Southern Piggyback Cell Stability Risk Assessment

Tamala Park Waste Management Facility

Prepared for Mindarie Regional Council

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ACRONYMNS

AEP	-	Annual Exceedance Probability
ANCOLD	-	Australian National Commission on Large Dams
BGL	-	Below Ground Level
BH	-	Borehole
BOM	-	Bureau of Meteorology
FDM		Finite Difference Method
FEM	-	Finite Element Method
FLAC		Fast Lagrangian Analysis of Continua
FoS	-	Factor of Safety
FS_{min}	-	Minimum Factor of Safety
GCL	-	Geosynthetic Clay Liner
GIS	-	Geographic Information System
ha	-	Hectare
HDPE	-	High Density Polyethylene
ICOLD	-	International Commission on Large Dams
kN/m^2	-	Kilo Newtons per metre squared
kN/m^3	-	Kilo Newtons per cubic metre
kPa	-	Kilo Pascal
LLDPE	-	Linear Low Density Polyethylene
m	-	metres
AHD	-	Australian Height Datum
MCE	-	Maximum Credible Earthquake
MSW	-	Municipal Solid Waste
NATA	-	National Association of Testing Authorities
NSHA	-	National Seismic Hazard Assessment for Australia
OBE	-	Operating Base Earthquake
PGA	-	Peak Ground Acceleration
PSR	-	Parallel Submerged Ratio
R&D	-	Research and Development
RF	-	Risk Factor
r_u	-	Pore-water pressure as a function of the overburden stress
SA	-	Spectral Acceleration
SEE	-	Safety Evaluation Earthquake
SRA	-	Stability Risk Assessment
TR	-	Technical Report
WMF	-	Waste Management Facility

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1 Introduction

The Mindarie Regional Council (MRC) operates the Tamala Park Waste Management Facility (the Site) which is located on Lot 9020 on Plan 408820, 30km north of Perth, at 1700 Marmion Avenue, Clarkson, WA, 6030. The Site is one of the largest putrescible landfills (Category 64, Class II and III) in the Perth metropolitan area. The Site is licensed with the Department of Water and Environmental Regulation (DWER), Licence number L6963/1997/14.

In order to meet closure obligations, to provide an adequate closure landform in accordance with the Closure & Post Closure Management Plan[1], MRC are proposing to construct a new piggyback cell 'Southern Piggyback Cell' (SPC) between the capped Stage 1 landfill, operational Stage 2 landfill, and existing Northern Piggyback Cell (NPC) at the Site.

Talis Consultants Pty Ltd (Talis) was engaged to undertake a Stability Risk Assessment (SRA) to support the abovementioned SPC design and approvals application.

This report presents the findings of the SRA.

1.1 Report Context

There is no set guidance requirement for undertaking SRA's for solid waste facilities in Australia, 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 proposed Southern Piggyback Cell at the Site and presents the overall findings of the work.

1.2 Location and Topography

Tamala Park is situated 30km north of Perth, 10km from Wanneroo town site to the south-east, 3km from Burns Beach to the south-west, and 3km from Quinns Rocks and Mindarie Keys to the north-west. It comprises part of Lot 9020 on Plan 408820 at 1700 Marmion Avenue, Clarkson, WA, 6030. The landfill occupies an area of approximately 37 hectares (ha), of which approximately 11ha have been capped. The Site Boundary is shown in Licence L6963/1997/14 and depicted on Figure 1 presented in Appendix A. Access to the site is via Marmion Avenue, with internal roads providing access to the site facilities and landfill cells.

The residential Catalina Housing Estate and development area is located approximately 150m north of the premise boundary and 500m north of the active landfill site (Stage 2, Phases 2 and 3). In addition, the Kinross residential area is located approximately 20m south of the Premises boundary and 650m south of the active landfill (Stage 2, Phase 3).

There are a number of sensitive land uses within the vicinity of the site including groundwater, public drinking water source, the Indian Ocean, Neerabup Lakes and Bush Forever Site 323 (links Burns Beach to Neerabup National Park. These are described in more detail within the CPCMP[1].

[1] Tamala Park Closure and Post-Closure Management Plan, Talis Consultants Pty Ltd Ref: TW21026 - Tamala Park Closure and Post-Closure Management Plan_2.0, 23 June 2021

At present the Site's surface elevation slopes from approximately 50m Australian Height Datum (AHD) in the northern portion of the site to approximately 40m AHD at the eastern and southern boundaries, but to 32.5mAHD along the western edge of the landfill.

The landfill includes Stage 1 which was excavated and filled from 1991 to 2004 in two phases (south and north). Stage 1 is unlined and therefore no has leachate collection, is capped and contains a landfill gas extraction system. Stage 2 is lined with a capacity of approximately 10 million tonnes with three filling phases. It is estimated that Stage 2 will be filled by 2028/2029.

Stage 1 of the landfill comprises closed and rehabilitated 'North' and 'South' areas. Stage 1 is located in the eastern section of the site and rehabilitated levels range between 10m to 50m AHD.

Stage 2 is located to the west of Stage 1 and comprises Phase 1 in the north and Phases 2 and 3 in the south. Phase 1 is predominantly capped and restored and rises from approximately 10 m AHD at the northern extent of the landfill to 53m AHD at the southern extent adjacent to the active Phase 2 area.

The topography and layout of the existing phases is shown on Drawing W-100, in Appendix A.

1.3 Geology

The geology of the Site consists predominantly of sand of the Quindalup Dune system overlying low grade sand and limestone of the Tamala Limestone formation (the base of which is approximately -35 m AHD). Geoscience Australia (1:2.5 million scale) classifies surface geology across the site as being "Dunes, sand plan with dunes, coastal dunes"

1.4 Hydrogeology

An unconfined aquifer system is present within the sand and limestone of the superficial formations that underlie the site. An extensive network of groundwater monitoring bores is located up-gradient, down-gradient and to the north of the Site. Groundwater monitoring results indicated levels ranging from 0.4m above AHD (Australian Height Datum) to 0.9mAHD (CSIRO, March 2017), these vary seasonally. The general inferred direction of groundwater flow within the superficial formation aquifer is in a westerly direction towards the ocean. GHD updated the Conceptual Site Model in December 2019 and indicated that abstraction bores operated by the Water Corporation might be influencing groundwater gradient to swing in a more northerly direction.

1.5 Climate

Being in a temperate zone, rainfall is seasonal with higher rainfall generally in the months of May to September. Table 1-1: presents a summary of rainfall records, from 1970 to 2021.

Table 1-1: Rainfall Overview in Millimetres (1970-2021) from Perth Metro Station

Aspect	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Average	19	14	20	35	88	127	142	124	83	38	24	10	**730.9**
90^{th} Percentile	46	35	53	71	138	206	191	167	111	589	53	20	**867.8**
Highest	139	137	70	154	191	251	279	186	144	96	92	77	**904.8**

The mean annual rainfall for the Site is calculated as 730.9 millimetres (mm) with the highest recorded annual rainfall at 904mm, which occurred in 1995.

The highest mean temperature is 31.5°C, occurring in February, whilst the lowest mean temperature is 7.9°C occurring in July. Table 1-2 shows the average maximum and minimum temperatures at the Perth Metro weather station for years 1994 to 2020.

Table 1-2: Maximum and Minimum Temperatures from Perth Metro Station

Aspect	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Mean Max. Temp. (°C)	31.2	31.5	29.6	25.9	22.3	19.5	18.5	19.1	20.5	23.4	26.7	29.4	**24.8**
Mean Min Temp. (°C)	18.1	18.3	16.8	13.8	10.4	8.6	7.9	8.3	9.6	11.6	14.3	16.4	**12.9**

The wind direction generally ranges from east-northeast in the morning (9am), changing direction to west-southwest in the afternoon (3pm). Winds at the Site are typically moderate in the morning and the afternoon. The wind rose for morning and afternoon winds can be seen in Figure 1-1.

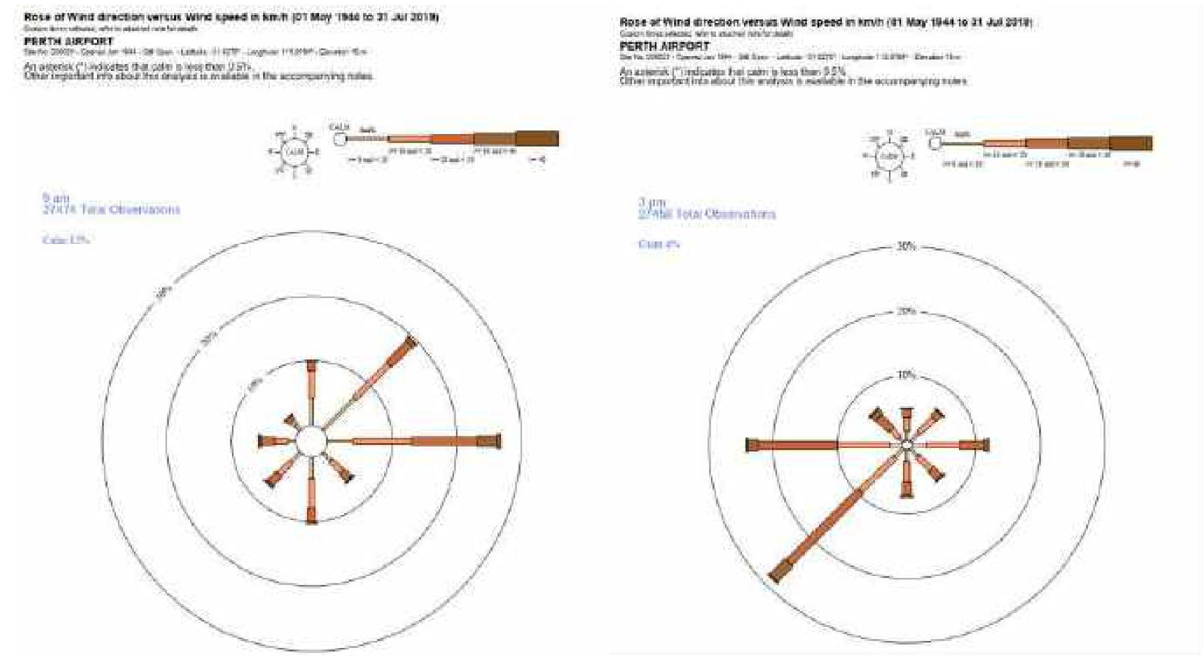

Figure 1-1: 9am (left) and 3pm (right) Wind Rose for Perth Metro Station

1.6 Conceptual Site Model

The conceptual stability site model has been developed from information contained in the Mandatory Auditors Report[2], and MRC records.

1.6.1 Stage 1

Unlined 'dilute and attenuate' landfill encompassing 12 cells. Filled from 1991 to 2004 with waste depths extending to 41m. Lowest level of the unlined landfill is anecdotally reported as 2m AHD.

[2] Mandatory Auditor's Report, 'Tamala Park Landfill, 1700K Marmion Avenue, Tamala Park', Senversa Pty Ltd, October 2023.

The Stage 1 basal and sideslope subgrade consists of the Tamala Limestone where extensive extraction activities have taken place to form the below ground landfill void.

The Stage 1 sideslope subgrade consists of gradients ranging from 1V4.8H to 1V:28H. The Stage 1 sideslope has been temporarily capped, it is unknown when the cap was installed, and the soil depth over the 'pvc' temporary cap varies. The 'sand' cover will be excavated from the slope and the underlying pvc cap removed and disposed of in the active landfill area during construction of the piggyback lining.

1.6.2 Stage 2

The Stage 2 basal and sideslope subgrade consists of the Tamala Limestone where extensive extraction activities have taken place to form the below ground landfill void.

Stage 2 encompassing 10 cells, broken into three phases.

- Phase 1 (northern edge of Stage 2) with each section/cell containing HDPE base liner and clay wall liner, leachate collection system and landfill gas extraction system. Phase 1 was capped in 2011 with a gas extraction system installed.
- Phase 2 (Stage 2A - cells 16, 17, 21, 22 and Stage 2B – cells 26, 27, 31, 32) with HDPE lined floor and walls with leachate collection sumps.
- Phase 3 with HDPE geomembrane and geosynthetic clay liner. The side wall commences with a clay liner and changes to a 5-layer system comprising cushion geotextile, geosynthetic clay liner, LLDPE geomembrane, Slippage geotextile B (non-woven) and Slippage Geotextile A (woven).
- Northern Piggyback Liner - (Existing)
  - Limestone Fill
  - Spanning Layer (Geotextile) & Screened Limestone Fill
  - Cushion Geotextile
  - 2mm LLDPE Geomembrane
- Southern Piggyback Liner (Proposed – in ascending order through the system)
  - 300mm Regulation layer
  - Underliner Gas Collection/Venting System
  - 500mm Crushed Limestone Fill with geogrid reinforcement layer
  - Geosynthetic Clay Liner
  - 2mm LLDPE double textured Geomembrane
  - Cushion/Protection Geotextile
  - 300mm Leachate Drainage Layer or Soil/Limestone/Sand Protection Layer

1.6.3 Rehabilitation Design

The Victorian Environment Protection Agency (EPA), Best Practice Environmental Management 'Siting, Design, Operation and Rehabilitation of Landfills', 2015 (BPEM) Landfill Guidelines have been adopted and supported by Tamala Park 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.

1.6.4 Final Profile

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

- The CPCMP[1] prepared by Talis, June 2021;
- 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 Site.

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;
- Slopes of not less than 1V:20H and no greater than 1V:4H are proposed for the final restoration profile. During the filling of adjacent phases temporary slopes will be progressively filled against and in the medium term will permanently buttress the temporary slopes. It should be noted that the steepest slope regarded as Best Practice is 1V:5H and should be employed unless it can be demonstrated that the long-term stability of said slope is not compromised. The CPCMP SRA[3] demonstrated that the slightly steeper 'pre-settlement' slope angle to be employed at Tamala Park will remain stable in the long term with the currently proposed engineering design;
- 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 pre-settlement elevation will not exceed 60m AHD.

1.6.5 Capping System

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

[3] Capping Stability Risk Assessment, Tamala Park Waste Management Facility. Reference:- TW21026 – Tamala Park Stability Assessment_2.0, Talis Consultants Pty Ltd, June 2021

- 200mm thick Regulation Layer consisting of available indigenous soils suitable for purpose;
- Gas Collection layer comprising a Geocomposite (Geonet);
- A 1.5mm Linear Low-Density Polyethylene (LLDPE) textured geomembrane;
- Sub-surface drainage layer comprising a drainage Geocomposite;
- Revegetation layer at least 1,000mm thick of clean locally sourced soil and; and
- A 200mm thick topsoil/growth medium.

1.6.6 Waste Mass Model

The existing Stage 2 landfill is composite lined with a leachate management system established at the beginning of landfilling operations, filling is ongoing and the waste has been deposited in a series of phases, stages and lifts across the entire footprint. Currently, the unlined Stage 1 North and South, as well as Stage 2 Phase 1 has been capped. The topographical surface of the waste has achieved an elevation up to 45mAHD as shown in Drawing W-100, although as previously stated it will peak at about 60m AHD as indicated in Drawing C-101, when final fill heights are achieved (see Appendix A).

The base of the Stage 2 landfill lies at approximately 5m AHD, with basal gradients designed encourage any residual leachate to be channelled to the extraction points. The depth of the base below current topographic waste level is in the region of 25 to 35m.

As a guide the basal lining system for Stage 2 consists of the following elements (from bottom to top):

- Compacted Subgrade Layer
- Geosynthetic Clay Liner (GCL)
- 2.0mm Geomembrane Liner
- Protection/Cushion Geotextile Layers
- Leachate Drainage Layer – comprising a 300mm thick layer of non-calcareous aggregate
- Separation/Filter Geotextile Layer

For the purpose of the waste mass model, the future temporary waste slopes at the edge of each stage/phase are modelled at a gradient of 1V:3.0H, to maximum 20m vertical height.

The waste shall ultimately be placed in line with the pre-settlement restoration levels (Drawing C-101, Appendix A) at a maximum permanent gradient of approximately 1V:4H.

No site-wide leachate monitoring data has been made available to determine the true leachate levels across the currently filled landfill. Notwithstanding this, for the purpose of this assessment, the pore-water pressure in the waste mass has been taken as a function of the overburden stress (r_u).

A r_u value of 0.2 has been utilised for the Stage 2 waste mass in all limit equilibrium assessments. 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.

1.6.7 Landfill Gas Management

A landfill gas management system has been installed within the site by Energy Developments Limited (EDL) who own and operate the equipment. The gas is used to fuel gas engines to create electricity which is sold to the grid. Further details surrounding the Landfill Gas Management System are provided in the CPCMP[1].

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

Little investigation data was made available beneath the southern piggyback landfill footprint to ascertain the geological sequence following the historic quarrying operations. The basal subgrade consists of the Tamala Limestone where extensive extraction activities have taken place to form the below ground landfill void.

The base of the Stage 2 landfill (circa 5m AHD) and Stage 1 Landfill (anecdotally reported to be circa 2m AHD) and lies between 27 and 54 metres below the top of restoration profile. The current filling profile suggests that the underlying limestone beneath the southern piggyback area has been surcharged by a minimum of 25-27m of waste. The rock (underlying unlined intact Tamala Limestone) will not be subject to excessive deformation from the stresses imposed by the proposed waste mass.

Assessment of basal deformability has therefore been screened out.

2.1.2 Basal Heave

Basal heave arises where underlying groundwater pressures increase to overcome the imposed loads from the overburden and waste mass. The groundwater levels appear to show little variance, between 0 and 1mAHD. With existing overburden comprising approximately 4m of limestone and 27 (min) of waste) the risk of basal heave is insignificant.

Assessment of basal heave has therefore been screened out.

2.1.3 Cavities in the subgrade

No underground mining activity is known to have occurred in the vicinity of the landfill footprint. Although, the Tamala Limestone is known for numerous syngenetic karst features including pinnacles, caves, dolines (collapse and solution). The Environmental Geology Series (Gozzard, 1982a[4], 1982b[5] and 1982c[6]) identifies Tamala Limestone LS1 as a medium to coarse grained Aeolian calcareous sandstone to sandy limestone (calcarenite). LS2 in the Environmental Geology Series (Gozzard, 1982a[4], 1982b[5] and 1982c[6]) marks a karst belt displaying sinkholes and caves, with LS2 reported to have a greater carbonate content (approx. 80%).

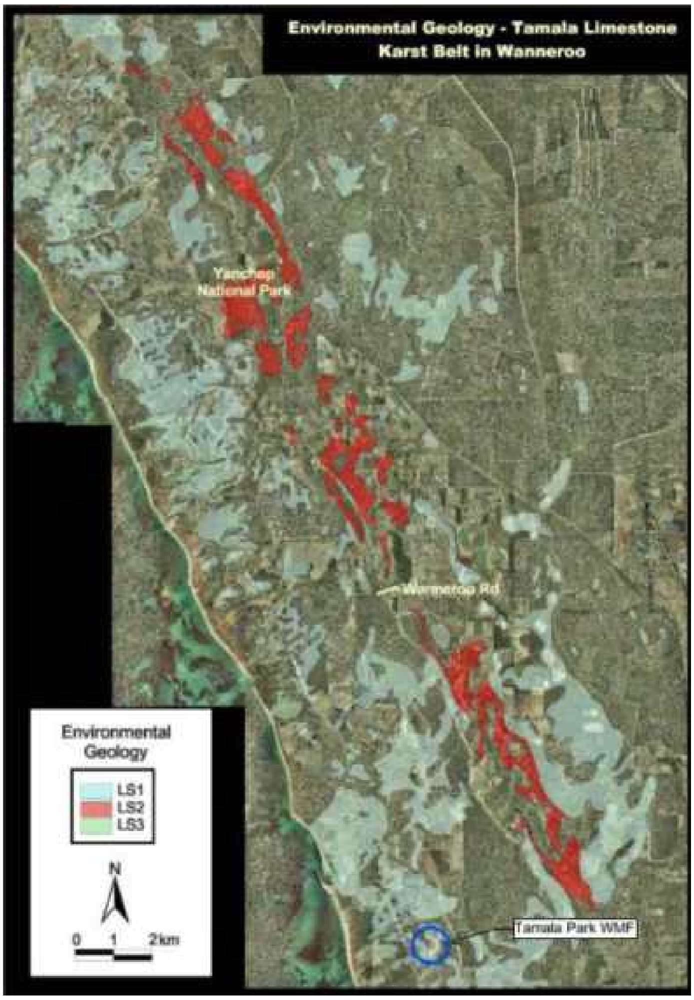

Figure 2-1: Location of Tamala Park WMF with respect to 'Figure 2.1 Tamala Limestone Karst Belt in Wanneroo[7]'

Figure 2-1 is reproduced from the Geoscience Australia 'Review of Karst Hazards in the Wanneroo Area' (Czaky, 2003[7]) identifying the location of the Tamala Park WMF with respect to the south-west to north-east karst belt, which represents an interdune swale within the Spearwood Dune System. Czaky (2003)[7] reported that the LS2 area is prone to karst features due to low topography enabling groundwater to interact with the limestone. On the neighbouring dunes, the limestone sits higher and limits limestone-groundwater contact.

[4] Gozzard, J.R. (1982a) Muchea Sheet 2034 I and part 3134 IV, Perth Metropolitan Region, Environmental Geology Series, Geological Survey of Western Australia.
[5] Gozzard, J.R. (1982b) Perth Sheet 2034 II and part 2034 III and 2134 II, Perth Metropolitan Region, Environmental Geology Series, Geological Survey of Western Australia.
[6] Gozzard, J.R. (1982c) Yanchep Sheet 2034 IV, Perth Metropolitan Region, Environmental Geology Series, Geological Survey of Western Australia.
[7] Csaky, D, (2003) Review of Karst Hazards in the Wanneroo Area, Perth, Western Australia. Minerals and Geohazards Division Perth Cities Project. Geoscience Australia.

