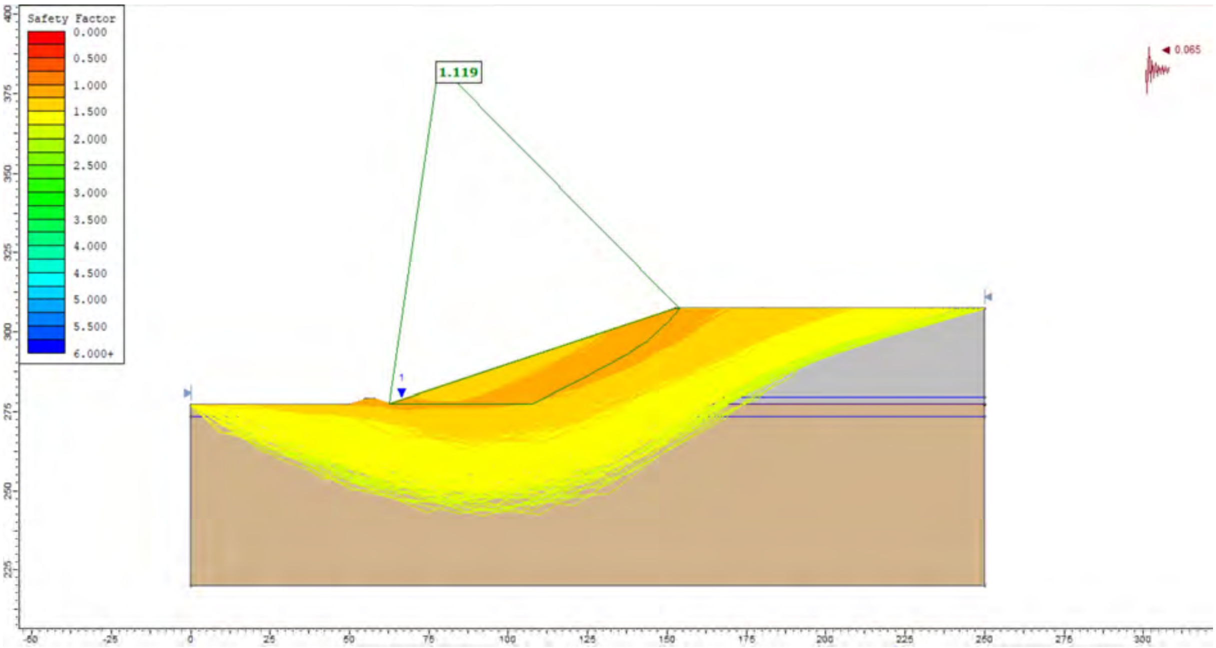
OBE Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.1 in Near Slope Waste, Ru=0.1 in Remaining Waste

Displaying Failure Mechanisms up to FOS =1.8



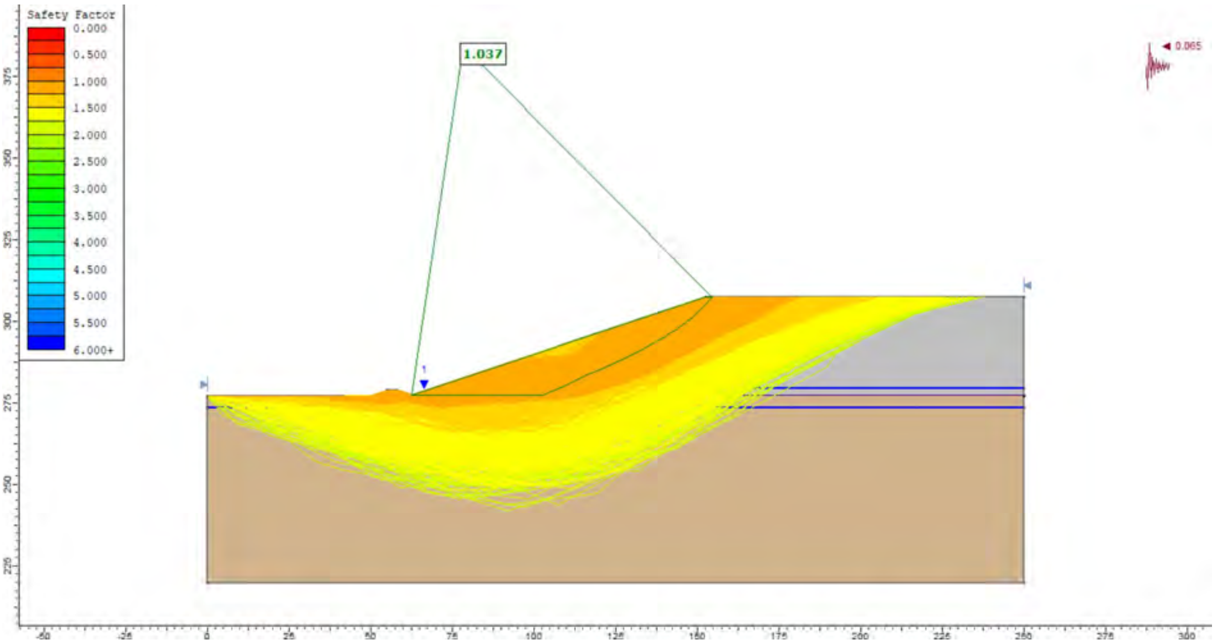
OBE Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.1 in Near Slope Waste, Ru=0.2 in Remaining Waste

