Capping Stability Risk Assessment

Salt Valley Road Landfill – Closure and Rehabilitation

Prepared for Opalvale Pty Ltd

8 March 2021

Project Number: TW20122

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1 Introduction

Talis Consultants Pty Ltd (Talis) was engaged by Opal Vale Pty Ltd (Opal Vale) to undertake a Capping Stability Risk Assessment (SRA) to support the closure and Post-Closure Management Plan (CPCMP) for their Class II Salt Valley Road Landfill facility (the Site).

Opalvale Pty Ltd (Opalvale) operates the Salt Valley Road Class II Landfill (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.

This document describes the way the assessment was carried out for the rehabilitation and capping at the Site and presents the overall findings of the work.

It should be noted that in December 2014 Golder Associates (Golders) carried out a number of technical studies in support of the Development Application for the Site (Appendix 5 - "Opal Vale 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 restoration it primarily focussed on the entire facility and in particular the basal lining system. The Golders' studies were based on the engineering design prepared by IW Projects (IWP).

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 in APPENDIX A. 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 C-001 in APPENDIX B. 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 3km to 5km 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 is situated in the Williamson's Pit. 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 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.

Temporary waste slopes are approximately 1V:3H along the southern and western extents of Cell 1 where the depth of waste is the deepest.

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 C-001.

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 in the Works Approval Supporting Documentation – 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 near surface 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.

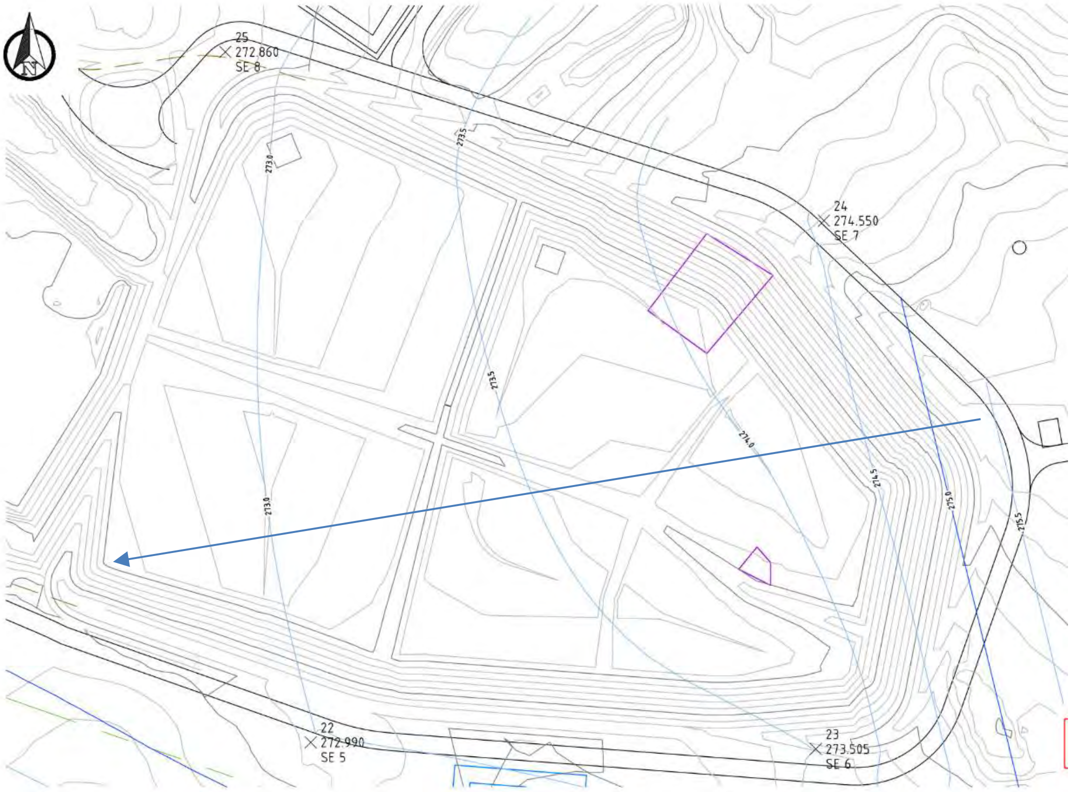
The landfill is underlain at depth by granite and gneiss, but the boreholes show that the near surface soils to be a highly weathered zone comprising sandy clays and silty clays.

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 278.96m AUD (BH SE4) to 267.46 m AUD (BH SE1).

The generalised localised groundwater flow direction was inferred to be from the north east to south west of the landfill. Figure 1-1 presents the local groundwater regime beneath the site using the highest levels recorded since 2014. As can be seen the groundwater gradient is steeper along the eastern boundary, but flattens considerably over the central and western end of the landfill

Figure 1-1: Groundwater Flow Contours (m AHD) and Inferred flow Direction



This is repeated in Drawing C-401.

1.4.1 Groundwater

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

Table 1-1: Groundwater Depth from Historical Hydrogeological Monitoring

Bore ID	Minimum Static Water Level (m AHD)	Maximum 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	274.6
BH SE8	271.6	272.9
BH SE9	270.0	271.4

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

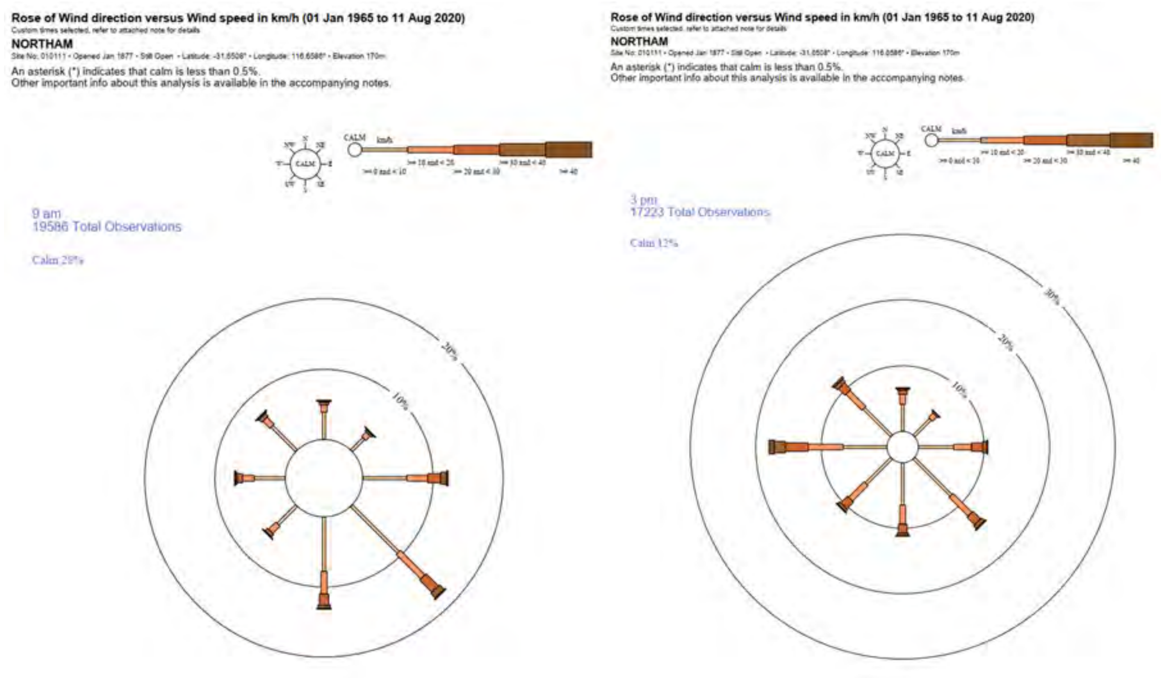
Over the majority of the base the Stage 1 landfill the highest elevation of groundwater is between 273m and 274m AUD.

1.4.2 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 Closure Plan.

1.5.1 Rehabilitation Design

The NSW Landfill Guidelines have been adopted and supported by Opal Vale 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 Development Plan prepared by IW Projects Pty Ltd (IWP)and the CPCMP, Talis, Feb 2021, which this document supports;
- 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 cells.

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 Stage 1 landfill (Cells 5 and 6). 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. Combined 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 Waste Mass Model

As the existing cell is basally lined, a leachate management system has been established at the beginning of landfilling operations. This consists of creating basal gradients overlain with a leachate collection layer designed to convey all the fluid to one of two low points (in the north-western corners of cells 1 and 5) 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 proposed Stage 2 extension on the western extent of Stage 1 are proposed/modelled at a gradient of 1V:3.0H, to maximum 38m vertical height and on the southern slope to a maximum height of 25.5m.

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 312mAHD. The proposed restoration profile is presented on Drawing C-108.

There is no site wide leachate monitoring to determine the true leachate levels across the currently filled landfill. Having stated this, there is no fill placed in the majority of the landfill as yet. 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 and 0.1 has been utilised for the waste mass in all limit equilibrium assessments. 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, there is little opportunity for perched leachate to occur and, if it does, it is 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.

1.5.5 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 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.

Additionally, the basal containment has been assessed by Golders in carrying out their technical studies.

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

2.1.2 Basal Heave

The design carried out by IWP, supported by Stass and demonstrated the underlying maximum seasonal potentiometric head recorded is at an elevation of 273m AHD 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, excavated as part of the initial development works within the residual quarry excavation. All sideslopes are infilled and buttressed with waste deposits. 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.

2.2.1 Deformability

As per the basal subgrade (Section 2.1.1) the sideslope subgrade comprises insitu and reformed sand. It is therefore considered 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 recorded by the monitoring wells shows the phreatic surface to lie between 273m and 274m AUD over the majority of the base for Cells 1 to 6. The focus of this SRA is 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 278m AUD, 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:

- Compacted Subgrade Layer – A nominal 300mm 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 designed by IWP and assessed by Golders. Moreover, the system has been approved for development. Therefore, further consideration is not considered necessary.

Assessment of the base has consequently been screened out.

2.3.2 Sideslope Lining Stability

The side slope liner is essentially the same as illustrated in section 2.3.1 and also formed part of the original design and approval system.

Assessment of the side slopes has therefore been screened out.

2.3.3 Waste Mass Screening

For the purpose of the waste mass model, the 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.

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 sides of the facility meeting at a west to east ridgeline at an elevation of 312m AHD.

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 current 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 covered further minimises the risk of perched systems. Additionally, the higher proportion of construction waste will promote a more porous waste mass. However, Opalvale wishes to undertake some leachate recirculation, so 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. A R_u value of 0.0 and 0.1 has been utilised for the waste mass during further assessments for the temporary slope and includes a value of 0.2 for the permanent slopes.

2.3.4 Capping Screening

2.3.4.1 *Groundwater*

The underlying maximum seasonal potentiometric head recorded is at an approximate elevation between 273m AUD and 274m AUD across the majority of the base, between 9m and 22m beneath the landfill development area around the northern, southern and eastern sides of the landfill. Apart from the western temporary slope where the separation distance is around 5m, the groundwater is unlikely to affect the capping system proposed for the site.

A groundwater regime will be included as part on the ongoing analyses with this SRA.

2.3.4.2 *Compressible Waste and Cavities in Waste*

No external factors will be present to cause anything other than waste deformations/compressibility normally associated with waste settlement. Good working practices should be adopted ensure that large objects with the potential to collapse are not deposited within the upper layers of the waste profile and all waste deposits are well compacted. Further investigation is not considered to be required. It is proposed that the final waste surface be graded and inspected prior to placement of the regulation layer. This practice will eliminate the potential for near-surface cavities to be present, and therefore is not considered to require further assessment.

2.3.4.3 *Stability*

The proposed pre-settlement slope gradient will need to be considered with respect to long-term conditions. All materials employed within the landfill and its engineering are quick draining and as such, short term, undrained conditions are not deemed appropriate for consideration. Stability of the capping system requires assessment with regard to interface shear strengths of the adjacent materials.

In terms of the potential influence of gas pressures on the capping stability, a gas collection and extraction system will be installed at the site, to control gas pressures under the cap and eliminate the potential for any significant pressure to build up beneath the capping system. The issue of gas pressure beneath the cap will be considered in terms of interface (veneer) stability.

2.3.4.4 *Construction*

Construction vehicles shall not be allowed to operate directly on top of the geosynthetic cap and wheeled 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 cover materials/soils are spread upslope as per good practice to prevent tension/damage within the lower geosynthetics.

The potential effects of construction plant activity on the capping 1V:5H slope gradient during placement of restoration soils should be considered as geosynthetics are to be used in the capping system.

2.3.4.5 *Wind Uplift*

There is the possibility that the temporary capped western slope could be uplifted by the wind if a geosynthetic capping system is used. However, Opalvale have elected to temporarily cap the slope with site-won soils and therefore wind uplift is not considered to require further assessment.

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3 Stability Risk Assessment Modelling

3.1 Modelling Approach & Software

A stability assessment undertaken represents the considered scenarios for the different modelled phases of the landfill 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 embankment.
- Limit equilibrium analyses for the derivation of factors of safety for the capping stability analysis.

The stability analysis program SLIDE2 (Version 9.008) from RocScience has been used to undertake the limit equilibrium using the Bishop simplified and Morgenstern-Price for potential circular failure slopes and Morgenstern-Price and Spencer 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 and the potential presence of perched leachate. The limit equilibrium analyses for the waste mass modelling have therefore been undertaken using circular analysis for the Ru=0 with no seismic Loading scenarios where anisotropy is not present and non-circular analysis for most other situations.

The scenarios assessed are considered to be the critical worst case (highest) slopes. Notably the western 1V:3H slope that will be temporarily capped prior to the future phases being developed and the 25.5m high 1V:5H permanently capped northern, southern and eastern 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

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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.

primary search method which should always be tried first for a slope model with non-circular failure modes. (RocScience 2016).'

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.
- Drained shear strength of the various soil strata, the interfaces between the geosynthetic components and the waste.

Laboratory testing on two sets of soil samples 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 Table 3-1.

Table 3-1and Table 3-2present a summary of the principal geotechnical characteristics of these two sets of data. The two soil types tested relate to the proposed materials that were to be employed in the cover soils above the low permeability geomembrane. The first soil, labelled SOIL, in the lab testing is to be used in the main body of the restoration soils. The second soil, labelled SAND, was originally considered as a drainage medium placed immediately above the geomembrane.

Laboratory or *insitu* testing of the strata beneath the landfill were not undertaken as the focus of this report is the restoration profile. The previous report prepared by Golders details the stability of the landfill in relation to the basal geology. As such, the geotechnical properties used in the Golders report have been employed herein for the basal geological layers.

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
LF_SAND_LLDPE_TALIS2012_DDST3	Sand/LLDPPE Interface	31.96	6.3	21.36	2.65
LF_SOIL_TALIS2012_DDST3	Soil/Geotextile Interface	22.29	9.58	17.22	1.49

Table 3-2: Summary of Basic Geotechnical Properties

Sample	Soil	Maximum Dry Density (Mg/m^3)	Optimum Moisture Content (%)	D_{10} (mm)
LF_SOIL_TALIS2012	Light Brown Silt and Fine to Medium Sand	1.72	17.5	0.0045
LF_SAND_TALIS2012	Light Brown Fine to Medium Sand	1.95	10.5	0.085

A triaxial test was conducted on the SOIL which was shown to yield a coefficient of permeability of 1.129×10^{-7} m/s. Using Hazen's Rule, the effective particle size (D_{10}) can be empirically related to the permeability. For the SOIL, the coefficient of permeability is calculated to be 2.205×10^{-7} m/s which is pretty close to the measured value from the triaxial test.

The permeability of the more porous SAND can be assessed similarly by Hazen's rule and produces a value of 7.225×10^{-5} m/s. As stated previously this material was hoped to be used as the sub-SOIL drainage layer, however, the performance may, judging by the Hazen's Rule value, not be as a high as anticipated and needed. As such, the use of a suitably specified Geonet may be required to maintain a dry interface between the geomembrane and the covers soils. There are a large number of different manufacturers producing geonets and therefore at this stage the comprehensive testing regime needed was not carried.

TR1[1] has conducted a literature study on interface friction for geonets against a textured geomembrane. For this SRA, the values recommended therein has been employed. As part of the detailed design or prior to tendering the construction contract appropriate laboratory testing should be carried out between the potential geonets likely to be used and proposed geomembrane. This will be needed to confirm that the real world shear strength achieves the minimum standard set by the design and the values set out in TR1[1], namely a peak effective angle of friction of 11 degrees with a cohesion of 3kPa. As can be seen by comparing the appropriate values in Table 3-1using a geonet will create a significantly low interface friction.

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.

The geotechnical parameters for limit equilibrium analysis include the shear strength and unit weight of each material within the model, plus porewater. Shear strength has largely been defined using the effective shear strength parameters of cohesion, (c'), and the angle of shearing resistance, (ø').

In terms of waste strength, conservative values of effective shear strength parameters as derived from a study of geotechnical properties of municipal waste by Van Impe and Bouazza[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

[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.

depending on composition and strain. The landfill at Salt Valley Road will accept a range of municipal, commercial and 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.

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
Restoration Soils	19	5	28 (21)	Fine Sand
Waste	10	5	25	Mixed Putrescible Waste.
Basal Soils*	20	0	24	Fine Sand

*from Golders Report "Opal Vale – Technical Studies in Support of the Design, December 2014"

For the closed form analysis, interface design parameters are presented in Table 3-4, friction angles and cohesion are as published in R&D technical report TR1[6], and Talis historic shear box data.

Table 3-4: Closed Form Interface Design Parameters

Interface	Peak		Post Peak	
	c' (kPa)	ø'	c' (kPa)	ø'
Restoration Soil	5	28	5	22
Restoration Soil/Geonet (Drainage Geocomposite)	3	22	2	17
Geonet/Textured LLDPE Geomembrane	3	11	2.4*	9.1
Geomembrane/Subgrade	5	32	2.6	21.2

*TR1 post peak cohesion reported as 9.2kPa. However, there were only 3 relevant tests considered. Therefore the Cohesion is reduced to reflect a degree of strain softening at the interface - reduced to 80% of peak cohesion 2.4kPa.

TR1[6], reports textured geomembrane to geonet (drainage geocomposite) interface results of 11 degrees and 3 kPa peak, and 9.1 degrees and 9.2 kPa for residual. The TR1 post peak cohesion values have been lowered from 9.2 kPa to 80% of peak cohesion conditions (2.4 kPa) to account for a degree on strain softening at the interface.

Site specific interface friction tests are recommended to be undertaken on final selected geosynthetic products prior to incorporation into any capping and restoration work during detailed design works.

[6] N Dixon and D R V Jones, 'Stability of Landfill Lining Systems: Report No. 1 Literature Review, R&D Technical Report P1-385/TR1', Environment Agency UK, 2002

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 (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 3-1.

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 3-2.

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

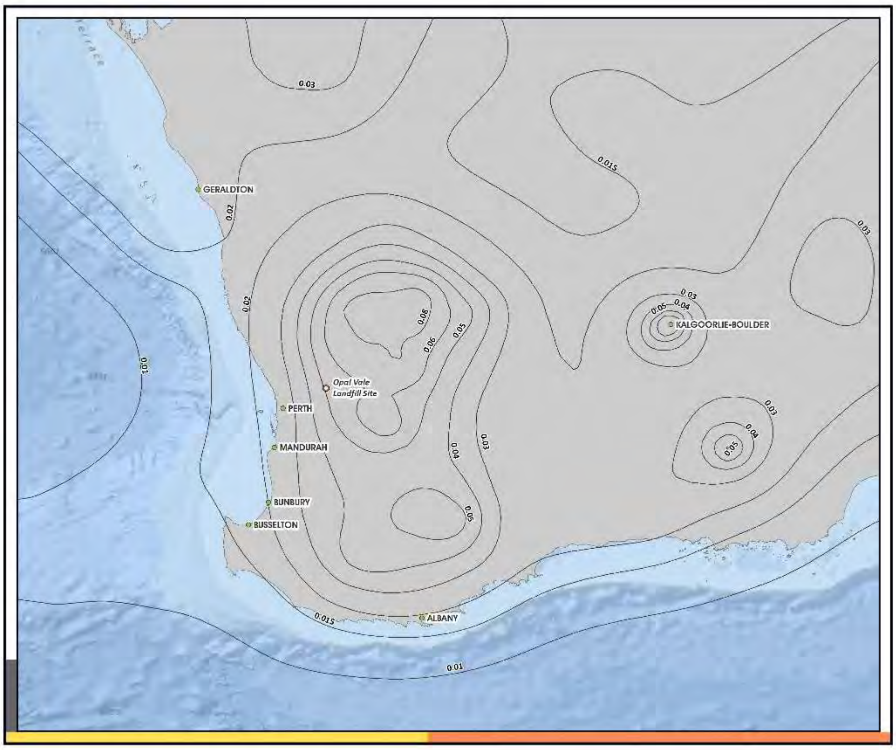


Figure 3-1: NSHA18 – 10% Probability of Exceedance in 50 years (1:475 AEP) contours

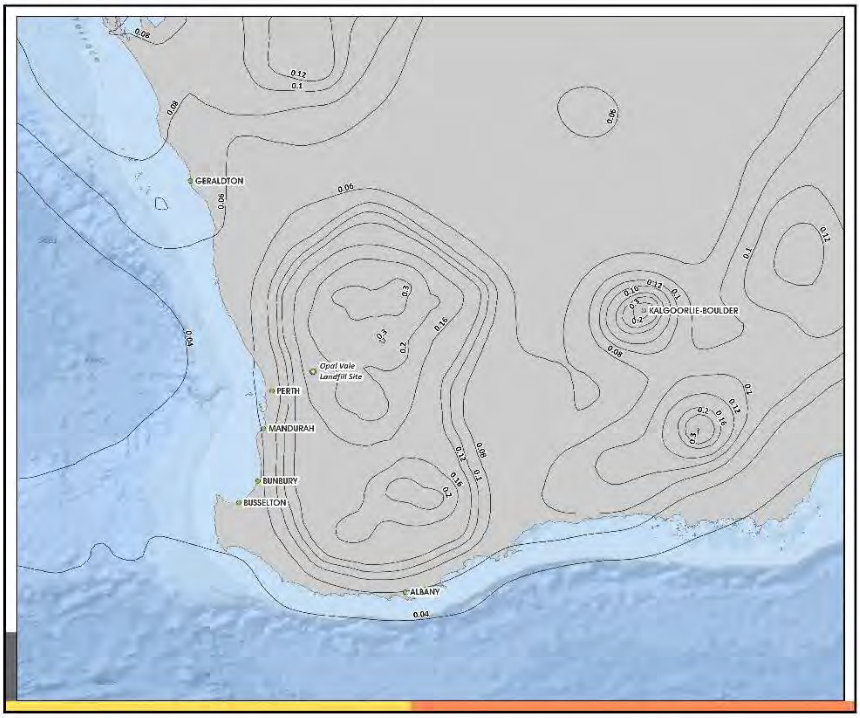


Figure 3-2: NSHA18 – 2% Probability of Exceedance in 50 years (1:475 AEP) contours

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 Spencer and Morgenstern-Price non-circular forms of analysis, for the 1V:3.0H temporary waste slopes adjacent to future Stage 2. 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. For the purpose of the stability assessment an inferred excavation profile has been utilised. As described in Section 1.5.4, 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 and gas that could increase pore fluid pressure within the waste.

The Waste Mass Stability summary is presented in Table 3-5.

Table 3-5: Waste Mass Analysis Results

Scenario	Method	Factor of Safety	Comments
Temporary Capping Slope No Seismic Loading	Drained Circular	1.735	1V:3.0H temporary waste slope. R_u value of 0.0 applied to waste. Acceptable (FoS > 1.3)
Temporary Capping Slope No Seismic Loading	Drained Non-Circular	1.581	1V:3.0H temporary waste slope. R_u value of 0.1 applied to waste. Acceptable (FoS > 1.3)
Temporary Capping Slope with Seismic Loading (OBE - 1:475)	Drained Circular	1.421	1V:3.0H temporary waste slope. R_u value of 0.0 applied to waste. Acceptable (FoS > 1.1)
Temporary Capping Slope with Seismic Loading (OBE - 1:475)	Drained Non-Circular	1.299	1V:3.0H temporary waste slope. R_u value of 0.1 applied to waste. Acceptable (FoS > 1.1)
Temporary Capping Slope with Seismic Loading (SEE/MCE - 1:1000 AEP)	Drained Circular	1.157	1V:3.0H temporary waste slope. R_u value of 0.0 applied to waste. Acceptable (FoS > 1.0)
Temporary Capping Slope **with Seismic Loading (SEE/MCE - 1:1000 AEP)**	**Drained** **Non-Circular**	**1.062**	**1V:3.0H temporary waste slope. R_u value of 0.1 applied to waste.** **Acceptable (FoS > 1.0)**

The lowest calculated factor of safety for the temporary capped waste slopes geometric scenario are greater than 1.3, 1.1 and 1.0 for the static, OBE and SEE/MCE AEP scenarios, and are therefore considered acceptable.

It should be acknowledged that it is anticipated that the temporary status for the slope will last just a few years. As such, not being a permanent condition the inclusion of the 1:1000 AEP is perhaps regarded as unnecessary because of the unlikelihood that such a severe seismic event would occur in such a short time frame, but is included for comparison.

The model results for temporary waste slopes are presented in APPENDIX D.

3.6 Capping Stability Analysis

There are two topographic scenarios which need to be considered, namely: a) North to South Section perpendicular through the crown and down the other side of the profile achieving the maximum height difference of 25.5m, and b) North West to South East Section along the ridgeline down to the ground surface. For the latter the height difference between the ridgeline and the ground is just over 13m because the waste mass is placed against the former side of the quarry.

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 and Morgenstern Price and Spencer non-circular forms of analysis. A summary of the results for the North to South Section is presented in Table 3-6.

Table 3-6: Summary of Stability Analysis for Capping Profile – General North to South Slope

Scenario	Method	Factor of Safety	Comments
Permanent Capping Slope General North to South Section No Seismic Loading	Drained Circular	2.965	1V:5H Capping Profile – 25.5m vertical height. R_u 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	2.695	1V:5H Capping Profile – 25.5m vertical height. R_u 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	2.444	1V:5H Capping Profile – 25.5m vertical height. R_u 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 Circular	2.209	1V:5H Capping Profile – 25.5m vertical height. R_u 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	2.010	1V:5H Capping Profile – 25.5m vertical height. R_u 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	1.803	1V:5H Capping Profile – 25.5m vertical height. R_u 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 Circular	1.672	1V:5H Capping Profile – 25.5m vertical height. R_u 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	1.507	1V:5H Capping Profile – 25.5m vertical height. R_u 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	1.356	1V:5H Capping Profile – 25.5m vertical height. R_u 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

The results of the analyses for the North West to South East Section is presented in Table 3-7.

Table 3-7: Summary of Stability Analysis for Capping Profile – Maximum North West to South East Slope

Scenario	Method	Factor of Safety	Comments
Permanent Capping Slope Peak North West to South East Section No Seismic Loading	Drained Circular	3.002	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.0 applied to waste Acceptable (FoS > 1.5)
Permanent Capping Slope Peak North West to South East Section No Seismic Loading	Drained Non-Circular	2.726	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.1 applied to waste Acceptable (FoS > 1.5)
Permanent Capping Slope Peak North West to South East Section No Seismic Loading	Drained Non-Circular	2.464	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.2 applied to waste Acceptable (FoS > 1.5)
Permanent Capping Slope Peak North West to South East Section With Seismic Loading (OBE - 1:475AEP)	Drained Circular	2.234	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.0 applied to waste Acceptable (FoS > 1.1)
Permanent Capping Slope Peak North West to South East Section With Seismic Loading (OBE - 1:475AEP)	Drained Non-Circular	2.036	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.1 applied to waste Acceptable (FoS > 1.1)

Permanent Capping Slope Peak North West to South East Section With Seismic Loading (OBE - 1:475A EP)	Drained Non-Circular	1.839	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.2 applied to waste Acceptable (FoS > 1.1)
Permanent Capping Slope Peak North West to South East Section With Seismic Loading (SEE/MCE - 1:1000 AEP)	Drained Circular	1.707	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.0 applied to waste Acceptable (FoS > 1.0)
Permanent Capping Slope Peak North West to South East Section With Seismic Loading (SEE/MCE - 1:1000 AEP)	Drained Non-Circular	1.559	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.1 applied to waste Acceptable (FoS > 1.0)
Permanent Capping Slope Peak North West to South East Section With Seismic Loading (SEE/MCE - 1:1000 AEP)	Drained Circular	1.406	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.2 applied to waste Acceptable (FoS > 1.0)

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.