The Site is located outside of the LS2 karst hazard zone, and extensive extraction activities have taken place at the site to the landfill basal levels. The SPC is in excess of 25m above the base of the previously filled Stage 1 landfill (waste placement commencing from 1991). No investigation data was made available beneath the landfill footprint and no evidence has been provided of any historic features during the previous extraction activities, or previous construction activities.

Assessment of cavities in the subgrade has therefore been screened out.

CQA procedures during excavations and subsequent regrading and placement and compaction of the basal sub-grade will eliminate the risk of near surface voids being present in the waste. Deformability of the waste is considered in the sideslope screening.

2.2 Sideslope Subgrade Screening

The side slopes will be formed on the existing Stage 1 landfill to the east and the previous residual quarrying/excavation extent to the south. All materials employed within the southern piggyback liner construction are considered to be ‘free-draining’ and as such, short term, undrained conditions are not deemed appropriate for modelling.

The measured coefficient of permeability of the crushed limestone is 4.153×10^{-5} m/s, with the measured coefficient of permeability of the 'subsoil' (sand) is 6.609×10^{-5} m/s. The US Army Corps of Engineers, 2003[8], states that during analyses of stability during construction and at end of construction, materials with values of permeability greater than 10^{-4} cm/sec (i.e. 1×10^{-6} m/s) usually will be fully drained throughout construction. Materials with values of permeability less than 10^{-7} cm/sec (i.e. 1×10^{-9} m/s) usually will be essentially undrained at the end of construction. In cases where appreciable but incomplete drainage is expected during construction, stability should be analysed assuming fully drained and completely undrained conditions, and the less stable of these conditions should be used as the basis for design.

The sideslope subgrade stability analysis will be undertaken in terms of drained conditions.

2.2.1 Deformability

The Stage 1 sideslope subgrade consists of gradients ranging from 1V4.8H to 1V:28H. The Stage 1 sideslope has been temporarily capped, it is unknown when the cap was installed, and the soil depth over the 'pvc' temporary cap varies. The 'sand' cover will be excavated from the slope and the underlying pvc cap removed and disposed of in the active landfill area during construction of the piggyback lining. The Stage 1 landfill was excavated and filled from 1991 to 2004 in two phases (south and north). It is therefore considered effective settlement in relation to the imposed stress from waste mass, will be minimal and largely taken place. The slope has previously had construction plant operating on the slope safely. Construction activities are therefore not considered to cause excessive deformation in the underlying waste mass.

The load imposed by the new waste on the historic waste deposits that forms the sideslope subgrade could give rise to settlements that could, in turn, affect the integrity of the lining system. The deformations with regards to long term strains will be assessed for the potential build-up of tension within the sideslope lining geosynthetics.

Deformations in relation to potential strains in the lining system is considered in Section 2.3.2.

[8] Slope Stability, Engineer Manual. US Army Corps of Engineers, EM 1110-2-1902 31 October 2003

2.2.2 Groundwater

The highest groundwater level is approximately 1m AHD which is approximately 4 metres below the base of the Stage 2 landfill and anecdotally 1m below the unlined Stage 1 basal levels. The southern piggyback liner profile suggests that the underlying limestone has been surcharged by a minimum of 27m of waste.

Assessment of groundwater on the sideslope has therefore been screened out.

2.2.3 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 to 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 lining. This practice will eliminate the potential for near-surface cavities to be present and therefore is not considered to require further assessment.

2.3 Basal & Sideslope Lining System Screening

2.3.1 Basal Lining Stability

The landfill base is >20m beneath the lowest tie-in point of the piggyback lining system

The Stage 2 basal lining engineering has now been filled against/buttressed and the risk of internal instability has consequently now been eliminated. The Stage 1 area is unlined with historic landfilling operations undertaken directly on the Tamala Limestone in the below ground landfill void. Therefore, there is no basal lining stability to be assessed under the proposed southern piggyback area.

Assessment of the basal lining has consequently been screened out.

2.3.2 Sideslope Lining Stability

The southern piggyback side slope liner will be placed to the full height from the Stage 2 landfill tie-in (west) to the eastern and southern extent. The liner will be tied into the existing northern piggyback liner to the north.

As discussed in Section 1.6.2, the sideslope lining consists of:

- 300mm Regulation layer
- Underliner Gas Collection/Venting System
- 500mm Limestone Fill with geogrid reinforcement layer
- Geosynthetic Clay Liner
- 2mm LLDPE Geomembrane
- Cushion/Protection Geotextile
- 300mm Leachate Drainage Layer or Soil/Limestone/Sand Protection Layer

The southern portion of the piggyback lining area will be constructed on the in situ Tamala Limestone with 1V:3H sideslopes. The southern sideslope lining consists of:

- 500mm Limestone Fill with geogrid reinforcement layer

- Geosynthetic Clay Liner
- 2mm LLDPE Geomembrane
- Cushion/Protection Geotextile
- 300mm Leachate Drainage Layer or Soil/Limestone/Sand Protection Layer

The leachate drainage blanket is to be placed above a cushion/protection geotextile over the extent of the east-west sideslope (stopping 2m from the perimeter as a gas break) and to 2m vertical height on the 1V:3H sideslope.

It is considered that the stability of the overall lining system, which will include the leachate drainage layer, will require assessment for both unconfined liner and construction activities. The stability of the unconfined lining system, this will be undertaken by closed form analysis, assessment of construction activities are discussed further in Section 2.3.2.3.

If the stability in the unconfined condition is satisfactory, it is clear that the stability and integrity of the side slope liner in the confined condition will be greater due to the buttressing effect of the waste placed in horizontal layers and will therefore be satisfactory. This issue is considered as being separate from the Waste Mass Analysis, which examines the influence of the confined side slope lining system on the overall stability of the waste mass.

The aspect of the side slope lining system performance with regards to long term strains needs to be assessed for the potential build-up of tension within the lining geosynthetics.

2.3.2.1 *Gas Pressure*

The build-up of gas pressure from the landfill is relevant to the stability of 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).

The waste composition at Tamala Park is predominantly municipal solid waste with negligible commercial and industrial and construction and demolition waste. Due to the moderate rainfall and temperate climate, the placed waste would normally be categorised as 'dry'. The historic waste deposits in the Stage 1 unlined landfill were deposited from 1991. There is a current active extraction system across the Stage 1 although it is anecdotally reported that there are limited yields from this gas field. An underliner gas collection system is proposed to alleviate the potential for any significant pressure to build up beneath the piggyback lining system.

The issue of gas pressure beneath the piggyback liner will be considered in terms of interface (veneer) stability.

2.3.2.2 *Wind Uplift*

Large areas of exposed geosynthetics employed within the piggyback liner could potentially be subject to high wind loads and as a result mechanical uplift. The majority of the piggyback liner will be covered/surcharged with a leachate drainage aggregate.

It is good practice to cover exposed geosynthetics progressively. However, the covering of the geosynthetics on the 1V:3H slope will be limited to the rate of rise of the adjacent waste lifts. Semi-permanent roped sandbag lines are therefore recommended to be installed on the exposed geosynthetics on the sideslope along the adjacent geotextile panel seams and should be monitored monthly.

Should the sandbags degrade, further surcharging of the slope will be required (with additional sand bags). The weather should be monitored during the operations and if significant periods of wind are forecast then the exposed geosynthetics should be adequately surcharged.

As such, wind uplift is not considered further within this SRA.

2.3.2.3 *Construction*

Construction vehicles shall not be allowed to operate directly on top of the geosynthetic lining system 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 sideslope during placement of drainage layer should be considered as geosynthetics are to be used in the piggyback lining system.

2.4 Waste Mass Screening

Placement of waste onto the southern piggyback will be undertaken in horizontal layers as the soil protection layer is installed on the lining system and the waste lifts are built up.

The maximum future temporary waste faces are considered to be located on the existing Stage 2 landfill area while the southern piggyback liner is constructed. For the purpose of the waste mass model, the temporary waste slopes formed between operational Stage 2 landfill area and the southern piggyback liner are proposed/modelled with a benched profile of overall gradient of 1V:3.0H to a maximum height of 20m.

In the case of unconfined (temporary) waste faces, the stability of the unconfined waste mass may be affected by leachate pore pressure in the waste mass. 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 0.1 has been utilised for the historic Stage 1 waste and a r_u value of 0.2 for Stage 2 waste where leachate recirculation has been undertaken and the waste mass to be deposited in the future.

No assessments will be undertaken for ‘no phreatic’ or ‘normal operating conditions leachate heads’, as the landfill base is >20m beneath the lowest tie-in point of the piggyback liner. If the worst-case scenario assessments of the pore-water pressure in the waste mass as a function of the overburden stress are acceptable, adequate factors of safety will be maintained for lower risk scenarios and have therefore been screened out.

The waste shall ultimately be placed in line with the pre-settlement restoration levels at a maximum gradient of approximately 1V:4H slopes on the western side, but much flatter on the eastern and southern flanks of the facility. The global permanent waste profile slopes have been assessed as part of the previously submitted Closure & Post Closure Management Plan[1] and Capping SRA[3] and are screened out of this assessment.

2.4.1.1 *Leachate Collection System*

Leachate collection from the base of landfill will be provided by a leachate collection pipe installed within the gravel leachate drainage blanket. All basal pipework will be designed for a maximum 6%

deflection to resist the static forces of the waste. Leachate will be extracted and monitored using a proprietary HDPE vertical telescopic riser and secondary sideslope risers.

The Marton Geotechnical Services (MGS) vertical telescopic riser is designed to accommodate axial loads during waste settlement caused by 'skin friction' and eliminate axial stress.

The reduced load imposed by waste settlement on the riser and hence the foundation pad, is not considered to warrant any further assessment.

2.5 Capping Screening

A Closure & Post Closure Management Plan[1] was prepared to support a licence amendment (submitted in September 2021) at the Tamala Park WMF. A capping stability risk assessment[3] was prepared to support both the Closure & Post Closure Management Plan and licence amendment.

As capping stability risk has been covered in a separate report, stability of the capping has been screened out of this report.

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/strains 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 temporary waste slopes.
- Finite difference method (FDM) analyses for the determination of potential tension/strains within the geosynthetic lining system.
- Closed-form analyses for the unconfined lining system.

The stability analysis program SLIDE2 (Version 9.038) 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.

Slide2's Auto Refine Search was utilised as the primary search method to define the critical 'circular' slip surfaces for the East-West and North-South sideslopes within SRA. 'The Auto Refine Search method uses a simple but effective algorithm for iteratively refining the search area on the slope, until the critical surface is located'. (RocScience 2016).'

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

TR1[9] states 'circular surfaces are seldom appropriate in the study of landfills, with recorded failures for lined landfill sites defined by Koerner and Soong[10], 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 temporary waste slopes have therefore been undertaken using non-circular analysis for all situations.

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.

FLAC2D, Version 9.00.173 (Itasca Consulting Group Inc, 2022) 2D finite difference method (FDM) analysis software has been used for the geomembrane liner integrity assessment. FLAC2D is a program for general-purpose FDM analysis of geotechnical structures for civil and mining applications.

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

The locations of the typical analysed sections for the southern piggyback liner are shown on Figure 3-1.

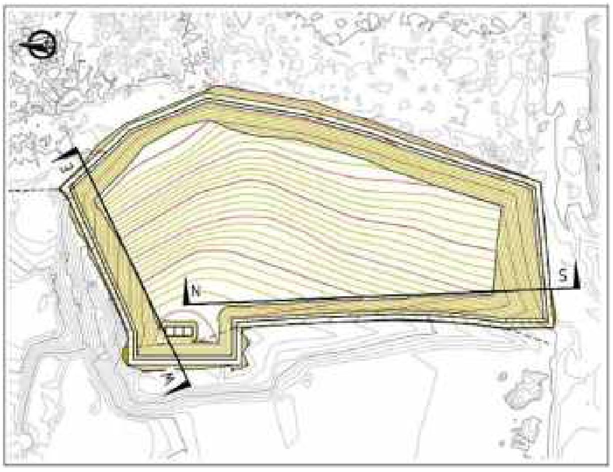

Figure 3-1: Location of East-West (1V:5.5H]) and North-South (1V:3H) sideslopes

3.2 Data Parameters

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

- Material unit weight.
- Drained shear strength, including the interfaces between the geosynthetic components and the waste.

Laboratory testing was undertaken by E-Precision Laboratory and TRI Australasia, in their National Association of Testing Authorities (NATA) accredited laboratory. The results of laboratory characterisation testing are presented in Appendix B and Table 3-1.

Table 3-2 presents a summary of the principal geotechnical characteristics for two sets of soil data. The two soil types tested relate to the proposed materials used on site.

Table 3-1: Summary of Basic Geotechnical Properties

Sample	Plastic Limit (%)	Maximum Dry Density (Mg/m^3)	Optimum Moisture Content (%)	Uniformity Coefficient	D_{10} (mm)
LIMESTONE	10.4	1.82	12.0	20	0.025
SUBSOIL	6.2	1.72	17.0	10	0.04

A triaxial test was conducted on the LIMESTONE which was shown to yield a coefficient of permeability of 4.153x10^{-5} m/s. Using Hazen's Rule, the effective particle size (D_{10}) can be empirically related to the permeability. For the LIMESTONE, the coefficient of permeability is calculated to be 6.25x10^{-6} m/s which is an order of magnitude less than the measured value from the triaxial test.

The measured permeability of the SUBSOIL is 6.609x10^{-5} m/s and by Hazen's Rule produces a value of 1.60x10^{-5} m/s which is 4 times lower.

In considering the material characteristics and assessed performance, the use of a suitably specified underliner gas collection network will be required to manage the potential gas pressure beneath the piggyback liner in terms of interface (veneer) stability.

Table 3-2 presents the results of a series of shear strength tests designed to determine the friction properties of the soils. The testing was undertaken as part of the Capping SRA[3].

Table 3-2: 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)
'Crushed' LIMESTONE	Inherent Strength	45.6	6.9	40.7	0.2
SUBSOIL/ SANDS	Inherent Strength	44.1	19.2	39.8	5.0

Table 3-3 presents the results of a series of tests designed to determine the interface friction properties between the geosynthetics and the soils. They represent the various elements of the piggyback lining system.

Table 3-3: Summary of Interface Friction Testing

Interface	Peak Angle of Shearing Resistance ø' (°)	Peak Effective Cohesion c′ (kPa)	Post Peak Angle of Shearing Resistance ø' (°)	Post Peak Effective Cohesion c′ (kPa)
Geotextile/Geomembrane**	25.1	0	14.6	2
	23.5	3	13.9	3
	26.7	1	14.6	4
	27.5	0	16.4	1
Adopted	**23**	**1**	**14.0**	**2**
Geomembrane**/GCL	28.6	1	16.6	8.2
	27.4	3	14.7	10
	28.8	0.3	10.6*	0
	26.9	3	15.1	8.8
Adopted	**26**	**2**	**13**	**5**
GCL/Subgrade	35.1	13	1.6*	49.4
	34.8	16	14.7	34.7
	34.6	15	29.3	14.7
	31.8	21	17.3	0
Adopted	**30**	**5**	**14**	**1**

Note: Internal shearing reported. **Double textured LLDPE*

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[11] , these values being backed up in later work by Kavazanjian et al[12] and later confirmed in a research summary by Jotisankasa[13]. The values for c' and ø' adopted throughout the modelling were 5kPa and 25°, respectively. It should be appreciated that the shear strength of waste will vary considerably depending on composition and strain. The landfill at Tamala Park will accept primarily municipal wastes so the chosen parameters are considered to be a realistic lower bound values.

Considering the laboratory testing data and literature the following design parameters are used in the slope stability analysis. It should be noted that selected design parameters are lower than the laboratory testing undertaken and are therefore considered a conservative approach.

Table 3-4: Material Parameters

Material	Bulk Unit Weight γ (kN/m³)	Effective cohesion c' (kPa)	Angle of Shearing Resistance ø' (°)	Undrained Shear Strength Su (kPa)	Typical Description
Sand*	18	1	33 (26.4)	>60	Fine Sand
Drainage Aggregate**	18	1	33 (26.4)		20-40mm Aggregate
Engineered Limestone Fill	18	1	35 (28)	>100 (80)	Crushed Limestone Fill
Waste	10	5	25		Mixed Putrescible Waste.
Limestone	18	50 (40)	38 (30.4)		Tamala Limestone

*based on experience from other dune soils
**conservative value based on previous experience

RocScience Slide2 parameters for the proprietary 'Naue 30/30' geogrid have been reduced for interface friction and adhesion to 35° and 5kPa respectively.

The Finite Difference numerical integrity analyses of the liner utilised a plastic constitutive model with a Mohr–Coulomb failure criterion. Values of Young's modulus (E) and Poisson's ratio (v) were selected to represent settlements experienced in municipal landfills, with waste stiffness of E = 0.5MPa, and v =0.3 as utilised as the base conditions within the Jones and Dixon study[14]. For the historic waste, the published data on the self-boring pressuremeter tests in non-hazardous wastes carried out by Jones

[11] Van Impe, W. F. and Bouazza, A., 'Geotechnical properties of MSW', draft version of keynote lecture, Osaka, 1996.
[12] 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.
[13] Jotisankasa, A., 'Evaluating the Parameters that Control the Stability of Municipal Solid Waste Landfills', Master of Science Dissertation, University of London, September 2001.
[14] Jones, D.R.V. and Dixon, N. (2005). 'Landfill lining stability and integrity: the role of waste settlement', Geotextiles and Geomembranes, 23, pp 27-53.

et al[15], are referenced. The study indicated a minimum shear modulus (S) of approximately 1MPa, from which a bulk modulus (K) of approximately 2MPa was calculated, equating to a Young's modulus of around 2.4-2.6MPa. A conservative value of 2MPa was adopted.

The US Bureau of Reclamation Design Standard for Embankment Dams[16] quote typical HDPE geomembrane modulus values to be 65 kip/in^2, equivalent to 484 MPa (from Koerner, R.M., Designing with Geosynthetics, Sixth Edition, Vols. I and II, Xlibris Corporation, United States, 2012).

GRI GM17[17] standard specification stipulates for a 2mm LLDPE, a 2% Modulus (max.) of 840N/mm, equivalent to 420 kN/mm^2 (i.e. 420 kN/m^2 or 420 MPa). There are no modulus values quoted for HDPE in GRI GM13[18]. 'D^2' designer data[19] report a Young's Modulus of 475 MPa for LLDPE, with Ineos reporting the typical engineering properties of HDPE to range between 400 and 1000 kN/m^2, with a Poisson's ratio between 0.40 and 0.45.

Giroud's (1994) [20] research on geomembrane stress-strain curves shows the 2% secant modulus of a HDPE to be over 3.5 times greater than a secant modulus at yield. Merry & Bray (1997) [21] concluded that results from testing high-density polyethylene (HDPE) specimens indicated that the secant modulus and strength decrease considerably at strain rates appropriate for long-term field applications. Fowmes et al. (2007) [22] recommended that 2% modulus values should not be adopted in design as this may result in overestimation of the material stiffness at strains in excess of 2%.

A modulus of 120 MPa has therefore been adopted in the assessment for the geomembrane to account for a degree of conservativism and represent long-term field applications.

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15 Dixon, N., Jones, D.R.V. and Whittle, R.W., (1999), 'Mechanical properties of household waste: in-situ assessment using pressuremeters'. Proc. 7th Int. Waste Management and Landfill Symp. Cagliari.

16 US Department of the Interior, Bureau of Reclamation. Design Standards No. 13. Embankment Dams, Chapter 20: Geomembranes Phase 4 (Final). DS-13(20)-16.1. September 2018.

17 GRI - GM17 Standard Specification. Standard Specification for 'Test Methods, Test Properties and Testing Frequency for Linear Low Density Polyethylene (LLDPE) Smooth and Textured Geomembranes'. Geosynthetic Institute, Revision 14: March 17, 2021.

18 GRI - GM13 Standard Specification, '"Test Methods, Test Properties and Testing Frequency for Linear Low Density Polyethylene (LLDPE) Smooth and Textured Geomembranes', Geosynthetic Institute. Revision 14: March 17, 2021.

19 https://designerdata.nl/materials/plastics/thermo-plastics/linear-low-density-polyethylene

20 Giroud, J.P., Mathematical Model of Geomembrane Stress-Strain Curves with a Yield Peak. Geotextiles and Geomembranes. 13 (1994) 1 22. 1994.

21 Merry, S.M., Bray, J.D., Time-Dependent Mechanical Response of HDPE Geomembranes. J. Geotech. Geoenviron. Eng. 1997.123:57-65. January 1997.

22 Fowmes, G.J., Dixon, N., Jones D.R.V., Validation of a numerical modelling technique for multilayered geosynthetic landfill lining systems. Geotextiles and Geomembranes 26 (2008) 109–121. 2007

Table 3-5: FLAC2D Material Parameters

Materials	Young's Modulus (kPa)	Poisson's Ratio (v)	Friction Angle (degrees)	Cohesion (kPa)	Density (kN/m3)
Sand	40,000[23]	0.3	33	1	18
Engineered 'Crushed Limestone' Fill	250,000	0.3	35	1	18
Limestone	55.1E+6[24]	0.3	38	50	18
Waste (New)	500	0.3	25	5	10
Waste (Historic)	2000	0.3	25	5	10
Geomembrane	120,000	0.45		-	9.39

For the closed form analysis, interface design parameters are presented in Table 3-6, friction angles and cohesion are based on adopted parameters from the laboratory shear box data.