Displaying Failure Mechanisms up to FOS =1.8



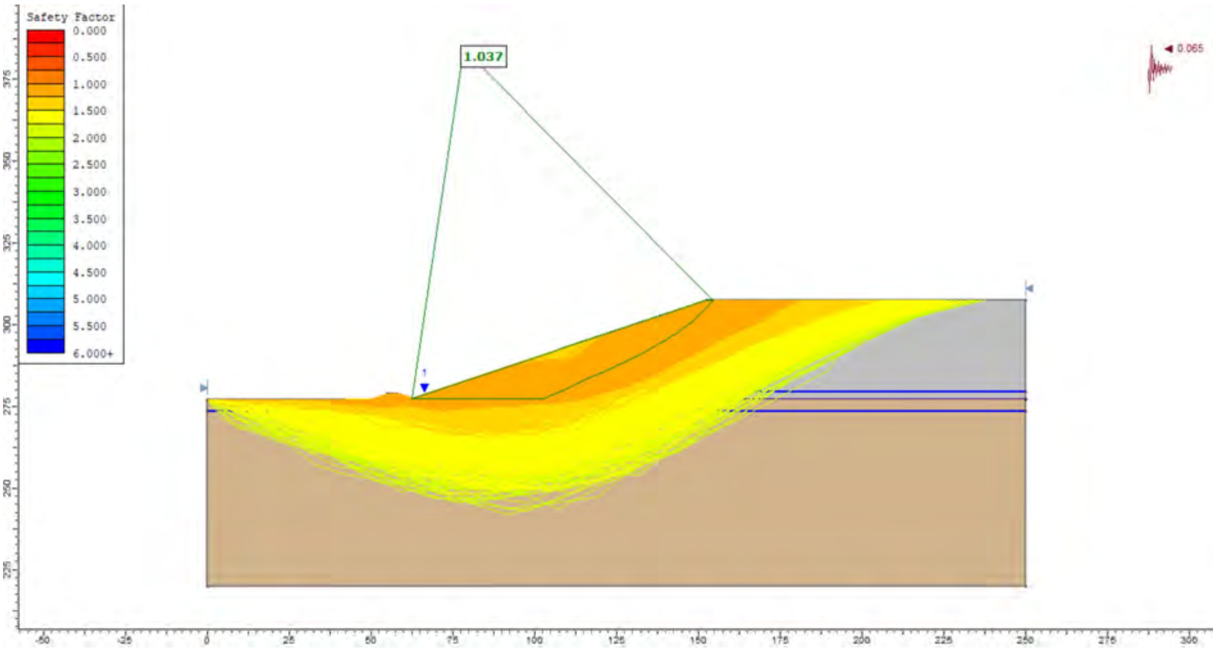
OBE Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.2 in Near Slope Waste, Ru=0.0 in Remaining Waste

Displaying Failure Mechanisms up to FOS =1.8



OBE Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.2 in Near Slope Waste, Ru=0.1 in Remaining Waste