In addition to the assessment of the stability of mass slopes, the most likely failure mechanism will be that along the interface of the weakest element of the restoration materials. By inspection, the weakest interface will be between the geonet and the geomembrane. A weak thin layer with corresponding shear strength characteristics equivalent to the geonet/geomembrane was inserted into the cover soils and the failure mechanism forced to travel along a plane within the weak layer. A conventional non-circular analysis was carried out instead of using the Cuckoo search method. This is straight forward to consider as the failure will travel along the longest length of the weak plane which is equivalent to a 13.25m height difference. Any shorter lengths would have a greater influence from the end effects meaning the factor of safety would be higher.

Using the Ru approach to simulate the presence of water on top of the geomembrane is a convenient approach as the failure plane is line and so would, by and large, the presence of water that had filtered through the cover soils. Of course, the design of the geonet should be such that the slope remains dry. The inclusion of the Ru is to provide some conservatism into the analysis.

The results of the analysis of the restoration profile along the weak layer is presented in Table 3-8.

Table 3-8: Summary of Stability Analysis for Capping Profile – Stability of Restoration Profile

Scenario	Method	Factor of Safety	Comments
Permanent Capping Slope Stability of Restoration Profile No Seismic Loading	Drained Non-Circular	2.035	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.0 applied to cover soils. Acceptable (FoS > 1.5)
Permanent Capping Slope Stability of Restoration Profile No Seismic Loading	Drained Non-Circular	2.024	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.1 applied to cover soils. Acceptable (FoS > 1.5)
Permanent Capping Slope Stability of Restoration Profile No Seismic Loading	Drained Non-Circular	2.014	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.2 applied to cover soils. Acceptable (FoS > 1.5)
Permanent Capping Slope Stability of Restoration Profile With Seismic Loading (OBE - 1:475 AEP)	Drained Non-Circular	1.539	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.0 applied to cover soils. Acceptable (FoS > 1.1)
Permanent Capping Slope Stability of Restoration Profile With Seismic Loading (OBE - 1:475 AEP)	Drained Non-Circular	1.531	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.1 applied to cover soils. Acceptable (FoS > 1.1)
Permanent Capping Slope Stability of Restoration Profile With Seismic Loading (OBE - 1:475 AEP)	Drained Non-Circular	1.523	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.2 applied to cover soils. Acceptable (FoS > 1.1)
Permanent Capping Slope Stability of Restoration Profile With Seismic Loading (SEE/MCE - 1:1000 AEP)	Drained Non-Circular	1.202	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.0 applied to cover soils. Acceptable (FoS > 1.0)
Permanent Capping Slope Stability of Restoration Profile With Seismic Loading (SEE/MCE - 1:1000 AEP)	Drained Non-Circular	1.196	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.1 applied to cover soils. Acceptable (FoS > 1.0)
Permanent Capping Slope Stability of Restoration Profile With Seismic Loading (SEE/MCE - 1:1000 AEP)	Drained Non-Circular	1.189	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.2 applied to cover soils. Acceptable (FoS > 1.0)

The lowest calculated factor of safety for the capping slopes geometric scenario are greater than 1.5, 1.1 and 1.0 for the static, OBE and SEE/MCE AEP scenarios, and are therefore considered acceptable.

The model results for capping slopes are presented in APPENDIX E.

3.6.1 Closed Form Analysis

A closed form analysis for the capping assessment has been undertaken and is presented in APPENDIX G.

For the 1V:5H capping profile, a waste regulation layer which will act as a gas collection layer will be overlain by a textured 1.5 mm LLDPE geomembrane and a geonet (drainage geocomposite) on top and 1.0 m of restoration soils protecting the geosynthetics. The geonet (drainage geocomposite) beneath the restoration soils around the landfill perimeter reduces the ability for pore pressures to build up within the system therefore a low parallel submerged ratio (PSR), as defined by Jones and Dixon[13], can be used.

The assessment was undertaken for both peak and post peak conditions.

The results of the capping stability analysis are presented in APPENDIX E 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:5H capping profile, and a PSR of 0.3, the minimum reported factor of safety is 1.74, along the restoration LLDPE/geonet interface. For assumed post peak strength conditions with a PSR of 0.3 the minimum FoS reported is 1.40, along also the LLDPE/geonet interface. As part of the analysis, no induced tensions are reported in the geosynthetics.

All FoS calculated are in excess of the minimum values for both peak and post peak scenarios, and therefore deemed acceptable.

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:5H geosynthetic capping slopes. The stability of a 1V:5H capping slope under the influence of construction plant operations has been assessed using the procedure proposed by Kerkes[15] and is presented in APPENDIX H.

It is assumed that the cover materials/soils are spread upslope as per normal good practice to prevent tension/damage within the lower geosynthetics.

The analysis shows that based on a 1.0m depth of cover soil a factor of safety of 1.57 against rupture of the geomembrane assuming the lowest peak shear strength conditions (11° & 3 kPa) at the geonet to LLDPE geomembrane interface at time of placement. The analysis has been undertaken assuming no limiting tension in the geomembrane and a typical unit of plant for such work such as a CAT D6N LGP bulldozer. The calculated factor of safety is above 1.3 which is considered acceptable.

[15] Kerkes, D.J., (1999),'Analysis of equipment loads on geocomposite liner systems', Proc. Geosynthetics 1999

A further analysis is presented for a layer depth of 0.3m, assuming the other conditions remain the same. This shows the factor of safety to be 0.82 which is clearly unacceptable. In fact, an acceptable factor of safety is not achieved until the initial layer thickness is at least 600mm. Of course, if the placement conditions change then additional calculations will be essential.

3.6.3 Gas Pressure

The build-up of gas pressure from the landfill is relevant to the stability of capping systems and the lining of existing waste slopes. Pore pressures generated by landfill gas can be shown to significantly reduce the effective normal stress on the lower geomembrane interface and can lead to instability (e.g. of a cover veneer). An assessment in accordance with the methodology proposed by Thiel[16] has been undertaken, based on the lowest interface shear strengths for the LLDPE geomembrane and geonet for both peak and post peak conditions (with post peak cohesion/adhesion also reduced to 3kPa in line with the previous sensitivity analysis).

The NSW EPA 'Hazardous Ground Gases'[17] publication states 'an active or recently-closed landfill can produce gas under significant pressure (typically 0.3–3 kPa)'. Thiel (2008) reports conceivable gas pressures for lowest, highest and most likely as 0, 4 and 1 kPa respectively.

The waste composition at Salt Valley is predominantly municipal solid waste, commercial and industrial, with some construction and demolition waste. Due to the moderate rainfall and temperate climate at Salt Valley, the placed waste can be categorised as 'dry'. Waste stabilization would occur very slowly under dry conditions and this process could continue for many decades, with lower rates of gas production.

For the purpose of the assessment a nominal gas pressure (Ug) of 2 has been utilised.

Analysis for the interface assessment with regards to gas pressure upon the capping system has shown, for the interfaces and the gas pressure considered, that a factor of safety of 1.673 exists for peak and 1.359 for post peak conditions on the steepest capping slope of 1V:5H, which is considered acceptable.

3.7 Sensitivity

The US Army Corps of Engineers, 1984 recommend the use of undrained conditions for cohesive soils and drained conditions for free draining granular materials, with a 20 percent strength reduction to allow for strain weakening during earthquake loading.

A sensitivity analysis was undertaken with reduced shear strength parameters (as shown in the parentheses in APPENDIX F), and undrained strength conditions for the sideslope with a seismic loading of 1:1000 AEP.

[16] Thiel, R. (1999). Design of a gas pressure relief layer below a geomembrane cover to improve stability, Proc. Geosynthetics '99, Boston, NAGS.
[17] NSW EPA 'Guidelines for the Assessment and Management of Sites Impacted by Hazardous Ground Gases', 2012.

In order to assess the sensitivity of the impact of a weak layer along interface of the lining system, a geosynthetic element with a density of 10 kN/m^3 and an interface friction of 9 degrees and cohesion of 2.4kPa was modelled. The results of the sensitivity results are presented in Table 3-9.

Table 3-9: Permanent Capping Slope Sensitivity Summary

Scenario	Method	Factor of Safety	Comments
Permanent Capping Slope Stability of Restoration Profile No Seismic Loading	Drained Non-Circular	1.643	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.0 applied to cover soils. Softened Interface. Acceptable (FoS > 1.1)
Permanent Capping Slope Stability of Restoration Profile No Seismic Loading	Drained Non-Circular	1.637	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.1 applied to cover soils. Softened Interface. Acceptable (FoS > 1.1)
Permanent Capping Slope Stability of Restoration Profile No Seismic Loading	Drained Non-Circular	1.631	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.2 applied to cover soils. Softened Interface. Acceptable (FoS > 1.1)
Permanent Capping Slope Stability of Restoration Profile With Seismic Loading (OBE - 1:475 AEP)	Drained Non-Circular	1.237	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.0 applied to cover soils. Softened Interface. Acceptable (FoS > 1.1)
Permanent Capping Slope Stability of Restoration Profile With Seismic Loading (OBE - 1:475 AEP)	Drained Non-Circular	1.233	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.1 applied to cover soils. Softened Interface. Acceptable (FoS > 1.1)
Permanent Capping Slope Stability of Restoration Profile With Seismic Loading (OBE - 1:475 AEP)	Drained Non-Circular	1.228	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.2 applied to cover soils. Softened Interface. Acceptable (FoS > 1.1)
Permanent Capping Slope Stability of Restoration Profile With Seismic Loading (SEE/MCE - 1:1000 AEP)	Drained Non-Circular	1.026	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.0 applied to cover soils. Softened Interface. Acceptable (FoS > 1.0)
Permanent Capping Slope Stability of Restoration Profile	Drained	0.980*	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.1

With Seismic Loading (SEE/MCE - 1:1000 AEP)	Non-Circular		applied to cover soils. Softened Interface. Acceptable (FoS > 1.0)
Permanent Capping Slope Stability of Restoration Profile With Seismic Loading (SEE/MCE - 1:1000 AEP)	Drained Non-Circular	0.934*	1V:5H Capping Profile – 13.25m vertical height. R_u Value of 0.2 applied to cover soils. Softened Interface. Acceptable (FoS > 1.0)

*See following paragraph

The analyses demonstrate acceptable factors of safety for drained conditions with reduced strength parameters and a weak post peak interface for all scenarios with a seismic loading up to an AEP of 1:1000years. As will be noted, the post peak shear properties of the interface between the geonet and geomembrane are associated with an unacceptable value for the Factor of Safety. As referred to previously, the interface friction has been taken from a range of tests included in TR1. It will be necessary during the detailed design stage of the project, or prior to construction when searching for suitable materials, that the proposed product combination achieves shear properties slightly highly than the mean presented in TR1. The minimum post peak shear properties that must be specified are Angle of Friction = 10 degrees and a cohesion Intercept of 2.7 kPa. Using these properties, the Factor of Safety is slightly above unity which is then acceptable.

Analyses are presented in APPENDIX F.

3.8 Assessment Summary

3.8.1 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, FS_{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.

Kavazanjian[19] infers the allowable seismic displacement should be based on factors for allowing detection and repair of breaches in the containment system on a project specific basis that should lead to development of rational, economical seismic design criteria for a solid waste landfill facility. Damaged landfill covers, above ground pipes and tanks, surface water control systems, and ancillary facilities are generally easy to detect and repair. Generic allowable calculated seismic displacement for cover systems are documented to be 300mm to 1m.

[18] RCRA Subtitle D (258) 'Seismic Design Guidance for Municipal Solid Waste Landfill Facilities'. US EPA, EPA600/R-95/051, April 1995.
[19] Kavazanjian, Edward 'Seismic Design of Solid Waste Containment Facilities' Proceedings of the Eight Canadian Conference on Earthquake Engineering, Vancouver, BC, June 1999, pp. 51-89

All FoS calculated during seismic conditions assessed are in excess of the minimum values for both peak and post peak scenarios, and therefore deemed acceptable assuming the post peak shear properties for the geonet/geomembrane interface achieves a minimum of 10 degrees (Phi) and 2.7 kPa (Cohesion). The final restoration profile is accessible via several trackways climbing the north and south slopes; therefore, it is deemed that should any deformation of the restoration profile occur, this can be readily repaired.

3.8.2 Basal & Sideslope Assessment

No basal and sideslope assessments were undertaken as these were screened out.

3.8.3 Waste Mass Stability

Temporary capped waste slopes are proposed/modelled at a gradient of 1V:3.0H, to maximum restoration height adjacent at the landfill crown. 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 312m AHD.

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 0.1 to represent the potential effects that leachate and gas could increase pore fluid pressure within the waste.

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 (300mm at the sump) through regular extraction.

The calculated factor of safety for the temporary waste slopes are greater than 1.3, 1.1 and 1.0 for the static, OBE and SEE/MCE AEP scenarios, and are therefore considered acceptable.

3.8.4 Capping Assessment

The capping profile assessed on the basis of a limit equilibrium analysis has demonstrated that a satisfactory factor of safety will be achieved for the proposed capping and restoration slopes for both short-term and long-term conditions.

With regards to the closed form analysis, the assessment of the stability of the capping system, with regards to interface friction and PSR, has demonstrated that a satisfactory factor of safety will be achieved for the proposed capping and restoration slopes for all scenarios assessed.

Analysis for construction plant upon the capping system has shown, for the plant considered, that a factor of safety of 1.57 exists for plant working on the steepest capping slope of 1V:5H for a layer thickness of 1m, during capping construction activities, which is considered acceptable. The placement of the restoration soils should not be undertaken at less than 600mm above the geonet/geomembrane interface. Should placement conditions be considered during the construction the Factor of Safety may be different.

Analysis of gas pressures in the low rainfall temperate climate at Salt Valley Landfill has demonstrated that for the interfaces considered in the capping system acceptable factors of safety are maintained.

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 Capping

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

Surface washout is not considered as part of this report, and although erosion protection may be required to prevent scouring of the restoration soils until vegetation can be established on the rehabilitated capping surface, it is recommended that if erosion/gullying is identified, it is remediated as soon as practicable to prevent damage to the capping system and exposure of the waste mass.

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 area around the Closure and Rehabilitation Area of the landfill site (Stage 1 – Cells 1 to 6). These conditions cannot be extrapolated across any other portion of the Site.

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 are significantly different from those assumed, the assessment should be reviewed.

The stability risk assessment has been prepared to support the Closure and Post-Closure Management Plan and the geotechnical assessment and the classification stated should not be regarded as a final engineering design.

APPENDIX A
Figures

Figure 01: Site Locality Plan

APPENDIX B
Drawings

Drawing C-001: Existing Site Layout and Topography

Drawing C-105: Proposed Fill Profile

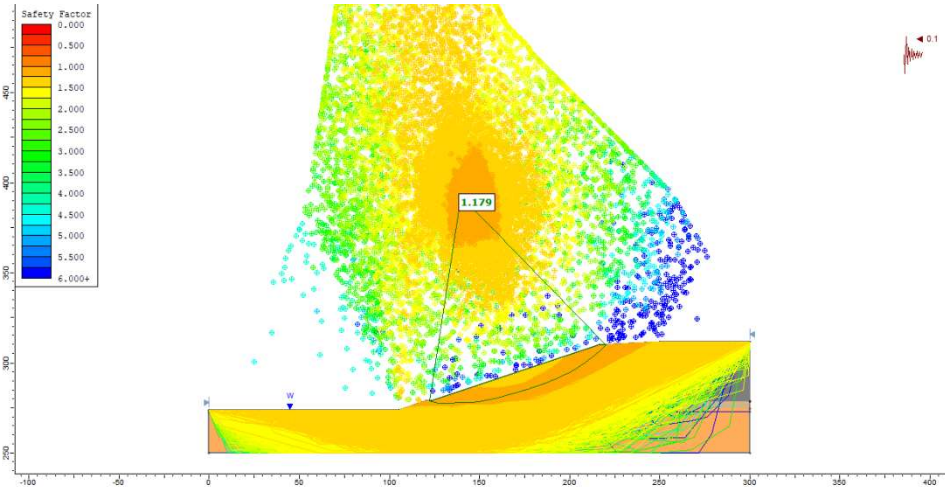
Drawing C-108: Proposed Landfill Restoration Profile & Surface Water Management

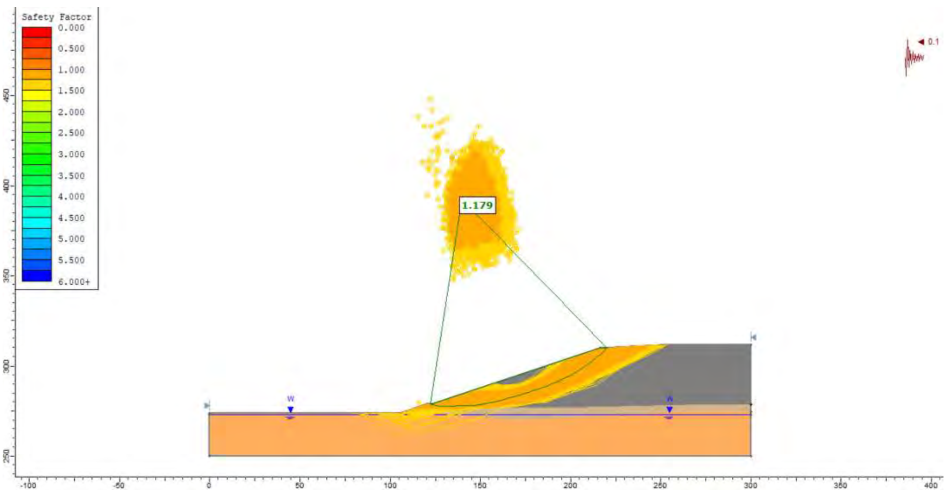
Drawing C-401: Hydrogeology Beneath Stage 1 Landfill Cells 1 to 6

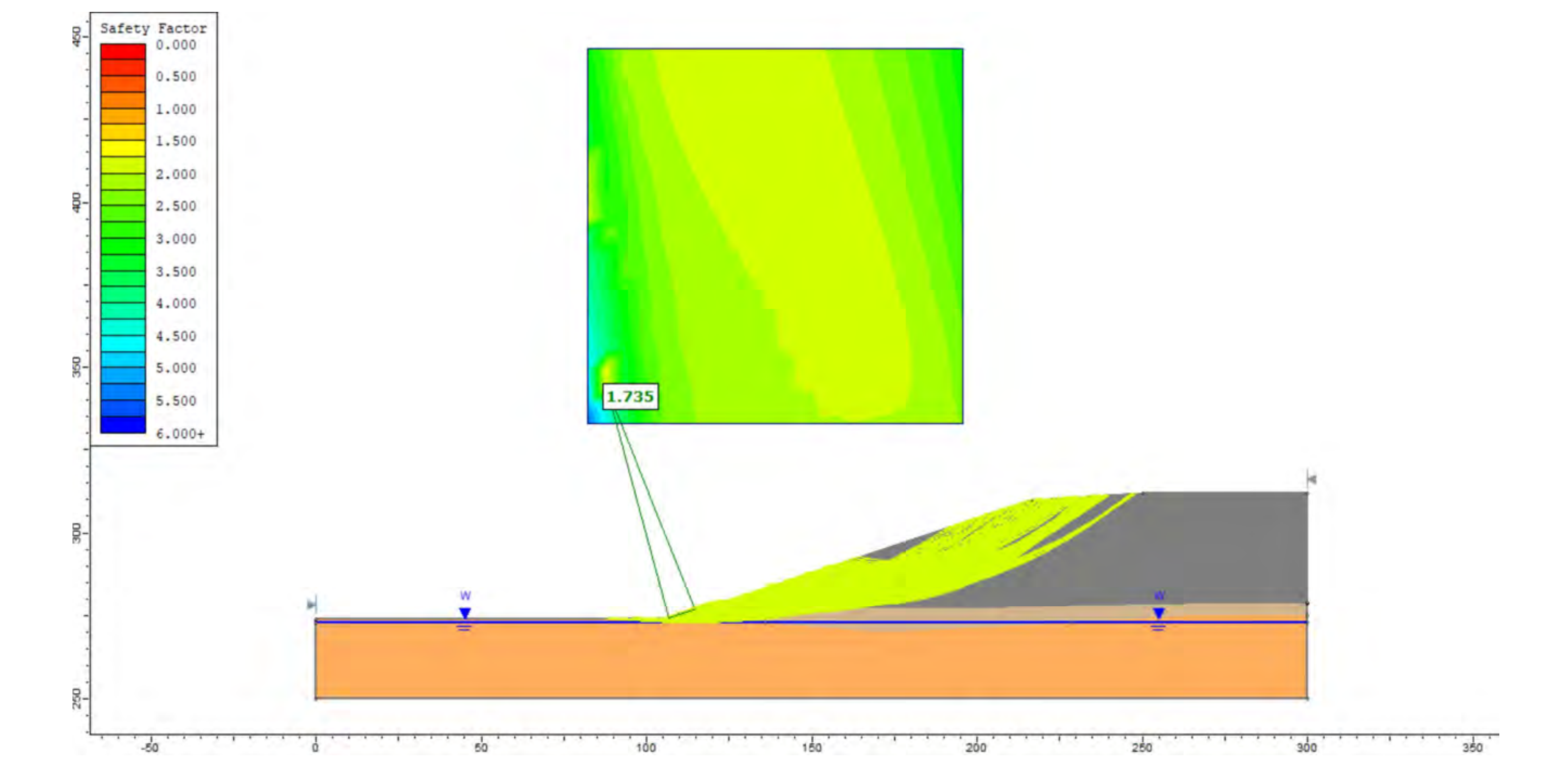
APPENDIX C
Laboratory Test Results

APPENDIX D
Waste Mass Stability Analysis

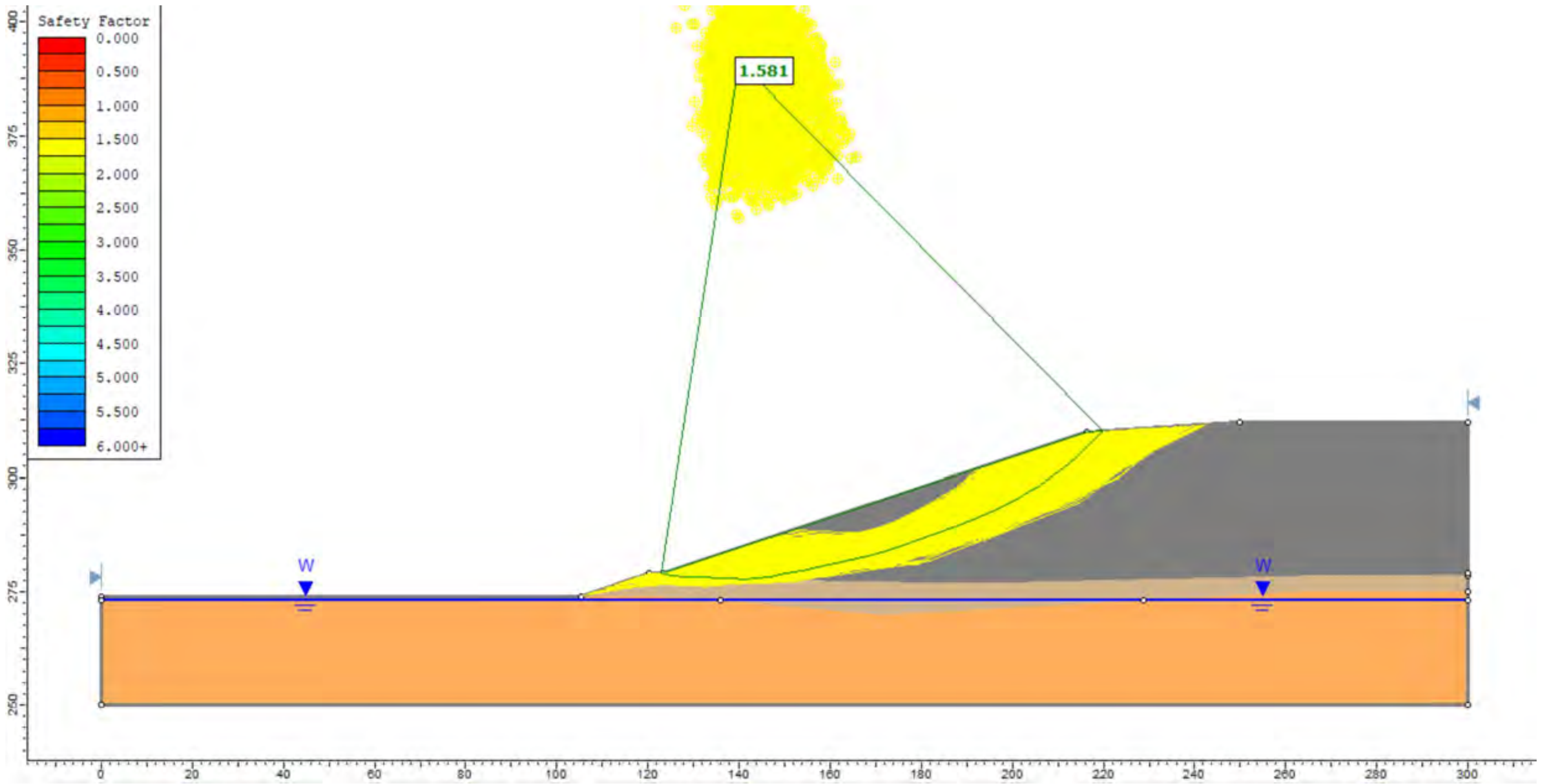
The following Analyses are shown as screen grabs from the Slide2 output. Numerous analyses are carried out for each scenario, as shown below (top), but because of the visual confusion this would cause, only the lowest are presented (below bottom).