Table 3-6: Closed Form Interface Design Parameters

Interface	Peak		Post Peak	
	c' (kPa)	ø'	c' (kPa)	ø'
Leachate Aggregate/Geotextile*	1	30	24	1
Geotextile/Geomembrane**	1	23	2	14
Geomembrane**/GCL	2	26	5	13
GCL/Engineered Fill 'Crushed Limestone'	5	30	1	14

* conservative value based on previous experience. ** double textured geomembrane

It should be acknowledged that although material/site specific data has been obtained. It will be necessary prior to the construction of the piggyback liner, and preferably before the contract is let, that additional corroboratory testing is undertaken in order to confirm that the above data is representative of the chosen geosynthetic materials.

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'[25], 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

[23] Geotechdata.info, Soil Young's modulus, http://geotechdata.info/parameter/soil-elastic-young-modulus.html (as of September 17.09.2013).
[24] Lake, L. Petroleum Engineering Handbook. Richardson, TX: Society of Petroleum Engineers, 2007. *Lower bound value adopted
[25] ICOLD (International Commission on Large Dams), Selecting Seismic Parameters for Large Dams Guidelines, 2009.

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

Total Risk Factor = RF Capacity + RF Height + RF Evacuation Reqts + RF Potential Downstream Damage
Total Risk Factor = 4 + 2 + 0 + 4 = 10
Total Risk Factor between 7-18 = Risk Class (Risk Rating) II (Moderate)

For a moderate ('significant' Class II risk class) category dam, ANCOLD July 2019 'Guidelines for Design of Dams and Appurtenant Structures for Earthquake[26] 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[27] 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 2023 National Seismic Hazard Assessment for Australia[28] (NSHA23) seismic design values, GIS data[29] indicates that the Site is located midway between the 0.02 and 0.03 Peak Ground Acceleration (PGA) contours for an annual probability of exceedance (AEP) of 1:475 at a Spectral Acceleration (SA) period of 0.0s, as shown on Figure 3-2.

[26] ANCOLD (Australian National Commission on Large Dams), Guidelines for Design of Dams and Appurtenant Structures for Earthquake, July 2019.
[27] Global Industry Tailings Standard on Tailings Management, ICMM, UN Environment Programme, PRI – Principles for Responsible Investment, GlobalTailingsReview.org, August 2020.
[28] 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
[29] https://data.gov.au/dataset/earthquake-hazard-risk-contour-map-national-geoscience-dataset

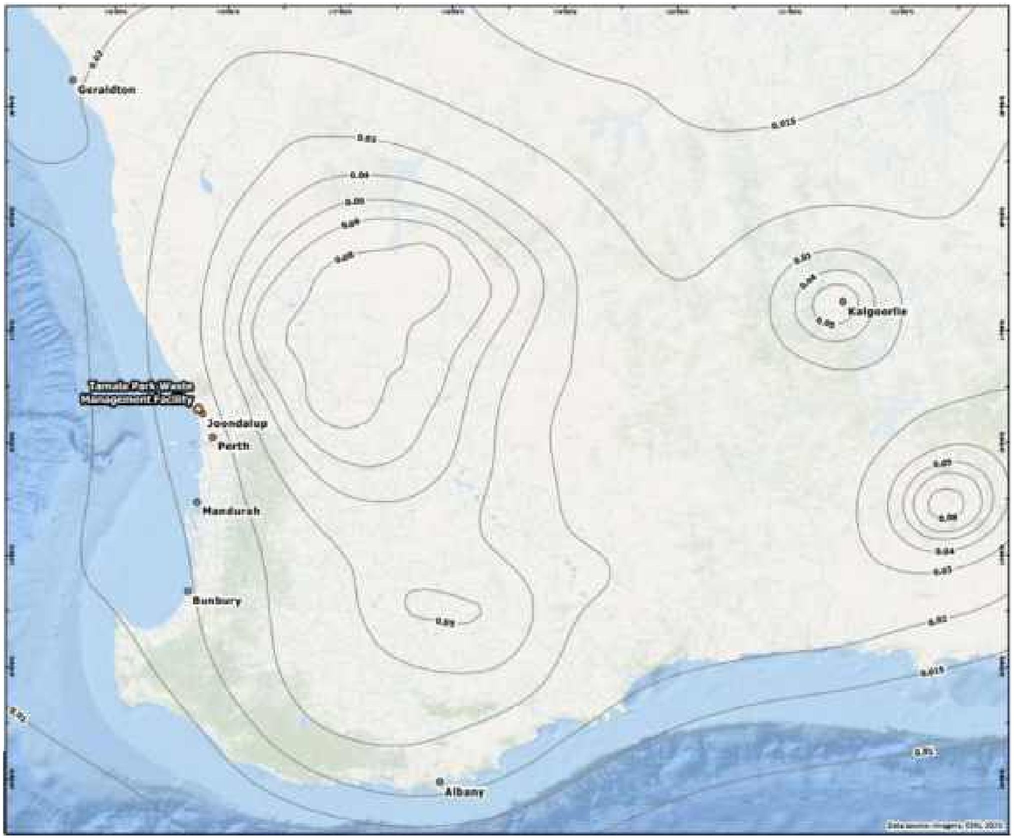

Figure 3-2: NSHA23 – 10% Probability of Exceedance in 50 years (1:475 AEP) contours

The NSHA18[28] seismic design values, GIS data[29] indicates that the Site is located midway between the 0.06 and 0.08 (=0.07g) Peak Ground Acceleration (PGA) contour intervals for an annual probability of exceedance (AEP) of 1:2475 at a Spectral Acceleration (SA) period of 0.0s, as shown on Figure 3-3.

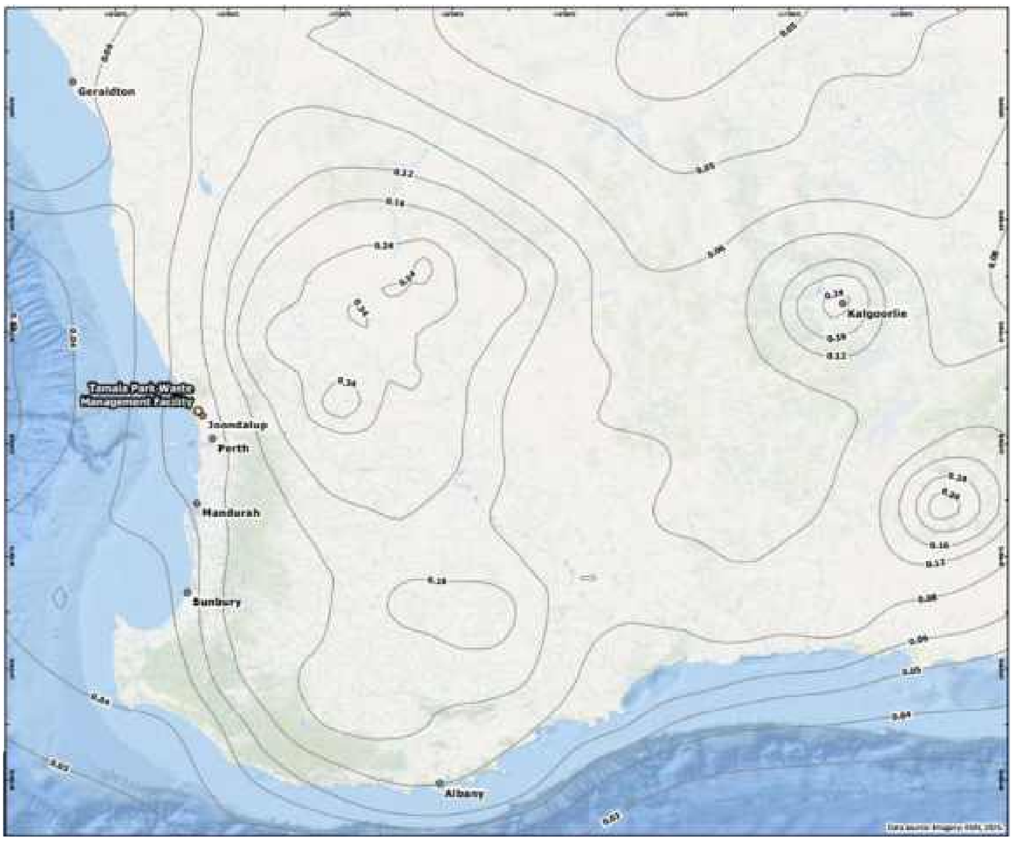

Figure 3-3: NSHA23 – 2% Probability of Exceedance in 50 years (1:2475 AEP) contours

Utilising a logarithmic interpolation between the conservative NHSA23 values of 0.025g and 0.07g for the 1:475 AEP and 1:2475 AEP respectively, a 1:1000 AEP equates to a PGA 0.059g.

AS1170.4[30] identifies the sub-soil class across the site Class C_e – Shallow Soil. The normalised response spectra 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 0.025g with an amplification of 1.3 relates to a horizontal seismic load coefficient of 0.0325g
- SEE/MCE. PGA 0.059g with an amplification of 1.3 relates to a horizontal seismic load coefficient of 0.077g

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

[30] AS1170.4, Australian Standard – 'Structural design actions Part 4: Earthquake actions in Australia'. 2nd Edition 2007.

stability assessments of projects in NSW and interstate. The UK Environment Agency document TRI2[31] states "Slopes should be designed to obtain factors of safety in the region of 1.3 to 1.5".

ANCOLD Guidelines on Tailings Dams[32] 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.

3.5 Modelling Results

3.5.1 Basal & Sideslope Subgrade Analysis

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

The load imposed by the new waste on the historic waste deposits that forms the sideslope subgrade could give rise to settlements that could, in turn, affect the integrity of the lining system. A FLAC2D model was developed to consider the degree of strains likely to develop within the lining system, in particular the LLDPE geomembrane. Deformations in relation to potential strains in the lining system are considered the critical element to be assessed in the report. The analysis is discussed in Section 3.5.7.

The subgrade analysis considered the East-West (1V:5.5H) sideslope to the lower tie-in to Stage 2 liner.

The typical arrangement for the East-West (1V:5.5H) subgrade sideslope is shown on Figure 3-4.

[31] 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

[32] ANCOLD, 'Guidelines on Tailings Dams – Planning, Design, Construction, Operation and Closure', May 2012

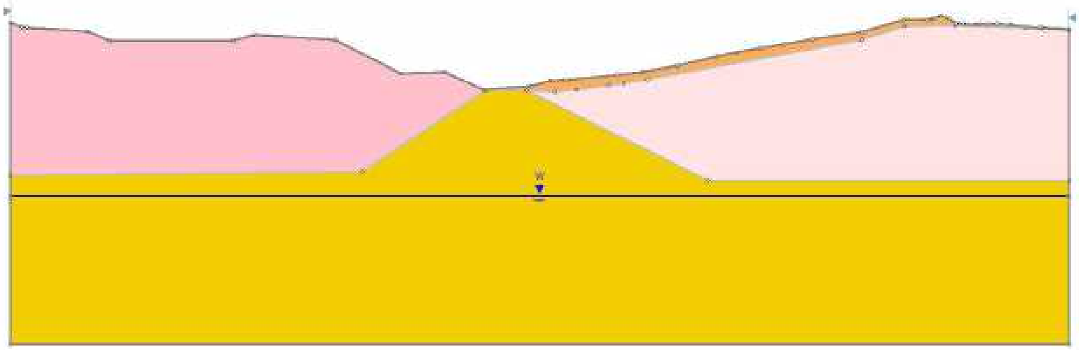

Figure 3-4: Typical East-West 1V:5.5H Subgrade Section

The summary of the circular and non-circular analysis for the East-West 1V:5.5H sideslope subgrade are presented in Table 3-7. Model Scenarios are presented in Appendix C.

Table 3-7: Summary of Stability Analysis for East-West Subgrade Sideslope

Scenario	Method	Factor of Safety	Comments
East-West Sideslope Subgrade No Seismic Loading	Drained Circular	2.364	1V:5.5H Sideslope Acceptable (FoS > 1.5)
East-West Sideslope Subgrade with Seismic Loading (OBE - 1:475 AEP)	Drained Circular	2.112	1V:5.5H Sideslope Acceptable (FoS > 1.1)
East-West Sideslope Subgrade with Seismic Loading (SEE - 1:1000 AEP)	Drained Circular	1.839	1V:5.5H Sideslope Acceptable (FoS > 1.0)
East-West Sideslope Subgrade No Seismic Loading	Drained Non-Circular	2.338	1V:5.5H Sideslope Acceptable (FoS > 1.5)
East-West Sideslope Subgrade with Seismic Loading (OBE - 1:475 AEP)	Drained Non-Circular	2.088	1V:5.5H Sideslope Acceptable (FoS > 1.1)
East-West Sideslope Subgrade with Seismic Loading (SEE - 1:1000 AEP)	Drained Non-Circular	1.819	1V:5.5H Sideslope Acceptable (FoS > 1.0)
East-West Sideslope Subgrade with Weak Interface Seismic Loading (SEE - 1:1000 AEP)	Drained Circular	1.599	1V:5.5H Sideslope Acceptable (FoS > 1.0)
East-West Sideslope Subgrade with Weak Interface Seismic Loading (SEE - 1:1000 AEP)	Drained Non-Circular	1.597	1V:5.5H Sideslope Acceptable (FoS > 1.0)

The analysis also considered the existing temporary 'pvc' cap to be a potential weak interface on the sideslope subgrade for SEE Loading as part of a sensitivity assessment.

The lowest calculated factor of safety for the East-West Subgrade Sideslope are greater than 1.5, 1.1 and 1.0 for the static, OBE and SEE AEP scenarios, and are therefore considered acceptable.

3.5.2 Basal & Sideslope Lining System Analysis

The analysis considered the north-south 1V:3H sideslope to the lower tie-in to Stage 2 liner, and the east-west piggyback liner towards the proposed extraction sumps.

The typical arrangement for the East-West (1V:5.5H) sideslope and North-South (1V:3H) sideslope shown on Figure 3-5 and Figure 3-6 respectively.

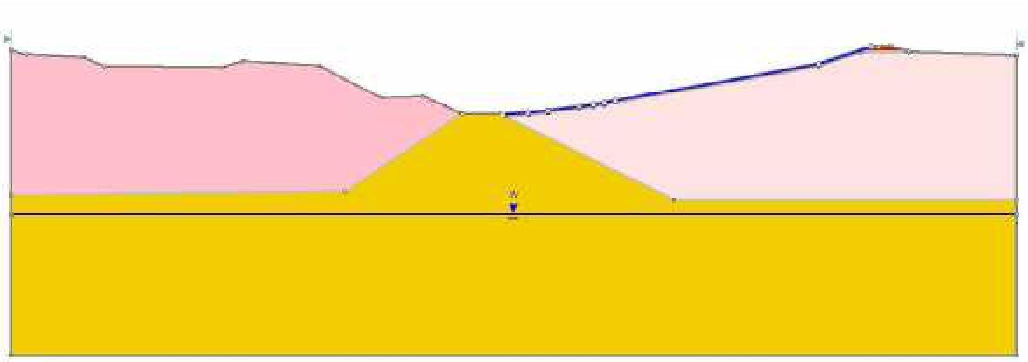

Figure 3-5: Typical East-West 1V:5.5H Section

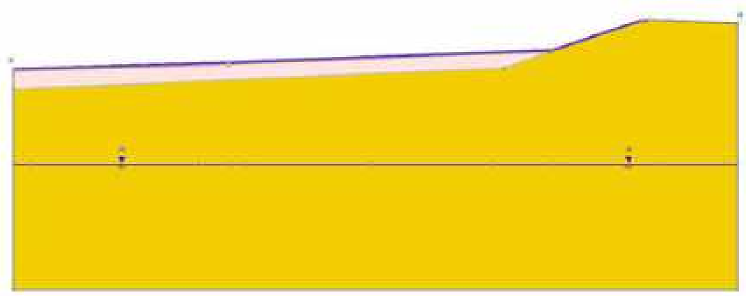

Figure 3-6: Typical North-South 1V:3H Section

The inferred piezometric surface is >25m below the lowest sidelope elevation of the proposed piggyback liner. A r_u value of 0.1 have been used for all materials with the exception of the Stage 2 putrescible waste ($r_u = 0.2$) where leachate recirculation has been conducted. The long-term (drained) condition has been assessed, should the placement of waste against the liner be delayed for some reason. As such, minimum allowable drained and conservative peak effective shear strength parameters are adopted for the various components for the sideslope. It is unlikely that post peak conditions will develop in the sideslope lining system as the waste will progressively buttress the slope in the long term during the phased operational filling.

The summary of the circular and non-circular analysis for the East-West 1V:5.5H sideslope are presented in Table 3-8, with summary of the circular and non-circular analysis of the North-South 1V:3H sideslope presented in Table 3-9. Model Scenarios are presented in Appendix C.

Table 3-8: Summary of Stability Analysis for East-West Sideslope

Scenario	Method	Factor of Safety	Comments
East-West Sideslope No Seismic Loading	Drained Circular	2.233	1V:5.5H Sideslope Acceptable (FoS > 1.5)

Scenario	Method	Factor of Safety	Comments
East-West Sideslope with Seismic Loading (OBE - 1:475 AEP)	Drained Circular	2.009	1V:5.5H Sideslope Acceptable (FoS > 1.1)
East-West Sideslope with Seismic Loading (SEE - 1:1000 AEP)	Drained Circular	1.855	1V:5.5H Sideslope [Grid Search] Acceptable (FoS > 1.0)
East-West Sideslope No Seismic Loading	Drained Non-Circular	2.099	1V:5.5H Sideslope Acceptable (FoS > 1.5)
East-West Sideslope with Seismic Loading (OBE - 1:475 AEP)	Drained Non-Circular	1.893	1V:5.5H Sideslope Acceptable (FoS > 1.1)
East-West Sideslope with Seismic Loading (SEE - 1:1000 AEP)	Drained Non-Circular	1.715	1V:5.5H Sideslope Acceptable (FoS > 1.0)

Table 3-9: Summary of Stability Analysis for North-South Sideslope

Scenario	Method	Factor of Safety	Comments
North-South Sideslope No Seismic Loading	Drained Circular	2.460	1V:3H Sideslope Acceptable (FoS > 1.5)
North-South Sideslope with Seismic Loading (OBE - 1:475 AEP)	Drained Circular	2.217	1V:3H Sideslope Acceptable (FoS > 1.1)
North-South Sideslope with Seismic Loading (SEE - 1:1000 AEP)	Drained Circular	1.948	1V:3H Sideslope Acceptable (FoS > 1.0)
North-South Sideslope No Seismic Loading	Drained Non-Circular	2.299	1V:3H Sideslope Acceptable (FoS > 1.5)
North-South Sideslope with Seismic Loading (OBE - 1:475 AEP)	Drained Non-Circular	2.120	1V:3H Sideslope Acceptable (FoS > 1.1)
North-South Sideslope with Seismic Loading (SEE - 1:1000 AEP)	Drained Non-Circular	2.089	1V:3H Sideslope Acceptable (FoS > 1.0)

The lowest calculated factor of safety for the East-West sideslope and North-South sideslope are greater than 1.5, 1.1 and 1.0 for the static, OBE and SEE AEP scenarios, and are therefore considered acceptable.

3.5.3 Unconfined Liner

The unconfined stability of the lining system, including the leachate drainage layer was assessed via closed form analysis. The closed form analysis utilised the material interface parameters presented in Table 3-6 and soil parameters as presented in Table 3-4. The assessment was undertaken for both peak and post peak conditions for the 1V:3H slope with the drainage aggregate placed to 2m vertical height and the 1V:8H slope with the drainage aggregate placed to 13m vertical height.

The leachate drainage aggregate internal shear strength presented is a lower bound conservative value based on previous experience, with post peak strengths reduced to 80% of peak strength as shown in the parentheses in Table 3-4. Scenarios have been modelled for parallel submerged ratios of 0 and 0.5 assuming dry and partially saturated conditions respectively.

The unconfined liner interface stability assessment is presented in Appendix D, and demonstrates that the unconfined liner interfaces are considered acceptable with regards to FoS and tension. The generation of potential tensions in the geosynthetic lining system are more applicable to the construction activities, these are considered further in Section 3.5.5.

3.5.4 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 of the piggyback. 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[33] has been undertaken, based on the lowest interface shear strengths for the LLDPE geomembrane and GCL for both peak and post peak conditions. The cover soil in this instance is the leachate drainage blanket at 0.3m thickness. For the FoS calculation the post peak cohesion from the interface friction values presented in Table 3-3 has been reduced to 2kPa.

The NSW EPA 'Hazardous Ground Gases'[34] 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 Tamala Park is predominantly municipal solid waste with negligible commercial and industrial and construction and demolition waste. Due to the moderate rainfall and temperate climate, the placed waste would normally be categorised as 'dry'. An underliner gas collection system is proposed to alleviate the potential of gas pressure building up beneath the piggyback lining system, initially by passively venting. There is a current active extraction system across the Stage 1 although it is reported that there are limited yields from this gas field. With reference to first-order decay, the gas yields will be lower in drier climates, and given that Stage 1 was filled from 1991 to 2004 in two phases (south and north), and is therefore past peak yield, the Stage 1 gas pressure is likely to be in the lower range.