Displaying Failure Mechanisms up to FOS =1.8

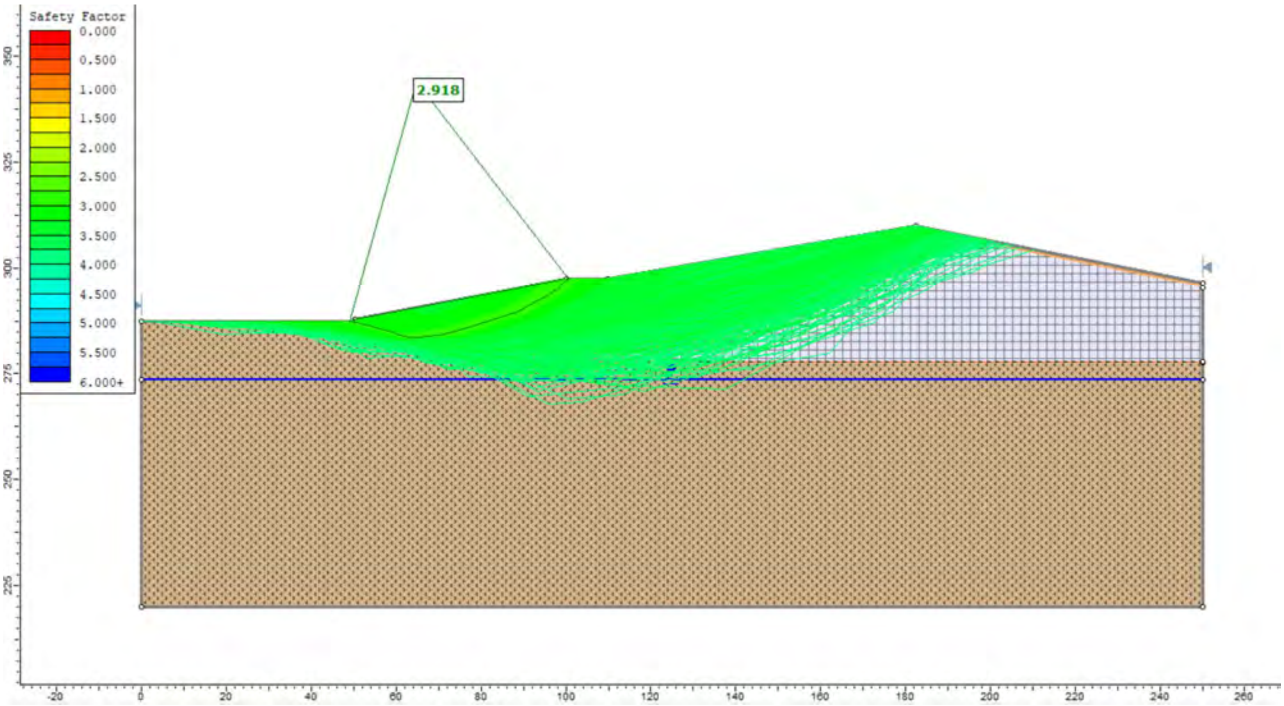


APPENDIX D
Capping Southern Slope Stability Analysis

Southern Slope with Berm

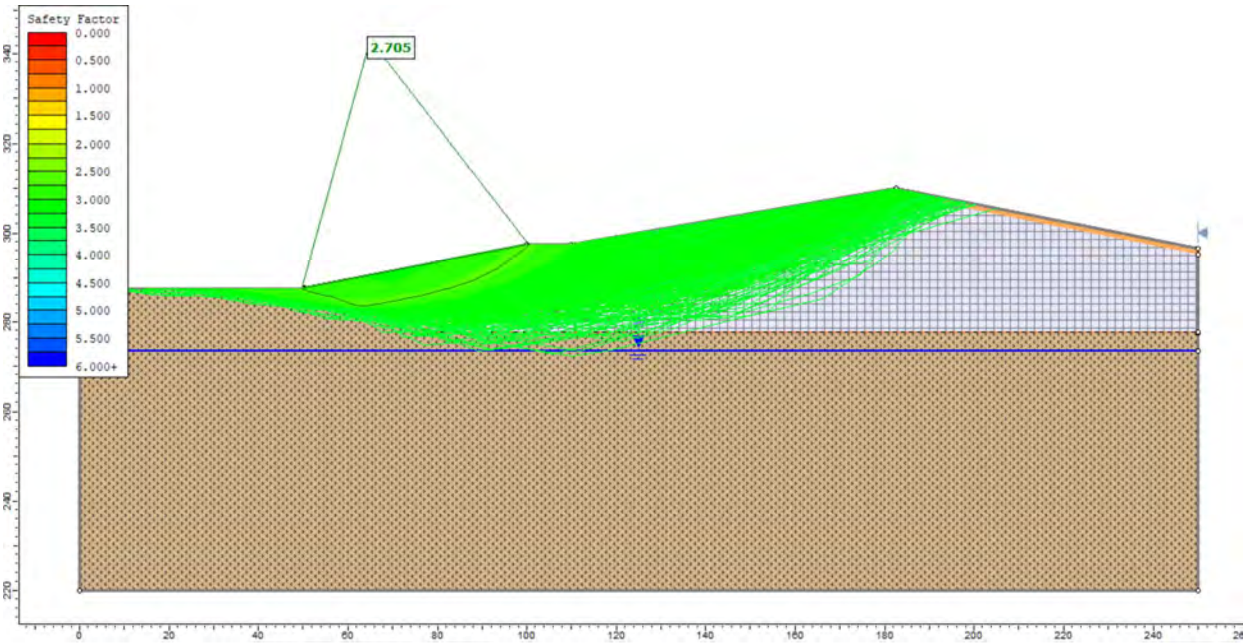
No Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.0 in Waste

Displaying Failure Mechanisms up to FOS =4.0



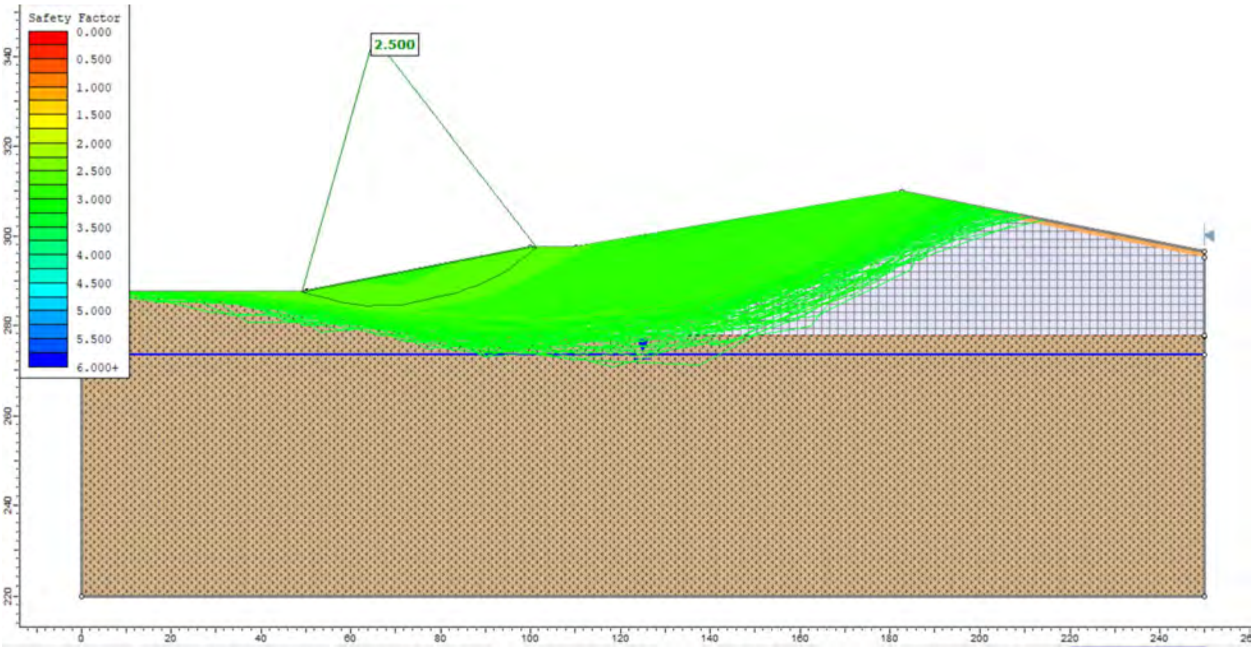
No Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.1 in Waste