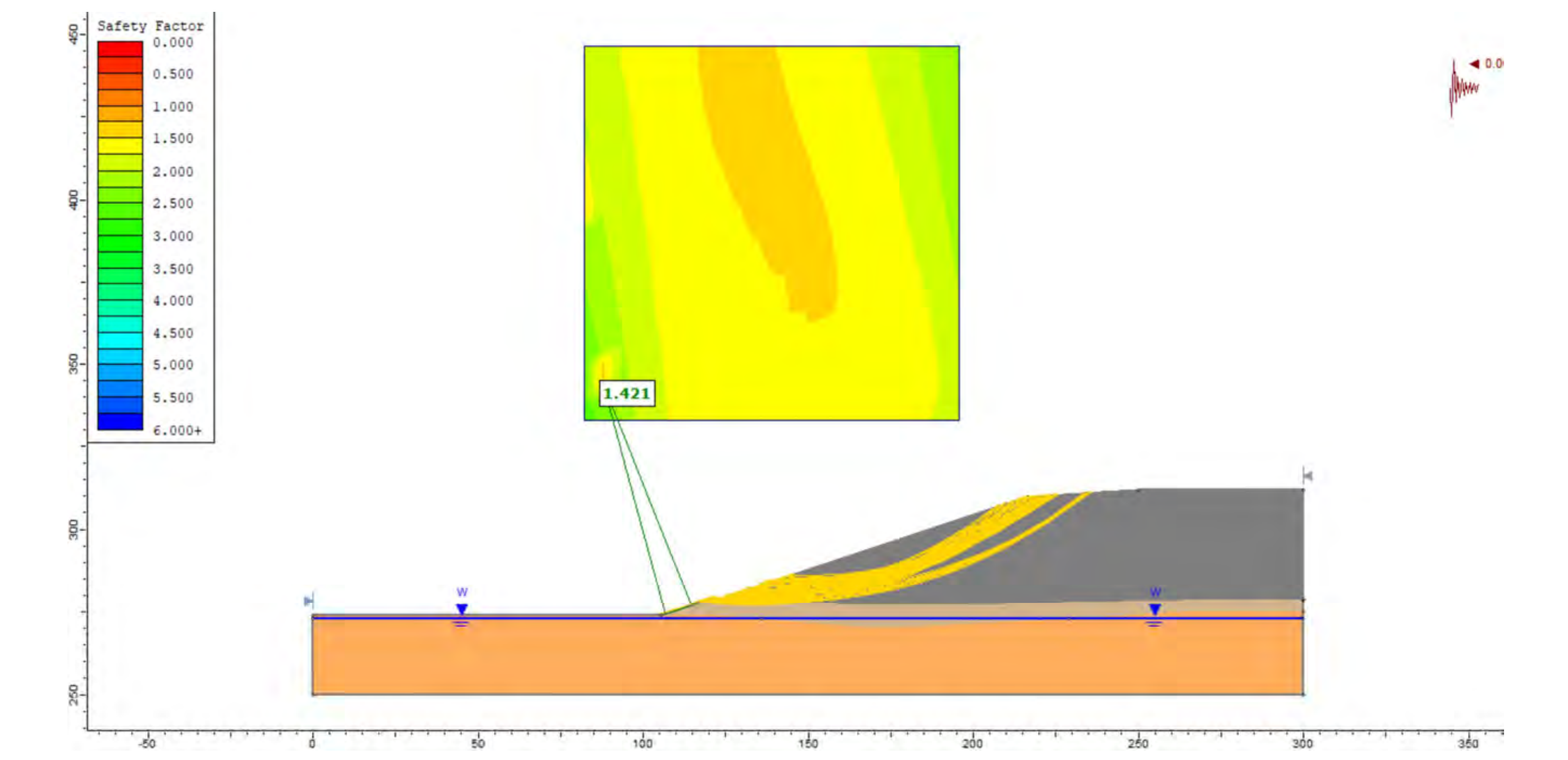




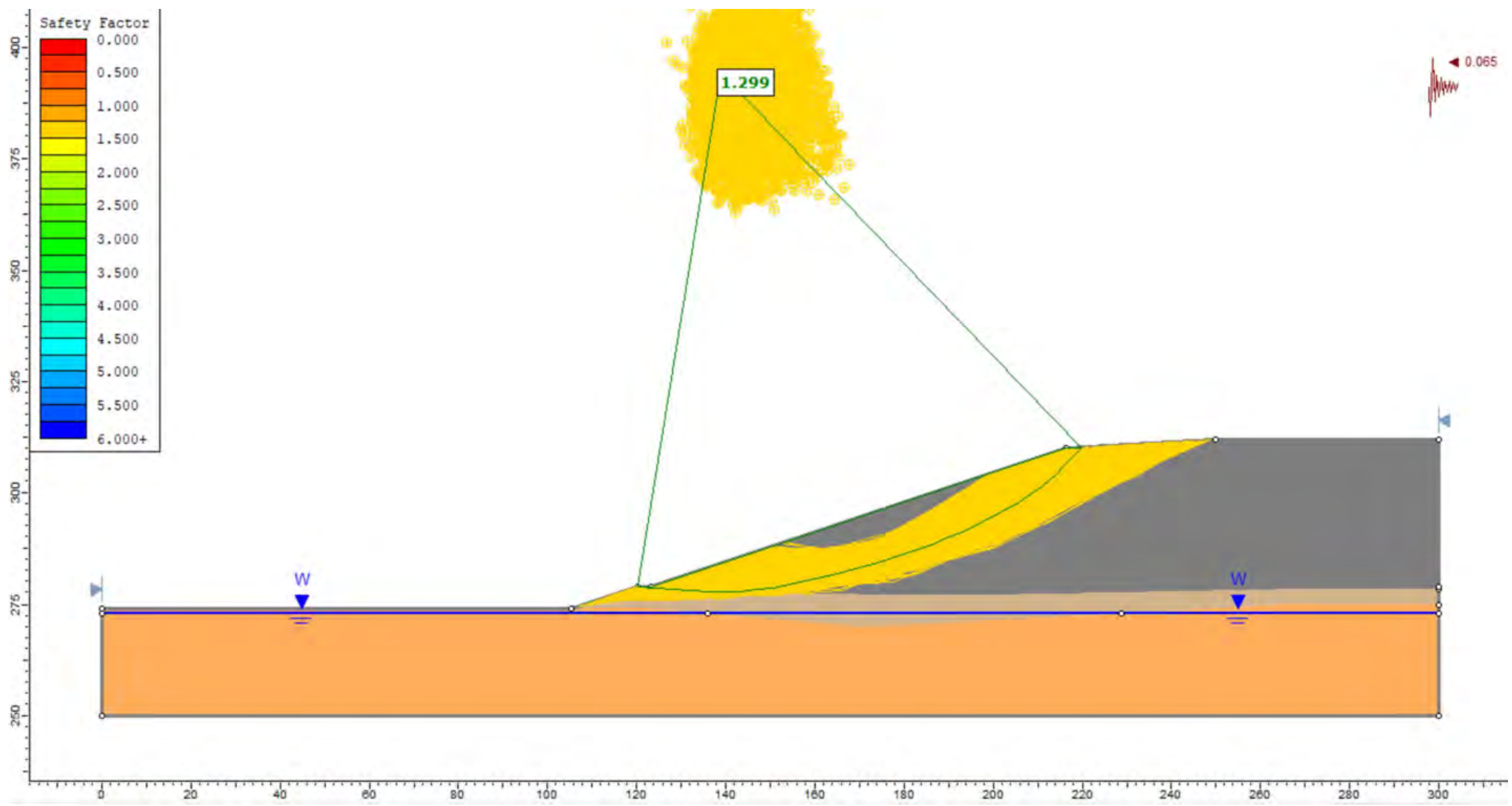
	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment		
	Analyses Description:	Temporary Capping Layer with Peak Shear Values		
	Section:	1 to 3 Western Slope	**Factors of Safety Illustrated**	1.7-2.0



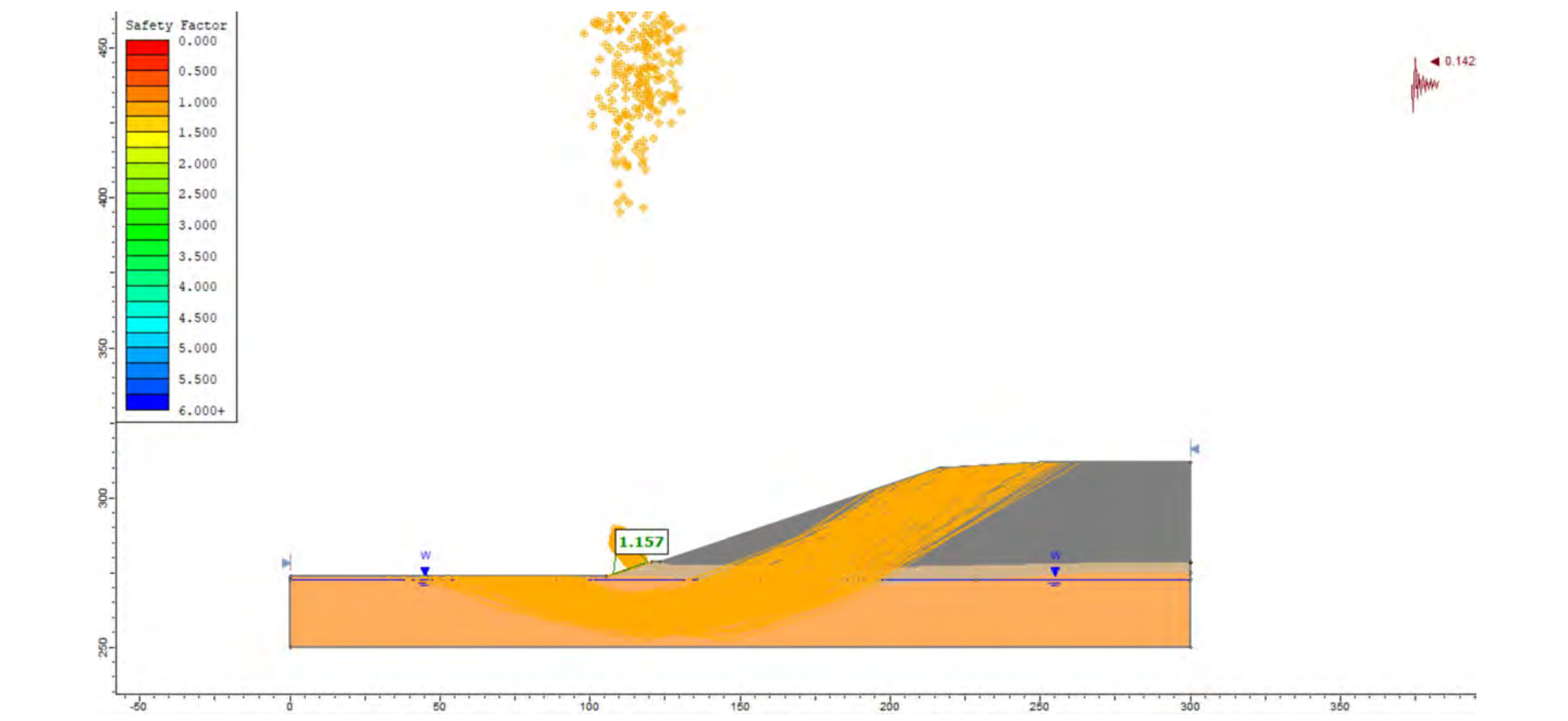
	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment		
	Analyses Description:	Temporary Capping Layer with Peak Shear Values and Ru=0.1		
	Section:	1 to 3 Western Slope	**Factors of Safety Illustrated**	1.5-1.7



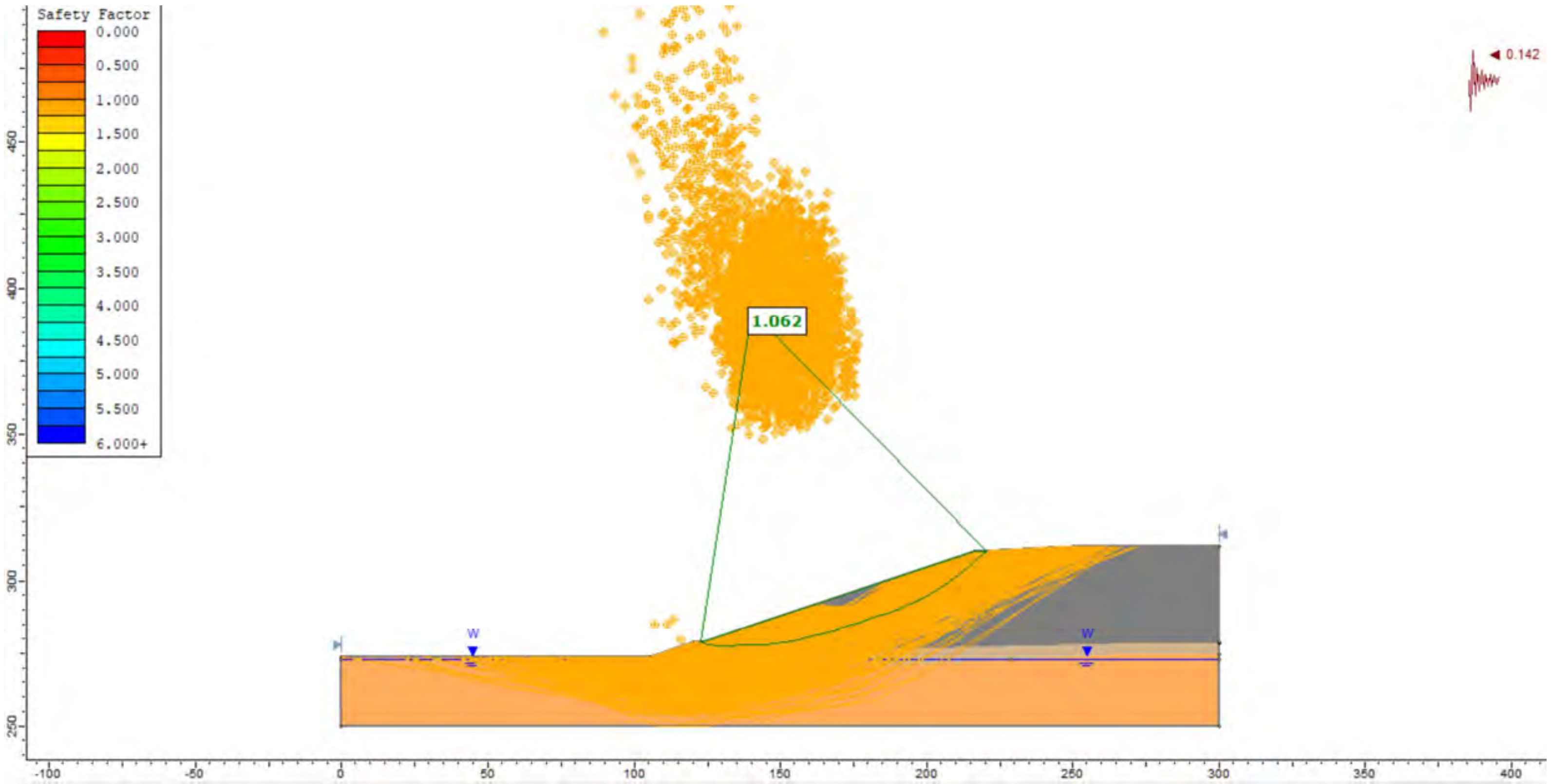
Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment			
Analyses Description:	Temporary Capping Layer with Peak Shear Values and Seismic Load (OBE- 1:475) and Ru=0.0			
Section:	1 to 3 Western Slope	**Factors of Safety Illustrated**	1.4-1.6	



	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment		
	Analyses Description:	Temporary Capping Layer with Peak Shear Values and Seismic Load (OBE- 1:475) and Ru=0.1		
	Section:	1 to 3 Western Slope	**Factors of Safety Illustrated**	1.2-1.4



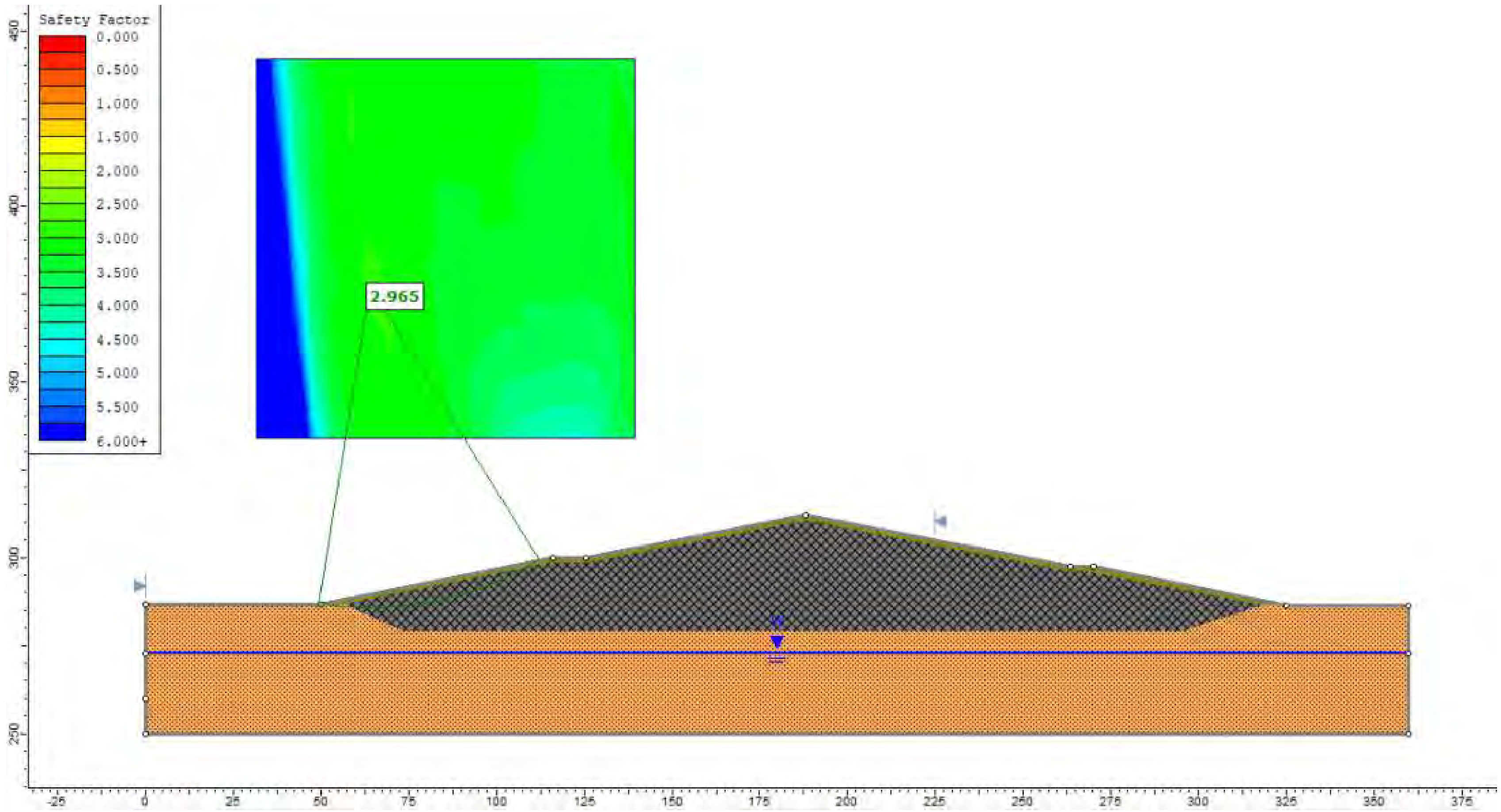
talis consultants	**Project:**	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment		
	Analyses Description:	Temporary Capping Layer with Peak Shear Values and Seismic Load (SEE/MCE – 1:1000 AEP)		
	Section:	1 to 3 Western Slope	**Factors of Safety Illustrated**	1.1-1.2



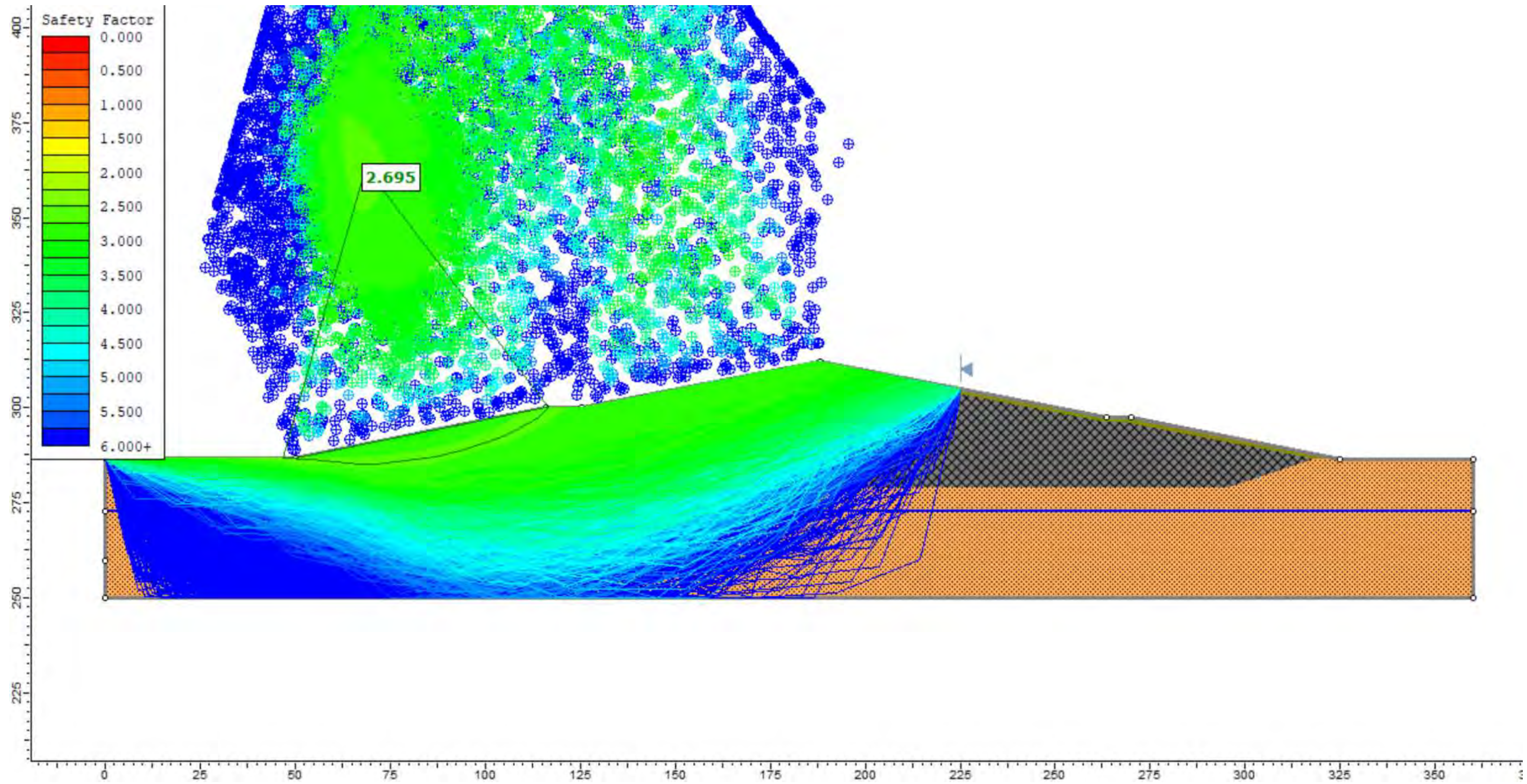
	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment		
	Analyses Description:	Temporary Capping Layer with Peak Shear Values and Seismic Load (SEE/MCE – 1:1000 AEP with Ru=0.1		
	Section:	1 to 3 Western Slope	**Factors of Safety Illustrated**	1.1-1.3

APPENDIX E

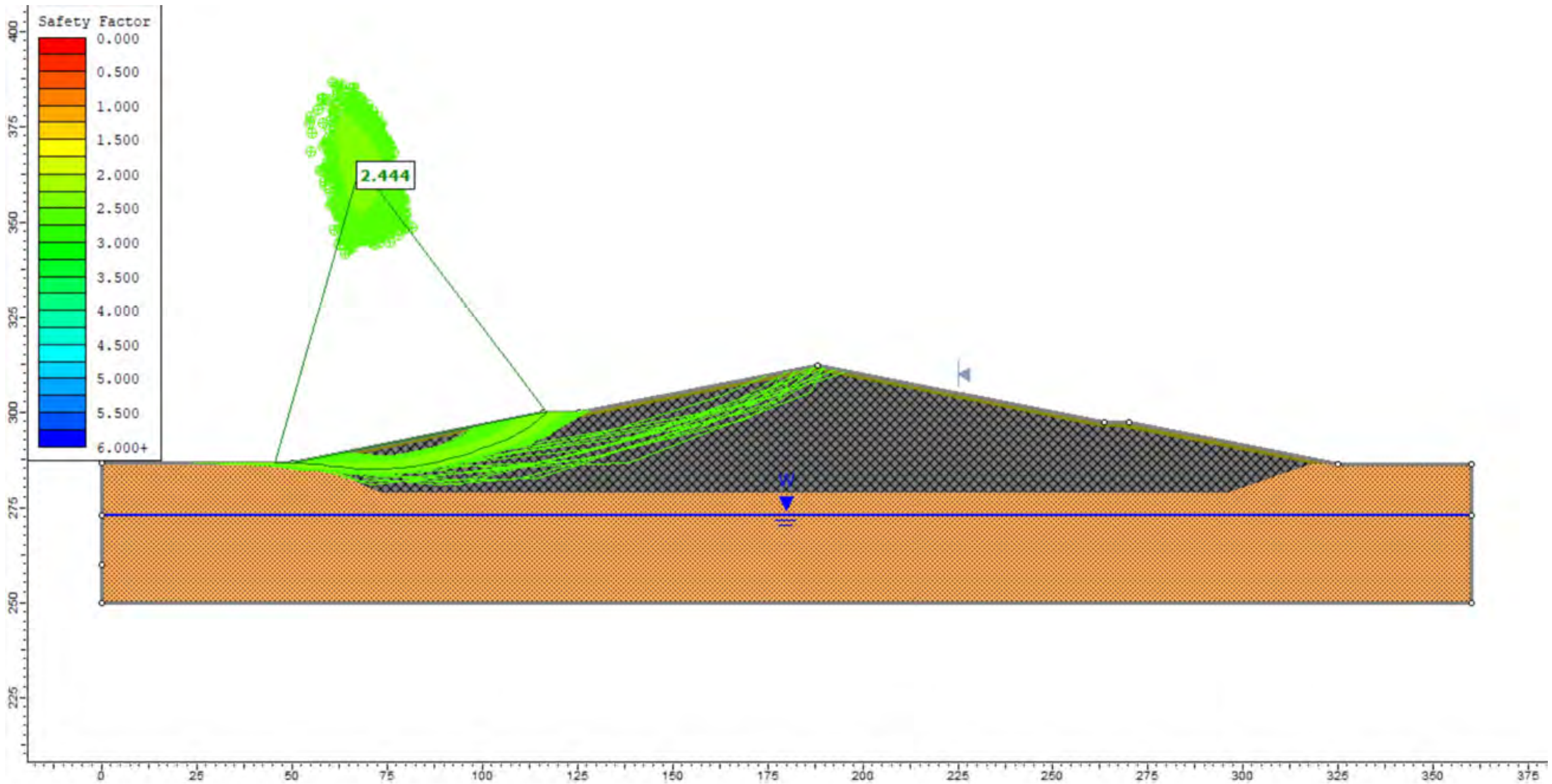
Capping Stability Analysis



	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment		
	Analyses Description:	Permanent Slope - Peak Shear Values		
	Section:	1 to 5 North to South Slope	**Factors of Safety Illustrated**	1.7-2.0



Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment			
Analyses Description:	Permanent Slope - Peak Shear Values with Ru=0.1			
Section:	1 to 5 North to South Slope	**Factors of Safety Illustrated**	All	