For the purpose of the assessment a nominal gas pressure (Ug) of 2 kPa has been utilised, and is considered more conservative that the most likely gas pressure (Ug of 1 kPa) as suggested by Thiel.

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

Monitoring of the underliner gas quality will be undertaken and if sufficient quantities of gas are available the system will be connected into the site wide gas collection system. Once the southern piggyback liner is buttressed with waste the risk of uplift and veneer instability due to gas pressure is removed.

Analysis for the interface assessment with regards to gas pressure upon the piggyback lining system has shown, for the interfaces and the gas pressure considered, that a factor of safety of 3.74 exists for peak and 2.86 for post peak conditions on the 1V:5.5H slope, which is considered acceptable. The interface assessment with regards to gas pressure is presented in Appendix E.

3.5.5 Plant Operations on Geosynthetics

Analysis has been carried out to determine the effects from construction plant on the placement of leachate drainage aggregate on a 1V:5.5H geosynthetic slope. The stability of a 1V:5.5H sideslope under the influence of construction plant operations has been assessed using the procedure proposed by Kerkes[35] and is presented in Appendix F.

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

Dynamic forces from construction activities such as braking and vibrations can induce tensions within the underlying the lining system, if sufficient working offsets are not maintained. The use of peak values would not account for any potential strength reduction or impact from the construction activities, therefore post peak interface shear strengths have been utilised as a conservative approach.

The analysis shows that based on a 1.0m depth of cover soil a factor of safety of 1.48 occurs against rupture of the geomembrane assuming the lowest post peak shear strength conditions (14.0° & 1 kPa) at the GCL to subgrade 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 D6R LGP bulldozer. The calculated factor of safety is above 1.3 which is considered acceptable.

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 0.79 which is clearly unacceptable. A factor of safety of 1.3 is not achieved until the initial layer thickness is at least 720mm. Of course, if the placement conditions change then additional calculations will be essential.

For slopes that are shallower than 1V:5.5H, the FoS would be inherently higher.

In order to minimise the potential of tension in the underlying geosynthetic lining system, it is recommended that the leachate drainage blanket is placed/spread upslope to the 300mm thickness with the aid of an excavator off 1m thick temporary access roads.

No assessment of the placement of the 300mm drainage aggregate/protection soils on the 1V:3H southern slope has been undertaken it is assumed that this will be undertaken incrementally with the aid of an excavator as horizontal waste lifts progress.

3.5.6 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 benched temporary waste slopes (max. height 20m). In the case of unconfined (temporary) waste faces, the stability of the unconfined waste

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

mass may be affected by any potential leachate head within the waste that could increase pore fluid pressure. For the purpose of the stability assessment an inferred filling profile has been utilised.

As described in Section 1.6.6, for the purpose of the assessment a r_u value of 0.1 has been utilised to represent the potential effect that leachate and gas that could increase pore fluid pressure within the Stage 1 historic unlined waste. A r_u value of 0.2 was utilised for the Stage 2 older waste where recirculation has been undertaken and the Stage 2 new waste yet to be to be deposited.

The temporary waste slope was also assessed with a r_u value of 0.3 for the Stage 2 existing and new waste as part of a sensitivity assessment for the SEE seismic loading scenario.

The final presettlement permanent waste slopes (approximately 1V:5H) for the East-West section were considered for static, OBE and SEE scenarios.

The Waste Mass Stability summary is presented in Table 3-10.

Table 3-10: Waste Mass Analysis Results

Scenario	Method	Factor of Safety	Comments
Temporary Waste Slope No Seismic Loading	Drained Non-Circular	1.409	1V:3.0H benched waste slope. r_u value of 0.2 applied to new waste. Acceptable (FoS > 1.3)
Temporary Waste Slope Seismic Loading (OBE - 1:475)	Drained Non-Circular	1.307	1V:3.0H benched waste slope. r_u value of 0.2 applied to new waste. Acceptable (FoS > 1.1)
Temporary Waste Slope Seismic Loading (SEE - 1:1000 AEP)	Drained Non-Circular	1.188	1V:3.0H benched waste slope. r_u value of 0.2 applied to new waste. Acceptable (FoS > 1.0)
Temporary Waste Slope Seismic Loading (SEE - 1:1000 AEP)	Drained Non-Circular	1.074	1V:3.0H benched waste slope. r_u value of 0.3 applied to new waste. Acceptable (FoS > 1.0)
Restoration Waste Slope No Seismic Loading	Drained Non-Circular	2.401	Approx. 1V:5.0H restoration waste slope. r_u value of 0.2 applied to new waste. Acceptable (FoS > 1.3)
Restoration Waste Slope Seismic Loading (OBE - 1:475)	Drained Non-Circular	2.213	Approx. 1V:5.0H restoration waste slope. r_u value of 0.2 applied to new waste. Acceptable (FoS > 1.1)

Restoration Waste Slope Seismic Loading (SEE - 1:1000 AEP)	Drained Non-Circular	1.843	Approx. 1V:5.0H restoration waste slope. r_u value of 0.2 applied to new waste. Acceptable (FoS > 1.0)

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

It should be acknowledged that due to the temporary status for the slope and as such, not being a permanent condition the inclusion of the 1:1000 AEP is perhaps regarded as highly conservative because of the unlikelihood that such a severe seismic event would occur in such a short time frame.

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

3.5.7 Liner Integrity Assessment

The future waste will be separated from the underlying subgrade by a composite lining system. The key issue for the proposed lining system in this area will be development of strains within the lining system due to loading and settlement of the overlying and underlying waste mass.

The waste mass was modelled at the final waste landform pre-settlement waste heights. The pre-settlement waste landform would likely impose the maximum forces on the composite lining system.

A FLAC2D finite difference model was developed to consider the degree of strains likely to develop within the lining system, in particular the LLDPE geomembrane.

The FLAC2D grid was fixed on the extremities in the x and y directions as they represent non-moveable boundaries. The area of interest was the side slope lining system. The waste mass was modelled directly upon the interface, which in turn overlies the 0.5m thick engineered limestone fill supporting layer. The waste landform in each scenario is deemed to represent a conservative scenario (full height) as during operations the waste will be placed in horizontal layers from the bottom up over an extended time-period incrementally loading the liner.

The geomembrane 'liner element' was modelled as a standard beam within the FLAC2D model with a Youngs Modulus of 120 MPa.

The two scenarios modelled with respect to the waste filled slopes represent the sections East-West along the approximate 1V:8H slope and North-South along the 1V:3H sideslope as per the limit equilibrium modelling, with waste fill topographic levels in general accordance with the final fill profile as presented on Drawing C-101 in Appendix A.

The outputs relating to the FLAC2D analysis are presented in Appendix H. The graphical outputs of the modelling present the material zones, displacement, displacement vectors and the 'Beam X- Force'. The FLAC2D 'Beam X-Force' represents the axial force on the beam structural element. The axial tensile force is represented by positive values while axial compressive force is represented by negative values, (SI unit of Newtons). The results of the scenarios modelled with regards to the liner integrity are presented in Table 3-11.

Table 3-11: FLAC2D Liner Integrity Assessment Results

Scenario	Max. Axial Force on Liner (kN)	Max. Axial Stress (kN/m2)	Axial Strain (%)
East-West Slope	7.33	3665	3.05
North-South Slope	1.08	540	0.45

The axial strain was calculated from the relationship of the maximum axial force on the liner and the geomembrane tensile modulus i.e. $\varepsilon_x = \frac{\sigma_x}{E}$ where E is the Modulus of Elasticity.

For example, for the East-West Slope with a 2mm LLDPE Geomembrane (0.002m) and Modulus of Elasticity of 120,000 kPa (120,000 kN/m^2) as per FLAC2D material parameters in Table 3-5.

$Axial\ Stress\ \sigma_x = \ Axial\ force/unit\ area$, thus

$Axial\ Stress\ \sigma_x = \ 7.33/(1\ x\ 0.002)$

$$\text{Axial Strain}\ \varepsilon_x = \frac{(7.33/(1\ x\ 0.002)}{120{,}000},\quad \varepsilon_x = \frac{3665}{120{,}000},\quad \varepsilon_x = \ 3.05\%$$

Minimum average tensile strength at break for a 2.0mm LLDPE geomembrane, in accordance with GRI GM17[18] standard specification is 21kN/m. The levels of tension indicated by the FLAC2D integrity analysis for the geomembrane component of the lining system for all scenarios are insufficient to allow the yield/break strength of the geosynthetic to be exceeded.

Maximum allowable strains for various geomembrane materials as specified in Victorian EPA Best practice environmental management, 'Siting, design, operation and rehabilitation of landfills'[36] and NSW EPA Environmental Guidelines: Solid Waste Landfills [37] were derived from Peggs (2003)[38]. The maximum allowable strains for LLDPE density<0.935 g/cm^3, LLDPE density.>0.935 g/cm^3, LLDPE randomly textured and LLDPE structured profile are 12%, 10%, 8% and 10% respectively.

Peggs (2003)[38] stated the measurement of strain is used as an indirect measure of the stress that exists in a geomembrane that might result in stress cracking. The objective of specifying these maximum allowable strain values is to limit the in-service stress to a sub-critical value where stress cracking will not be a problem in practice. It should be noted that LLDPE is not normally as susceptible to environmental stress cracking as HDPE.

The finite difference FLAC2D liner integrity analysis demonstrate that strains as presented in Table 3-10 are significantly lower than the maximum allowable LLDPE geomembrane strain values and are therefore considered acceptable.

3.6 Capping Stability Analysis

A Closure & Post Closure Management Plan[1] was prepared to support a licence amendment (submitted in September 2021) at the Tamala Park WMF. A capping stability risk assessment[3] was prepared to support both the Closure & Post Closure Management Plan and licence amendment.

As capping stability risk has been covered in a separate report, stability of the capping has been screened out of this report.

[36] Best practice environmental management. Siting, design, operation, and rehabilitation of landfills. EPA Victoria. Publication 788.3. August 2015.
[37] Environmental Guidelines: Solid Waste Landfills, NSW EPA, Second edition 2016.
[38] Peggs ID (2003). Geomembrane liner durability: Contributing factors and the status quo. Proceedings 1st UK National Geosynthetics Symposium, Nottingham, UK, pp.1-26.

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 using post peak shear strength to allow for strain weakening during earthquake loading.

All 'soil' materials employed within the southern piggyback liner construction are considered to be 'granular' and as such, undrained conditions are not deemed appropriate for modelling. A sensitivity analysis was undertaken with reduced drained shear strength parameters (as shown in the parentheses in Table 3-4), for the lowest FoS calculated on the North-South Sideslope under SEE seismic loading.

The results of the sensitivity results are presented in Table 3-12.

Table 3-12: North-South 1V:3H Slope Sensitivity Summary

Scenario	Method	Factor of Safety	Comments
North-South Sideslope SEE	Drained Non-Circular	1.420	1V:3H sideslope for circa 9m Reduced Shear Strength Values Acceptable (FoS > 1.0)
North-South Sideslope SEE	Drained Circular	1.564	1V:3H for sideslope circa 9m Reduced Shear Strength Values Acceptable (FoS > 1.0)

The analyses demonstrate acceptable factors of safety for drained conditions with reduced strength parameters with a seismic loading up to an AEP of 1:1000.

Analyses are presented in Appendix I.

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'[39] 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[40] 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

[39] RCRA Subtitle D (258) 'Seismic Design Guidance for Municipal Solid Waste Landfill Facilities'. US EPA, EPA600/R-95/051, April 1995.
[40] 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

facilities are generally easy to detect and repair. Generic allowable calculated seismic displacements for cover systems are documented to be 300mm to 1m.

All limit equilibrium FoS calculated during the seismic conditions assessed are in excess of the minimum values for both peak and post peak scenarios and therefore deemed acceptable.

3.8.2 Basal & Sideslope Assessment

The lowest topographic levels are approximately 27m above the inferred maximum potentiometric head across the site, therefore the risk of basal heave is not considered a viable failure mechanism.

The stability of the side slope subgrade has been analysed, and acceptable factors of safety have been determined.

The build-up of gas pressure from the landfill is relevant to the lining of existing waste slopes of the piggyback. Analysis for the interface assessment with regards to gas pressure upon the piggyback lining system has shown, for the interfaces and the gas pressure considered, the factors of safety are considered acceptable.

Analysis has been carried out to determine the effects from construction plant on the placement of leachate drainage aggregate on the 1V:5.5H geosynthetic slopes. It is assumed that the leachate drainage blanket is spread upslope as per normal good practice to prevent tension/damage within the lower geosynthetics. The analysis has shown that a minimum of 720mm of leachate aggregate is required to maintain a factor of safety of 1.3. It is however recommended that in order to minimise the potential of tension in the underlying geosynthetic lining system, that the leachate drainage blanket is placed/spread upslope to the 300mm thickness with the aid of an excavator off 1m thick temporary access roads.

No assessment of the placement of the 300mm drainage aggregate/protection soils on the 1V:3H southern slope has been undertaken it is assumed that this will be undertaken incrementally with the aid of an excavator as horizontal waste lifts progress.

The long-term stability of the side slope has been analysed, and acceptable factors of safety have been determined.

The finite difference FLAC2D liner integrity analysis demonstrate that strains developed in the LLDPE geomembrane from the waste mass modelled are significantly lower than the maximum allowable LLDPE geomembrane strain values and are therefore considered acceptable.

3.8.3 Waste Mass Stability

Placement of waste onto the southern piggyback will be undertaken in horizontal layers. The maximum future temporary waste faces are considered to be located on the existing Stage 2 landfill area while the southern piggyback liner is constructed. For the purpose of the waste mass model, the temporary waste slopes formed between operational Stage 2 landfill area and the southern piggyback liner are proposed/modelled with a benched profile to a maximum height of approximately 20m. The limit equilibrium analyses for the waste mass modelling have been undertaken using the Spencer and Morgenstern-Price non-circular forms of analysis.

In the case of unconfined (temporary) waste faces, the stability of the unconfined waste mass may be affected by increased pore pressures therefore, for the purpose of the assessment the pore-water pressure in the waste mass as a function of the overburden stress has been adopted to represent the potential effect that leachate and gas that could increase pore fluid pressure within the waste. A r_u

value of 0.1 has been utilised to represent the potential effect that leachate within the Stage 1 historic unlined waste. A r_u value of 0.2 has been utilised for the waste mass to be deposited in the future and for the Stage 2 waste where leachate recirculation has been undertaken.

The temporary waste slope was also assessed with a r_u value of 0.3 for the Stage 2 existing and new waste as part of a sensitivity assessment for the SEE seismic loading scenario.

The final presettlement permanent waste slopes (approximately 1V:5H) were considered for static, OBE and SEE and are considered acceptable.

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

The calculated factor of safety for the temporary waste slopes are considered acceptable.

3.8.4 Capping Assessment

A Closure & Post Closure Management Plan was prepared to support a licence amendment (submitted in September 2021)[1] at the Tamala Park WMF. A capping stability risk assessment[3] was prepared to support both the Closure & Post Closure Management Plan and licence amendment.

As capping stability risk has been covered in a separate report, stability of the capping has been screened out of this report.

3.8.5 Wind Uplift

Geosynthetics should not be left open over prolonged periods of time and exposed to inclement weather. Surcharging of the geosynthetics as part of best practice lining will need to be considered during construction activities. The majority of the piggyback liner will be covered/surcharged with a leachate drainage aggregate.

It is good practice to cover exposed geosynthetics progressively. However, the covering of the geosynthetics on the 1V:3H slope will be limited to the rate of rise of the adjacent waste lifts. Semi-permanent roped sandbag lines are therefore recommended to be installed on the exposed geosynthetics on the sideslope along the adjacent geotextile panel seams and should be monitored monthly. Should the sandbags degrade, further surcharging of the slope will be required (with additional sandbags). The weather should be monitored during the operations and if significant periods of wind are forecast then the exposed geosynthetics should be adequately surcharged.

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 construction works. Limited laboratory testing has been undertaken from current construction materials and geosynthetics. If the specific type and quantity of material is not available on site, then alternative designs will need to be assessed, or the stability assumptions reviewed. The most critical aspect of the piggyback is the interface friction properties between the geosynthetic layers. It is crucial that a comprehensive range of laboratory 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 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 proposed southern piggyback landfill development area. These conditions cannot be extrapolated across any other portion of the Site.

No Stage 1 basal elevation prior to landfilling has been made available. There has been no leachate monitoring data made available throughout the disposal area therefore true leachate levels and degree of saturation of the waste deposits are unknown. However, a conservative approach to fluid pressure within the waste has been adopted.

Groundwater elevation beneath the landfill development area has been inferred from the perimeter boreholes, continual monitoring should be undertaken to ensure that assumptions made in this assessment remain valid.

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 development approvals and the assessment and the classification stated should not be regarded as a final engineering design.

APPENDIX A
Drawings

Figure 1: Site Locality

Drawing W-100: Existing Site Layout and Topography

Drawing W-102: Formation Levels

Drawing W-201: Long Section A

Drawing W-202: Long Section B

Drawing C-101: Final Waste Fill Profile

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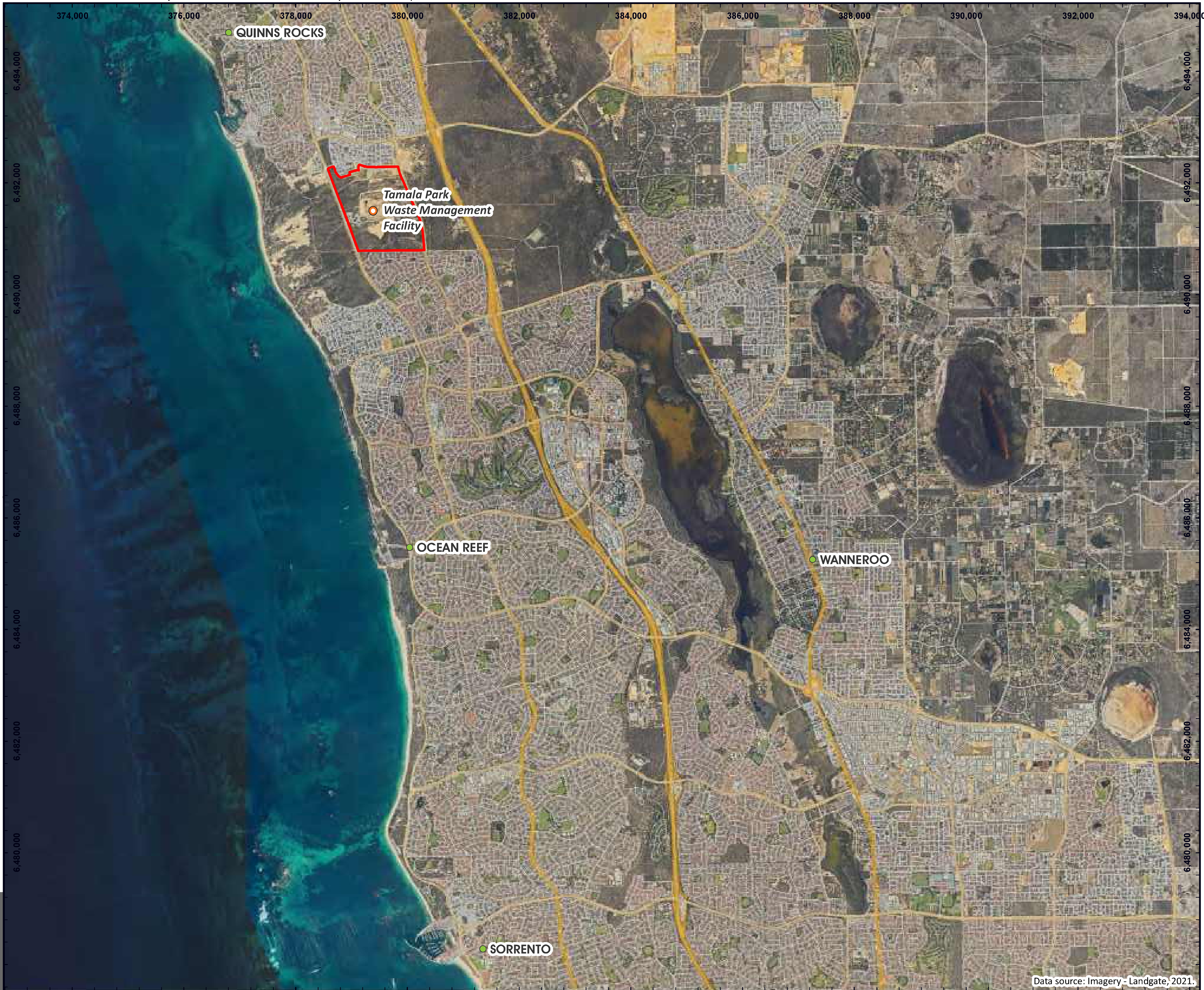
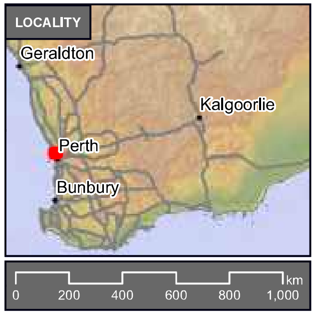