Displaying Failure Mechanisms up to FOS =3.5



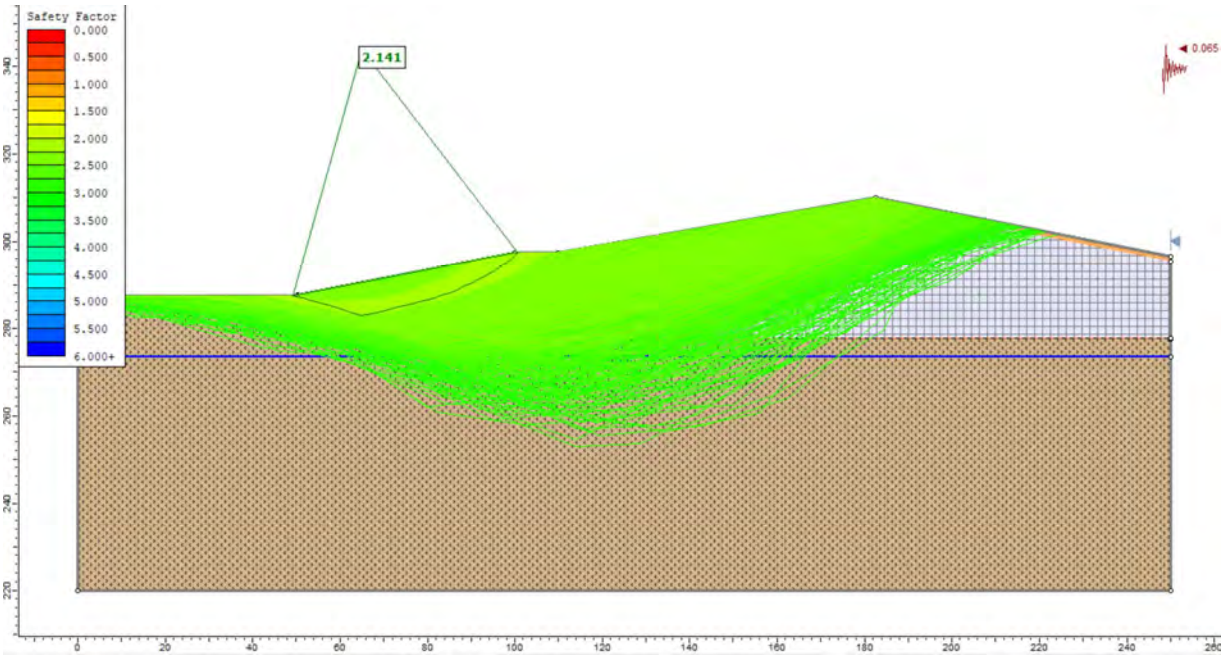
No Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.2 in Waste

Displaying Failure Mechanisms up to FOS =3.5



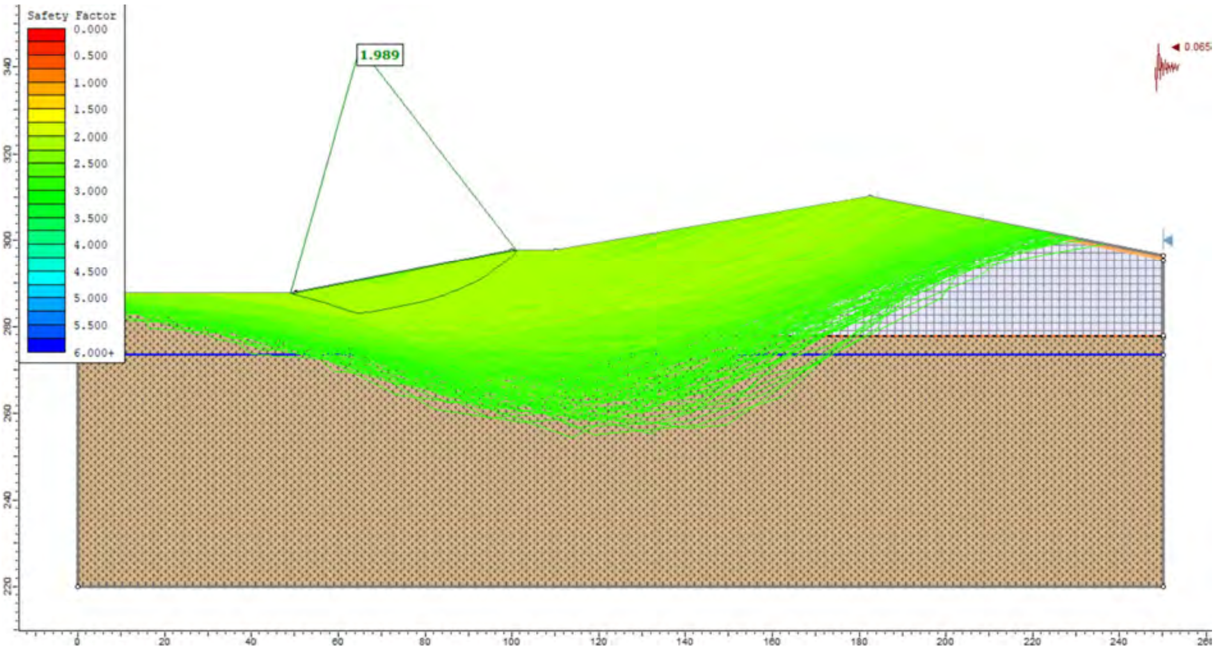
OBE Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.0 in Waste

Displaying Failure Mechanisms up to FOS =3.0



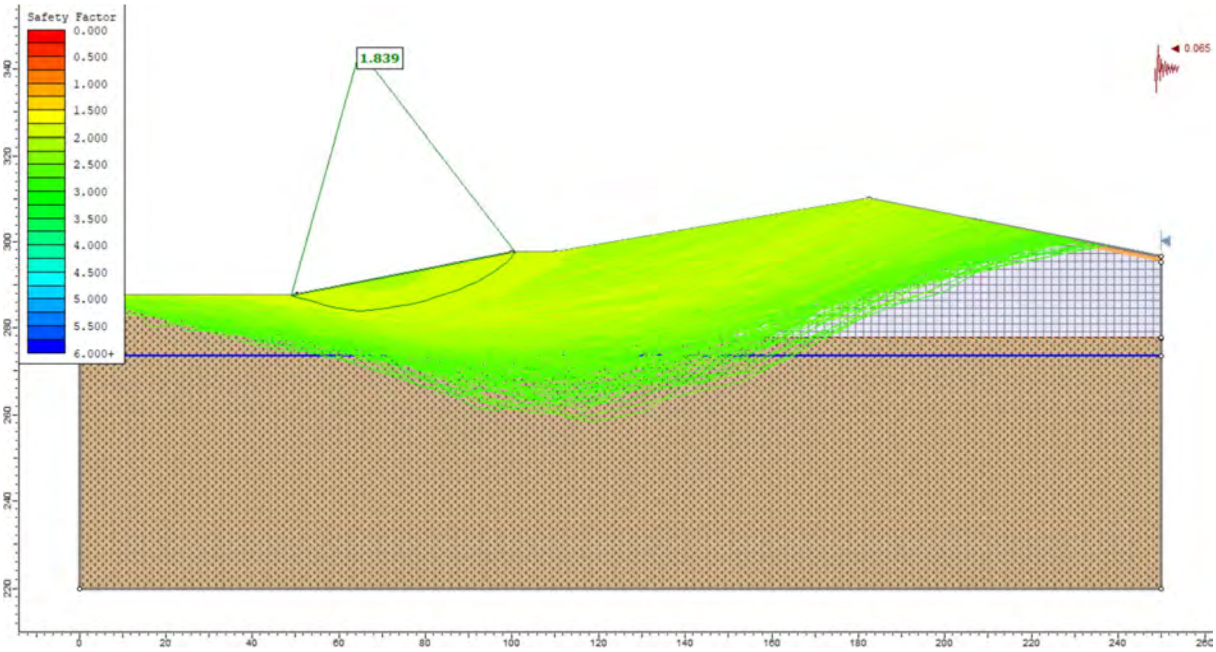
OBE Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.1 in Waste