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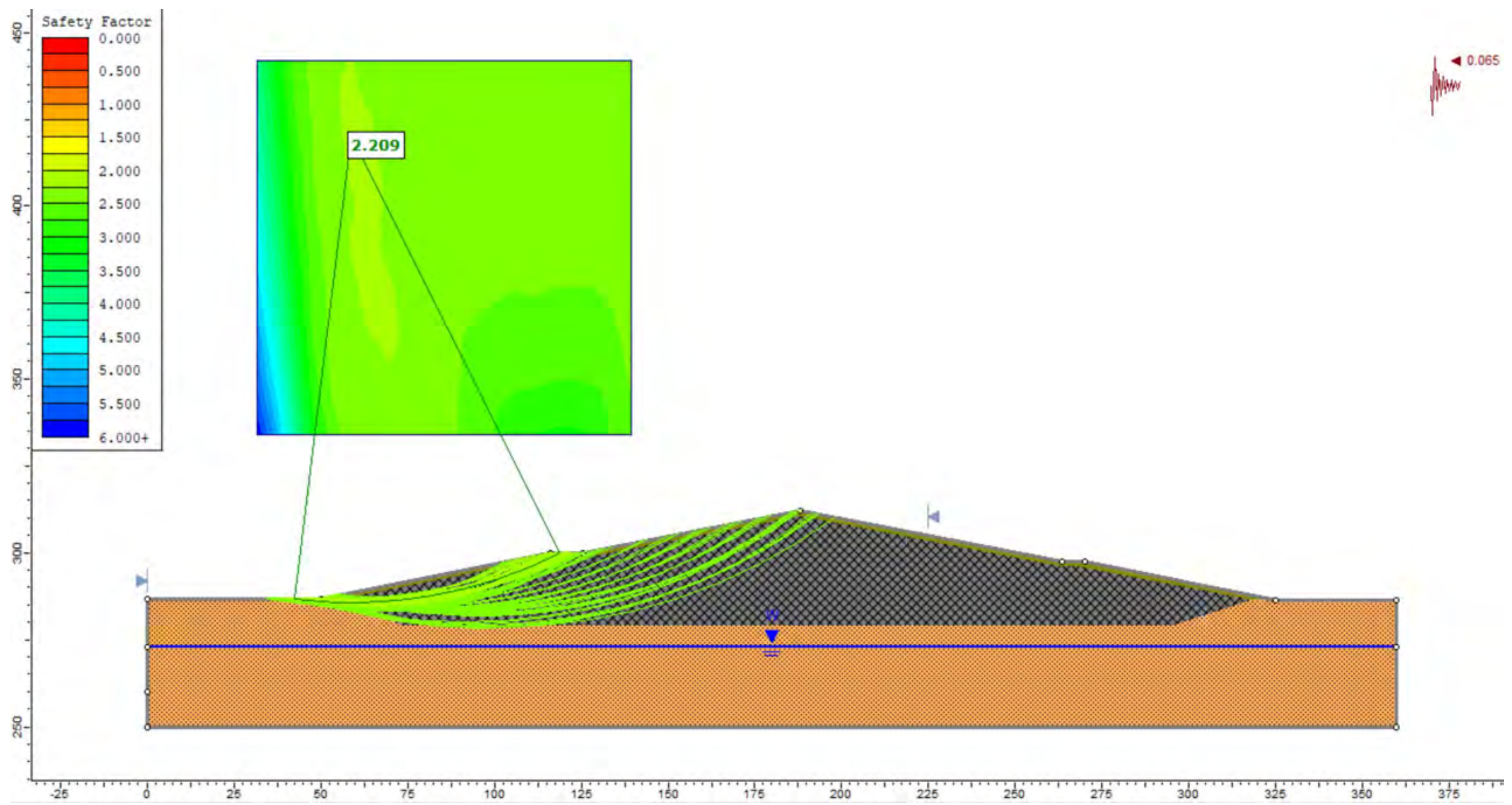
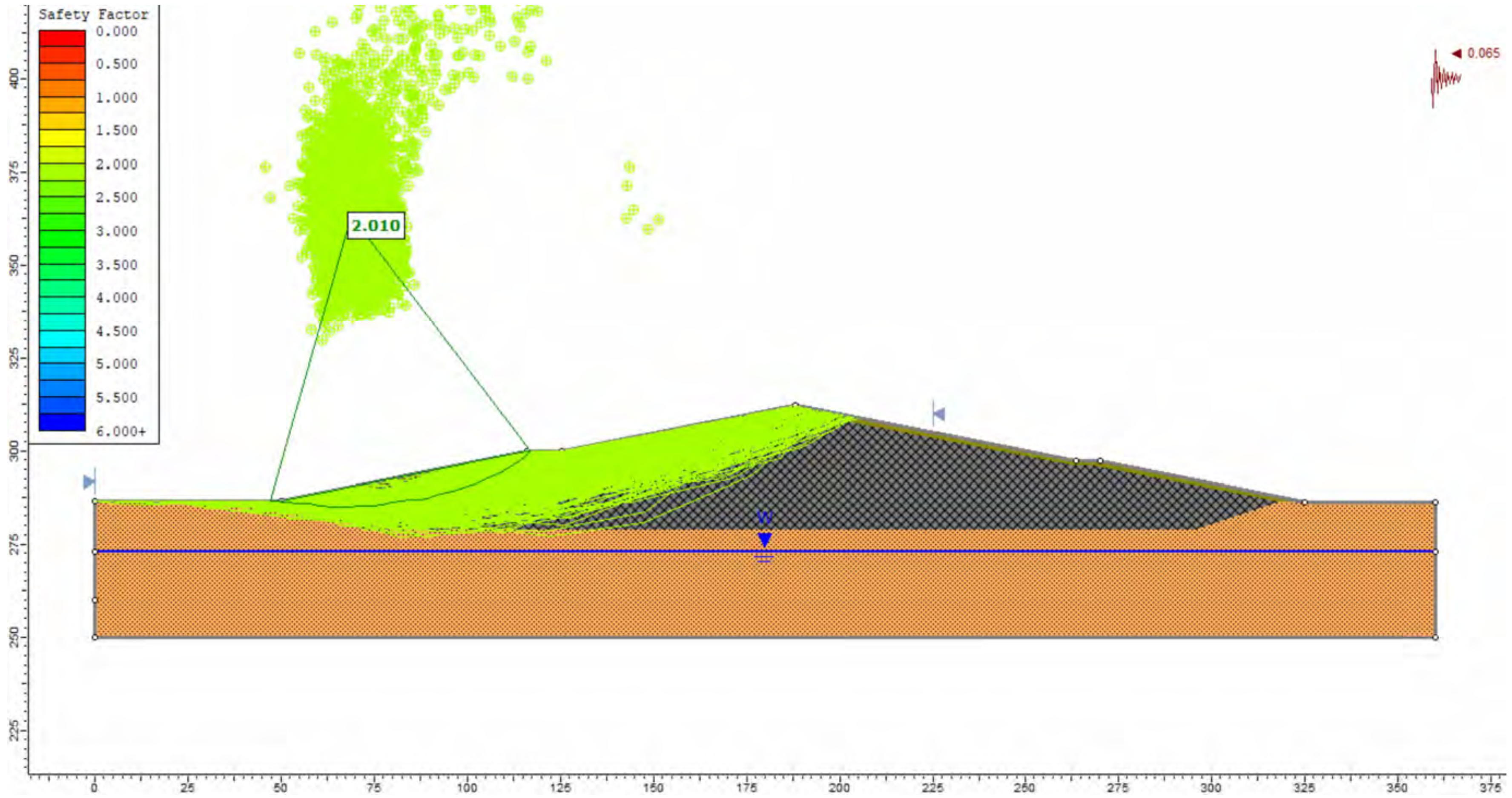
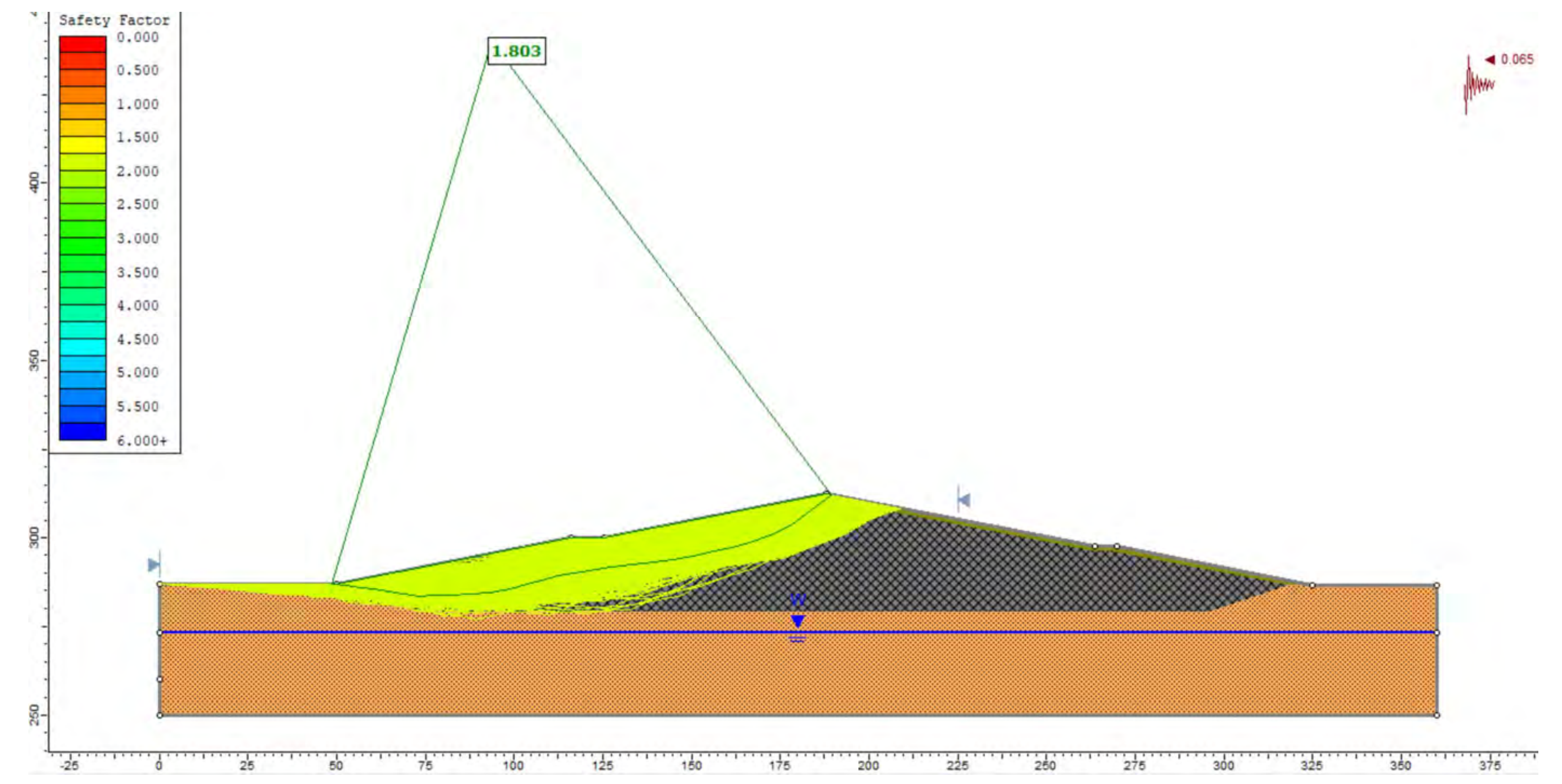
	Analyses Description:	Permanent Slope - Peak Shear Values with Ru=0.2		
	Section:	1 to 5 North to South Slope	**Factors of Safety Illustrated**	2.4-2.6



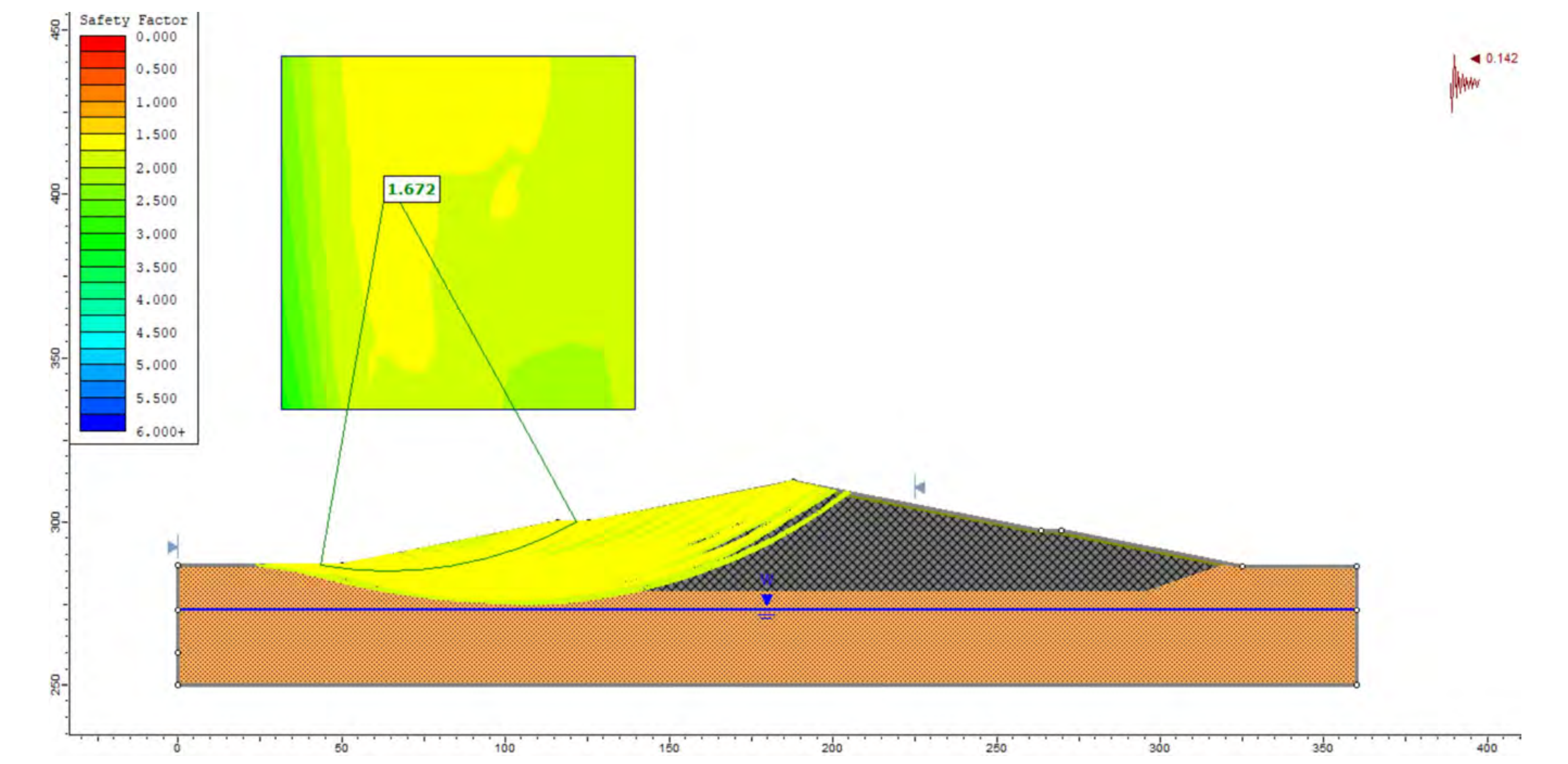
	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment		
	Analyses Description:	Permanent Slope - Peak Shear Values with Seismic Loading (OBE – 1:475 AEP)		
	Section:	1 to 5 North to South Slope	**Factors of Safety Illustrated**	2.2-2.4



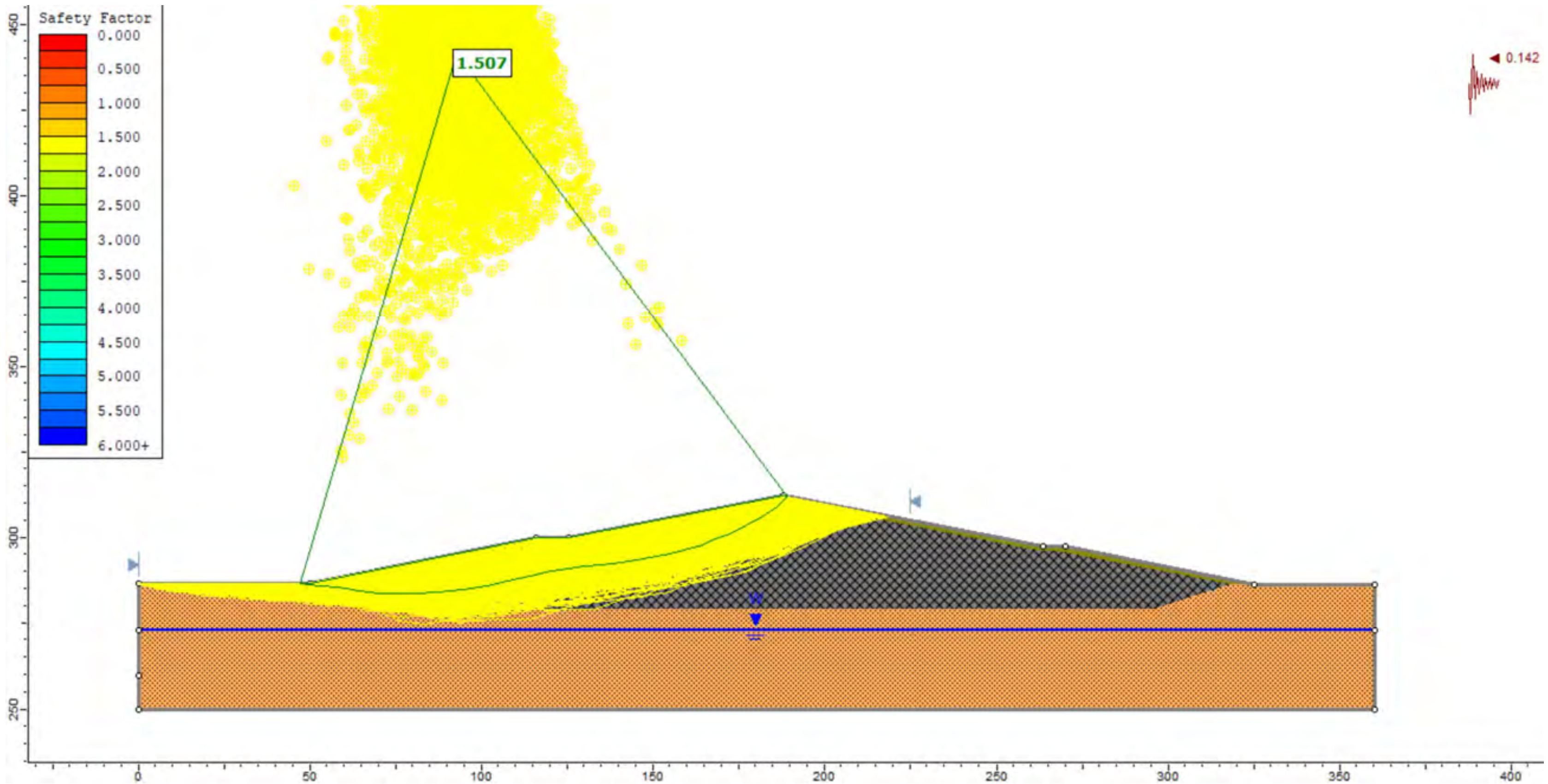
talis consultants	**Project:**	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment		
	Analyses Description:	Permanent Slope - Peak Shear Values with Seismic Loading (OBE – 1:475 AEP) with Ru = 0.1		
	Section:	1 to 5 North to South Slope	**Factors of Safety Illustrated**	2.0-2.2



talis consultants	**Project:**	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment		
	Analyses Description:	Permanent Slope - Peak Shear Values with Seismic Loading (OBE – 1:475 AEP) with Ru=0.2		
	Section:	1 to 5 North to South Slope	**Factors of Safety Illustrated**	1.8-2.0



	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment		
	Analyses Description:	Permanent Slope - Peak Shear Values with Seismic Loading (SEE/MCC – 1:1000 AEP)		
	Section:	1 to 5 North to South Slope	**Factors of Safety Illustrated**	1.6-1.8



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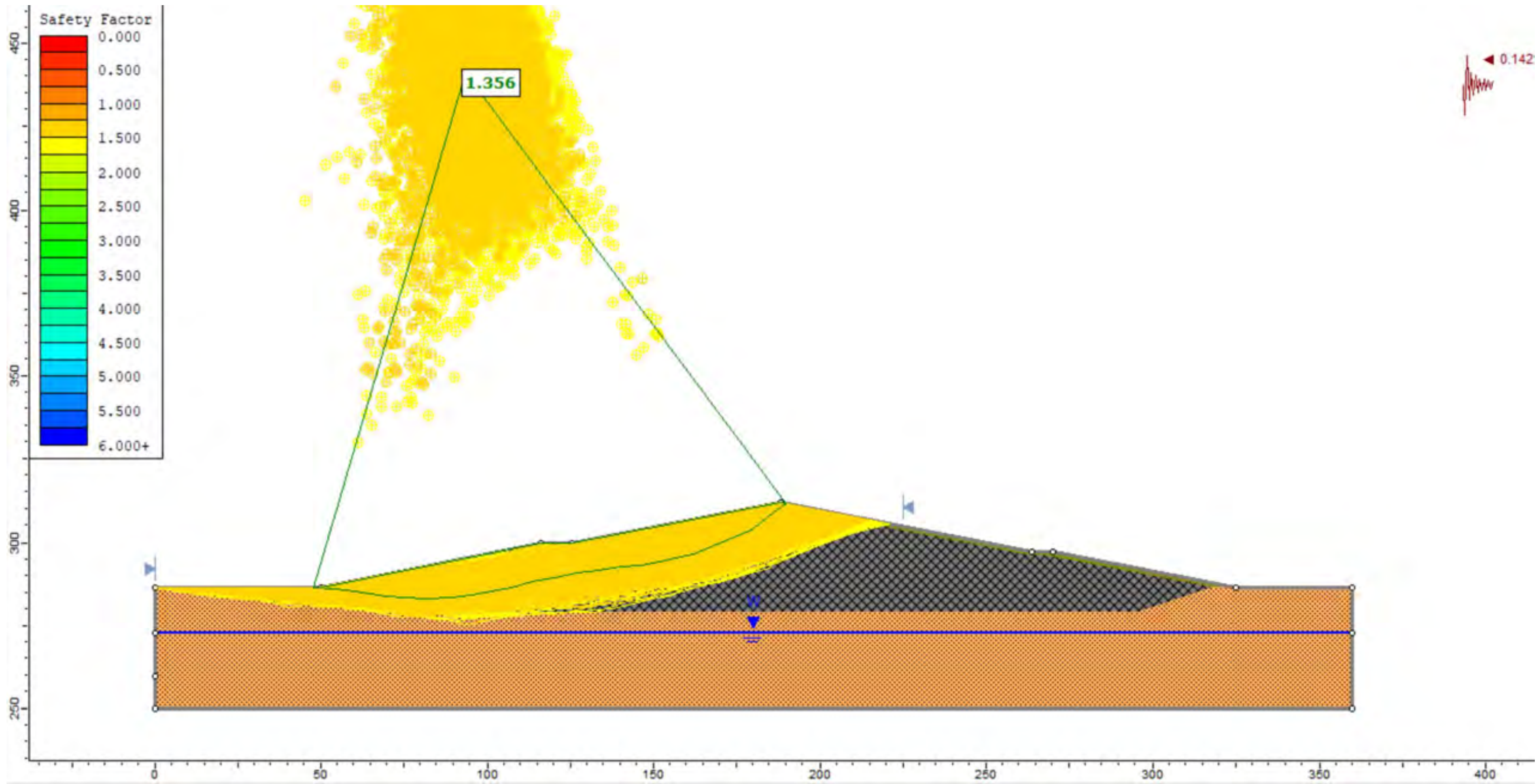
	Analyses Description:	Permanent Slope - Peak Shear Values with Seismic Loading (SEE/MCC – 1:1000 AEP) with Ru=0.1		
	Section:	1 to 5 North to South Slope	**Factors of Safety Illustrated**	1.5-1.7



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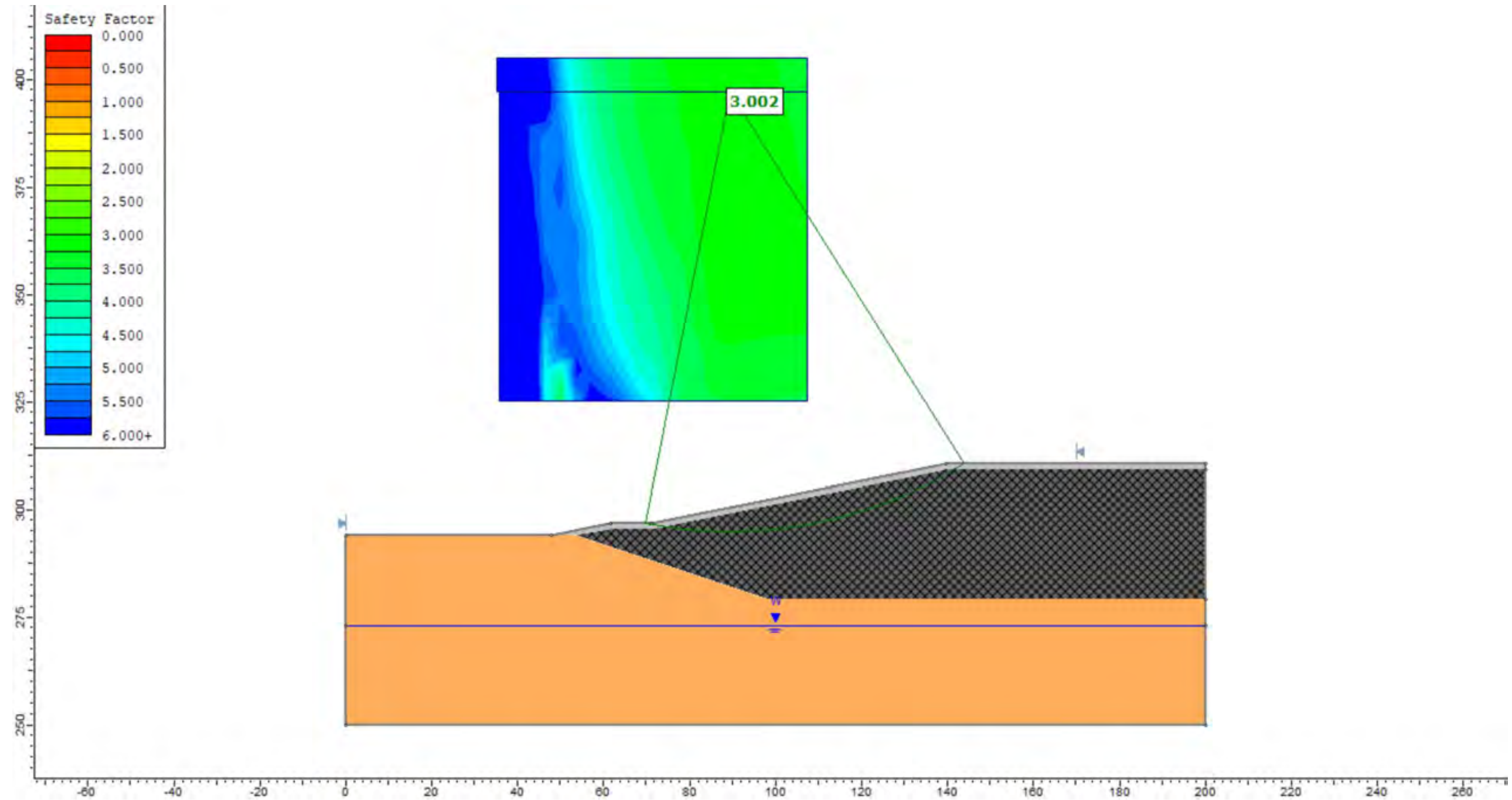
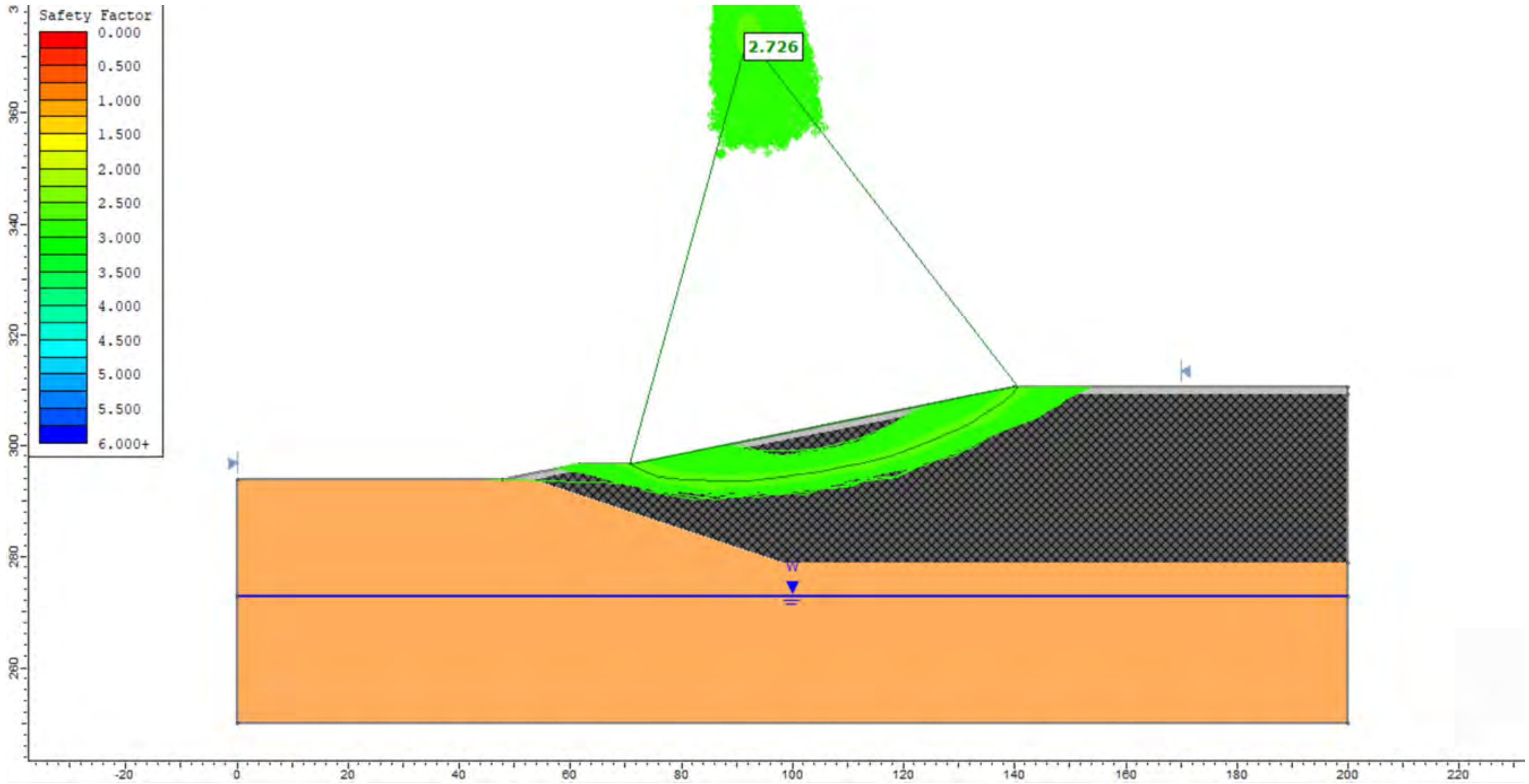
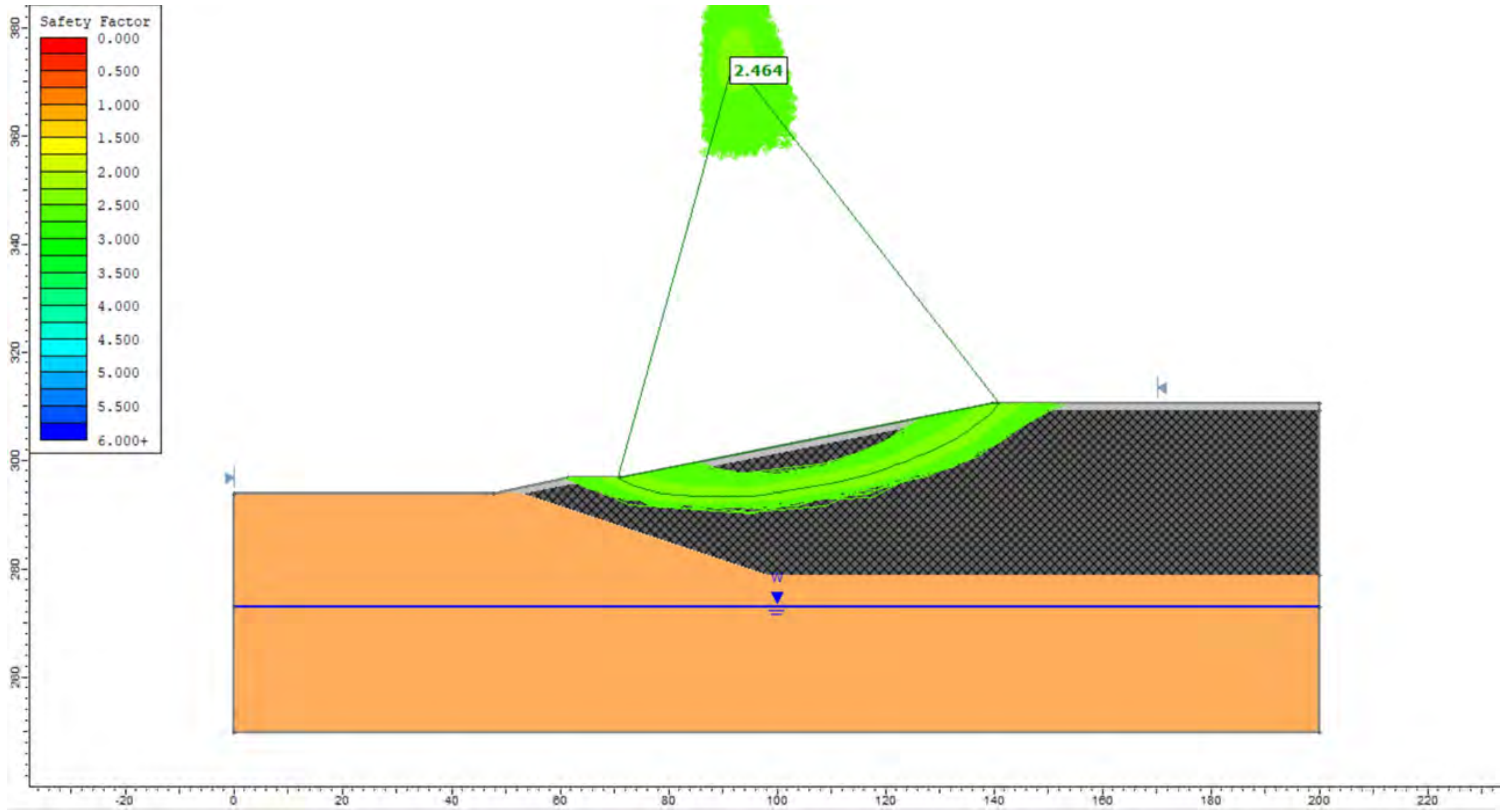
	Analyses Description:	Permanent Slope - Peak Shear Values with Seismic Loading (SEE/MCC – 1:1000 AEP) with Ru=0.2		
	Section:	1 to 5 North to South Slope	**Factors of Safety Illustrated**	1.3-1.55