LEGEND

- Site Boundary
- Freeway & Hiighways
- Main Roads
- Minor Roads
- Other

LOCALITY



LOCALITY

Tamala Park
Stability Risk Assessment

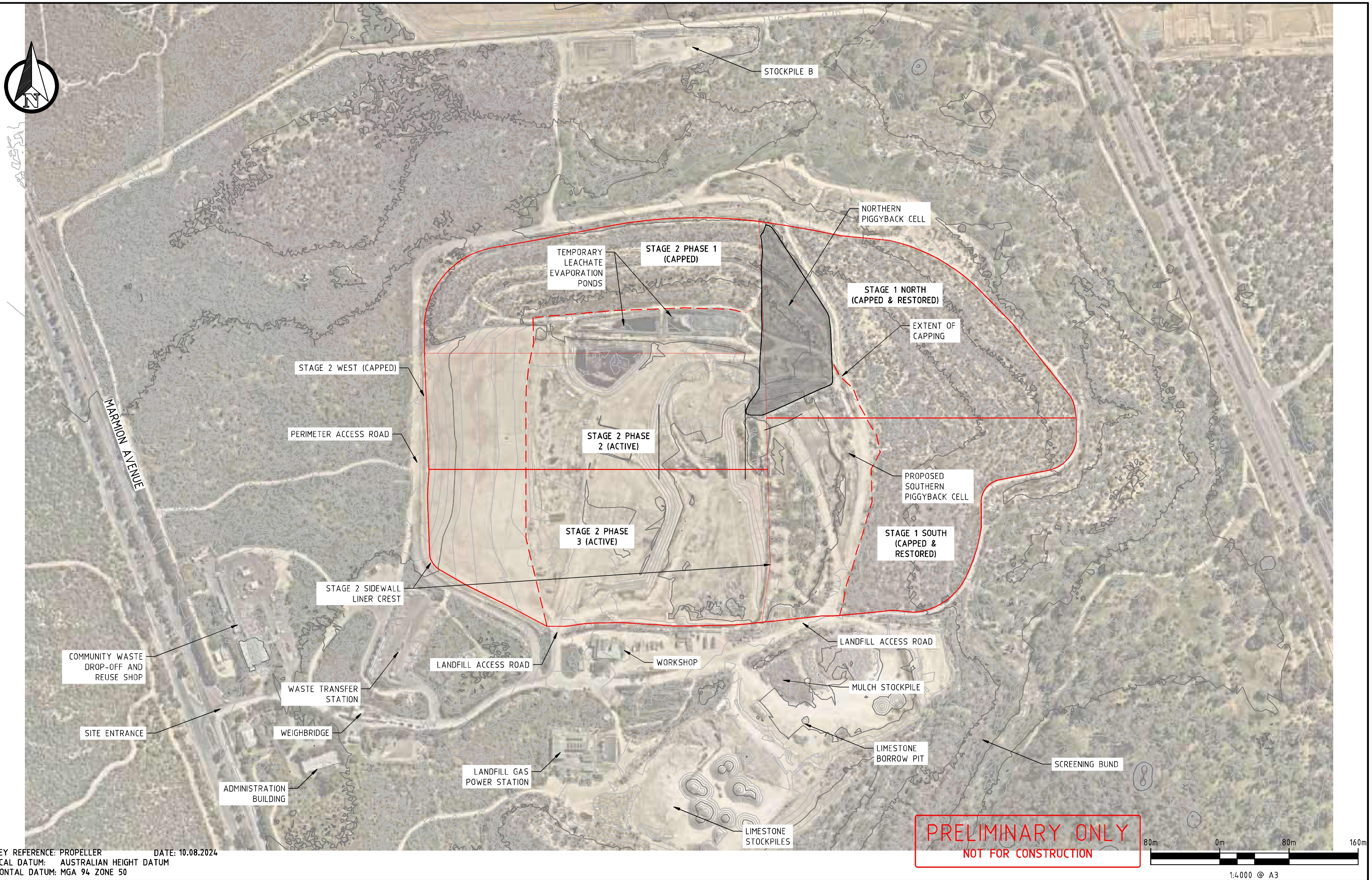
Mindarie Regional Council

0 500 1,000 1,500 2,000 2,500 Metres

Coordinate System: GDA 1994 MGA Zone 50
Projection: Transverse Mercator, Datum: GDA 1994
Scale @ A3: 1:64,660

Prepared:	N Johnston	Date:	23/06/2021
Reviewed:	C Stannard	Revision:	A
Project:	TW21026		

Figure 01



PRELIMINARY ONLY
NOT FOR CONSTRUCTION

SURVEY REFERENCE: PROPELLER DATE: 10.08.2024
VERTICAL DATUM: AUSTRALIAN HEIGHT DATUM
HORIZONTAL DATUM: MGA 94 ZONE 50

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Client: MINDARIE REGIONAL COUNCIL

NOTES

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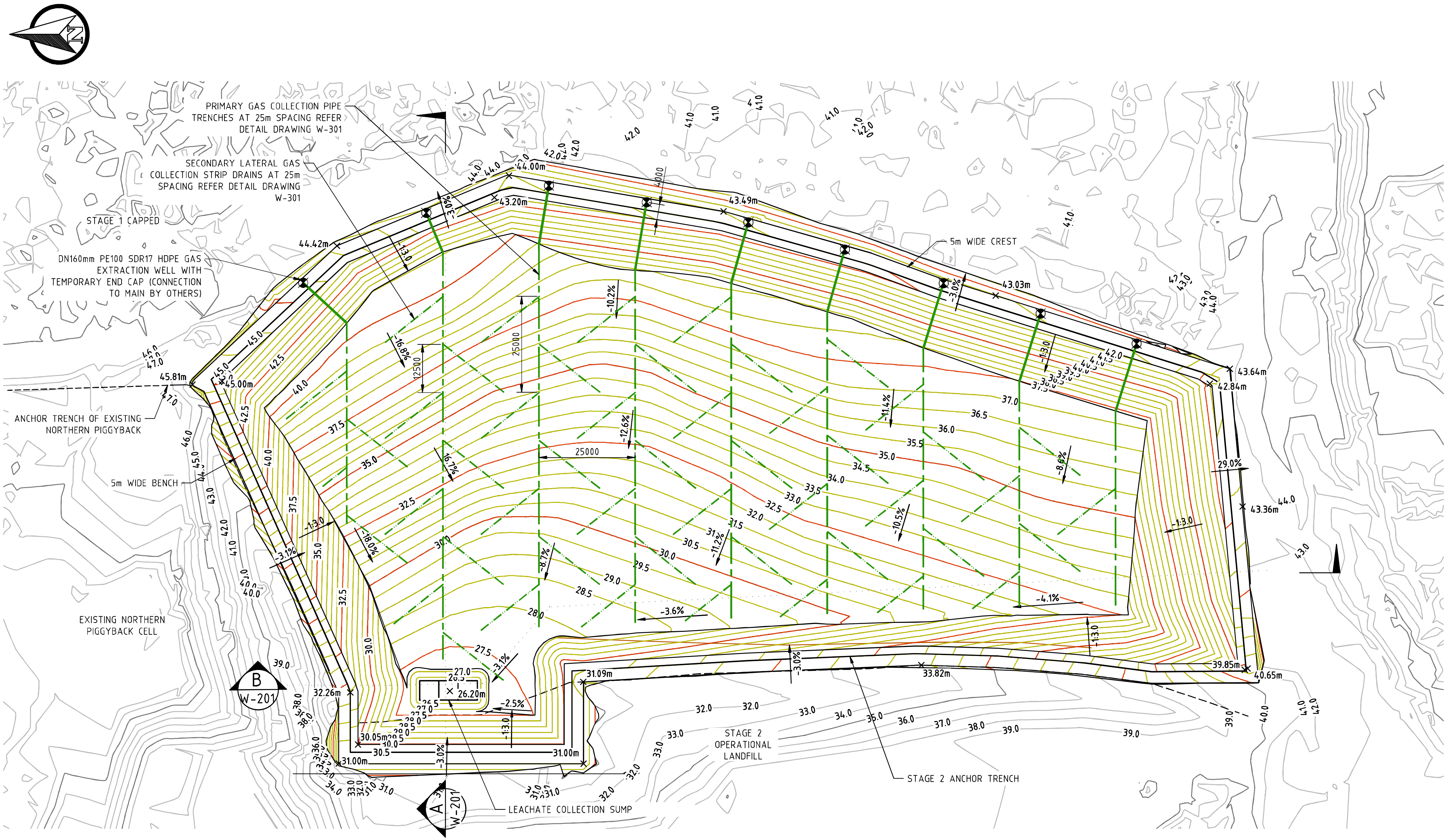
No.	Date	Drwn.	Chk.	Amendment / Issue	App.
A	04.11.2024	YJ	AB	PRELIMINARY ISSUE	

Project: TAMALA PARK SOUTHERN PIGGYBACK LANDFILL CELL

Title: EXISTING SITE LAYOUT & TOPOGRAPHY

Scale: AS SHOWN @ A3 Date: 04.11.2024
Drawn: YJ Checked: AB Approved: CS
Job No: TW23015 Drg. No: W-100 Rev: A
Filename: TW21026 - PIGGYBACK.DWG

Printed by Armand Bester on 07.11.2024 11:03 AM



FORMATION LEVELS AND SUB-CELL GAS COLLECTION LAYOUT

SCALE: 1:1000

LEGEND:

- GAS WELL DN 160mm SOLID HDPE PIPE
- PRIMARY GAS COLLECTION PIPE DN160mm PERFORATED PIPE
- SECONDARY LATERAL GAS COLLECTION STRIP DRAIN
- GAS WELL

PRELIMINARY ONLY
NOT FOR CONSTRUCTION

20m 0m 20m 40m
1:1000 @ A3

SURVEY REFERENCE: PROPELLER DATE: 10.08.2024
VERTICAL DATUM: AUSTRALIAN HEIGHT DATUM
HORIZONTAL DATUM: MGA 94 ZONE 50

talis consultants
www.talisconsultants.com.au T: 1300 251 070

Client: MINDARIE REGIONAL COUNCIL

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No.	Date	Drwn.	Chk.	Amendment / Issue	App.
A	04.11.2024	YJ	AB	PRELIMINARY ISSUE	

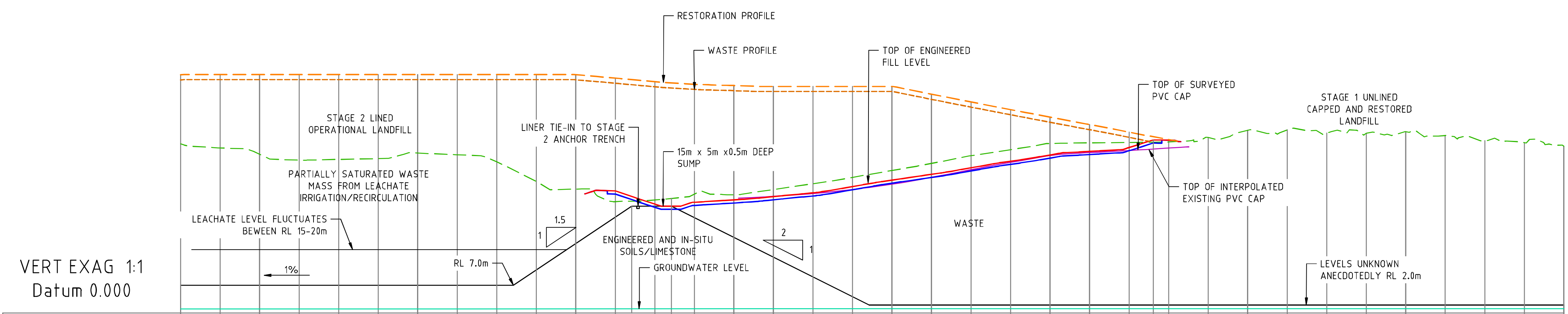
Project: TAMALA PARK SOUTHERN PIGGYBACK LANDFILL CELL

Title: FORMATION LEVELS AND SUB-CELL GAS COLLECTION LAYOUT

Scale: AS SHOWN @ A3 Date: 04.11.2024
Drawn: YJ Checked: AB Approved:
Job No: TW23015 Drg. No: W-102 Rev: A
Filename: TW21026 - PIGGYBACK.DWG

FILENAME: Y:\SECTIONS\WASTE\PROJECTS\TW2021\TW21026 - TAMALA PARK MRC CLOSURE PLAN\DRAWINGS\DRAWINGS\TW21026 - PIGGYBACK.DWG

Printed by Armand Bester on 07.11.2024 11:03 AM



CHAINAGE	EXISTING LEVELS	EXISTING PVC CAP LEVELS	COVER DEPTH	ENGINEERED FILL LEVELS	FORMATION LEVELS	LANDFILL BASE LEVELS	GROUNDWATER LEVELS	WASTE LEVELS	CAPPING LEVELS
0.000	42.775					6.990			
10.000	41.740					6.991	1.032	59.000	60.300
20.000	40.132					6.992	1.033	59.000	60.300
30.000	38.643					6.994	1.035	59.000	60.300
40.000	38.829					6.995	1.036	59.000	60.300
50.000	39.029					6.996	1.037	59.000	60.300
60.000	40.435					6.997	1.038	59.000	60.300
70.000	39.993					6.998	1.039	59.000	60.300
80.000	38.224					7.000	1.040	59.000	60.300
90.000	33.257					10.881	1.041	59.000	60.300
100.000	31.324					17.547	1.043	58.960	60.260
102.313				30.105	30.105			58.684	59.984
110.000	28.144			30.850	30.050	24.214	1.044	58.091	59.391
120.000	28.556			27.531	26.731	27.000	1.045	57.191	58.491
	28.947			27.000	26.200			124.179	
130.000	30.058			28.000	27.200	24.094	1.046	56.532	57.832
140.000	29.894			28.606	27.806	19.093	1.047	56.214	57.514
150.000	31.211	29.483	-2.331	29.410	28.610	14.092	1.048	56.000	57.300
160.000	32.670	30.219	-2.028	30.381	29.581	9.091	1.050	56.000	57.300
170.000	34.272	31.210	-2.330	31.952	31.152	4.090	1.051	56.000	57.300
180.000	36.017	32.551	-3.040	33.607	32.807	2.000	1.053	56.000	57.300
190.000	37.832	34.293	-3.365	35.053	34.253	2.000	1.054	54.014	55.314
200.000	39.748	36.228	-3.995	36.706	35.906	2.000	1.055	52.050	53.350
								51.066	52.366
210.000	41.279	38.087	-3.188	38.375	37.575	2.000	1.057	50.120	51.420
220.000	42.858	39.786	-3.691	39.951	39.151	2.000	1.058	48.220	49.520
230.000	43.004	40.558	-2.386	40.856	40.056	2.000	1.060	46.342	47.642
240.000	43.115	41.089	-1.940	41.827	41.027	2.000	1.061	44.427	45.727
	43.183	41.455	-1.728	43.751	42.951	2.000	1.061	246.101	
250.000	43.249	41.702	-1.392	43.673	43.676	2.000	1.062		43.881
253.043				43.301	43.301				
260.000	44.376					2.000	1.063		
270.000	46.216					2.000	1.065		
280.000	46.300					2.000	1.066		
290.000	45.507					2.000	1.067		
300.000	45.445					2.000	1.069		
310.000	45.813					2.000	1.071		
320.000	44.865					2.000	1.072		
330.000	43.581					2.000	1.073		
340.000	43.059					2.000	1.075		
350.000	42.187					2.000	1.076		

A W-102 LONG SECTION
SCALE: 1:1000

PRELIMINARY ONLY
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1:1000 @ A3 VERT
1:1000 @ A3 HORIZ
20m 0m 20m 40m

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www.talisconsultants.com.au T: 1300 251 070

Client: MINDARIE REGIONAL COUNCIL

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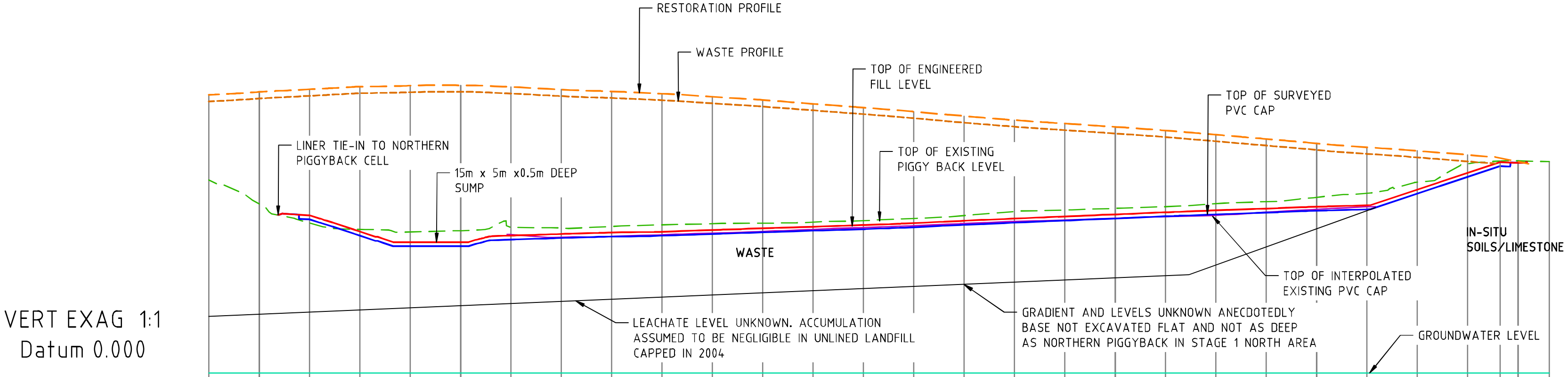
No.	Date	Drwn.	Chk.	Amendment / Issue	App.
B	18.09.2024	AB	CS	DESIGN UPDATED	
A	16.07.2024	YJ	AB	PRELIMINARY ISSUE	

Project: TAMALA PARK SOUTHERN PIGGYBACK LANDFILL CELL

Title: LONG SECTIONS SHEET 1 OF 2

Scale: AS SHOWN @ A3 Date: 16.07.2024
Drawn: YJ Checked: AB Approved:
Job No: TW23015 Drg. No: W-201 Rev: B
Filename: TW21026 - PIGGYBACK.DWG

FILENAME: Y:\SECTIONS\WASTE\PROJECTS\TW2021\TW21026 - TAMALA PARK MRC CLOSURE PLAN\DRAWINGS\DRAWINGS\TW21026 - PIGGYBACK.DWG



EXISTING LEVELS	39.322	34.572	30.800	29.520		29.014	29.259		30.568	29.863	30.244		29.986	30.490		30.566	30.872	31.182	31.483	32.511	32.933	33.603	33.672	34.265	34.536		35.486	35.802	36.633	38.861	42.543	43.073	43.108	42.960
EXISTING PVC CAP LEVELS									28.514	27.970	28.167		28.466	28.796		29.121	29.511	29.902	30.250	30.686	31.103	31.386	31.735	32.052	32.312		32.926	33.531	33.994					
COVER DEPTH									-2.054	-1.893	-2.077		-1.520	-1.694		-1.445	-1.361	-1.280	-1.232	-1.824	-1.830	-2.216	-1.937	-2.213	-2.223		-2.560	-2.271	-2.638					
ENGINEERED FILL LEVELS			32.301	29.076		27.000	27.000		28.293	28.608	28.896		29.057	29.404		29.731	30.055	30.377	30.768	31.237	31.661	32.077	32.493	32.909	33.284		33.654	34.013	34.284	37.391	40.705	42.848	42.736	
FORMATION LEVELS			31.501	28.276		26.200	26.200		27.493	27.808	28.096		28.257	28.604	28.658	28.931	29.255	29.577	29.968	30.437	30.861	31.277	31.693	32.109	32.484	32.566	32.854	33.213	33.484	36.591	39.905	42.048	42.734	
GROUNDWATER LEVELS		1.050	1.049	1.047	1.047	1.046	1.045	1.044	1.044	1.044	1.045	1.045	1.043	1.042		1.041	1.040	1.039	1.038	1.037	1.037	1.036	1.035	1.035	1.034		1.034	1.033	1.032	1.031	1.031	1.031	1.031	1.031
WASTE LEVELS		55.470	55.998	56.515		56.711	56.781		56.460	55.940	55.585		55.131	54.502 54.504		53.872	53.119	52.364	51.560	50.680	49.880	49.205	48.543	47.825	46.994 46.887		46.141	45.282	44.499	43.796	42.993	256.466		
CAPPING LEVELS		56.770	57.298	57.815		58.011	58.081		57.760	57.240	56.885		56.431	55.802 55.804		55.172	54.419	53.664	52.860	51.980	51.180	50.505	49.843	49.125	48.294 48.187		47.441	46.582	45.799	45.096	44.293		42.956	
CHAINAGE	0.000	10.000	20.000	30.000		40.000	50.000		60.000	70.000	80.000		90.000	100.000		110.000	120.000	130.000	140.000	150.000	160.000	170.000	180.000	190.000	200.000		210.000	220.000	230.000	240.000	250.000		260.000	266.199

B LONG SECTION
W-102

SCALE: 1:1000

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1:1000 @ A3 VERT
1:1000 @ A3 HORIZ
20m 0m 20m 40m

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Client: MINDARIE REGIONAL COUNCIL