Displaying Failure Mechanisms up to FOS =3.0



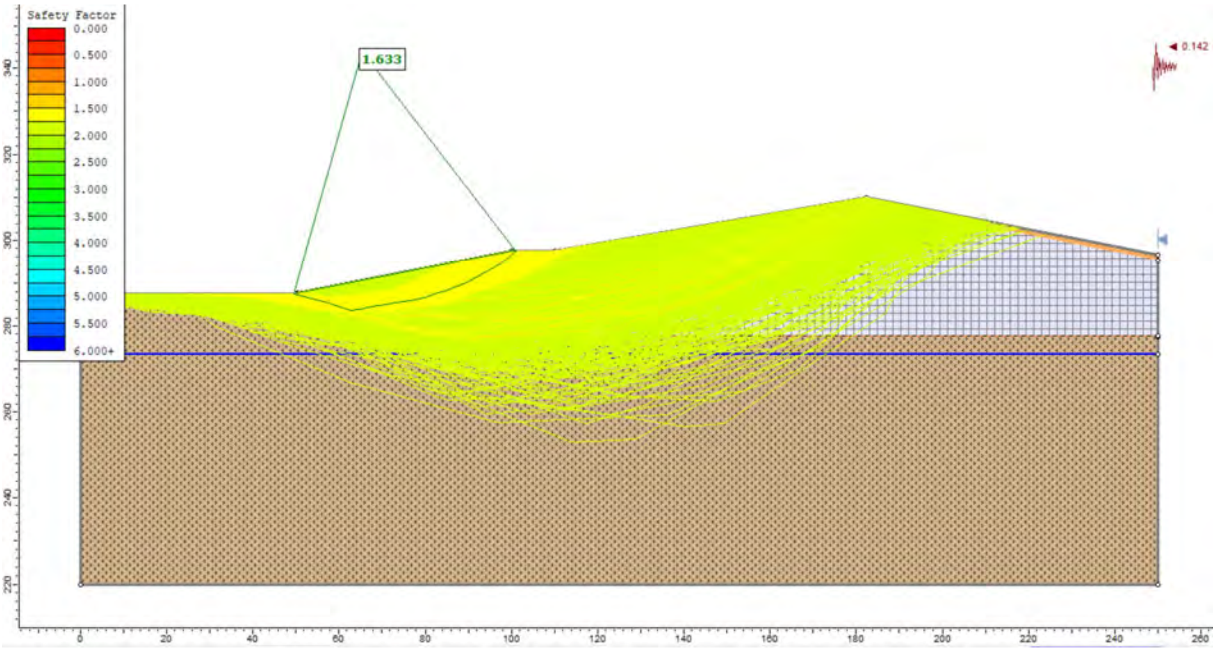
OBE Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.2 in Waste

Displaying Failure Mechanisms up to FOS =2.8



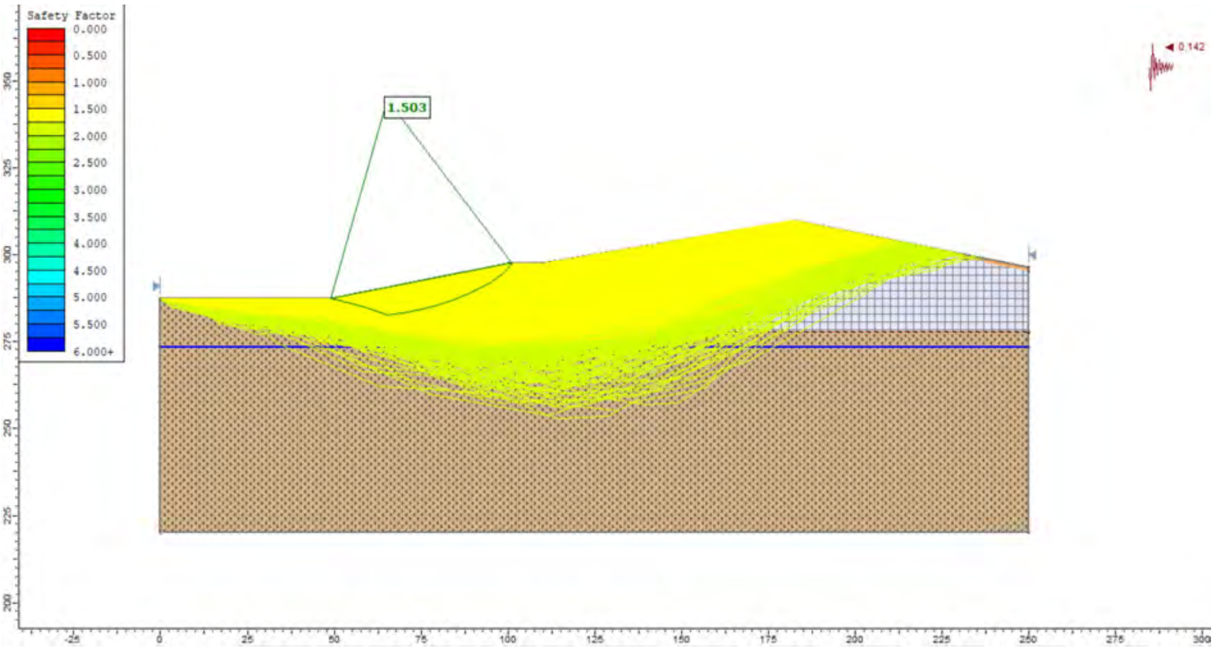
SEE Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.0 in Waste