	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment
	Analyses Description:	Permanent Slope - Peak Shear Values
	Section:	1 to 5 North West to South East Slope



	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment		
	Analyses Description:	Permanent Slope - Peak Shear Values with Ru = 0.1		
	Section:	1 to 5 North West to South East Slope	**Factors of Safety Illustrated**	2.7-2.9



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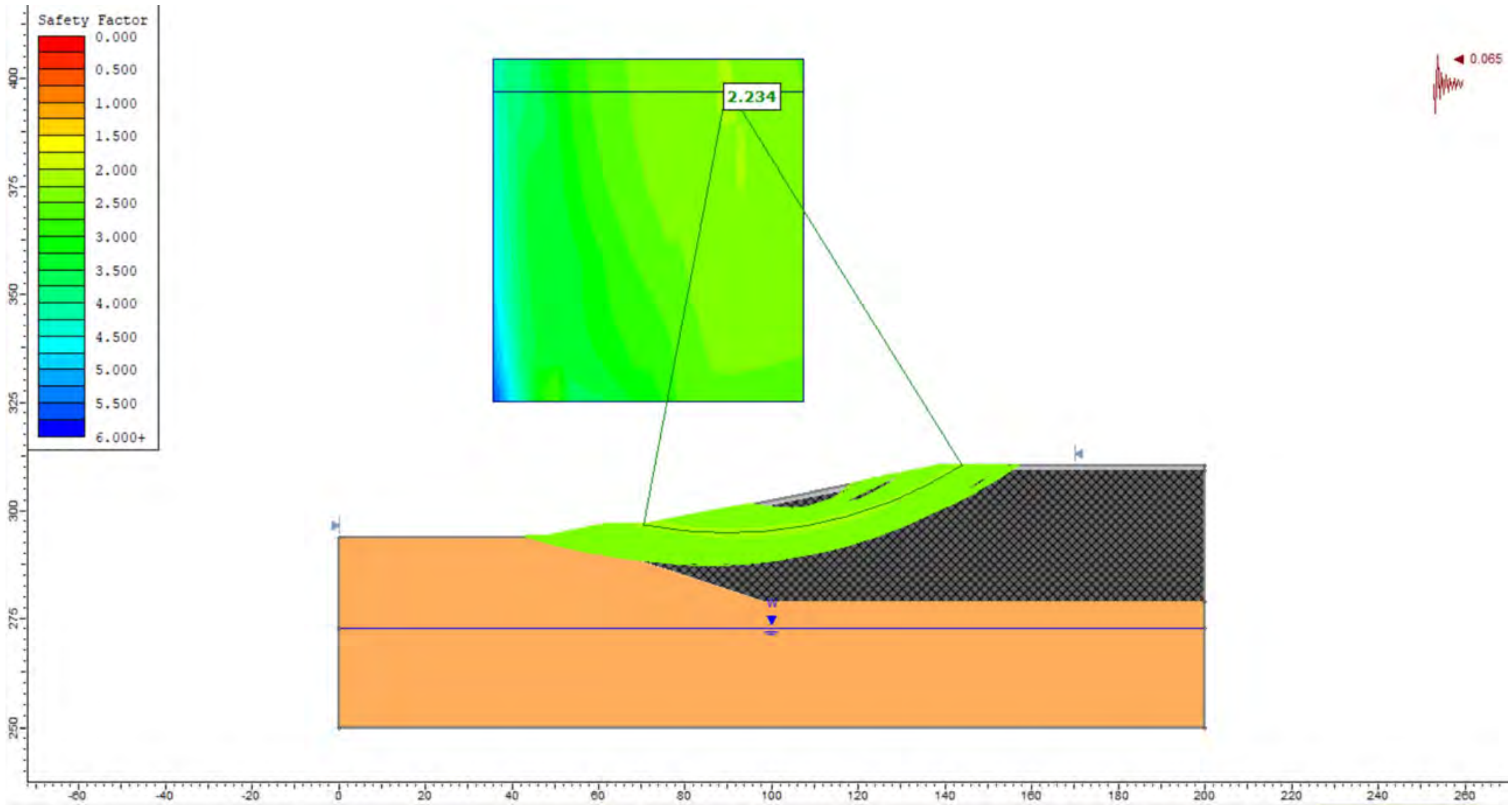
	Analyses Description:	Permanent Slope - Peak Shear Values with Ru = 0.2		
	Section:	1 to 5 North West to South East Slope	**Factors of Safety Illustrated**	2.4-2.6



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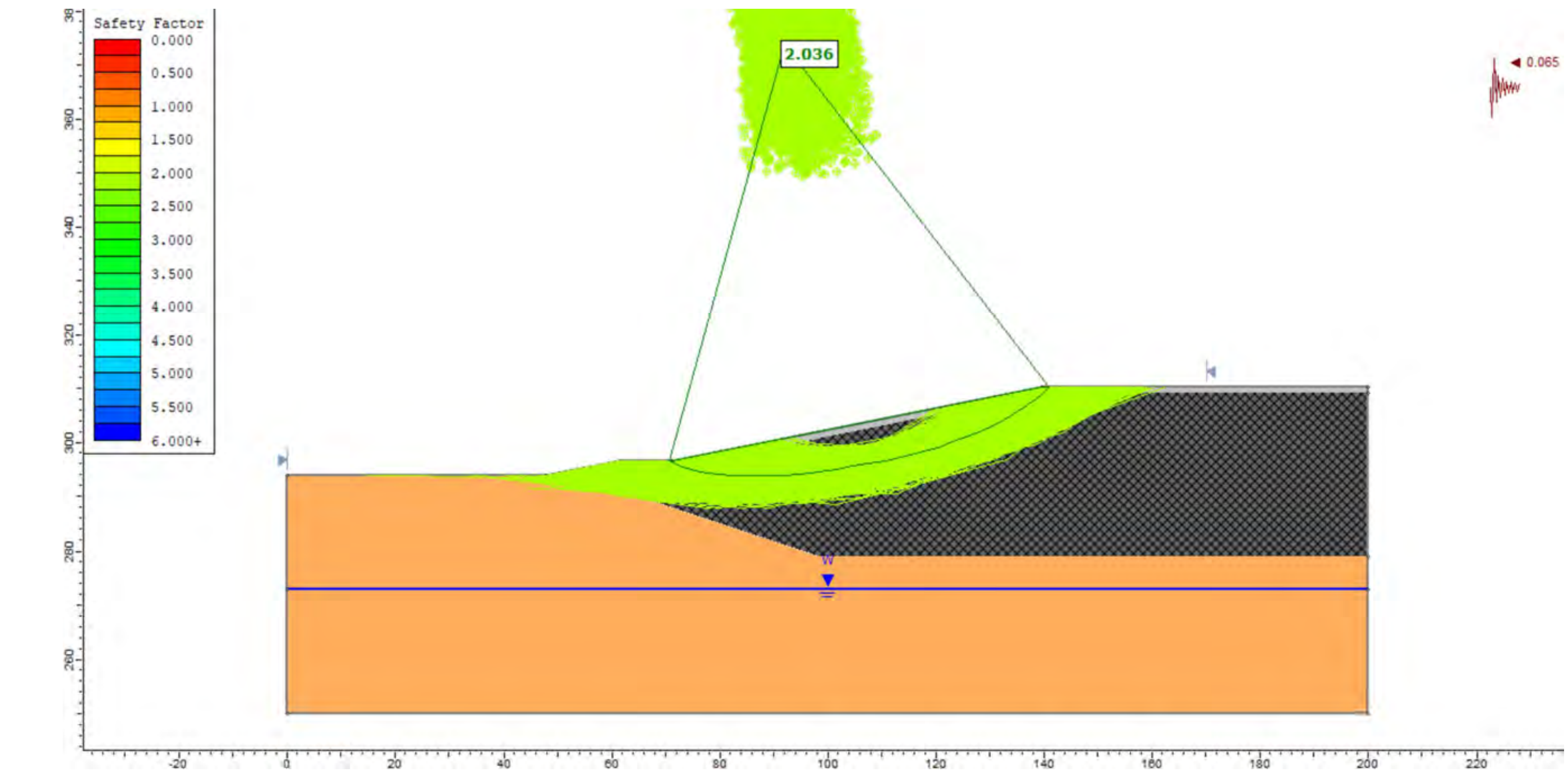
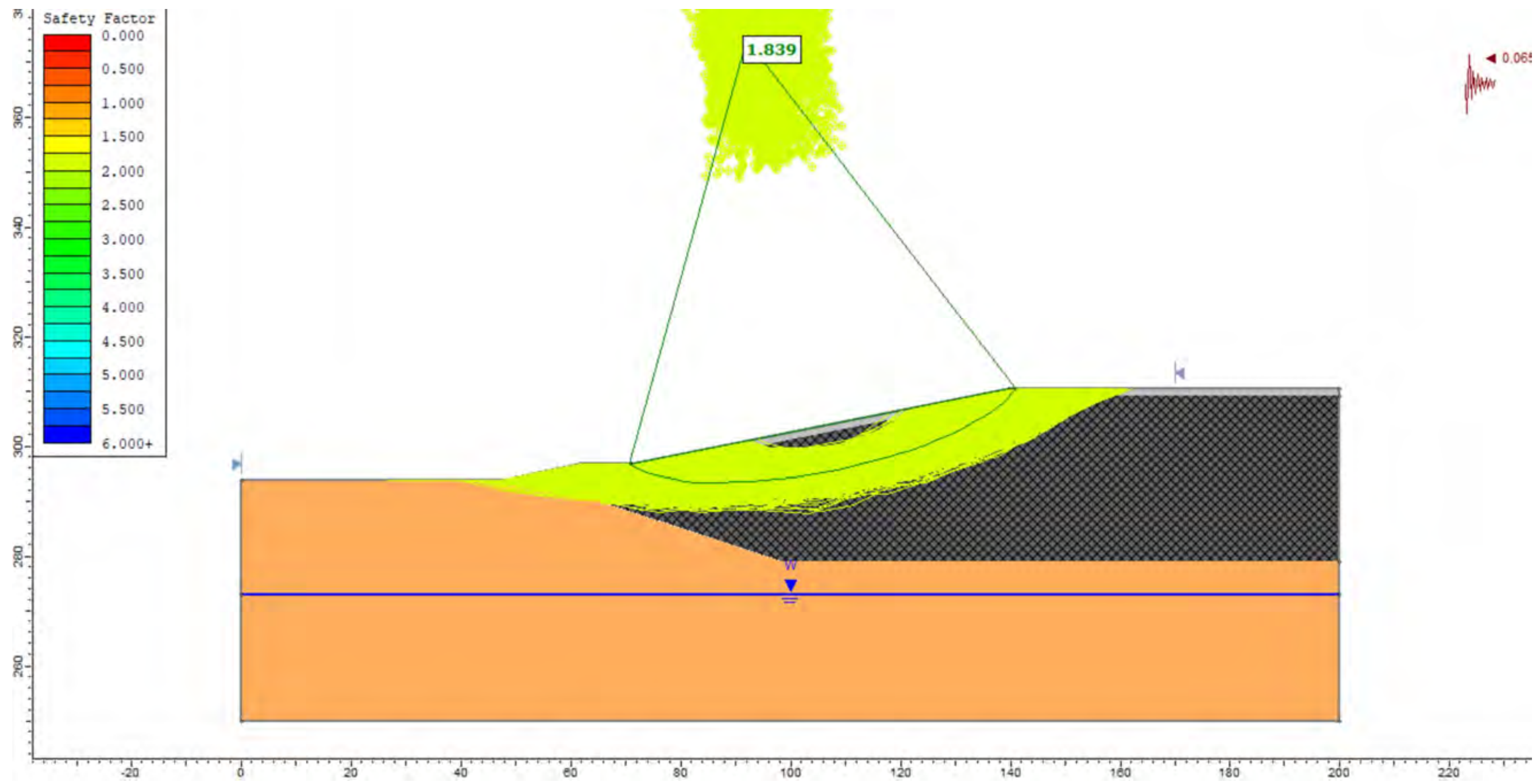
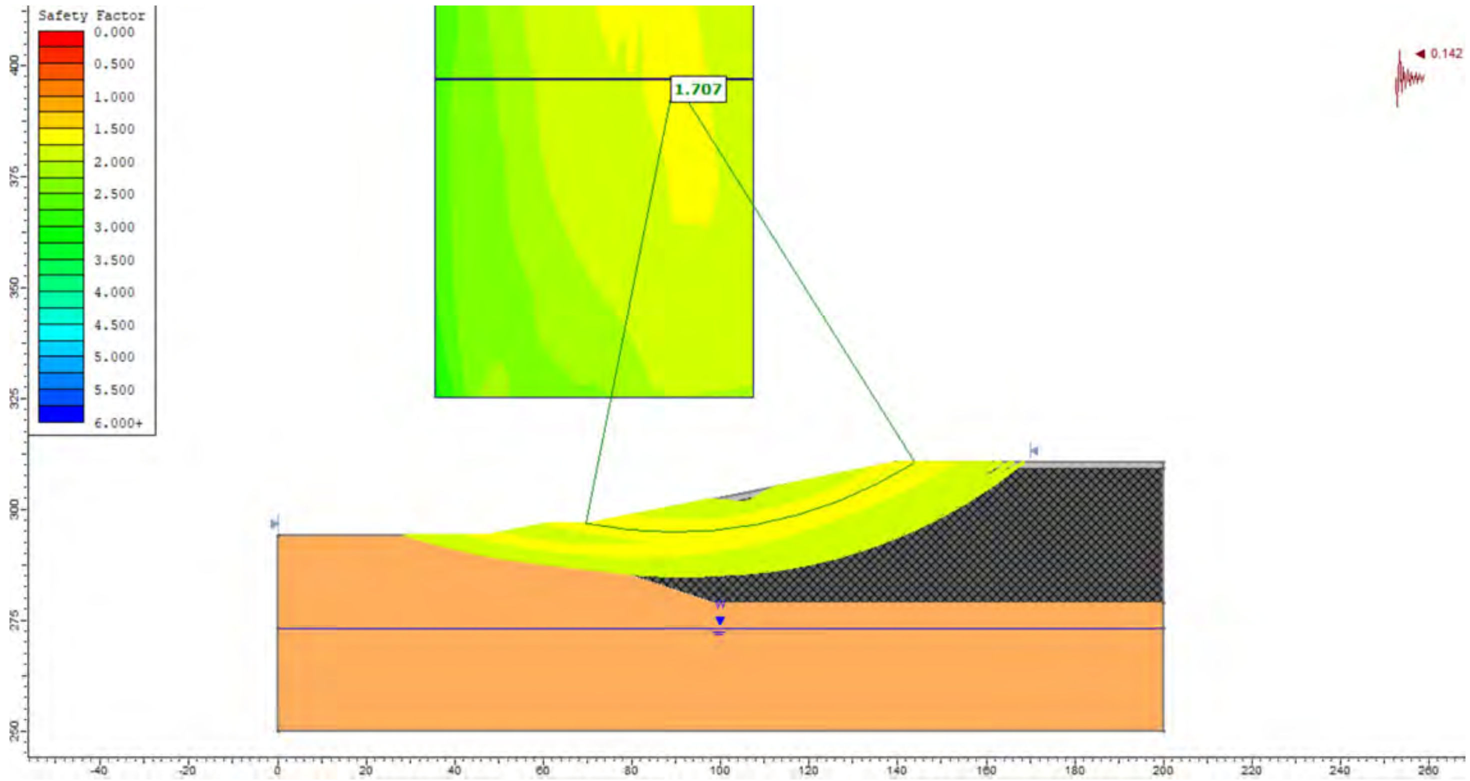
	Analyses Description:	Permanent Slope - Peak Shear Values with Seismic Loading (SEE – 1:475 AEP)		
	Section:	1 to 5 North West to South East Slope	**Factors of Safety Illustrated**	2.2-2.4



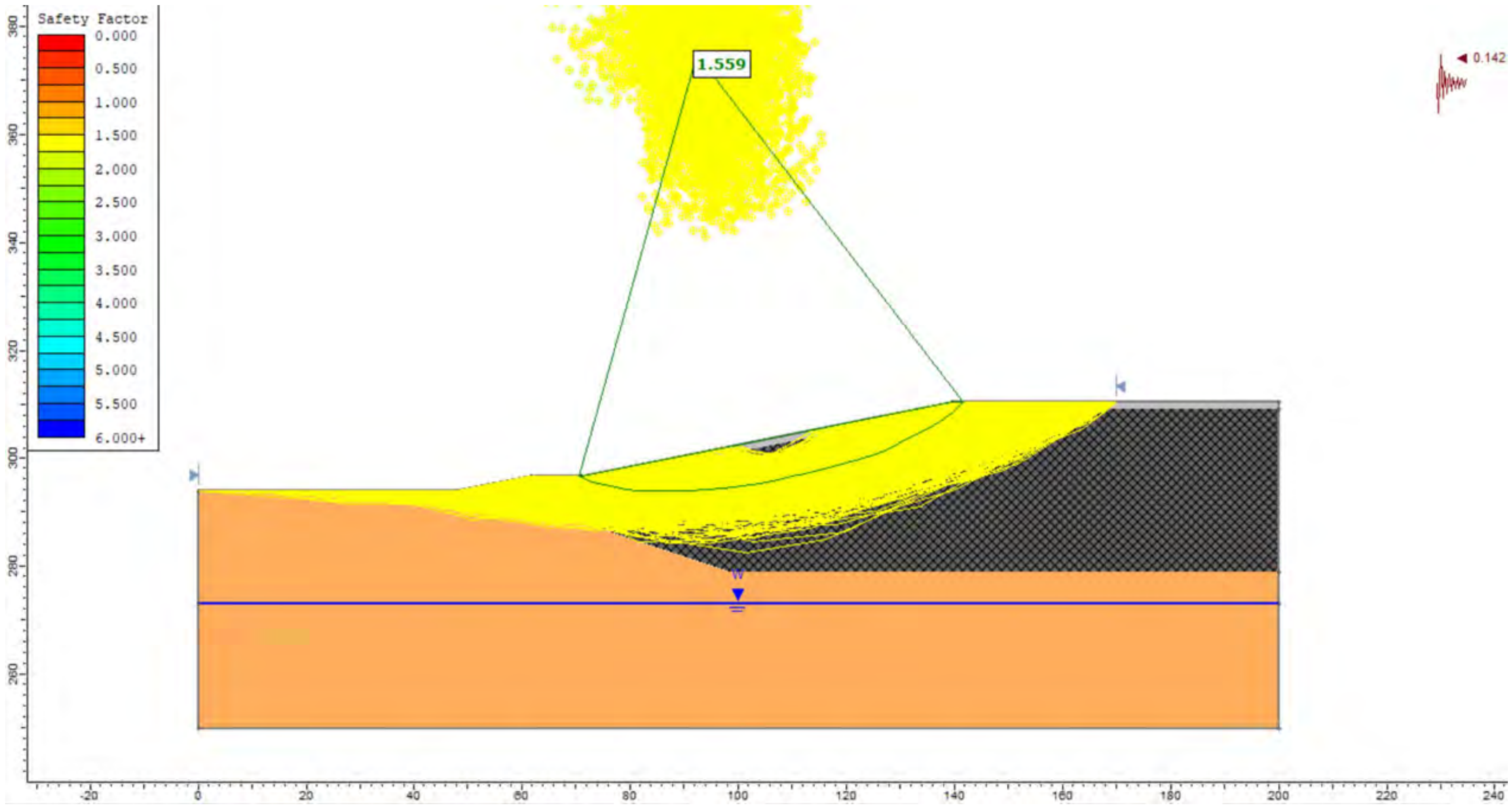
Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment			
Analyses Description:	Permanent Slope - Peak Shear Values with Seismic Loading (SEE – 1:475 AEP) with Ru = 0.1			
Section:	1 to 5 North West to South East Slope	**Factors of Safety Illustrated**	2.0-2.2	



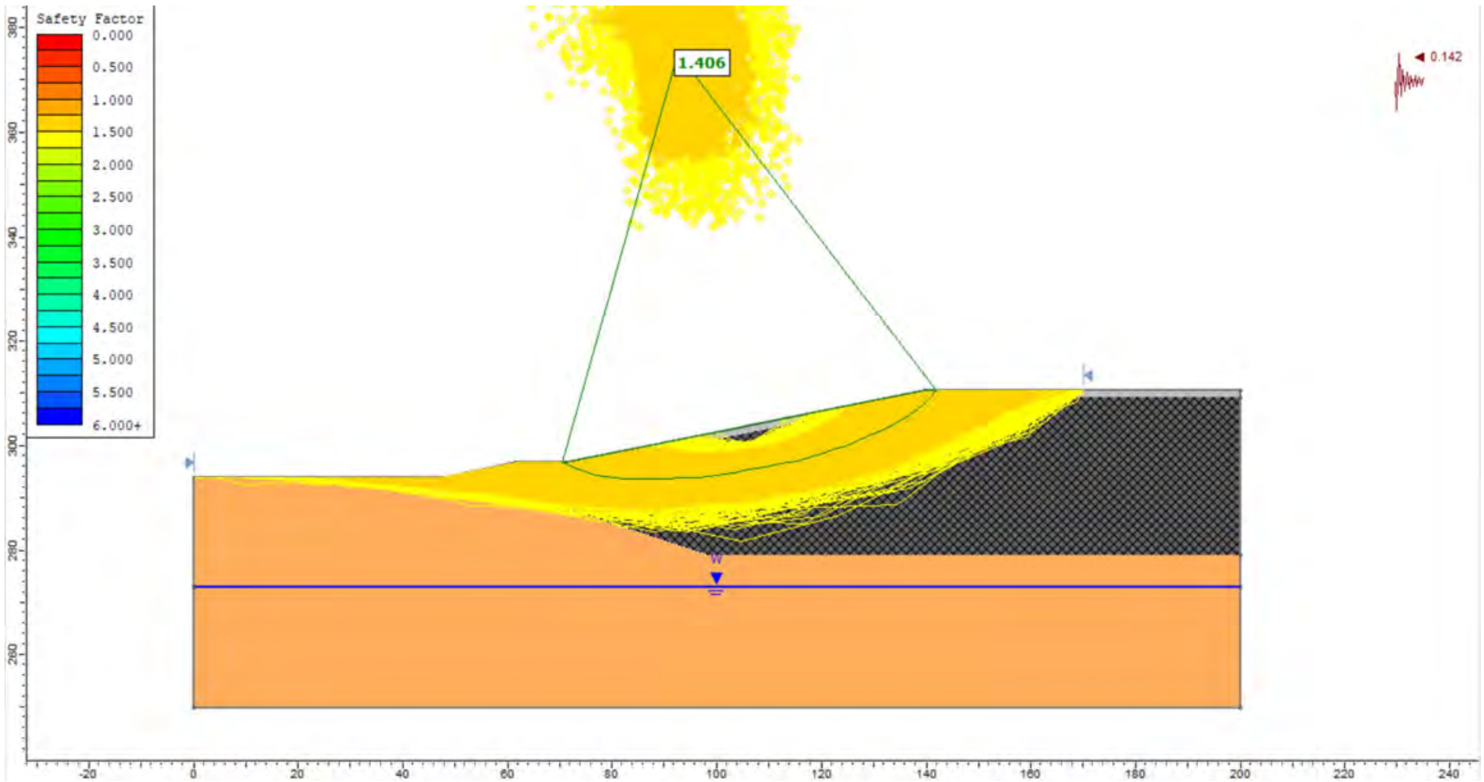
	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment		
	Analyses Description:	Permanent Slope - Peak Shear Values with Seismic Loading (SEE – 1:475 AEP) Ru = 0.2		
	Section:	1 to 5 North West to South East Slope	**Factors of Safety Illustrated**	1.8-2.0



talis consultants	**Project:**	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment		
	Analyses Description:	Permanent Slope - Peak Shear Values with Seismic Loading (SEE/MCC – 1:1000 AEP)		
	Section:	1 to 5 North West to South East Slope	**Factors of Safety Illustrated**	1.7-1.9



Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment			
Analyses Description:	Permanent Slope - Peak Shear Values with Seismic Loading (SEE/MCC – 1:1000 AEP) Ru = 0.1			
Section:	1 to 5 North West to South East Slope	**Factors of Safety Illustrated**	1.5-1.75	