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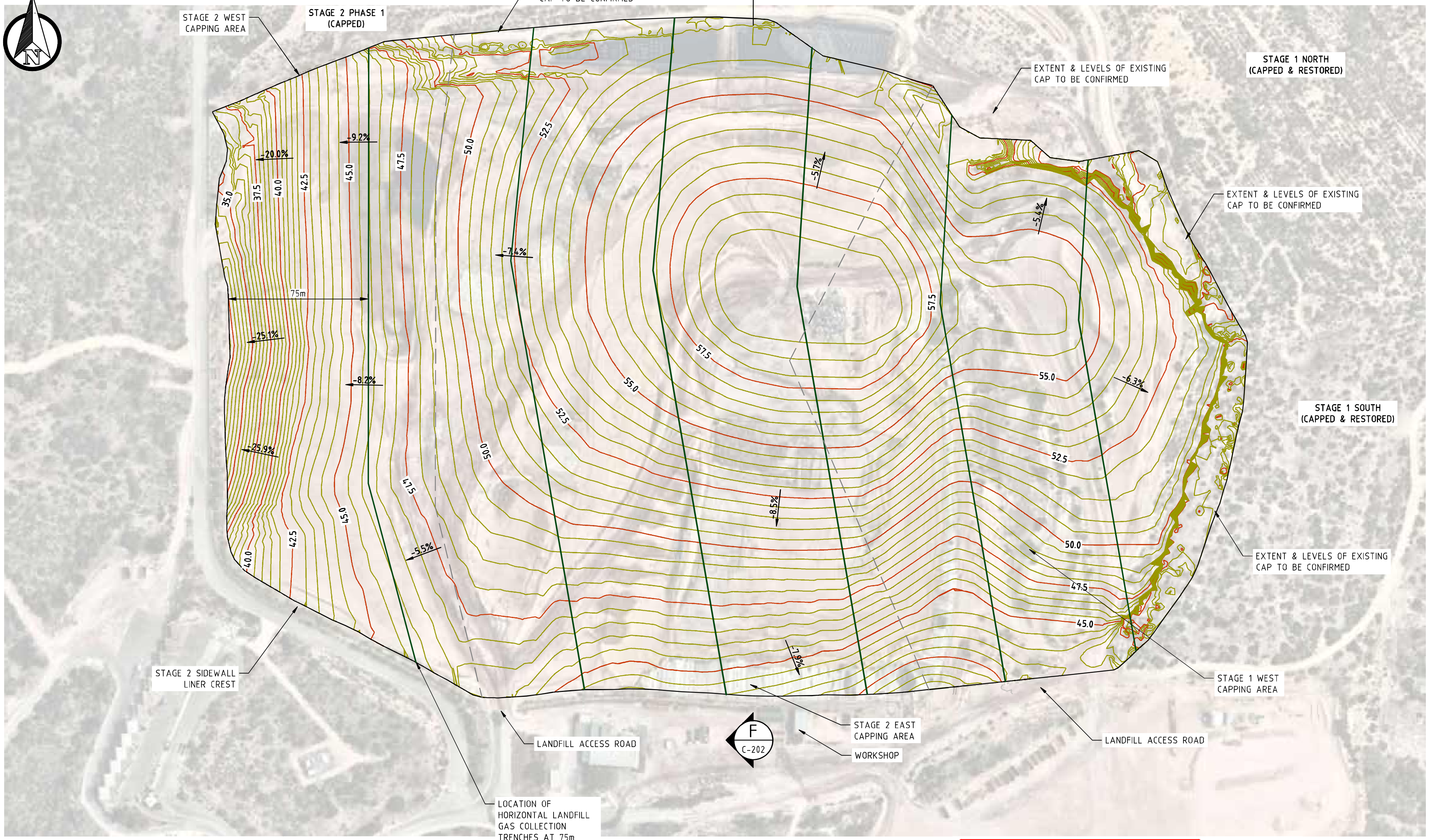
No.	Date	Drwn.	Chk.	Amendment / Issue	App.
A	04.11.2024	YJ	AB	PRELIMINARY ISSUE	

Project: TAMALA PARK SOUTHERN PIGGYBACK LANDFILL CELL

Title: LONG SECTIONS SHEET 2 OF 2

Scale: AS SHOWN @ A3 Date: 04.11.2024
Drawn: YJ Checked: AB Approved:
Job No: TW23015 Drg. No: W-202 Rev: A
Filename: TW21026 - PIGGYBACK.DWG



LEGEND:

HORIZONTAL LANDFILL GAS COLLECTION TRENCHES

SURVEY DATE: 17 FEBRUARY 2021
VERTICAL DATUM: AUSTRALIAN HEIGHT DATUM
HORIZONTAL DATUM: MGA 94 ZONE 50

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40m 0m 40m 80m
1:2000 @ A3

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PERTH
604 Newcastle Street Leederville WA 6007
PO Box 454 Leederville WA 6903

Client: MINDARIE REGIONAL COUNCIL

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No.	Date	Drwn.	Chk.	Amendment / Issue	App.
A	10.06.2021	AB	CS	PRELIMINARY ISSUE	

Project: TAMALA PARK CLOSURE PLAN

Title: FINAL WASTE FILL PROFILE

Scale: 1:2000 @ A3	Date: 03.06.2021	
Drawn: AH	Checked: CS	Approved:
Job No: TW21026	Drg. No: C-101	Rev: A

Filename: TW21026 - SET.DWG

APPENDIX B
Laboratory Test Results

Perth
Unit 3, 34 Sphinx Way
Bibra Lake
WA 6163
Ph: (08) 9418 8742
E-mail: [redacted]
Mob: [redacted]

E-PRECISION LABORATORY

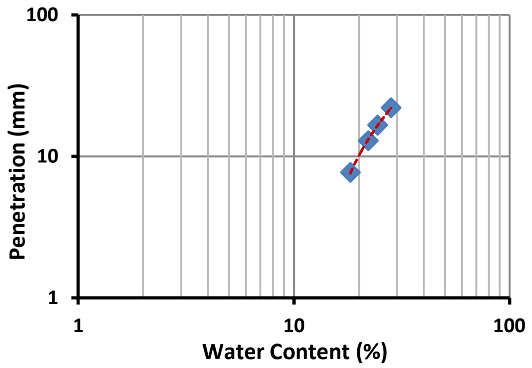
ATTERBERG LIMITS TEST REPORT

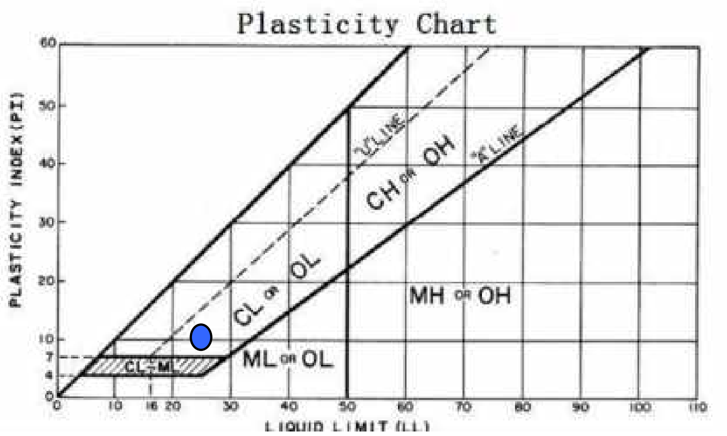
Test Method: BS1377 AS1289.2.1.1 7.1.1 3.1.1 3.2.1 3.4.1

Client:	Talis Consultants	Date Tested:	05/05/2021
Project:	Tamala Park Landfill, Mindarie	Lab:	EPLAB
Sample No:	Limestone	Job Number:	TALIS
Lab ID:	LIMESTONE_ATT		
Depth(m):	-	Room Temperature at Test:	20°C
Tested by:	Kohei	Sample Description:	-
Moisture Content (%):	-	Wet Density (t/m³):	-
		Dry Density (t/m³):	-

Liquid Limit (%):	24.77
Plastic Limit (%):	14.39
Plasticity Index (%):	10.38
Liquidity Index (%):	-
Shrinkage Limit (%):	12.31
Linear Shrinkage(%):	1.62

Results Chart





Notes: The sample/s were tested oven dried, dry sieved and in a 125-250mm mould

Stored and Tested the Sample as received
Samples supplied by the Client

Authorised Signature: [redacted]

Perth
Unit 3, 34 Sphinx Way
Bibra Lake
WA 6163
Ph: (08) 9418 8742
E-mail: [redacted]

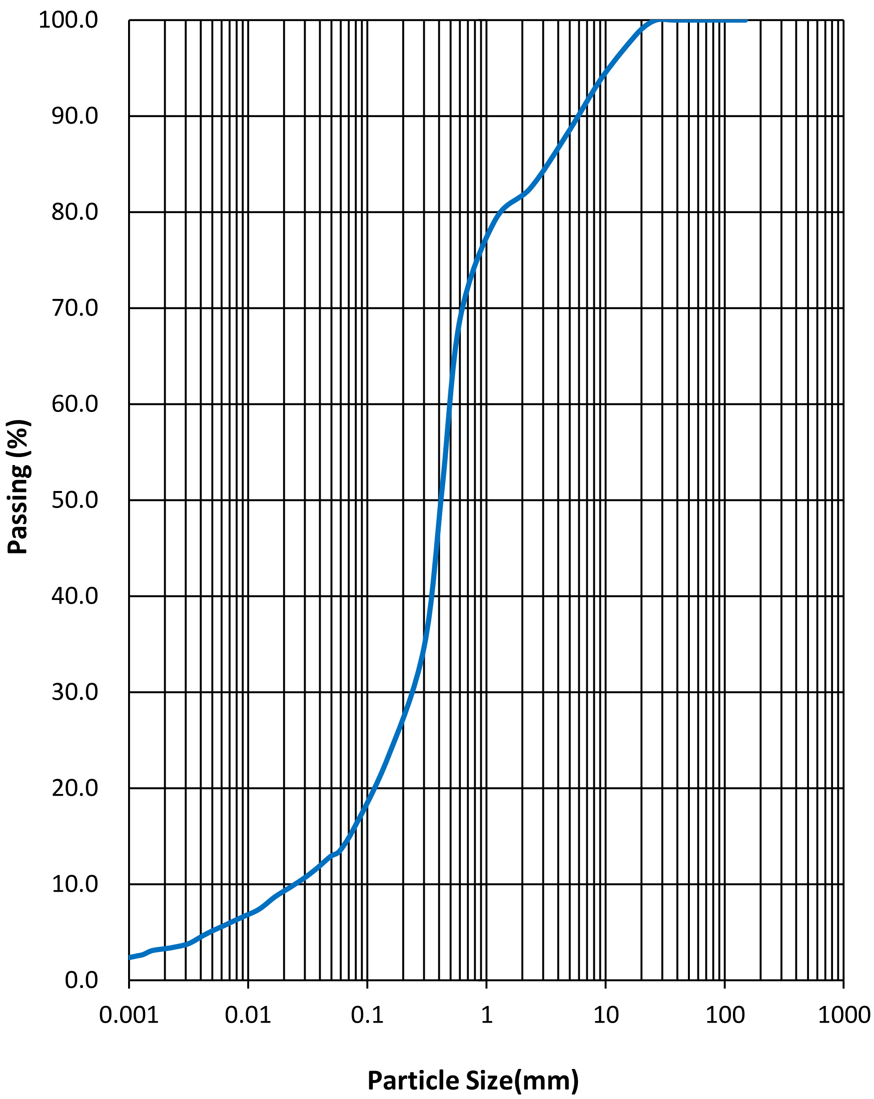
PARTICLE SIZE DISTRIBUTION TEST REPORT

Test Method: AS 1289 3.6.3 3.5.1

Client:	Talis Consultants	Date Tested:	06/05/2021
Project:	Tamala Park, Mindarie	EP Lab Job Number:	TALIS
Sample No:	Limestone Sample	Depth(m):	-
Lab ID:	LIMESTONE_PSD	Room Temperature at Test:	19°C
Tested by:	Kohei	2.36mm Particle Density (t/m³):	2.68
Checked by:	Phil	Moisture Content (%):	0.94

Sieve Size (mm)	Passing %
150	100.0
75	100.0
53	100.0
37.5	100.0
26.5	100.0
19	98.8
9.5	94.1
4.75	88.1
2.36	82.6
1.18	79.1
0.6	68.8
0.425	51.0
0.3	34.7
0.15	23.3
0.075	15.5
0.05834	13.5
0.04890	12.9
0.03481	11.3
0.02474	10.0
0.01689	8.7
0.01239	7.4
0.00879	6.5
0.00623	5.7
0.00442	4.8
0.00314	3.8
0.00222	3.4
0.00157	3.1
0.00130	2.7
0.00113	2.5
0.00101	2.4

PSD Graph



Notes:
Stored and Tested the Sample as received
Samples supplied by the Client

Authorized Signature: [redacted]

Perth
16 Gympie Way, Willetton
WA 6155
Ph: (08) 9418 8742
E-mail: Phillip.li@eprecisionlab.com
Mob: 0422 814 231

E-Precision Laboratory

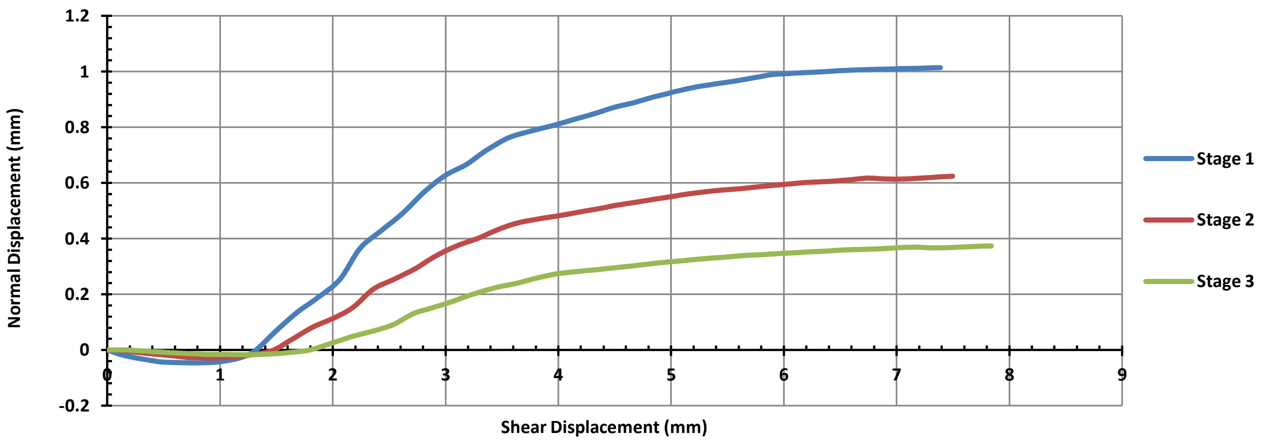
SINGLE-STAGE DRAINED DIRECT SHEAR TEST REPORT

Method: AS1289.6.2.2 / In-house Method

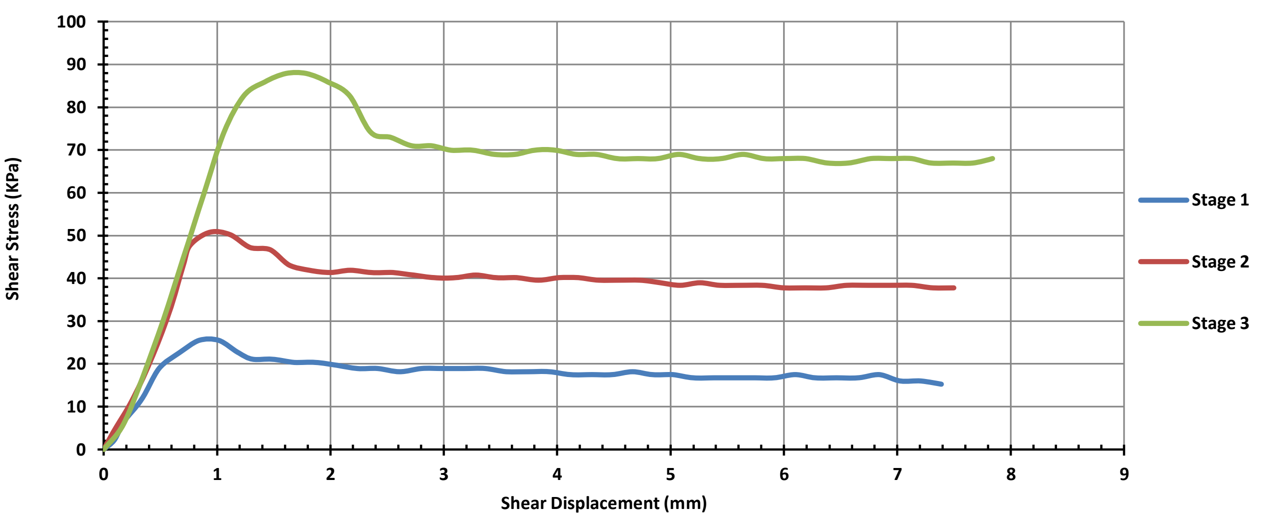
Client:	Talis Consultants	Date Tested:	23/05/2021
Project:	Tamala Park Landfill, Mindarie	EP Lab Job Number:	TALIS
Sample No:	Limestone Sample	Lab:	EPLab
Sample ID:	LIMESTONE_DDST		
Depth (m):	-	Room Temperature at Test:	20°

Type of Test:	Single Stage Intact Drained Shear	**Sample Description:**	-
Dimensions (mm):	100 x 100 (Circular)	**Shear Plane Dip Angle (°):**	N/A
Rate of Strain (mm/min):	0.025	**Initial Bulk Density (t/m³):**	1.79
Failure Criteria:	Horizontal Shear	**Initial Moisture Content (%):**	3.87

Normal Displacement Vs Shear Displacement Plot



Shear Stress Vs Shear Displacement Plot



Integrity Precision Innovation

Perth
16 Gympie Way, Willetton
WA 6155
Ph: (08) 9418 8742
E-mail: [redacted]

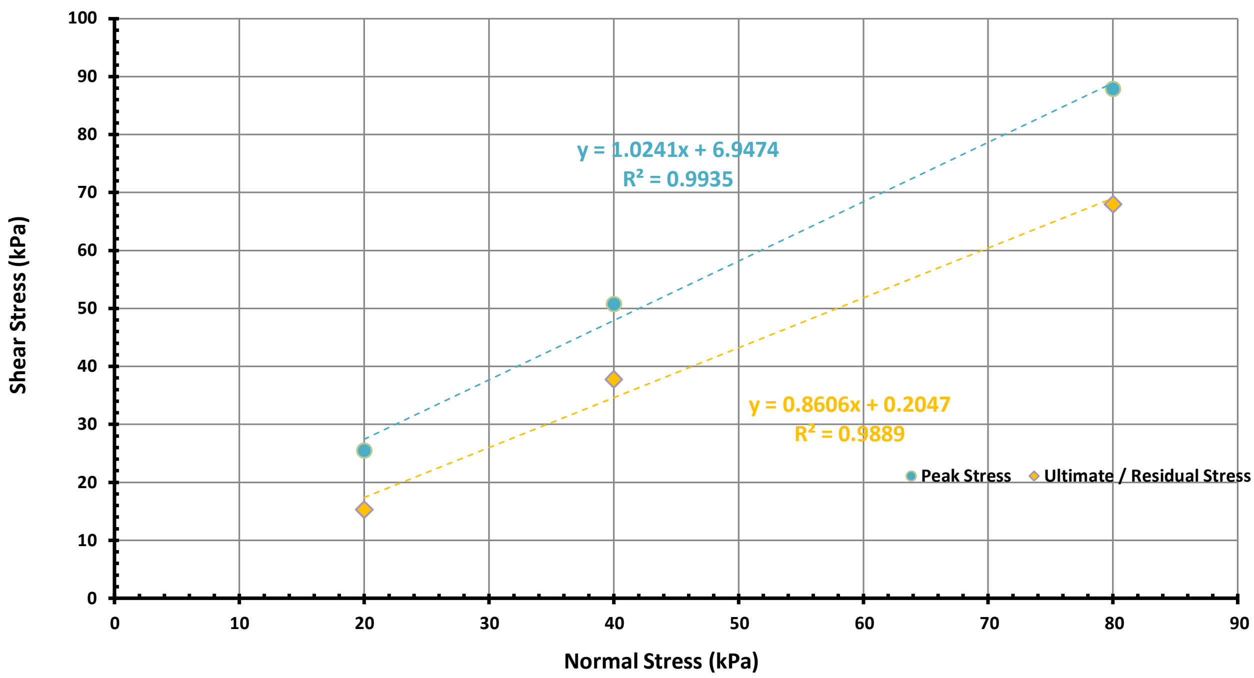
E-Precision Laboratory

SINGLE-STAGE DRAINED DIRECT SHEAR TEST REPORT

Method: AS1289.6.2.2 / In-house Method

Client:	Talis Consultants	Date Tested:	23/05/2021
Project:	Tamala Park Landfill, Mindarie	EP Lab Job Number:	TALIS
Sample No:	Limestone Sample	Lab:	EPLab
Sample ID:	LIMESTONE_DDST		
Depth (m):	-	Room Temperature at Test:	20°

(Peak / Ultimate) Normal Stress Vs Shear Stress (Effective Stress)



Defect Surface:	N/A
Dip Angle (°):	N/A

			Normal Stress (kPa)		Shear Stress (kPa)	
Peak	**Shear Angle (°)**	45.57	Stage 1	20	Stage 1	26
	Cohesion (kPa)	6.95	Stage 2	40	Stage 2	51
	R^2	0.9935	Stage 3	80	Stage 3	88

			Normal Stress (kPa)		Shear Stress (kPa)	
Ultimate / Residual	**Shear Angle (°)**	40.70	Stage 1	20	Stage 1	15
	Cohesion (kPa)	0.20	Stage 2	40	Stage 2	38
	R^2	0.9889	Stage 3	80	Stage 3	68

Integrity Precision Innovation

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Ph: (08) 9418 8742
E-mail: [redacted]