Displaying Failure Mechanisms up to FOS =2.0



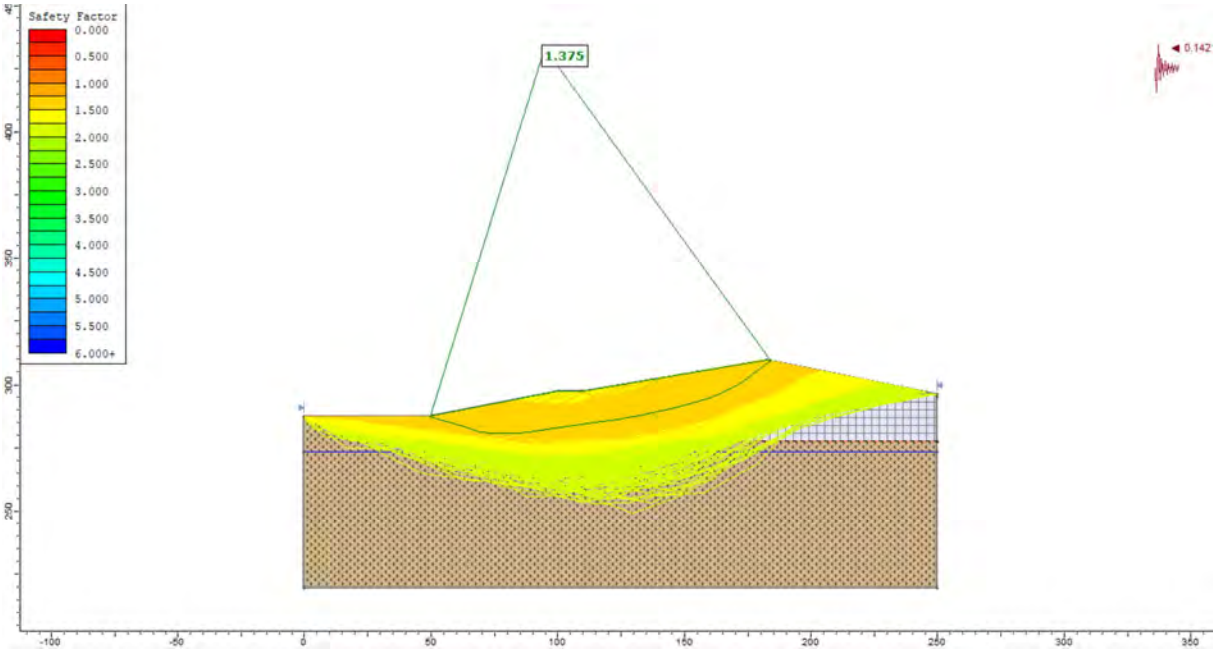
SEE Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.1 in Waste