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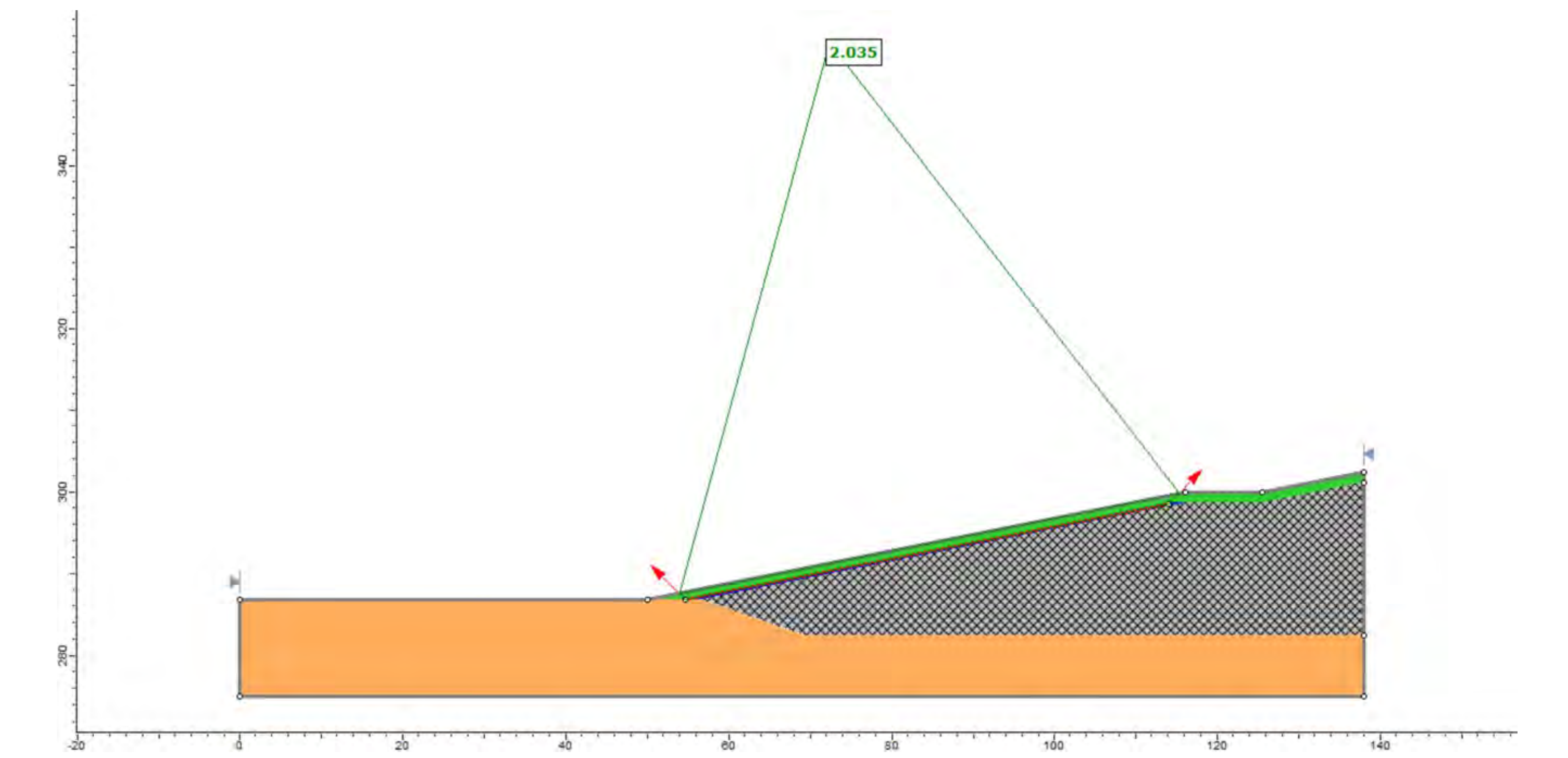
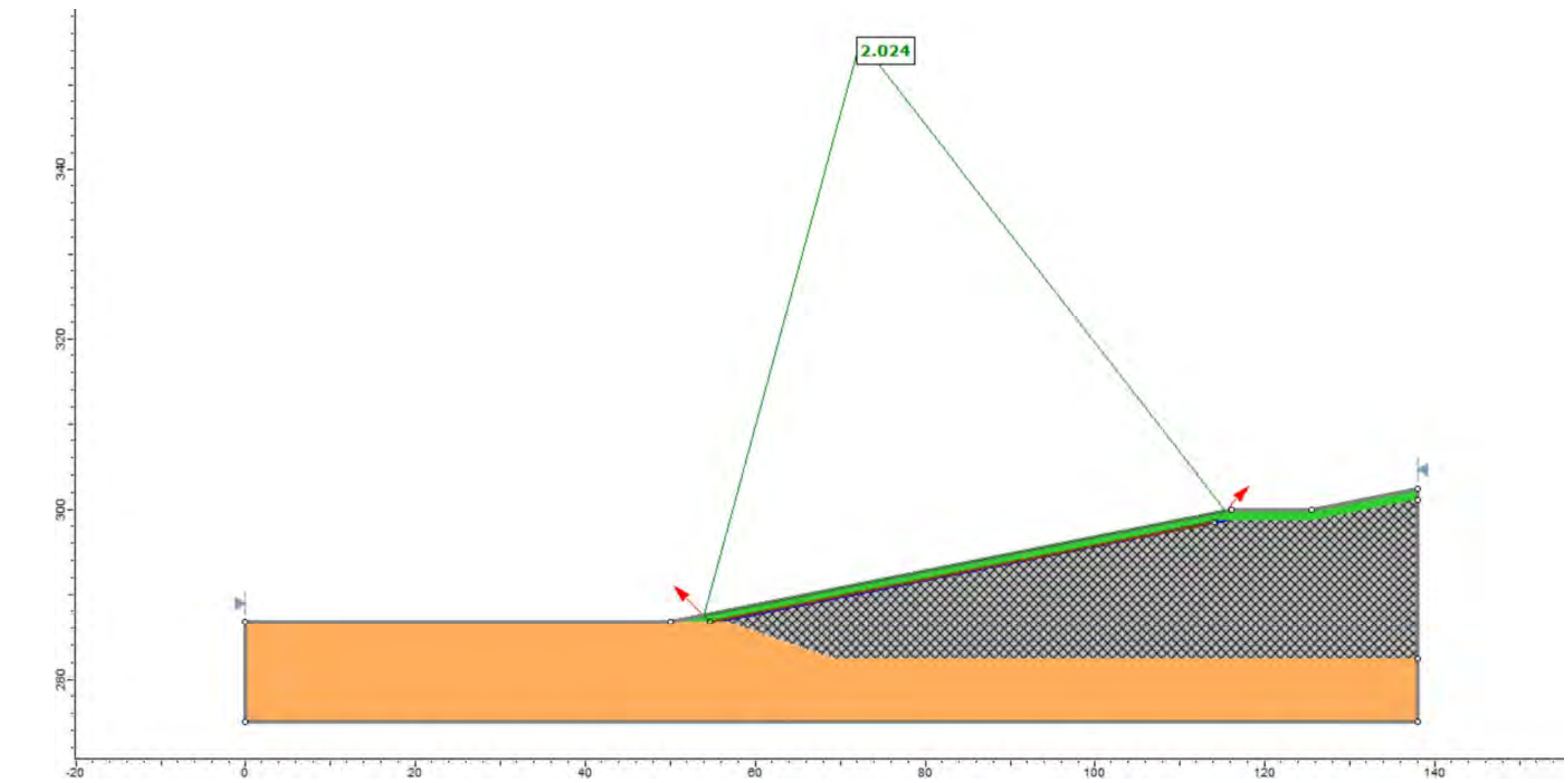
	Section:	1 to 5 North West to South East Slope	**Factors of Safety Illustrated**	1.4-1.6



	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment
	Analyses Description:	Low Shear Geonet/LLDPE Interface with Peak Values
	Section:	1 to 5 Slope



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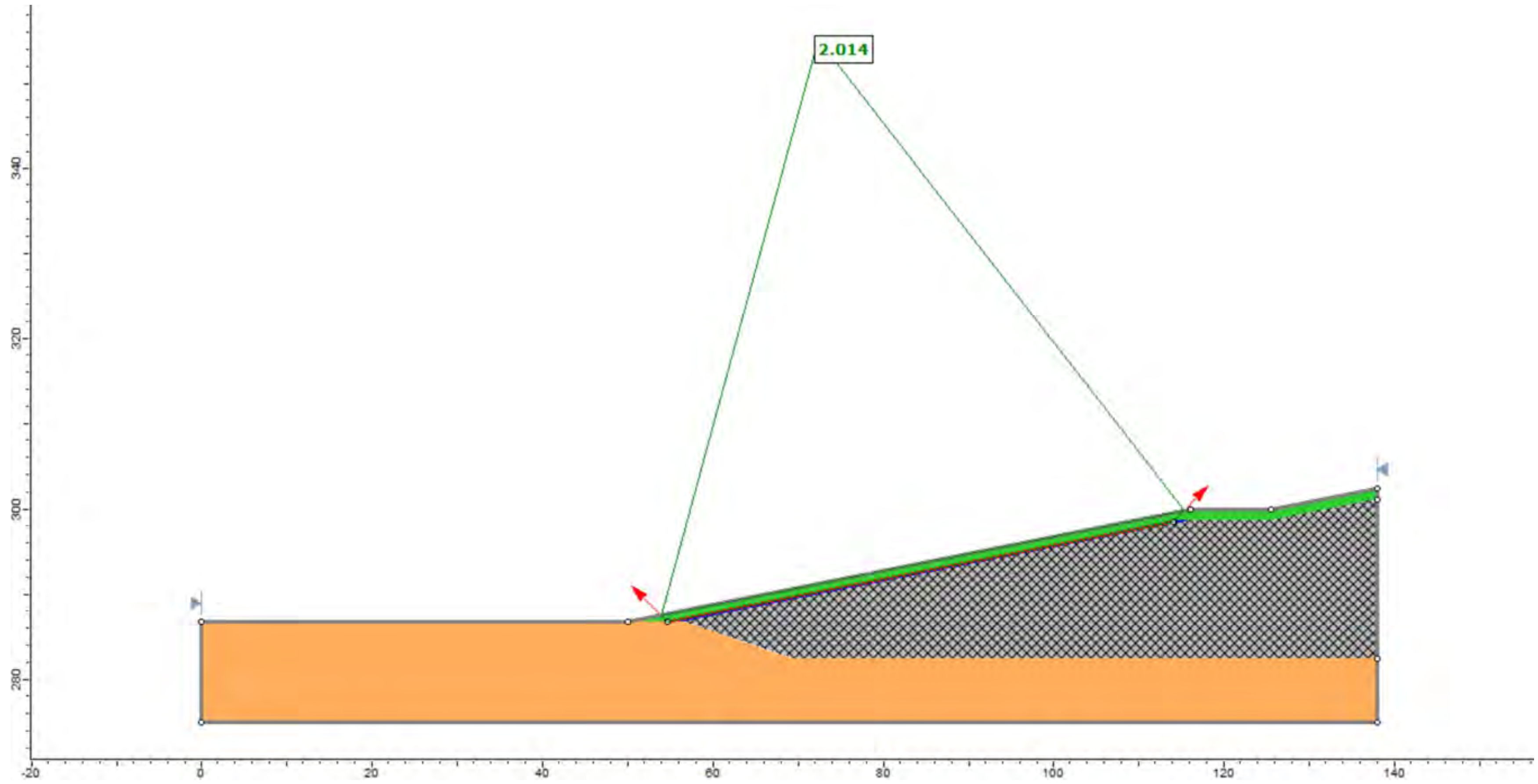
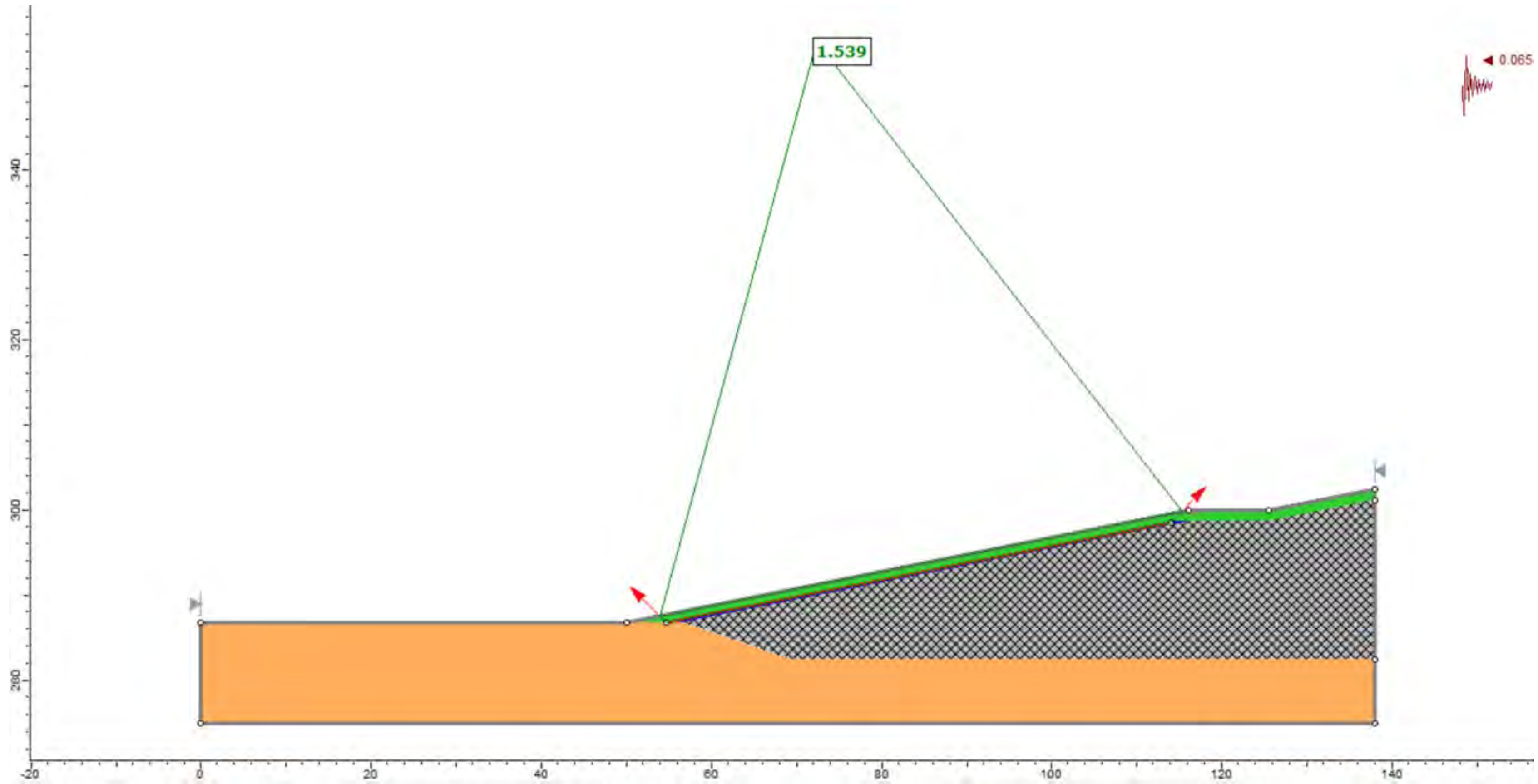
	Analyses Description:	Low Shear Geonet/LLDPE Interface with Peak Values with Ru=0.1
	Section:	1 to 5 Slope



	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment
	Analyses Description:	Low Shear Geonet/LLDPE Interface with Peak Values with Ru = 0.2
	Section:	1 to 5 Slope



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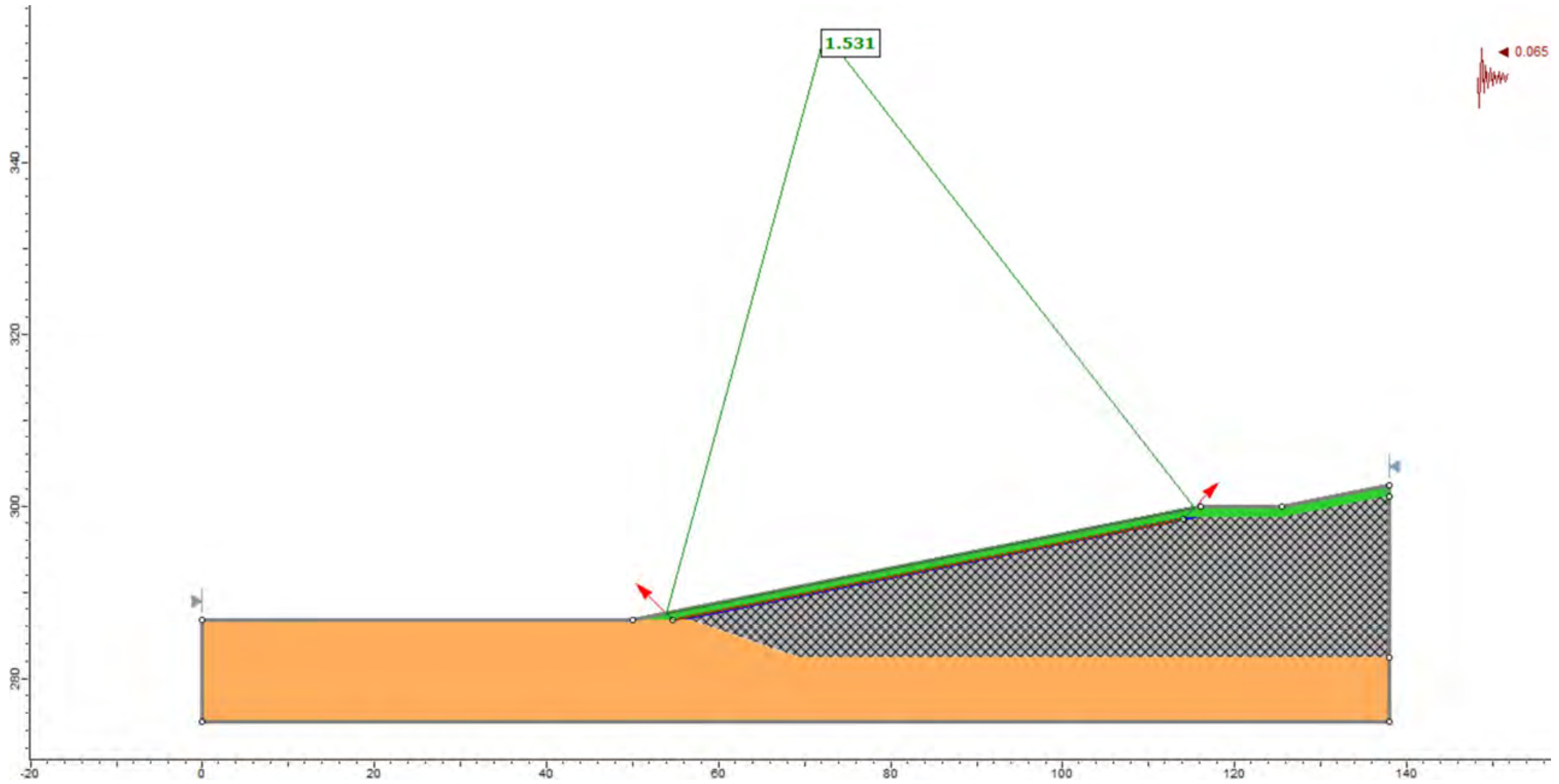
	Analyses Description:	Low Shear Geonet/LLDPE Interface with Peak Values with Seismic Loading (OBE – 1:475 AEP)
	Section:	1 to 5 Slope



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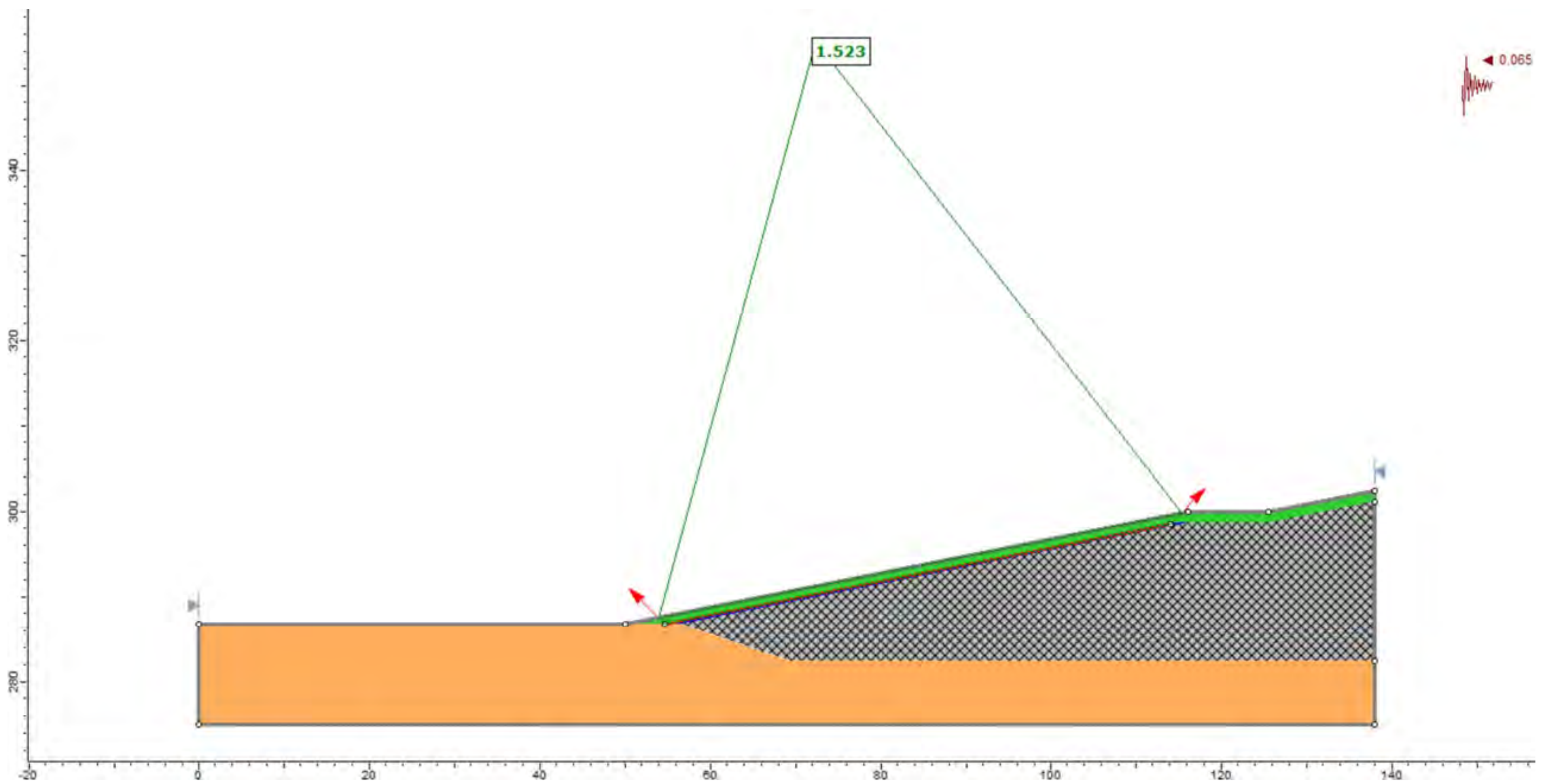
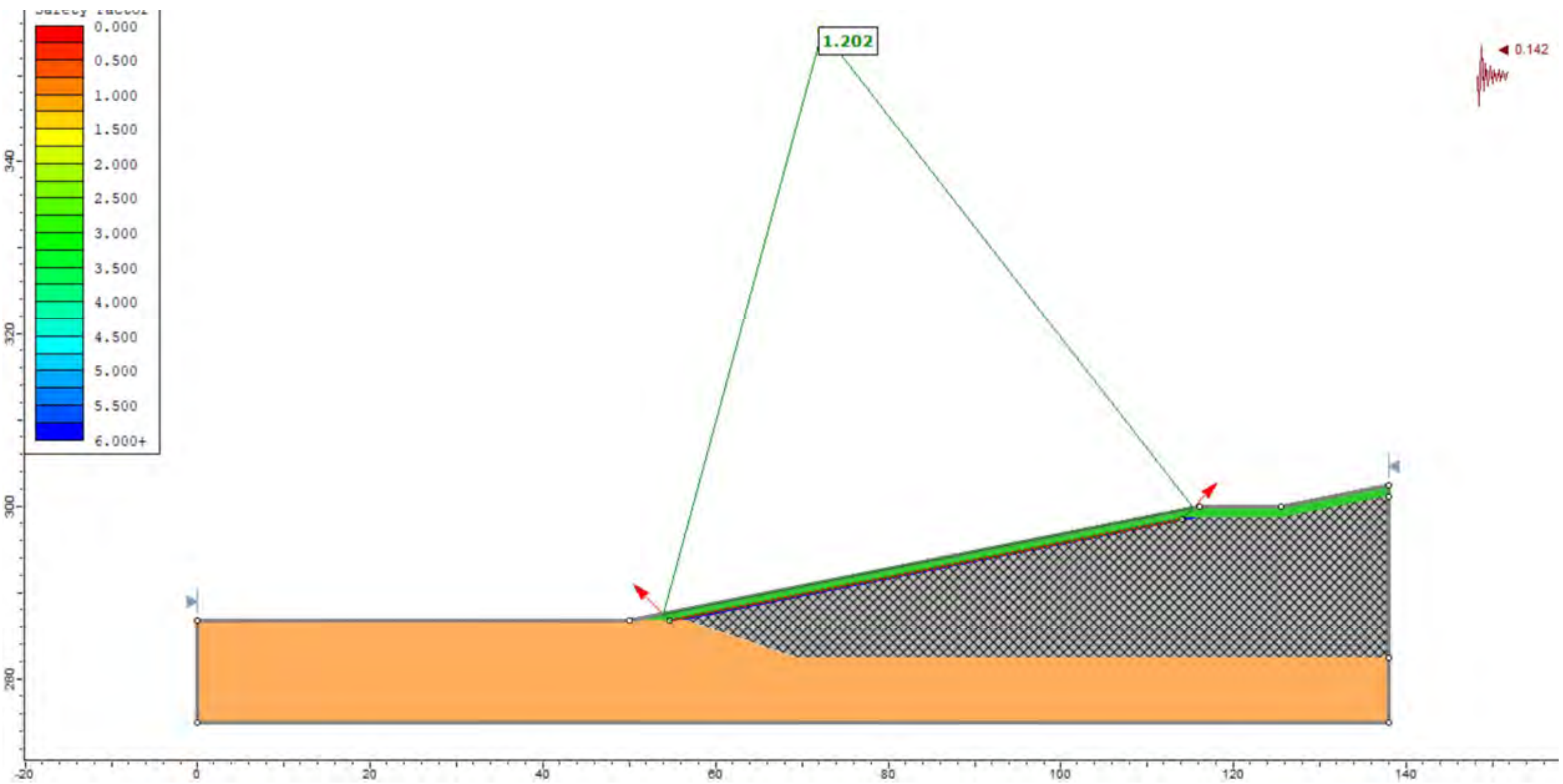
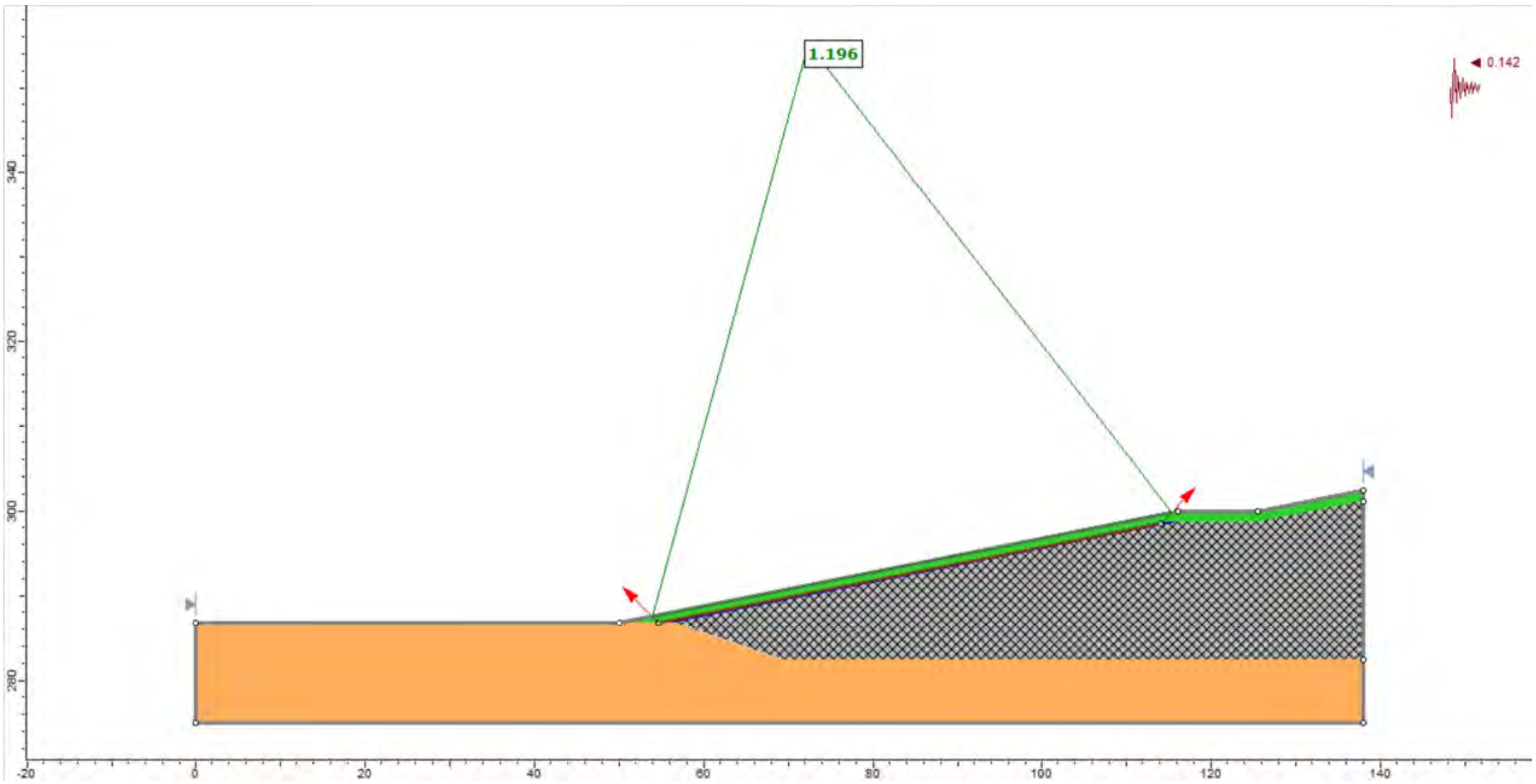
	Analyses Description:	Low Shear Geonet/LLDPE Interface with Peak Values with Seismic Loading (OBE – 1:475 AEP) and Ru = 0.1
	Section:	1 to 5 Slope