E-Precision Laboratory

SINGLE-STAGE DRAINED DIRECT SHEAR TEST REPORT

Method: AS1289.6.2.2 / In-house Method

Client:	Talis Consultants	Date Tested:	23/05/2021
Project:	Tamala Park Landfill, Mindarie	EP Lab Job Number:	TALIS
Sample No:	Limestone Sample	Lab:	EPLab
Sample ID:	LIMESTONE_DDST		
Depth (m):	-	Room Temperature at Test:	20°

Photo of Sample Post Testing

Sample 1

Sample 2

Sample 3

Notes: ***Sample remolded to 95% MDD @ dry end of OMC as requested by client***

Stored and Tested the Sample as received

Samples supplied by the Client

Authorised Signature (Geotechnical Engineer): [redacted]

Perth
16 Gympie Way, Willetton
WA 6155
Ph: (08) 9418 8742
E-mail: [redacted]

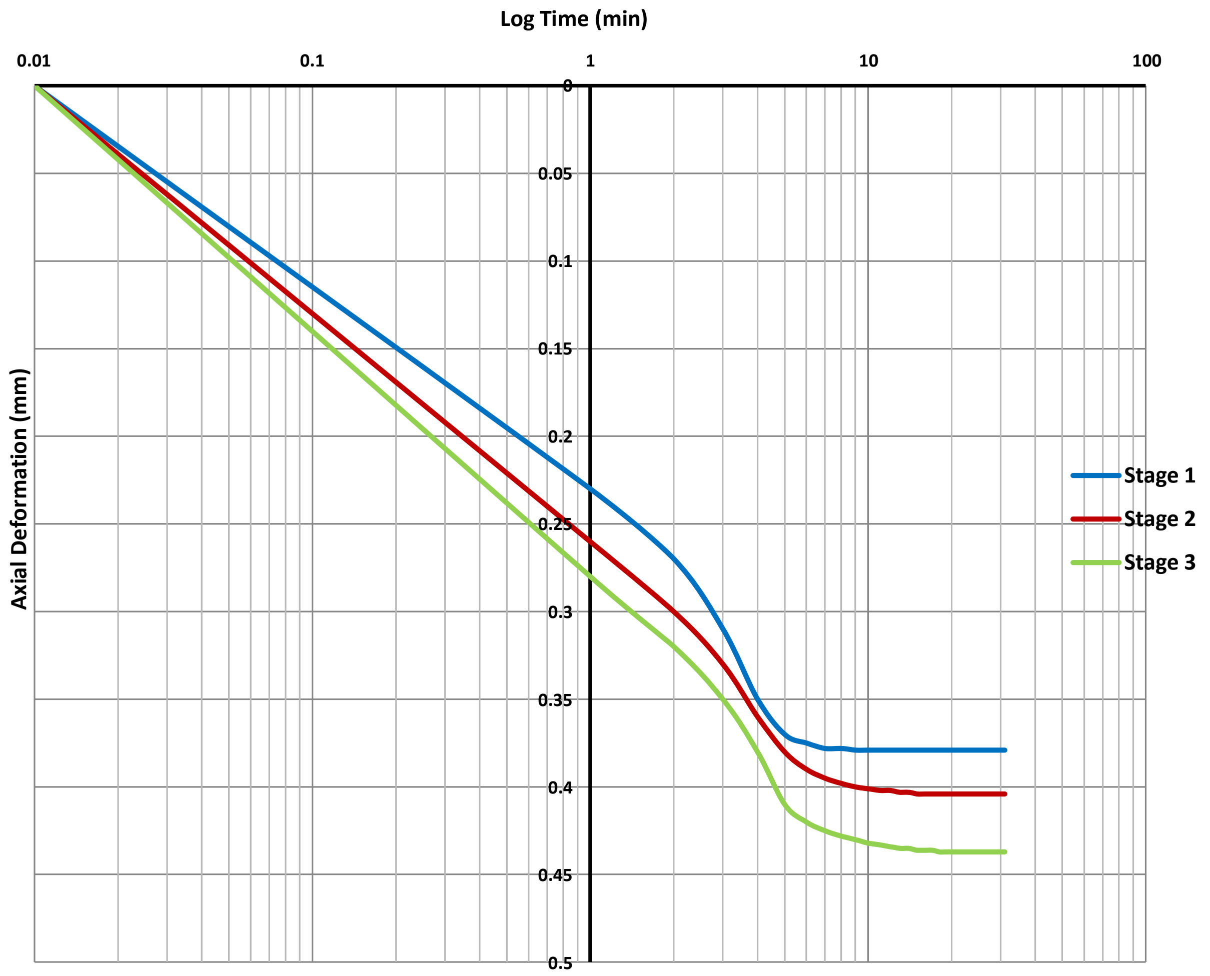
E-Precision Laboratory

SINGLE-STAGE DRAINED DIRECT SHEAR TEST REPORT

Method: AS1289.6.2.2 / In-house Method

Client:	Talis Consultants	Date Tested:	23/05/2021
Project:	Tamala Park Landfill, Mindarie	EP Lab Job Number:	TALIS
Sample No:	Limestone Sample	Lab:	EPLab
Sample ID:	LIMESTONE_DDST		
Depth (m):	-	Room Temperature at Test:	20°

Consolidation Vertical Deformation (mm) Vs Log Time (min)



Integrity Precision Innovation

Perth
Unit 3, 34 Sphinx Way
Bibra Lake
WA 6163
Ph: (08) 9418 8742
E-mail: [redacted]
Mob: [redacted]

E-Precision Laboratory

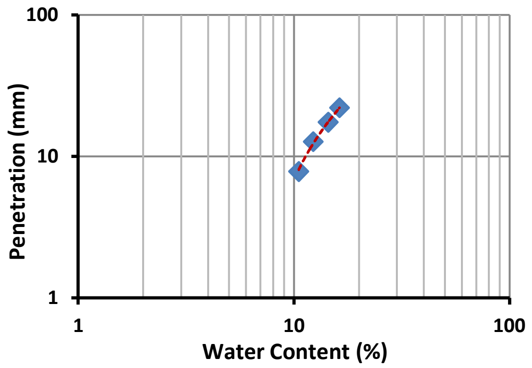
ATTERBERG LIMITS TEST REPORT

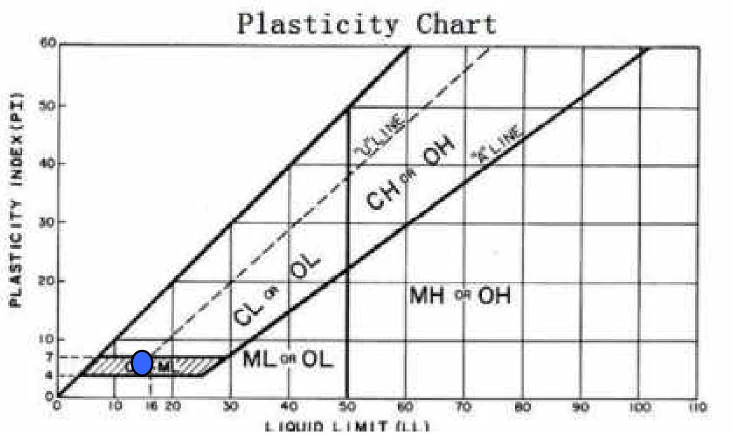
Test Method: BS1377 AS1289.2.1.1 7.1.1 3.1.1 3.2.1 3.4.1

Client:	Talis Consultants	Date Tested:	05/05/2021
Project:	Tamala Park Landfill, Mindarie	Lab:	EPLAB
Sample No:	Subsoil	Job Number:	TALIS
Lab ID:	SUBSOIL_ATT		
Depth(m):	-	Room Temperature at Test:	20°C
Tested by:	Kohei	Sample Description:	-
Moisture Content (%):	-	Wet Density (t/m³):	-
		Dry Density (t/m³):	-

Liquid Limit (%):	14.20
Plastic Limit (%):	8.05
Plasticity Index (%):	6.16
Liquidity Index (%):	-
Shrinkage Limit (%):	7.46
Linear Shrinkage(%):	3.39

Results Chart





Notes: The sample/s were tested oven dried, dry sieved and in a 125-250mm mould.

Stored and Tested the Sample as received

Samples supplied by the Client

Authorised Signature: [redacted]

Perth
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Bibra Lake
WA 6163
Ph: (08) 9418 8742
E-mail: [redacted]

E-Precision Laboratory

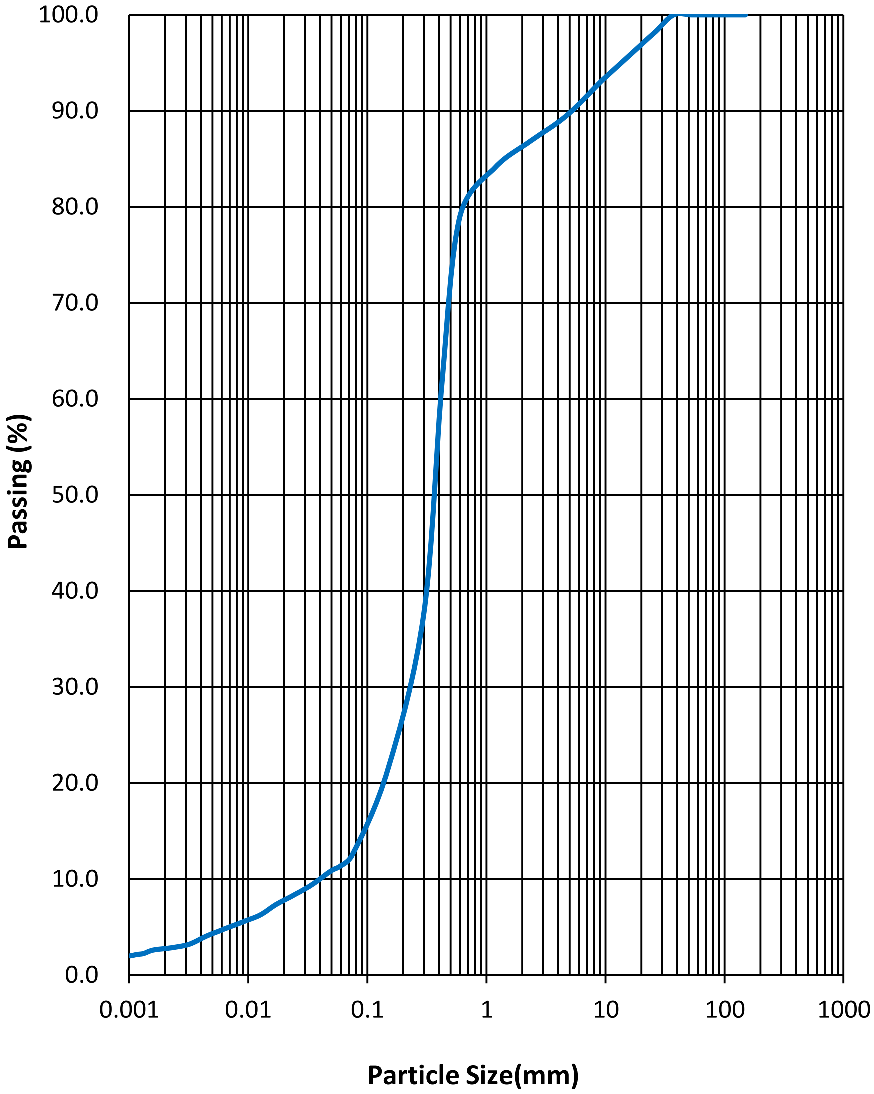
PARTICLE SIZE DISTRIBUTION TEST REPORT

Test Method: AS 1289 3.6.3 3.5.1

Client:	Talis Consultants	Date Tested:	06/05/2021
Project:	Tamala Park, Mindarie	EP Lab Job Number:	TALIS
Sample No:	Subsoil Sample	Depth(m):	-
Lab ID:	SUBSOIL_PSD	Room Temperature at Test:	19°C
Tested by:	Kohei	2.36mm Particle Density (t/m³):	2.65
Checked by:	Phil	Moisture Content (%):	3.36

Sieve Size (mm)	Passing %
150	100.0
75	100.0
53	100.0
37.5	100.0
26.5	98.3
19	96.6
9.5	93.2
4.75	89.5
2.36	86.9
1.18	84.1
0.6	79.0
0.425	62.0
0.3	37.8
0.15	21.5
0.075	12.5
0.05892	11.3
0.04939	10.8
0.03515	9.5
0.02499	8.4
0.01706	7.3
0.01252	6.2
0.00888	5.5
0.00630	4.8
0.00447	4.1
0.00317	3.2
0.00224	2.8
0.00159	2.6
0.00132	2.2
0.00114	2.1
0.00102	2.0

PSD Graph



Notes:
Stored and Tested the Sample as received
Samples supplied by the Client

Authorized Signature: [redacted]

Integrity Precision Innovation

Perth
16 Gympie Way, Willetton
WA 6155
Ph: (08) 9418 8742
E-mail: [redacted]

E-Precision Laboratory

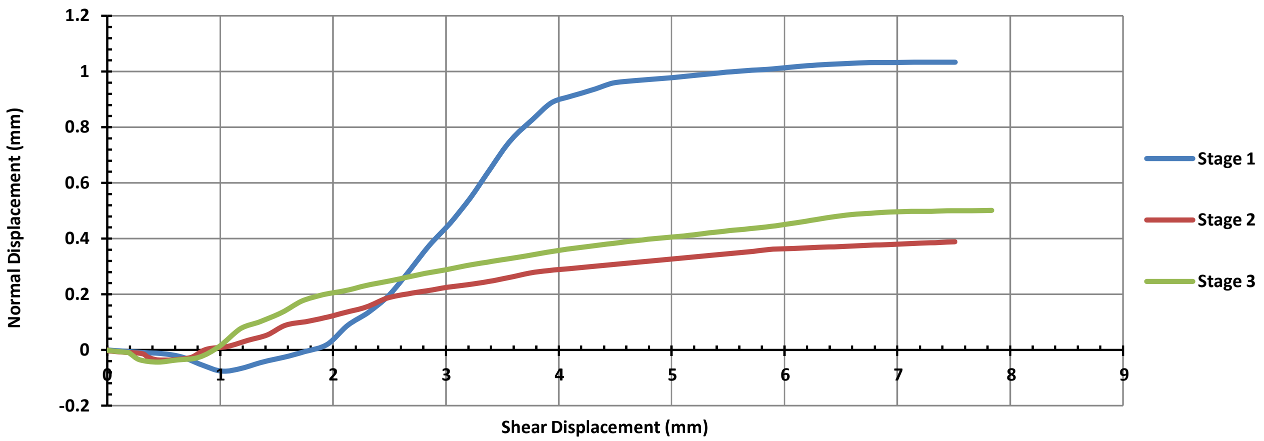
SINGLE-STAGE DRAINED DIRECT SHEAR TEST REPORT

Method: AS1289.6.2.2 / In-house Method

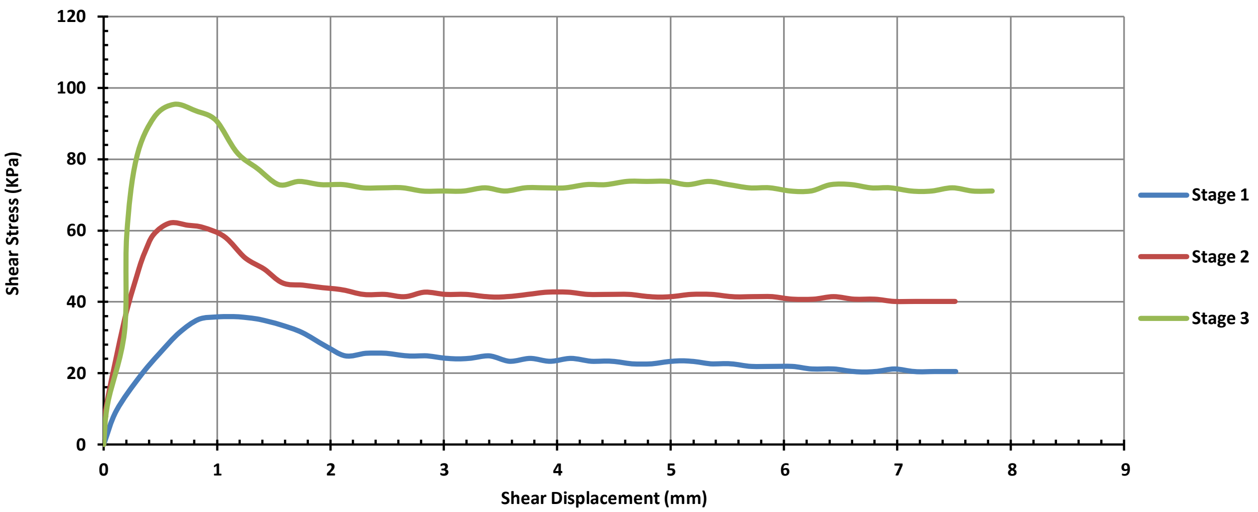
Client:	Talis Consultants	Date Tested:	20/05/2021
Project:	Tamala Park Landfill, Mindarie	EP Lab Job Number:	TALIS
Sample No:	Subsoil Sample	Lab:	EPLab
Sample ID:	SUBSOIL_DDST		
Depth (m):	-	Room Temperature at Test:	20°

Type of Test:	Single Stage Intact Drained Shear	**Sample Description:**	-
Dimensions (mm):	100 x 100 (Circular)	**Shear Plane Dip Angle (°):**	N/A
Rate of Strain (mm/min):	0.022	**Initial Bulk Density (t/m³):**	1.70
Failure Criteria:	Horizontal Shear	**Initial Moisture Content (%):**	3.65

Normal Displacement Vs Shear Displacement Plot



Shear Stress Vs Shear Displacement Plot



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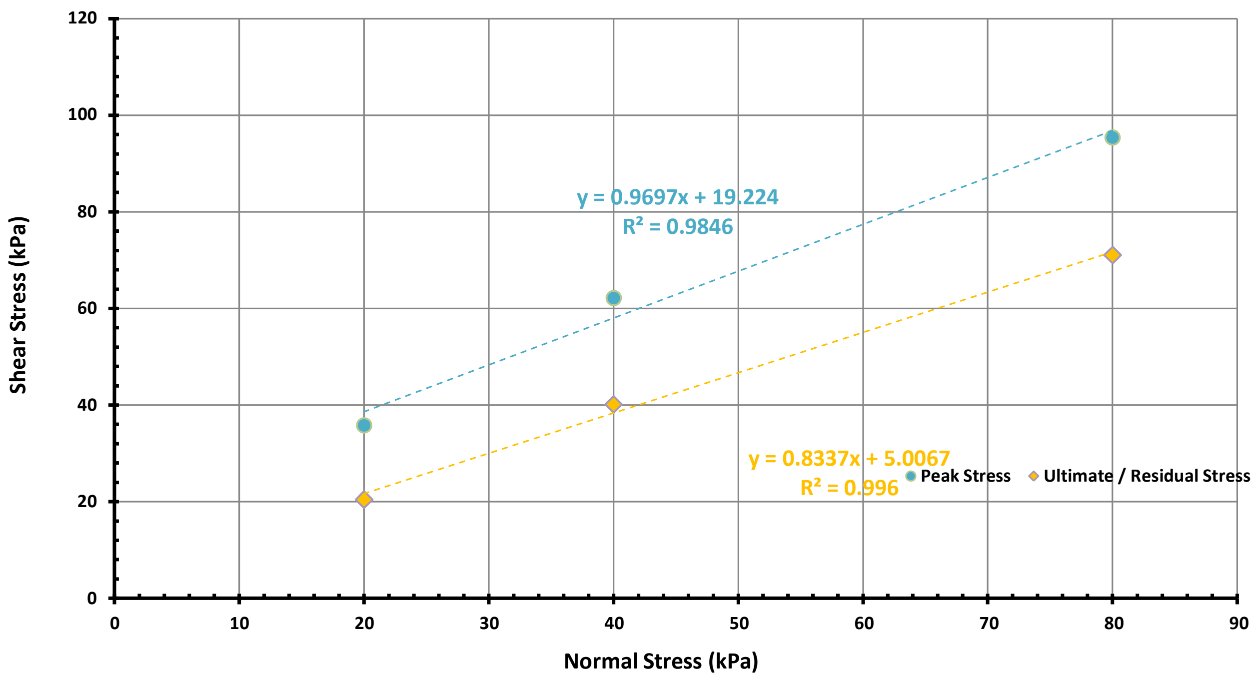
E-Precision Laboratory

SINGLE-STAGE DRAINED DIRECT SHEAR TEST REPORT

Method: AS1289.6.2.2 / In-house Method

Client:	Talis Consultants	Date Tested:	20/05/2021
Project:	Tamala Park Landfill, Mindarie	EP Lab Job Number:	TALIS
Sample No:	Subsoil Sample	Lab:	EPLab
Sample ID:	SUBSOIL_DDST		
Depth (m):	-	Room Temperature at Test:	20°

(Peak / Ultimate) Normal Stress Vs Shear Stress (Effective Stress)



Defect Surface:	N/A
Dip Angle (°):	N/A

			Normal Stress (kPa)		Shear Stress (kPa)	
Peak	**Shear Angle (°)**	44.13	Stage 1	20	Stage 1	36
	Cohesion (kPa)	19.22	Stage 2	40	Stage 2	62
	R^2	0.9846	Stage 3	80	Stage 3	95

			Normal Stress (kPa)		Shear Stress (kPa)	
Ultimate / Residual	**Shear Angle (°)**	39.83	Stage 1	20	Stage 1	20
	Cohesion (kPa)	5.01	Stage 2	40	Stage 2	40
	R^2	0.9960	Stage 3	80	Stage 3	71