Displaying Failure Mechanisms up to FOS =2.0



SEE Seismic Loading, Ru=0.2 in Leachate Drainage Layer, Ru=0.2 in Waste

Displaying Failure Mechanisms up to FOS =2.0



APPENDIX E
Closed Form Analysis

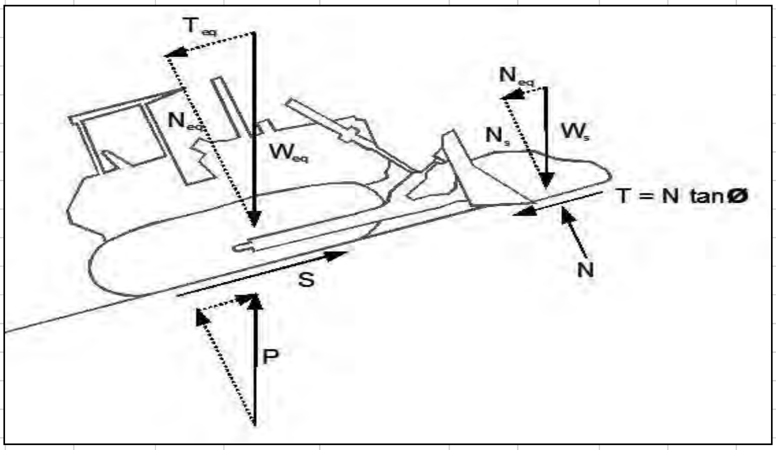
			Peak				Post Peak			
Opal Vale Closed Form Cell 3 Internal Side Slope Assessment										
Capping System Interface Stability Assessment										
Internal Slopeof Basal Lining System with 1:3 Side slope										
Input Parameters			**Sect 1**	**Sect 2**	**Sect 3**	**Sect 4**	**Sect 1**	**Sect 2**	**Sect 3**	**Sect 4**
b	Slope Angle	°	18.04	18.04	18.04	18.04	18.04	18.04	18.04	18.04
H	Slope height	m	4.50	4.50	4.50	4.50	4.50	4.50	4.50	4.50
h	Thickness of Leachate Collection Layer	m	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30
f	Friction angle of Leachate Collection Layer	°	30.00	30.00	30.00	30.00	24.00	24.00	24.00	24.00
c	Cohesion of Leachate Collection Layer	kPa	0.00	0.00	0.00	0.00	1.00	1.00	1.00	1.00
dct	Interface friction angle Leachate Collection Layer/GeoTextile	°	30.00	30.00	30.00	30.00	24.00	24.00	24.00	24.00
act	Apparent cohesion of Leachate Collection Layer/GeoTextile	kPa	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
dtg	Interface friction angle of Geotextile/HDPE	°	25.70	25.70	25.70	25.70	9.10	9.10	9.10	9.10
atg	Apparent cohesion of Geotextile/HDPE interface	kPa	6.90	6.90	6.90	6.90	2.40	2.40	2.40	2.40
dgs	Interface friction angle HDPE/GCL	°	19.50	19.50	19.50	19.50	15.60	15.60	15.60	15.60
ags	Apparent cohesion of HDPE/GGCL	kPa	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
dgf	Interface friction angle GCL/Subgrade	°	25.70	25.70	25.70	25.70	25.70	25.70	25.70	25.70
agf	Apparent cohesion of GCL/Subgrade	kPa	6.90	6.90	6.90	6.90	6.90	6.90	6.90	6.90
PRS	Parallel Submerged Ratio		**0.00**	**0.10**	**0.20**	**0.30**	**0.00**	**0.10**	**0.20**	**0.30**
g_d	Dry unit weight of Leachate Drainage Layer	kN	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
g_{sat}	Saturated weight of Leachate Drainage Layer	kN	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
h_w	Thickness of saturated cover soil	m	**0.00**	**0.03**	**0.06**	**0.09**	**0.00**	**0.03**	**0.06**	**0.09**
W_A	Weight of active wedge	kN	**75.72**	**75.72**	**75.72**	**75.72**	**75.72**	**75.72**	**75.72**	**75.72**
W_P	Weight of passive wedge	kN	**2.75**	**2.75**	**2.75**	**2.75**	**2.75**	**2.75**	**2.75**	**2.75**
U_n	Resultant pore water pressure perpendicular to slope	kN	**0.00**	**4.13**	**8.23**	**12.30**	**0.00**	**4.13**	**8.23**	**12.30**
U_h	Resultant pore water pressure on interwedge surface	kN	**0.00**	**0.00**	**0.02**	**0.04**	**0.00**	**0.00**	**0.02**	**0.04**
N_{Aab}	Effective force normal to failure plane of active wedge above impermeable layer	kN	**71.99**	**67.87**	**63.77**	**59.70**	**71.99**	**67.87**	**63.77**	**59.70**
N_{Abb}	Effective force normal to failure plane of active wedge below impermeable layer	kN	**71.99**	**72.00**	**72.00**	**72.01**	**71.99**	**72.00**	**72.00**	**72.01**
U_v	Resultant vertical pore water pressure acting on passive wedge	kN	**0.00**	**0.01**	**0.06**	**0.12**	**0.00**	**0.01**	**0.06**	**0.12**
L	Slope Length	m	**14.53**	**14.53**	**14.53**	**14.53**	**14.53**	**14.53**	**14.53**	**14.53**
Leachate Drainage Layer/GeoTextile Interface										
	Quadratic Equation Parameters	a	22.30	22.30	22.30	22.30	22.30	22.30	22.30	22.30
		b	-45.30	-43.03	-40.75	-38.48	-35.90	-34.15	-32.40	-30.64
		c	7.43	7.01	6.58	6.16	4.42	4.17	3.91	3.67
	Factor of Safety Against Failure		**1.85**	**1.75**	**1.65**	**1.55**	**1.48**	**1.40**	**1.32**	**1.24**
	Tension	**kN**	**-112.91**	**-111.56**	**-110.04**	**-108.32**	**-24.32**	**-23.06**	**-21.66**	**-20.07**
			No Tension	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**
Geotextile/HDPE Interface										
	Quadratic Equation Parameters	a	22.30	22.30	22.30	22.30	22.30	22.30	22.30	22.30
		b	-134.06	-132.16	-130.26	-128.36	-49.55	-48.92	-48.27	-47.62
		c	24.12	23.77	23.41	23.06	6.40	6.31	6.22	6.13
	Factor of Safety Against Failure		**5.83**	**5.74**	**5.66**	**5.57**	**2.08**	**2.06**	**2.03**	**2.00**
	Tension	**kN**	**-3.05**	**-2.71**	**-2.35**	**-1.98**	**1.63**	**1.94**	**2.26**	**2.61**
			No Tension	**No Tension**	**No Tension**	**No Tension**	**Tension**	**Tension**	**Tension**	**Tension**
HDPE/GeonetGCL Interface										
	Quadratic Equation Parameters	a	22.30	22.30	22.30	22.30	22.30	22.30	22.30	22.30
		b	-30.02	-30.01	-29.99	-29.95	-24.54	-24.53	-24.51	-24.48
		c	4.56	4.56	4.56	4.56	2.77	2.77	2.77	2.77
	Factor of Safety Against Failure		**1.17**	**1.17**	**1.17**	**1.17**	**0.97**	**0.97**	**0.97**	**0.97**
	Tension	**kN**	**-113.63**	**-113.62**	**-113.60**	**-113.55**	**-114.76**	**-114.75**	**-114.73**	**-114.69**
			No Tension	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**
GGCL/Subgrade Interface										
	Quadratic Equation Parameters	a	22.30	22.30	22.30	22.30	22.30	22.30	22.30	22.30
		b	-134.06	-134.05	-134.03	-133.99	-133.71	-133.70	-133.68	-133.65
		c	24.12	24.12	24.12	24.12	18.60	18.60	18.60	18.60
	Factor of Safety Against Failure		**5.83**	**5.83**	**5.83**	**5.82**	**5.85**	**5.85**	**5.85**	**5.85**

N.B. This calculation assumes friction angles and cohesion as published in R&D TECHNICAL REPORT P1-385/TR1, typical Naue GCL Shear box data and Talis Shear box data.
R&D TECHNICAL REPORT P1-385/TR1, reports textured geomembrane to geonet interface results of 11 degrees and 3kpa peak, and 9.1 degrees and 9.2 kpa for residual.
Geomembrane-Geonet Post peak cohesion reduced to 2kPa to reflect strain softening at the interface
Interface Friction tests to be undertaken on proposed geosynthetic products prior to any construction works