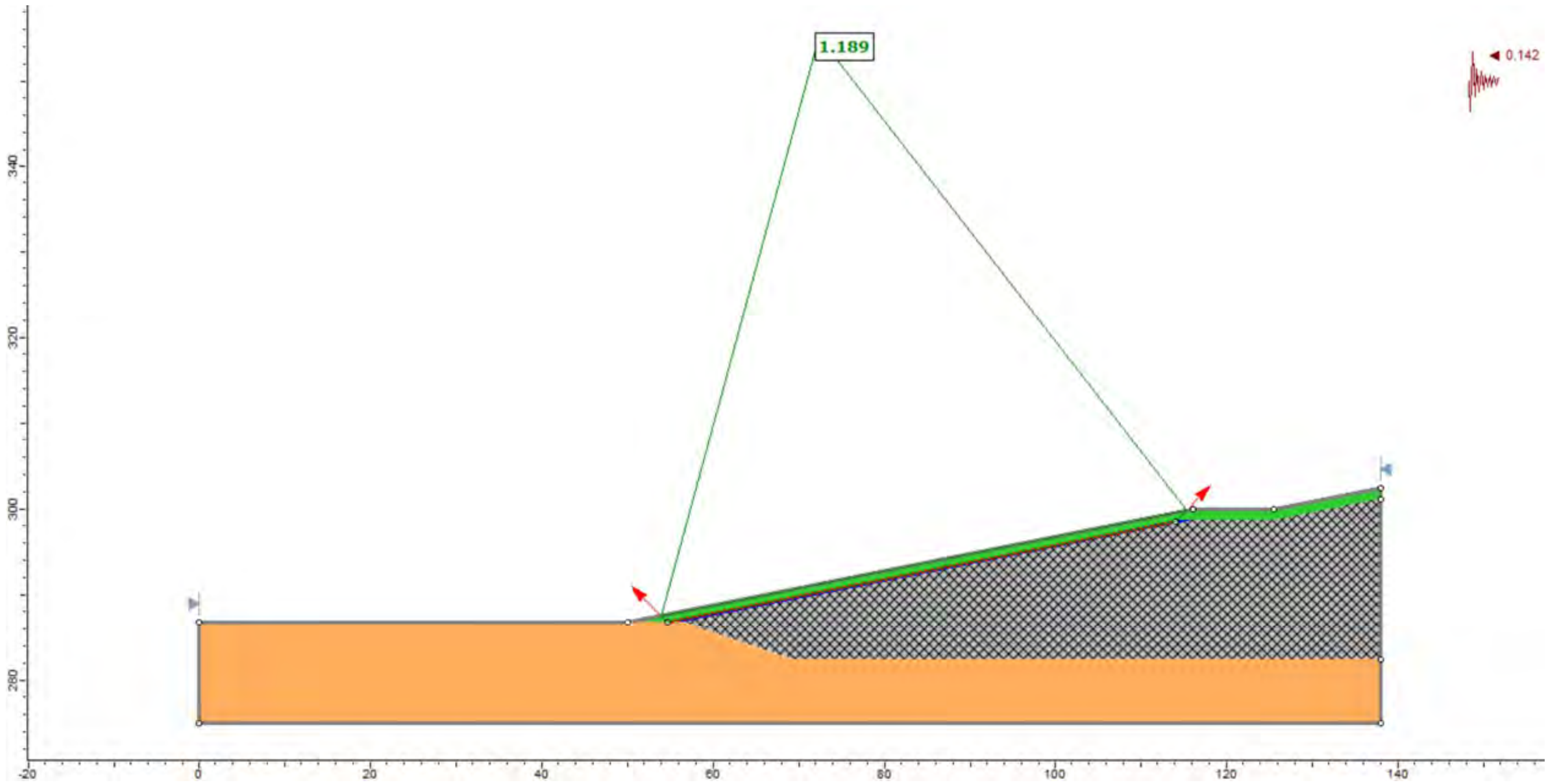
	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment
	Analyses Description:	Low Shear Geonet/LLDPE Interface with Peak Values with Seismic Loading (OBE – 1:475 AEP) and Ru=0.2
	Section:	1 to 5 Slope



	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment
	Analyses Description:	Low Shear Geonet/LLDPE Interface with Peak Values with Seismic Loading (SEE/MCC – 1:1000 AEP)
	Section:	1 to 5 Slope



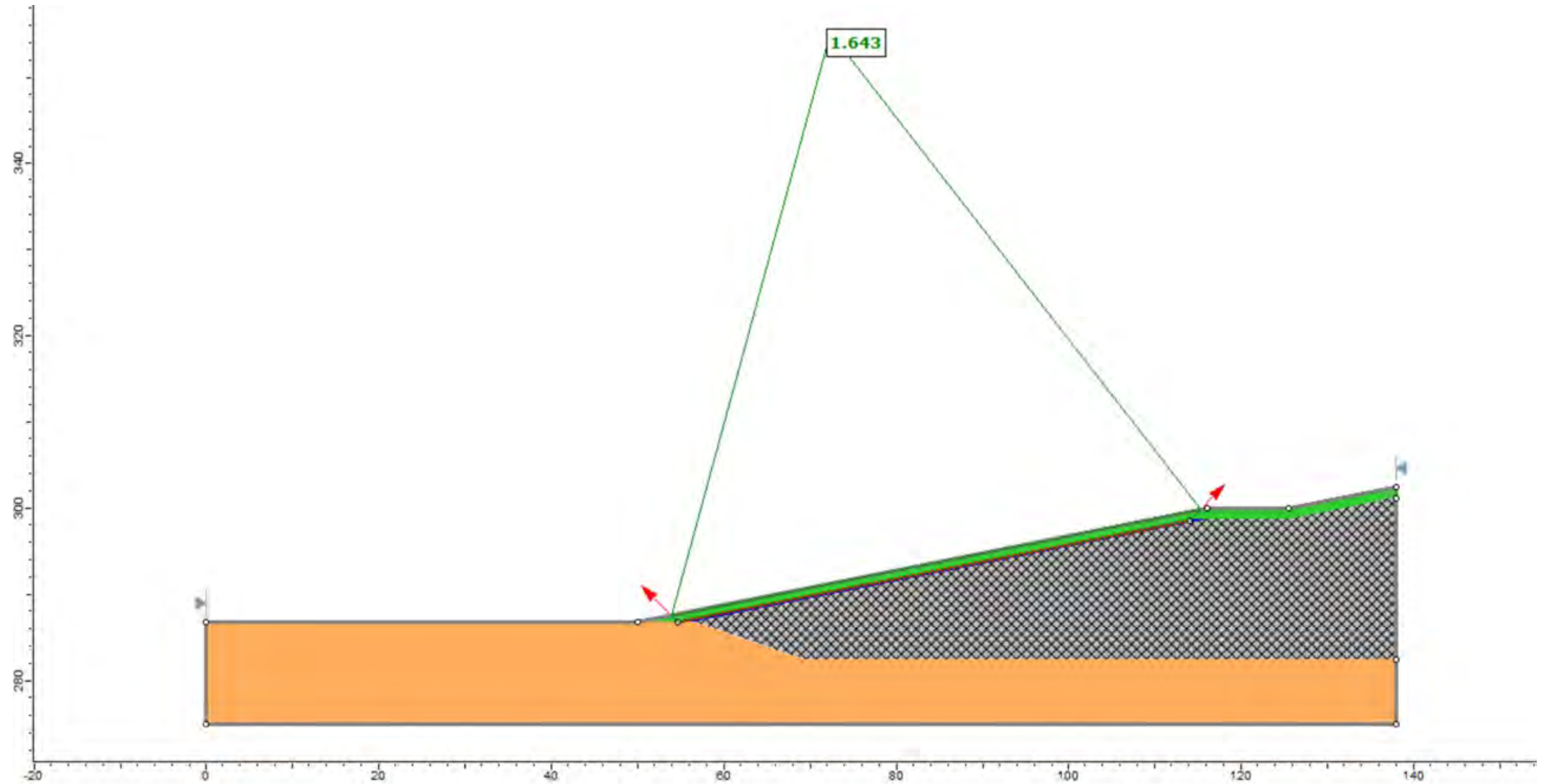
	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment
	Analyses Description:	Low Shear Geonet/LLDPE Interface with Peak Values with Seismic Loading (SEE/MCC – 1:1000 AEP) and Ru = 0.1
	Section:	1 to 5 Slope



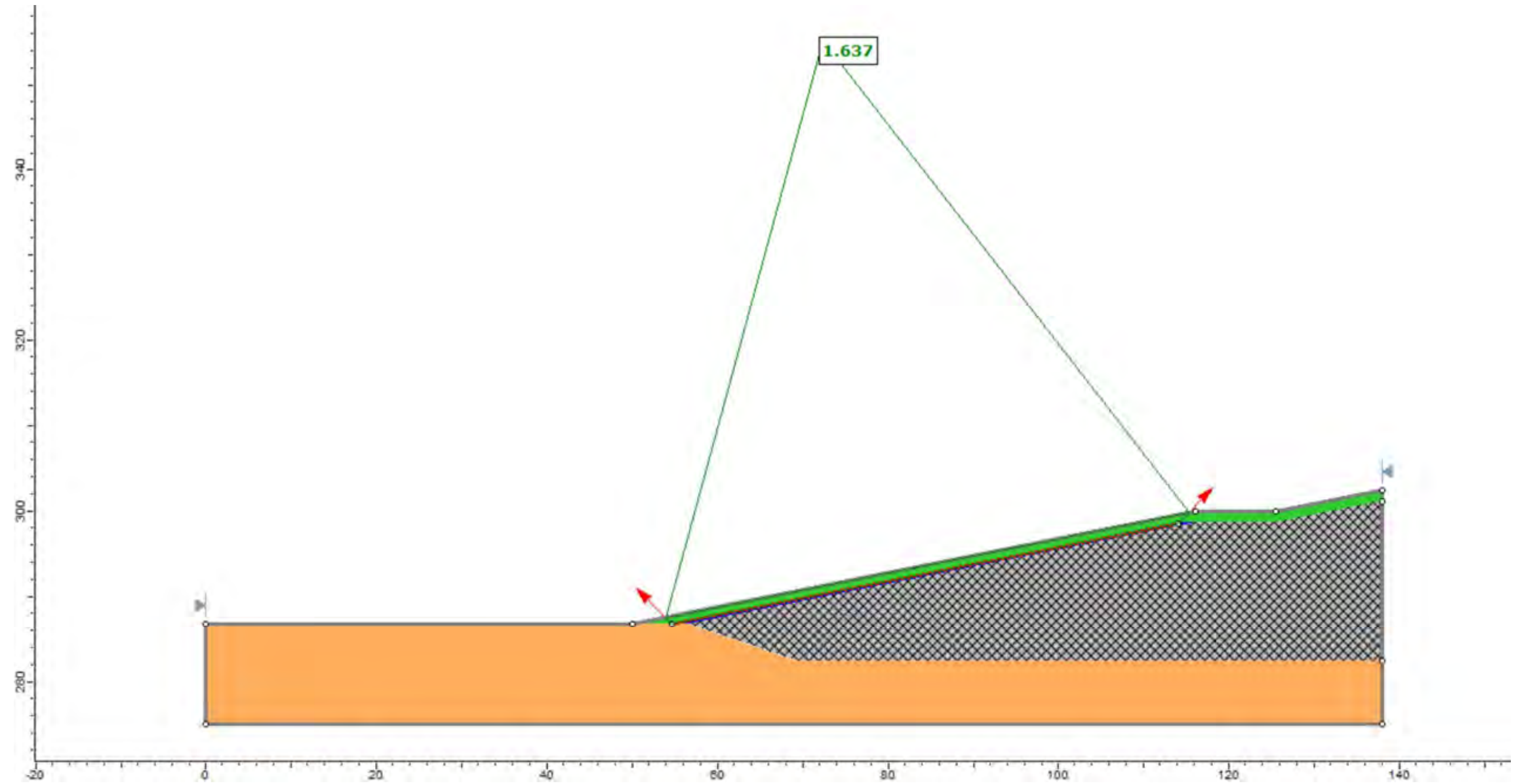
	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment
	Analyses Description:	Low Shear Geonet/LLDPE Interface with Peak Values with Seismic Loading (SEE/MCC – 1:1000 AEP) and Ru = 0.2
	Section:	1 to 5 Slope

APPENDIX F

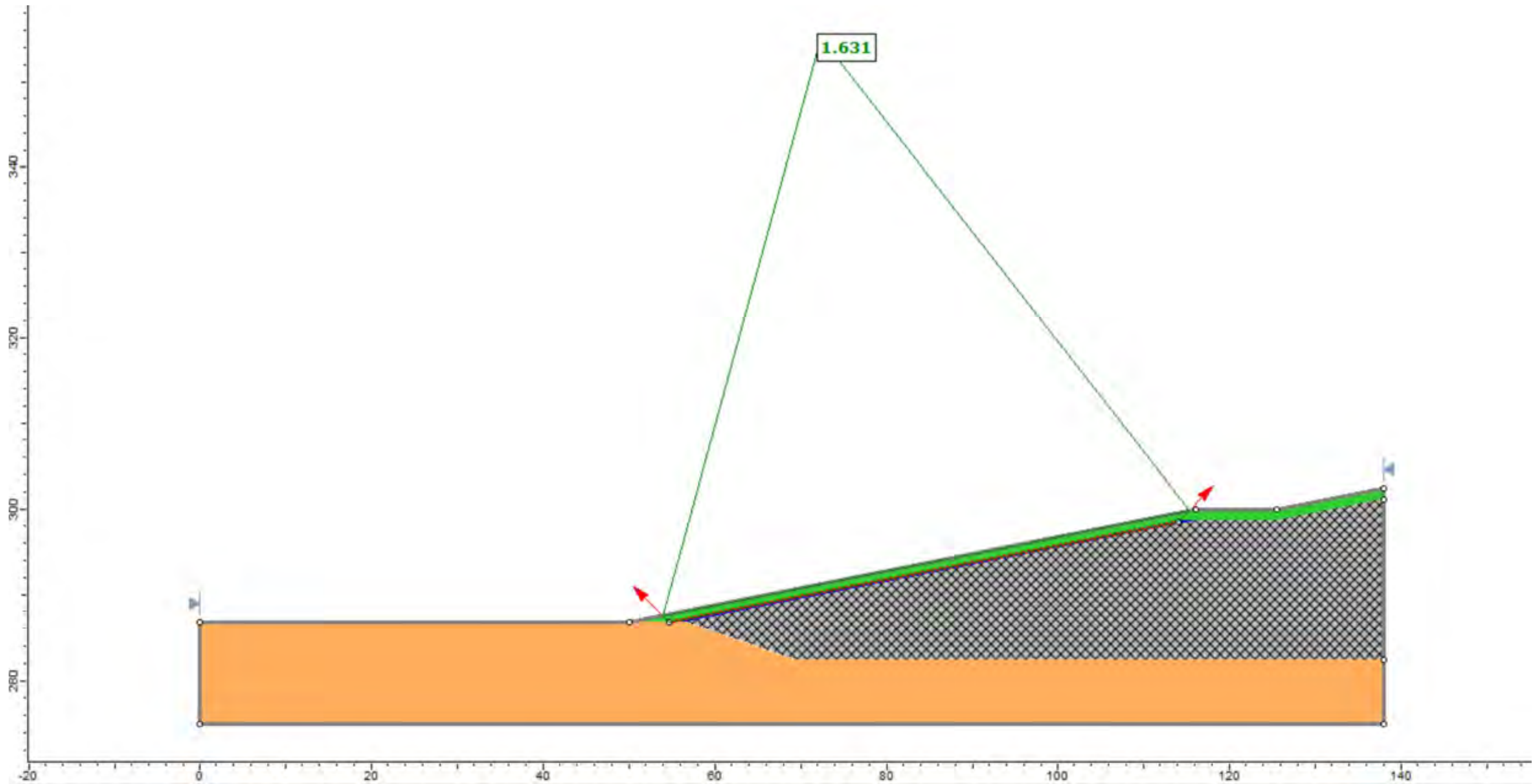
Sensitivity Analysis



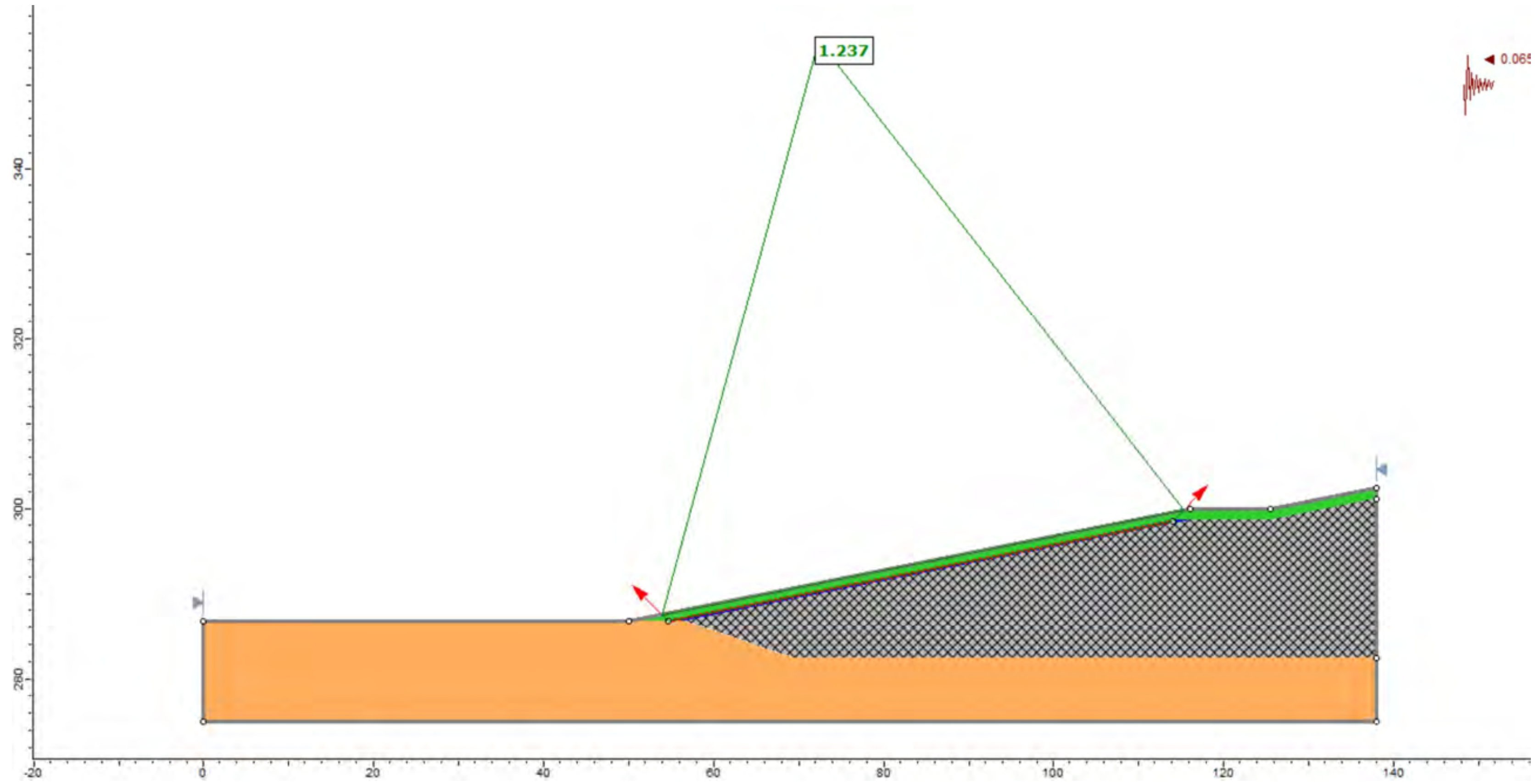
	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment
	Analyses Description:	Low Shear Geonet/LLDPE Interface with Post Peak Values
	Section:	1 to 5 Slope



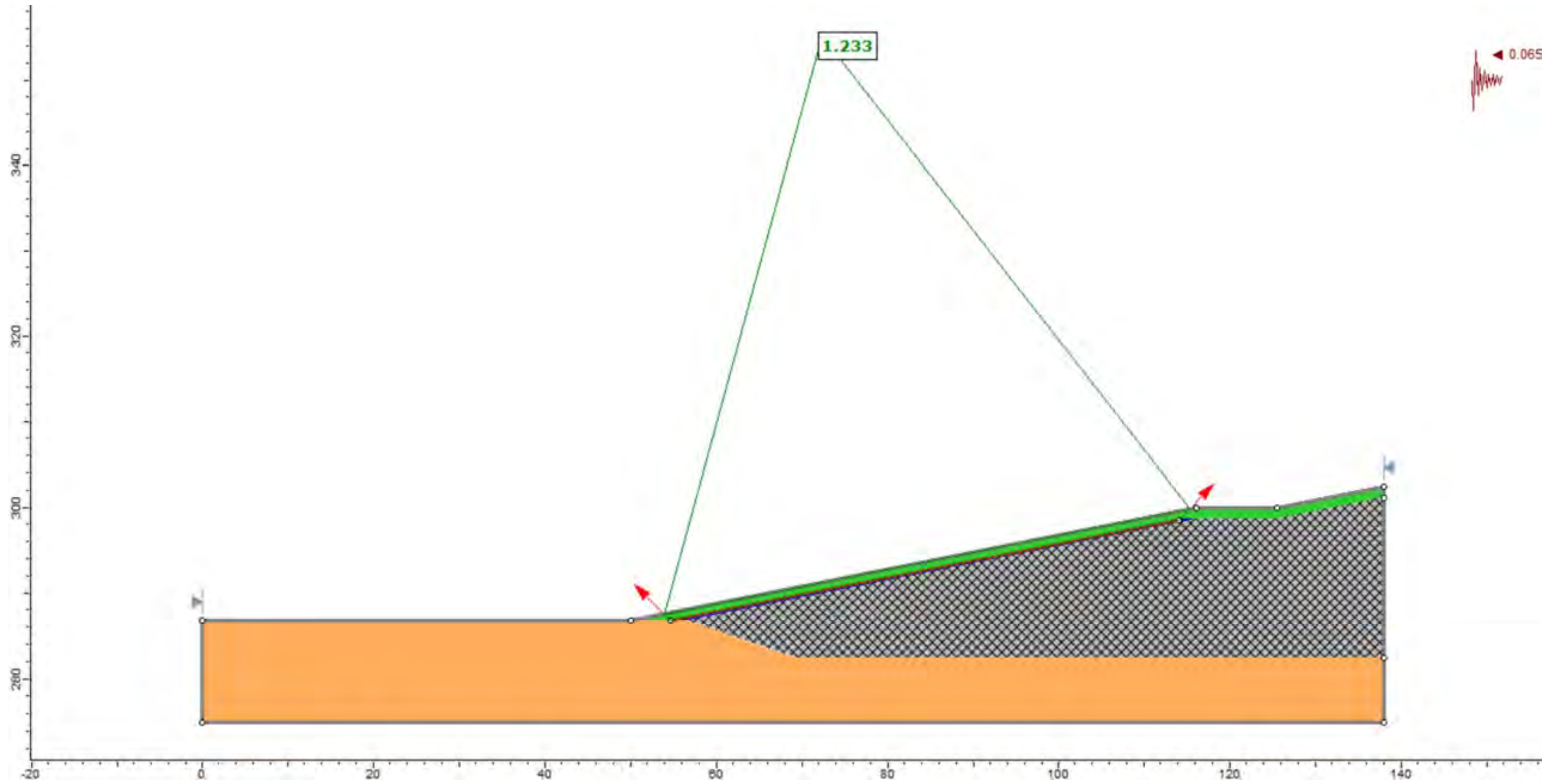
	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment
	Analyses Description:	Low Shear Geonet/LLDPE Interface with Post Peak Values with Ru=0.1
	Section:	1 to 5 Slope



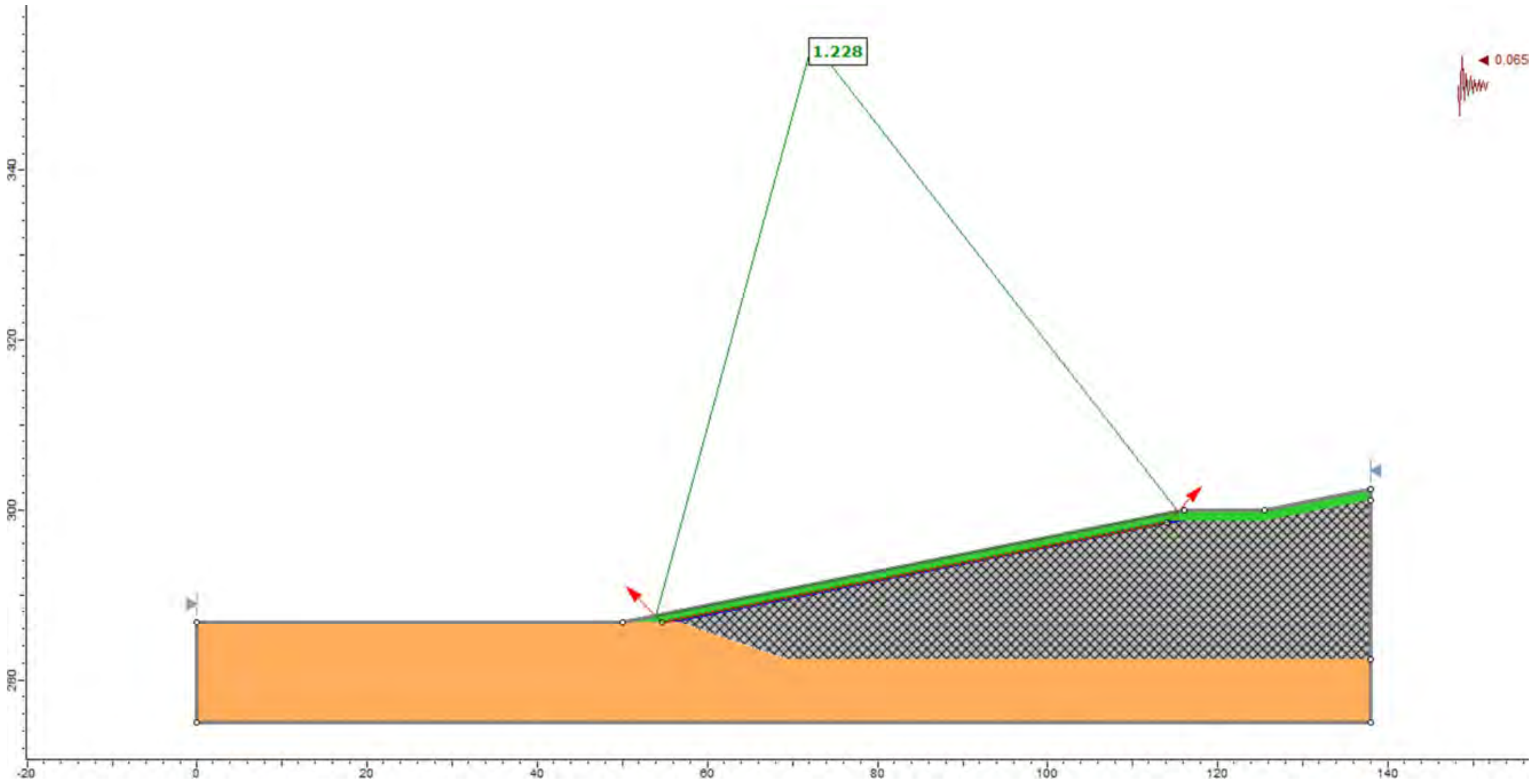
	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment
	Analyses Description:	Low Shear Geonet/LLDPE Interface with Post Peak Values with Ru = 0.2
	Section:	1 to 5 Slope



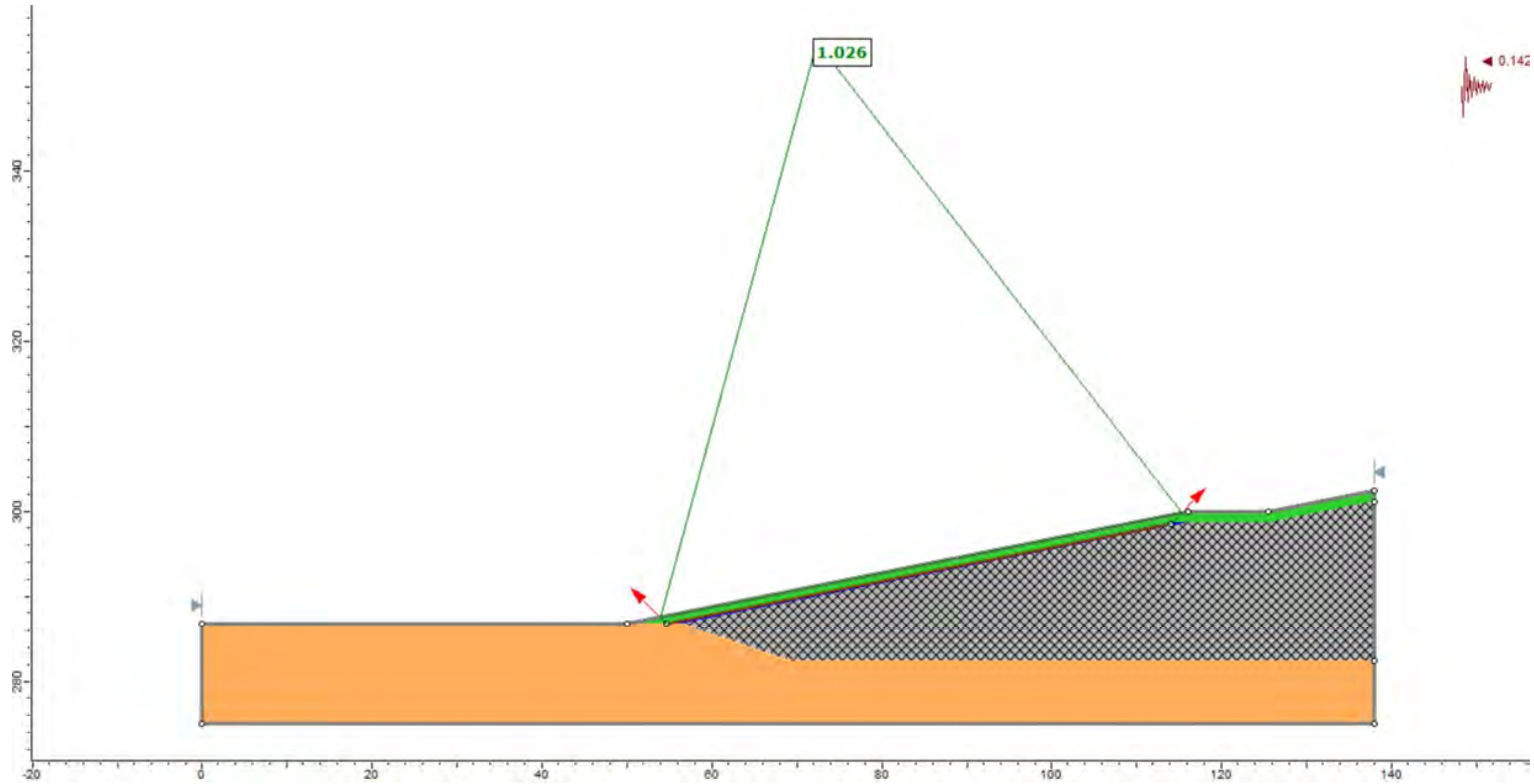
	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment
	Analyses Description:	Low Shear Geonet/LLDPE Interface with Post Peak Values with Seismic Loading (OBE – 1:475 AEP)
	Section:	1 to 5 Slope



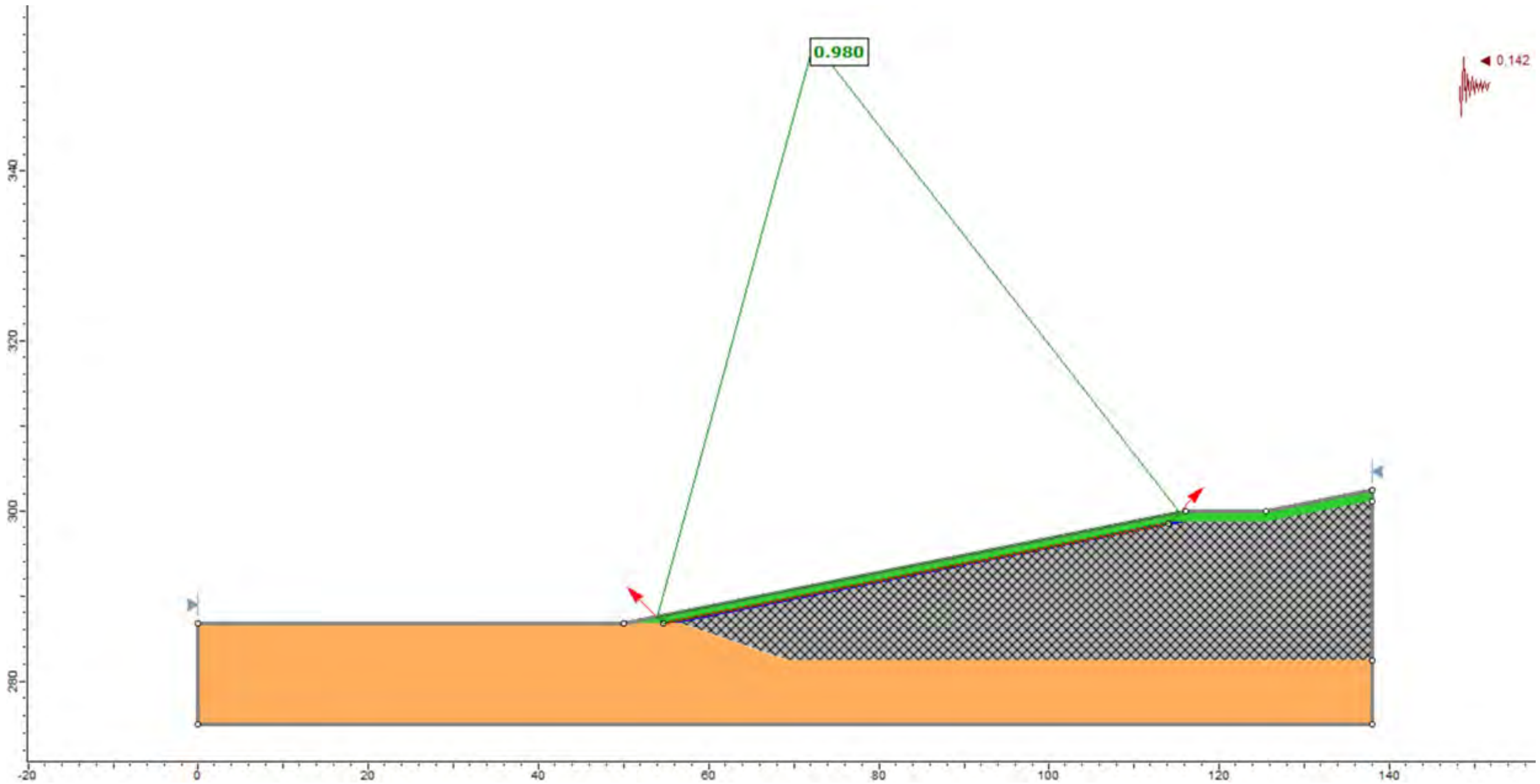
	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment
	Analyses Description:	Low Shear Geonet/LLDPE Interface with Post Peak Values with Seismic Loading (OBE – 1:475 AEP) and Ru = 0.1
	Section:	1 to 5 Slope



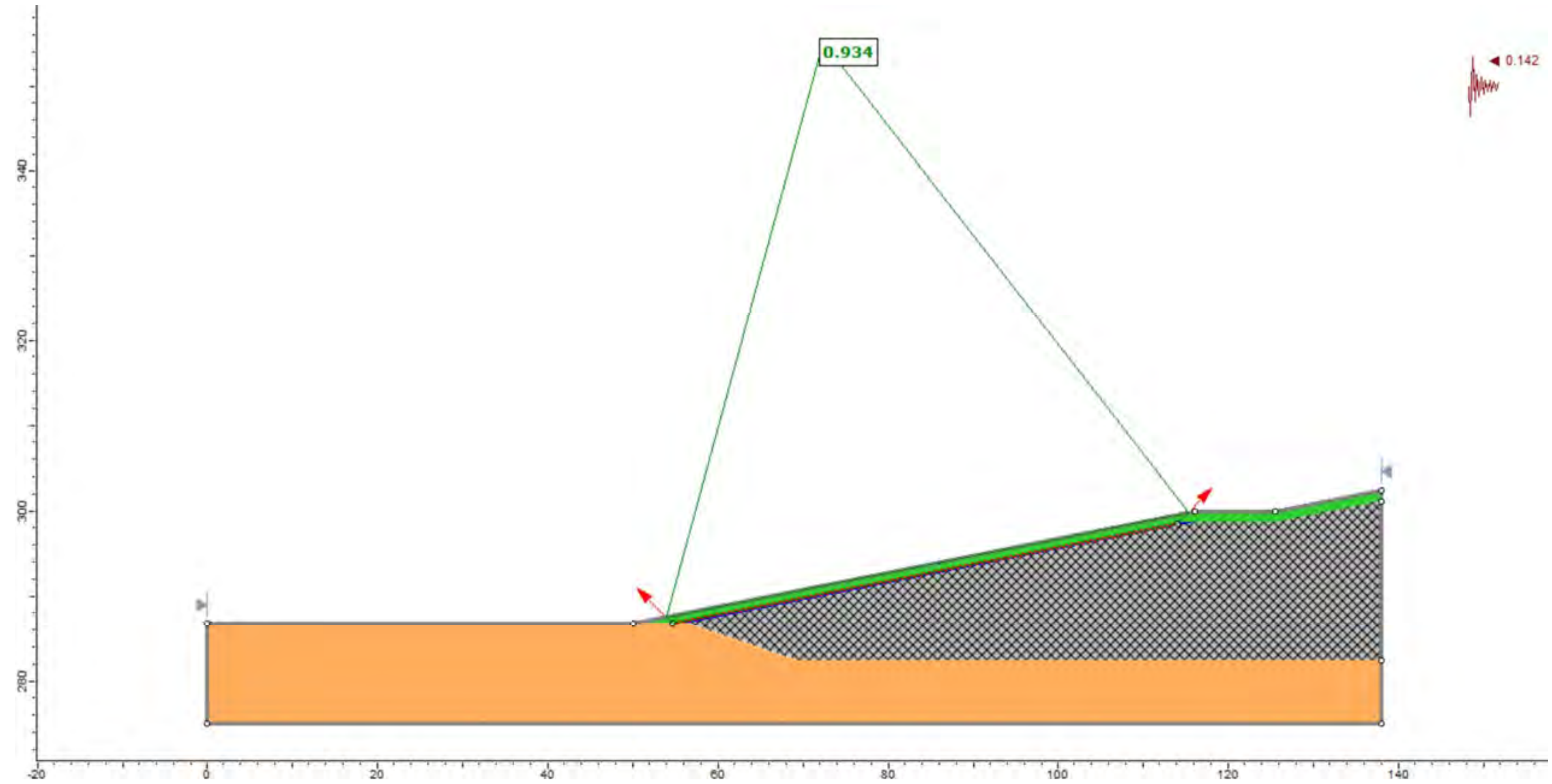
	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment
	Analyses Description:	Low Shear Geonet/LLDPE Interface with Post Peak Values with Seismic Loading (OBE – 1:475 AEP) and Ru=0.2
	Section:	1 to 5 Slope



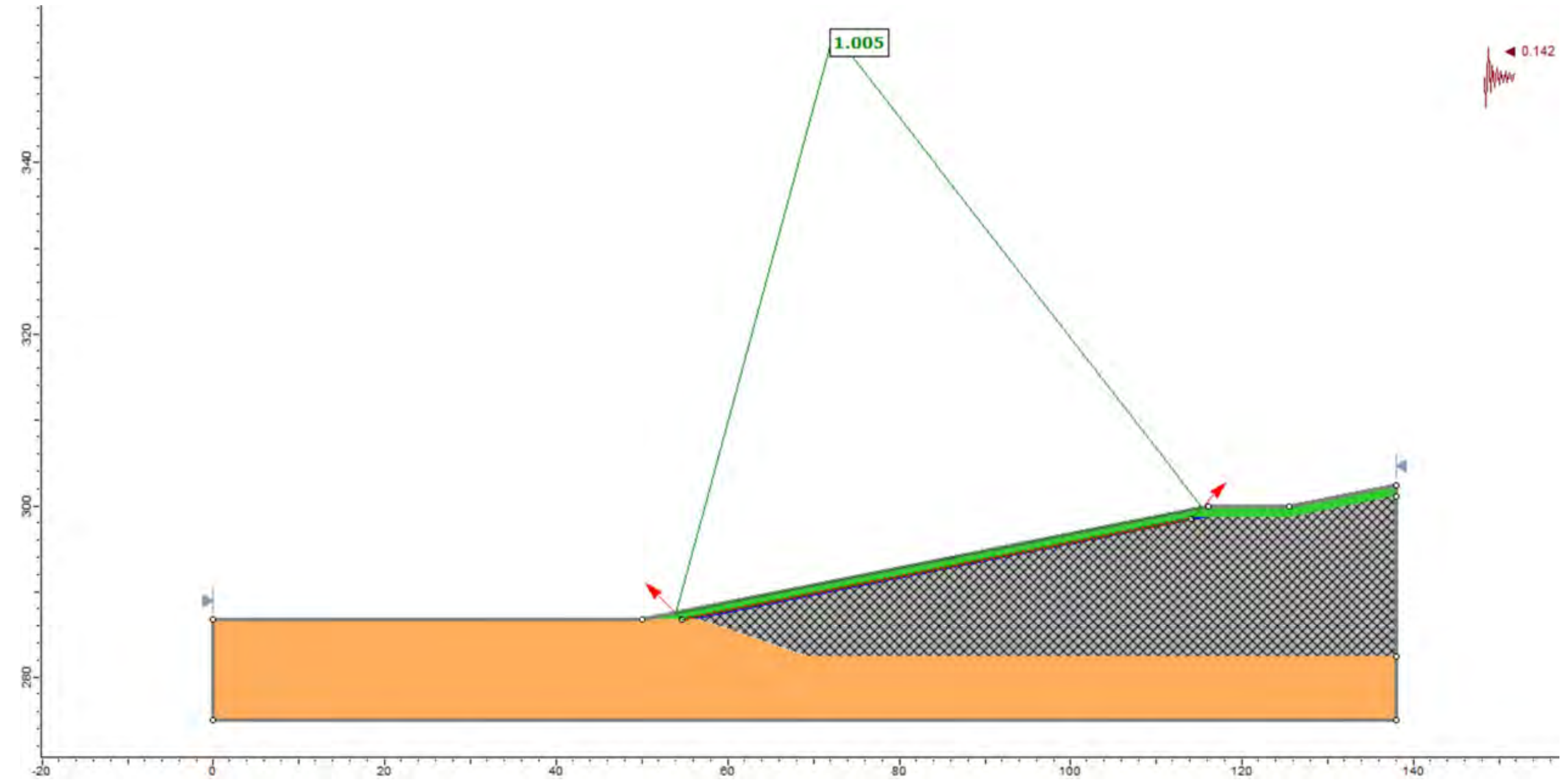
	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment
	Analyses Description:	Low Shear Geonet/LLDPE Interface with Post Peak Values with Seismic Loading (SEE/MCC – 1:1000 AEP)
	Section:	1 to 5 Slope



	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment
	Analyses Description:	Low Shear Geonet/LLDPE Interface with Post Peak Values with Seismic Loading (SEE/MCC – 1:1000 AEP) and Ru = 0.1
	Section:	1 to 5 Slope



	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment
	Analyses Description:	Low Shear Geonet/LLDPE Interface with Post Peak Values with Seismic Loading (SEE/MCC – 1:1000 AEP) and Ru = 0.2
	Section:	1 to 5 Slope



	Project:	Salt Valley Road Class II Landfill Closure Plan – Stability Risk Assessment
	Analyses Description:	Low Shear Geonet/LLDPE Interface with Post Peak Values with Seismic Loading (SEE/MCC – 1:1000 AEP) and Ru = 0.2, with Increased Shear Properties – Cohesion = 2.7kpa & Phi = 10 degrees
	Section:	1 to 5 Slope

APPENDIX G
Closed Form Analysis

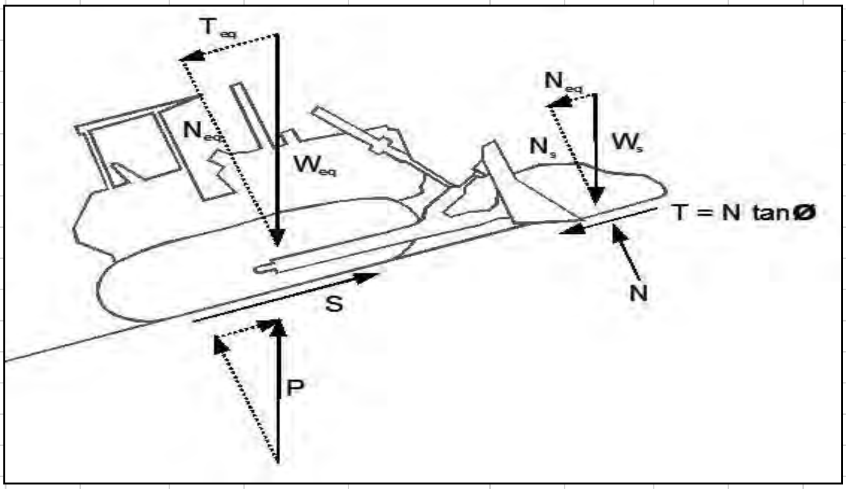
Opal Vale Closed Form Capping Assessment
Capping System Interface Stability Assessment
4 Interfaces @1:5 Side slope

			Peak				Post Peak			
Input Parameters			**Sect 1**	**Sect 2**	**Sect 3**	**Sect 4**	**Sect 1**	**Sect 2**	**Sect 3**	**Sect 4**
b	Slope Angle	°	11.31	11.31	11.31	11.31	11.31	11.31	11.31	11.31
H	Slope height	m	13.25	13.25	13.25	13.25	13.25	13.25	13.25	13.25
h	Thickness of Restoration soils	m	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
f	Friction angle of Restoration soil	°	28.00	28.00	28.00	28.00	21.00	21.00	21.00	21.00
c	Cohesion of Restoration soil	kPa	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
dct	Interface friction angle Restoration Soil/Geonet	°	22.00	22.00	22.00	22.00	18.00	18.00	18.00	18.00
act	Apparent cohesion of Restoration Soil/Geonet	kPa	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
dtg	Interface friction angle of Geonet/LLDPE	°	11.00	11.00	11.00	11.00	9.10	9.10	9.10	9.10
atg	Apparent cohesion of Geonet/LLDPE interface	kPa	3.00	3.00	3.00	3.00	2.40	2.40	2.40	2.40
dgs	Interface friction angle LLDPE/Soil	°	27.40	27.40	27.40	27.40	25.50	25.50	25.50	25.50
ags	Apparent cohesion of LLDPE/Soil	kPa	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
dgf	Interface friction angle GeoText/Subgrade	°	22.20	22.20	22.20	22.20	17.20	17.20	17.20	17.20
agf	Apparent cohesion of GeoText/Subgrade	kPa	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
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 cover soil	kN	19.00	19.00	19.00	19.00	19.00	19.00	19.00	19.00
g_{sat}	Saturated weight of cover soil	kN	21.00	21.00	21.00	21.00	21.00	21.00	21.00	21.00
h_w	Thickness of saturated cover soil	m	**0.00**	**0.10**	**0.20**	**0.30**	**0.00**	**0.10**	**0.20**	**0.30**
W_A	Weight of active wedge	kN	**1234.27**	**1247.73**	**1261.09**	**1274.34**	**1234.27**	**1247.73**	**1261.09**	**1274.34**
W_P	Weight of passive wedge	kN	**49.40**	**49.45**	**49.61**	**49.87**	**49.40**	**49.45**	**49.61**	**49.87**
U_n	Resultant pore water pressure perpendicular to slope	kN	**0.00**	**65.99**	**131.48**	**196.45**	**0.00**	**65.99**	**131.48**	**196.45**
U_h	Resultant pore water pressure on interwedge surface	kN	**0.00**	**0.05**	**0.20**	**0.45**	**0.00**	**0.05**	**0.20**	**0.45**
N_{Aab}	Effective force normal to failure plane of active wedge above impermeable layer	kN	**1210.30**	**1157.52**	**1105.16**	**1053.23**	**1210.30**	**1157.52**	**1105.16**	**1053.23**
N_{Abb}	Effective force normal to failure plane of active wedge below impermeable layer	kN	**1210.30**	**1223.51**	**1236.64**	**1249.68**	**1210.30**	**1223.51**	**1236.64**	**1249.68**
U_v	Resultant vertical pore water pressure acting on passive wedge	kN	**0.00**	**0.25**	**1.00**	**2.25**	**0.00**	**0.25**	**1.00**	**2.25**
L	Slope Length	m	**67.56**	**67.56**	**67.56**	**67.56**	**67.56**	**67.56**	**67.56**	**67.56**
Soils/Geonet Interface										
	Quadratic Equation Parameters	a	237.36	239.95	242.53	245.08	237.36	239.95	242.53	245.08
		b	-531.01	-510.26	-489.46	-468.60	-422.80	-406.10	-389.38	-372.63
		c	50.99	48.77	46.56	44.37	29.60	28.31	27.03	25.76
	Factor of Safety Against Failure		**2.14**	**2.03**	**1.92**	**1.81**	**1.71**	**1.62**	**1.53**	**1.45**
	Tension	**kN**	**-209.33**	**-196.30**	**-181.78**	**-165.57**	**-124.34**	**-110.70**	**-95.57**	**-78.72**
			No Tension	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**
Geonet/LLDPE Interface										
	Quadratic Equation Parameters	a	237.36	239.95	242.53	245.08	237.36	239.95	242.53	245.08
		b	-480.95	-471.05	-461.01	-450.83	-386.28	-378.11	-369.84	-361.48
		c	45.67	44.60	43.54	42.48	26.80	26.16	25.53	24.91
	Factor of Safety Against Failure		**1.93**	**1.86**	**1.80**	**1.74**	**1.55**	**1.50**	**1.45**	**1.40**
	Tension	**kN**	**-420.24**	**-417.87**	**-414.98**	**-411.51**	**-366.42**	**-363.31**	**-359.66**	**-355.43**
			No Tension	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**
LLDPE/soil Interface										
	Quadratic Equation Parameters	a	237.36	239.95	242.53	245.08	237.36	239.95	242.53	245.08
		b	-666.69	-673.56	-680.18	-686.53	-603.26	-609.56	-615.65	-621.55
		c	65.42	66.13	66.84	67.55	43.46	43.93	44.40	44.87
	Factor of Safety Against Failure		**2.71**	**2.71**	**2.70**	**2.70**	**2.47**	**2.47**	**2.46**	**2.46**
	Tension	**kN**	**-272.65**	**-275.36**	**-277.99**	**-280.53**	**-146.31**	**-147.73**	**-149.08**	**-150.36**
			No Tension	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**
GeoText/Subgrade Interface										
	Quadratic Equation Parameters	a	237.36	239.95	242.53	245.08	237.36	239.95	242.53	245.08
		b	-535.83	-541.28	-546.48	-551.42	-404.56	-408.69	-412.63	-416.39
		c	51.50	52.07	52.62	53.18	28.20	28.51	28.82	29.12
	Factor of Safety Against Failure		**2.16**	**2.16**	**2.15**	**2.15**	**1.63**	**1.63**	**1.63**	**1.63**

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.
However, Geomembrane-Geonet Post peak cohesion reduced to 2.4kPa to reflect strain softening at the interface
A statistical represetative number of Interface Friction tests to be undertaken on proposed geosynthetic products prior to any construction works
4 interfaces employed although Geotextile/Soil interface will not be included in the final design

APPENDIX H
Plant Operations on Geosynthetic Cap

STABILITY ASSESSMENT FOR PLANT OPERATIONS ON GEOSYNTHETIC CAP



Unit weight of soil cover	19.00	kN/cu.m
Depth of soil cover (1st lift, D)	1.00	m
Dozer type	CAT D6N LGP	
Total dozer weight	174.47	kN
Track length (L)	3.12	metres
Track width (W)	0.84	metres
Width of dozer blade (Wb)	4.08	metres
Height of soil pile (Hb)	0.80	metres
Length in front of blade (Lb)	1.00	metres
Weight of soil being spread	62.02	kN
Slope angle, alpha	11.31	degrees
Soil cover friction angle	28.00	degrees
Interface friction angle	11.00	degrees
Interface adhesion	3.00	kN/sq.m
Unit tension (geosynthetic)	0.00	kN/m
Factor of safety	**1.57**	

Forces

N(1)	=	98.80	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)	=	187.01	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)	=	41.89	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)	=	33.56	kN	$N_{4.} = N_1(\sin\beta + \tan\phi_m \cos\beta)$
N(5)CB	=	8.62	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	=	8.62	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

APPENDIX I
Gas Pressure Interface Analysis

Capping System Interface Stability Analysis - Landfill Gas

Data			
Peak Shear			
Cohesion of Lower Geomembrane Interface		3	kPa
Friction Angle of Lower Geomembrane Interface		11	degs
Post Peak Shear			
Cohesion of Lower Geomembrane Interface		2.4	kPa
Friction Angle of Lower Geomembrane Interface		9.1	degs
Thickness of Cover Soils		1	m
Slope Angle (1 in x)		5	x
Gas Pressure Beneath the Geomembrane		2	kpa
Average Unit Weight of Cover Soil		19	kN/m3)

For Peak Shear Strength	Factor of Safety	**1.673**	FOS to be greater than 1.3
For Post Peak Shear Strength	Factor of Safety	**1.359**	FOS to be greater than 1

Capping System Interface Stability Analysis - Landfill Gas

From Thiel (1999), the factor of safety for an infinite slope with gas pressure is given by:

$$\text{FoS} = \frac{\alpha' + (h\,y \cos \beta - U_g) \tan \delta'}{h\,y \sin \beta'}$$

where:

α is the cohesion intercept of the lower geomembrane interface
δ is the angle of shearing resistance of the lower geomembrane interface
h is the thickness of the cover soil above the geomembrane
β is the slope angle
Ug is the gas pressure beneath the geomembrane
y is the average unit weight of the cover soil

Note: The geonet will effectively dissipate gas pressures to the gas venting system. A nominal value for Ug of 2 is adopted. It is likely that the pressure will be below 0 if gas extraction is taking place.

Typical Ug Values* (kPa)	0	LCV	Lowest Conceivable Value
	4	HCV	Highest Conceivable Value
	1	MLV	Most Likely Value
	0.67	σ	Standard Deviation 'Three sigma rule'

Ug* from Thiel (2008) Slope Stability Sensitivities of Final Covers, The First Pan American Geosynthetics Conference.

Thiel, R. (1999). Design of a gas pressure relief layer below a geomembrane cover to improve stability, Proc. Geosynthetics '99, Boston, NAGS.

Assets | Engineering | Environment | Noise | Spatial | Waste

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