APPENDIX F
Plant Operations on Geosynthetic Cap

Leachate Collection Layer =0.3m thick, with No Tension in Geomembrane

STABILITY ASSESSMENT FOR PLANT OPERATIONS ON GEOSYNTHETIC CAP



Unit weight of soil cover	18.00	kN/cu.m
Depth of soil cover (1st lift, D)	0.30	m
Dozer type	CAT D2 LGP	
Total dozer weight	83.00	kN
Track length (L)	2.30	metres
Track width (W)	0.72	metres
Width of dozer blade (Wb)	2.90	metres
Height of soil pile (Hb)	0.90	metres
Length in front of blade (Lb)	0.80	metres
Weight of soil being spread	37.58	kN
Slope angle, alpha	18.42	degrees
Soil cover friction angle	30.00	degrees
Interface friction angle	19.50	degrees
Interface adhesion	3.00	kN/sq.m
Unit tension (geosynthetic)	0.00	kN/m
Factor of safety	**0.85**	

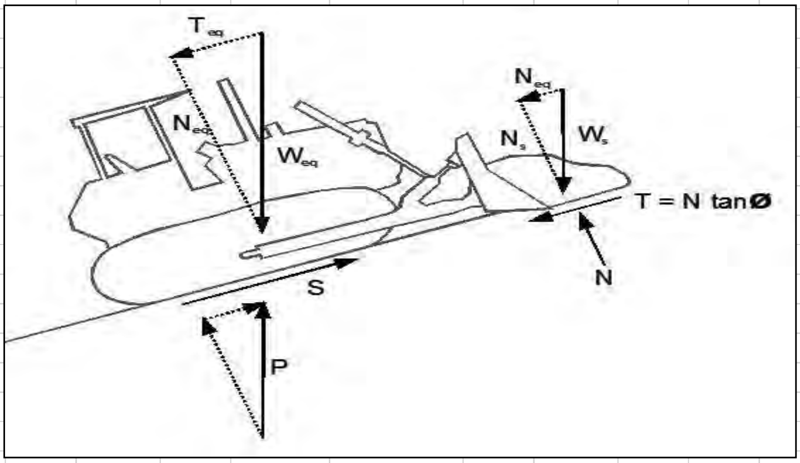
Forces

N(1)	=	4.77	kN	$N_1 = \dfrac{W_1}{\cos\beta - \tan\phi_m \sin\beta - (\sin\beta + \tan\phi_n \cos\beta)\tan\phi_m}$
N(2)	=	49.63	kN	$N_2 = \dfrac{W_2 + 0.5P - (\delta_n \rho_2 + \tau_G - 0.5S)(\sin\alpha - \cos\alpha \tan\phi_m)}{\cos\alpha + \tan\delta_n \sin\alpha + (\sin\alpha - \tan\delta_n \cos\alpha)\tan\phi_m}$
N(3)	=	0.92	kN	$N_3 = \dfrac{W_3}{\cos\theta + \tan\phi_n \sin\theta + (\sin\theta - \tan\phi_m \cos\theta)\tan\phi_m}$
N(4)	=	3.66	kN	$N_{4.} = N_1(\sin\beta + \tan\phi_m \cos\beta)$
N(5)CB	=	0.21	kN	$N_{5CB} = N_4 + N_2(\tan\delta_m \cos\alpha - \sin\alpha) + (\theta_m A_2 + T_\alpha - 0.5S)\cos\alpha$
N(5)AB	=	0.21	kN	$N_{5AB} = N_3(\sin\theta - \tan\phi_m \cos\theta)$

SLIDING BLOCK ANALYSIS WITH SURFACE LOADS (P & S) AND GEOTEXTILE TENSILE FORCE (Tg)
Method of Kerkes, D.J. (1999), "Analysis of equipment loads on geocomposite liner systems", Proc Geosynthetics 99,
BULLDOZER SPREADING SOIL UPSLOPE

Leachate Collection Layer =0.3m thick, with 5kPa Tension allowed in Geomembrane

STABILITY ASSESSMENT FOR PLANT OPERATIONS ON GEOSYNTHETIC CAP



Unit weight of soil cover	18.00	kN/cu.m
Depth of soil cover (1st lift, D)	0.30	m
Dozer type	CAT D2 LGP	
Total dozer weight	83.00	kN
Track length (L)	2.30	metres
Track width (W)	0.72	metres
Width of dozer blade (Wb)	2.90	metres
Height of soil pile (Hb)	0.90	metres
Length in front of blade (Lb)	0.80	metres
Weight of soil being spread	37.58	kN
Slope angle, alpha	18.42	degrees
Soil cover friction angle	30.00	degrees
Interface friction angle	19.50	degrees
Interface adhesion	3.00	kN/sq.m
Unit tension (geosynthetic)	5.00	kN/m
Factor of safety	**0.97**	

Forces

N(1)	=	3.63	kN	$N_1 = \dfrac{W_1}{\cos\beta - \tan\phi_m \sin\beta - (\sin\beta + \tan\phi_n \cos\beta)\tan\phi_m}$
N(2)	=	50.23	kN	$N_2 = \dfrac{W_2 + 0.5P - (\delta_n \rho_2 + \tau_G - 0.5S)(\sin\alpha - \cos\alpha \tan\phi_m)}{\cos\alpha + \tan\delta_n \sin\alpha + (\sin\alpha - \tan\delta_n \cos\alpha)\tan\phi_m}$
N(3)	=	0.95	kN	$N_3 = \dfrac{W_3}{\cos\theta + \tan\phi_n \sin\theta + (\sin\theta - \tan\phi_m \cos\theta)\tan\phi_m}$
N(4)	=	2.47	kN	$N_{4.} = N_1(\sin\beta + \tan\phi_m \cos\beta)$
N(5)CB	=	0.27	kN	$N_{5CB} = N_4 + N_2(\tan\delta_m \cos\alpha - \sin\alpha) + (\theta_m A_2 + T_\alpha - 0.5S)\cos\alpha$
N(5)AB	=	0.27	kN	$N_{5AB} = N_3(\sin\theta - \tan\phi_m \cos\theta)$

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Talis Consultants

Head Office
Level 1, 604 Newcastle Street,
Leederville
Western Australia 6007

PO Box 454,
Leederville
Western Australia 6903

NSW Office
5/62 North Street, Nowra
New South Wales, 2541

PO Box 1189, Nowra
New South Wales, 2541

P: 1300 251 070
E: info@talisconsultants.com.au