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E-Precision Laboratory

SINGLE-STAGE DRAINED DIRECT SHEAR TEST REPORT

Method: AS1289.6.2.2 / In-house Method

Client:	Talis Consultants	Date Tested:	20/05/2021
Project:	Tamala Park Landfill, Mindarie	EP Lab Job Number:	TALIS
Sample No:	Subsoil Sample	Lab:	EPLab
Sample ID:	SUBSOIL_DDST		
Depth (m):	-	Room Temperature at Test:	20°

Photo of Sample Post Testing

Sample 1

Sample 2

Sample 3

Notes: ***Sample remolded to 95% MDD @ dry end of OMC as requested by client***

Stored and Tested the Sample as received

Samples supplied by the Client

Authorised Signature (Geotechnical Engineer): [redacted]

Perth
16 Gympie Way, Willetton
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E-mail: [redacted]

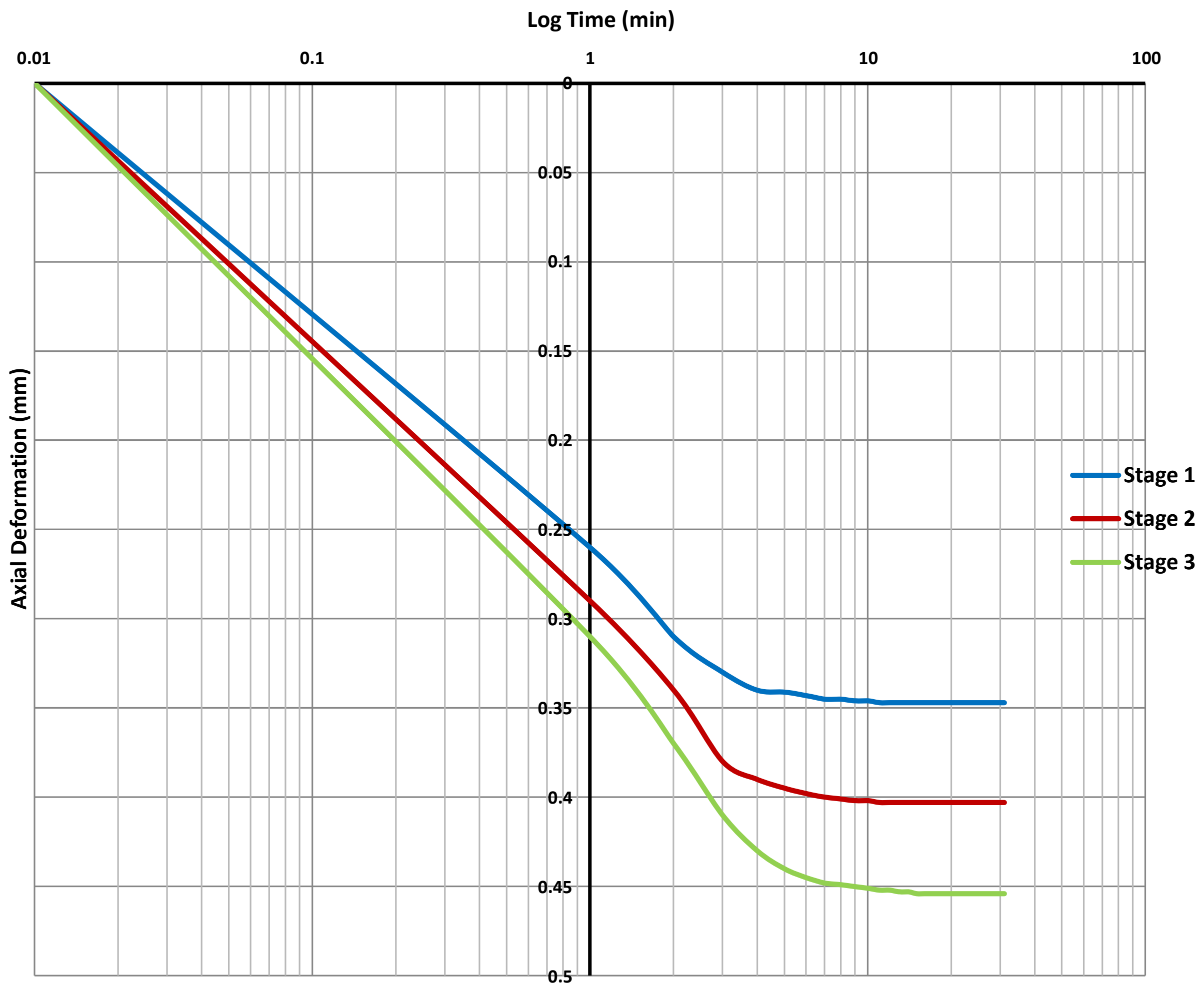
E-Precision Laboratory

SINGLE-STAGE DRAINED DIRECT SHEAR TEST REPORT

Method: AS1289.6.2.2 / In-house Method

Client:	Talis Consultants	Date Tested:	20/05/2021
Project:	Tamala Park Landfill, Mindarie	EP Lab Job Number:	TALIS
Sample No:	Subsoil Sample	Lab:	EPLab
Sample ID:	SUBSOIL_DDST		
Depth (m):	-	Room Temperature at Test:	20°

Consolidation Vertical Deformation (mm) Vs Log Time (min)



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Ph: (08) 9418 8742
Mob: [redacted]

E-PRECISION LABORATORY

CONSTANT HEAD PERMEABILITY TEST REPORT

Test Method: AS1289 6.7.1

Client:	Talis Consultants	Date Tested:	13/05/2021
Project:	Tamala Park Landfill, Mindarie	Date Reported:	16/05/2021
Lab:	EPLAB	EP Lab Job Number:	TALIS
Tested by:	Phil		
Checked by:	Phil		

Lab ID:	SUBSOIL_FH	LIMESTONE_FH		
Client ID:	Subsoil	LIMESTONE		
Depth (m):	-	-		
Sample Conditions:	Remolded 92% SMDD	Remolded 92% SMDD		
Surcharge Pressure (kPa):	12.5	12.5		
Initial Bulk Density (t/m³):	2.01	1.94		
Initial Moisture Content (%):	16.76	12.13		
Dry Density (t/m³):	1.72	1.73		
Saturation (Skempton's B):	1.00	1.00		
K_{20} (m/s):	**6.609 E^{-5}**	**4.153 E^{-5}**		

Notes:
Stored and Tested the Sample as received
Samples supplied by the Client

[redacted]

Authorised Signatory (Geotechnical Engineer):

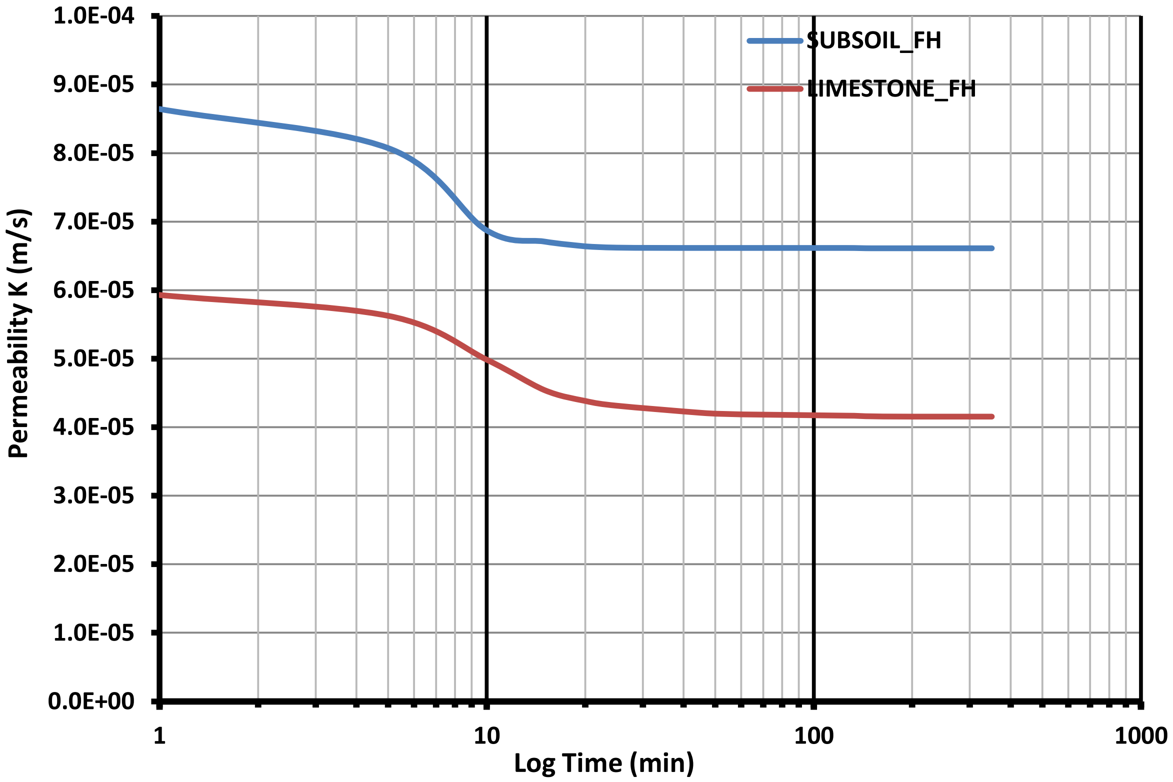
Perth
Unit 3, 34 Sphinx Way
Bibra Lake,
WA 6163
Ph: (08) 9418 8742
Mob:[redacted]

E-PRECISION LABORATORY

CONSTANT HEAD PERMEABILITY TEST REPORT

Test Method: AS1289 6.7.1

Client:	Talis Consultants	Date Tested:	13/05/2021
Project:	Tamala Park Landfill, Mindarie	Date Reported:	16/05/2021
Lab:	EPLAB	EP Lab Job Number:	TALIS



Notes:
Stored and Tested the Sample as received
Samples supplied by the Client

Authorised Signatory (Geotechnical Engineer): [redacted]

Integrity Precision Innovation

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E-mail: [redacted]

E-Precision Laboratory

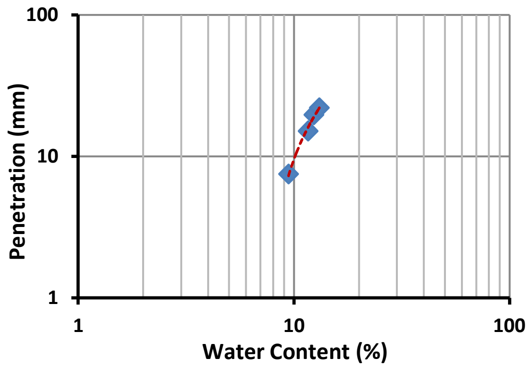
ATTERBERG LIMITS TEST REPORT

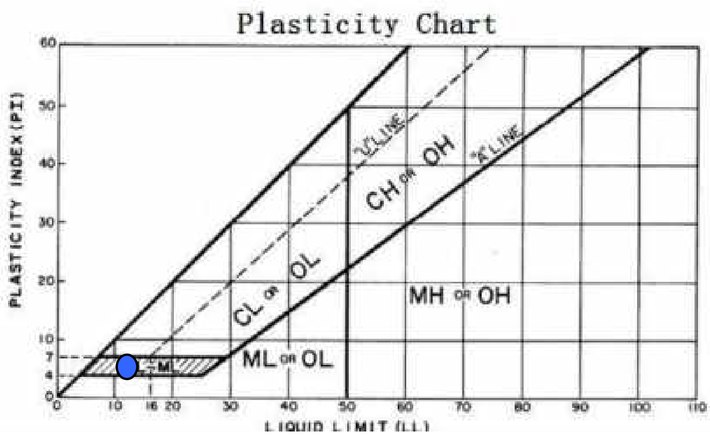
Test Method: BS1377 AS1289.2.1.1 7.1.1 3.1.1 3.2.1 3.4.1

Client:	Talis Consultants	Date Tested:	05/05/2021
Project:	Tamala Park Landfill, Mindarie	Lab:	EPLAB
Sample No:	Topsoil	Job Number:	TALIS
Lab ID:	TOPSOIL_ATT		
Depth(m):	-	Room Temperature at Test:	20°C
Tested by:	Kohei	Sample Description:	-
Moisture Content (%):	-	Wet Density (t/m^3):	-
		Dry Density (t/m^3):	-

Liquid Limit (%):	11.87
Plastic Limit (%):	8.12
Plasticity Index (%):	3.75
Liquidity Index (%):	-
Shrinkage Limit (%):	7.73
Linear Shrinkage(%):	1.75

Results Chart





Notes: The sample/s were tested oven dried, dry sieved and in a 125-250mm mould.

Stored and Tested the Sample as received

Samples supplied by the Client

Authorised Signature: [redacted]

Integrity Precision Innovation

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E-Precision Laboratory

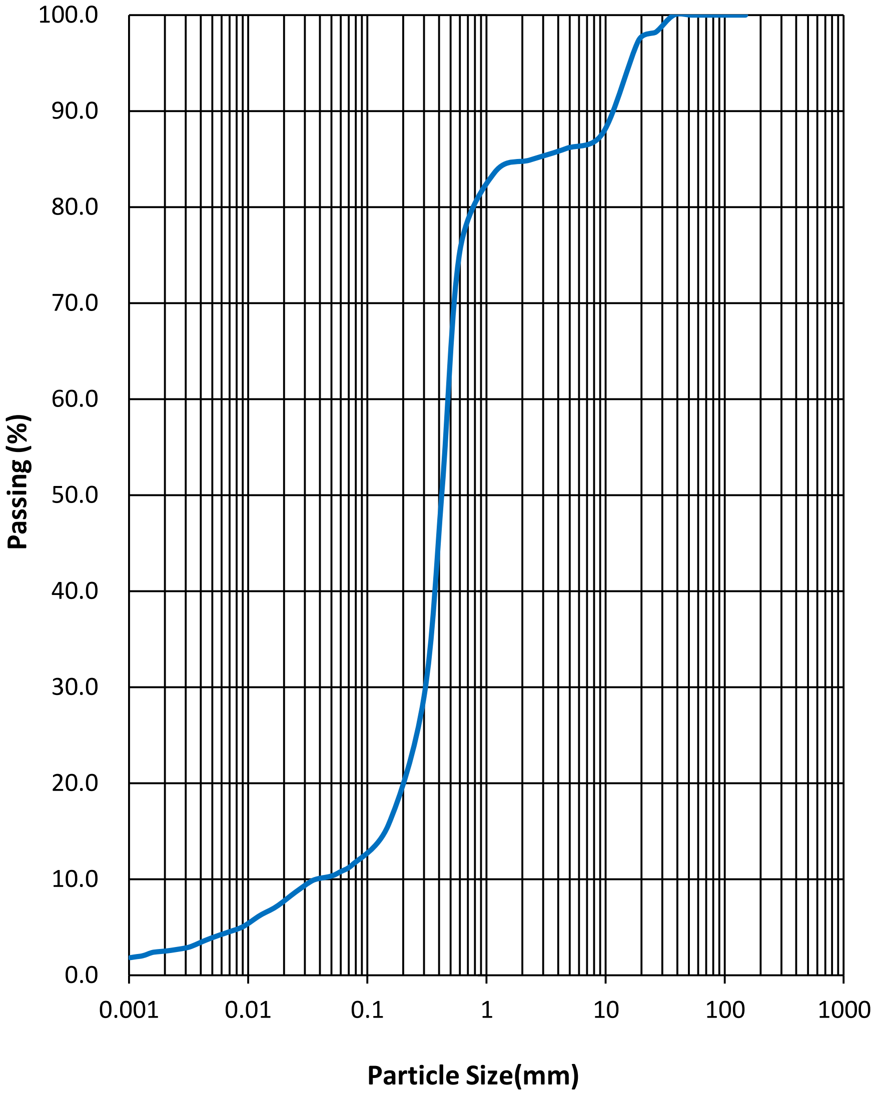
PARTICLE SIZE DISTRIBUTION TEST REPORT

Test Method: AS 1289 3.6.3 3.5.1

Client:	Talis Consultants	Date Tested:	06/05/2021
Project:	Tamala Park, Mindarie	EP Lab Job Number:	TALIS
Sample No:	Topsoil Sample	Depth(m):	-
Lab ID:	TOPSOIL_PSD	Room Temperature at Test:	19°C
Tested by:	Kohei	2.36mm Particle Density (t/m³):	2.61
Checked by:	Phil	Moisture Content (%):	2.18

Sieve Size (mm)	Passing %
150	100.0
75	100.0
53	100.0
37.5	100.0
26.5	98.2
19	97.4
9.5	87.7
4.75	86.1
2.36	84.9
1.18	83.6
0.6	75.4
0.425	50.3
0.3	29.0
0.15	15.6
0.075	11.5
0.05937	10.8
0.04977	10.3
0.03528	9.9
0.02511	8.7
0.01719	7.1
0.01261	6.2
0.00897	5.0
0.00636	4.4
0.00451	3.7
0.00320	2.9
0.00227	2.6
0.00160	2.4
0.00133	2.0
0.00116	1.9
0.00103	1.8

PSD Graph



Notes:
Stored and Tested the Sample as received
Samples supplied by the Client

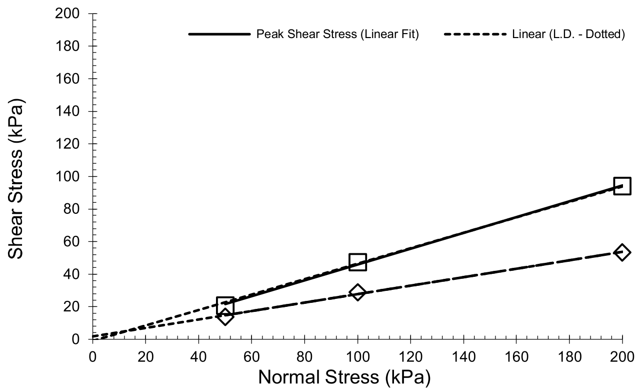
Authorized Signature: [redacted]

Interface Friction Test Report

Accredited for compliance with ISO/IEC 17025 & GAI-LAP Testing

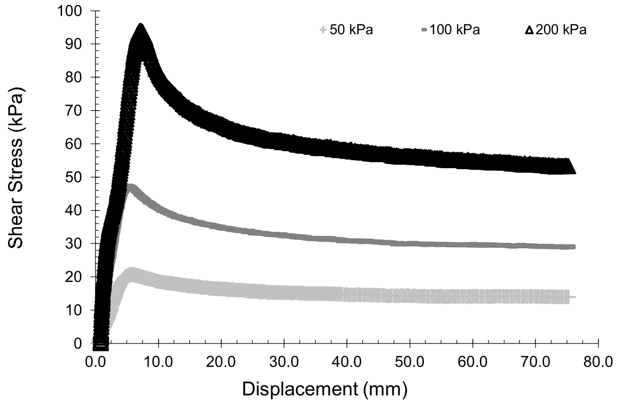
Client: **Talis Consulting** TRI Log #: A24-375
Project: **Tamala Park Cap** Test Method: ASTM D5321
Date: 5-10-2024 to 10-10-2024

Tested Interface: Geotextile vs Geomembrane 1



Test Results		
	Peak	**Large Displacement (@ 75 mm)**
Friction Angle (degrees):	**25.1**	**14.6**
Y-intercept or Adhesion (kPa):	**0**	**2**

Shearing occurred at the GT/GM interface . The large displacement friction angle regression analysis was adjusted to fit a zero y-intercept.



Test Conditions

Upper Box: Bidim A84 Nonwoven Geotextile
Roll No. 1743412

Lower Box: Solmax Double sided Textured LLDPE
Roll No. 0904-094695

Box Dimensions: 305 mm x 305 mm x 102 mm

Interface Conditioning: Interface soaked and loading applied for a minimum of 24 hours prior to shear.

Test Condition: Wet

Shearing Rate: 0.1 mm/minute

Test Data

Specimen No.	1	2	3
Bearing Slide Resistance (kPa)	1	1	2
Normal Stress (kPa)	50	100	200
Corrected Peak Shear Stress (kPa)	21	47	94
Corrected Large Displacement Shear Stress (kPa)	14	29	53
Peak Secant Angle (degrees)	22.5	25.3	25.2
Large Displacement Secant Angle (degrees)	15.5	16.1	14.9
Asperity (mm)	0.475	0.450	0.550

Warren Hornsey, Pr.Eng.

Director
Approved Signatory

END OF REPORT

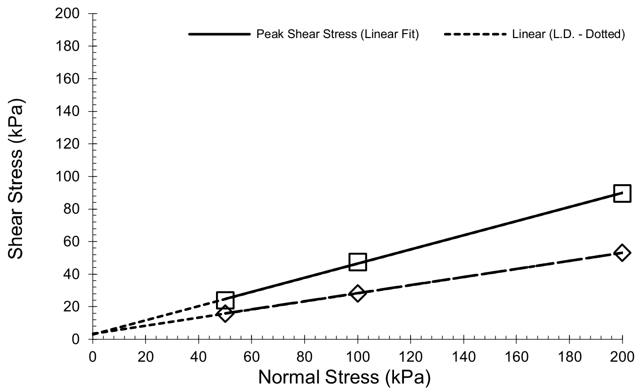
Interface Friction Test Report

Accredited for compliance with ISO/IEC 17025 & GAI-LAP Testing

Client: **Talis Consulting**
Project: **Tamala Park Cap**
Date: 29-10-2024 to 30-10-2024

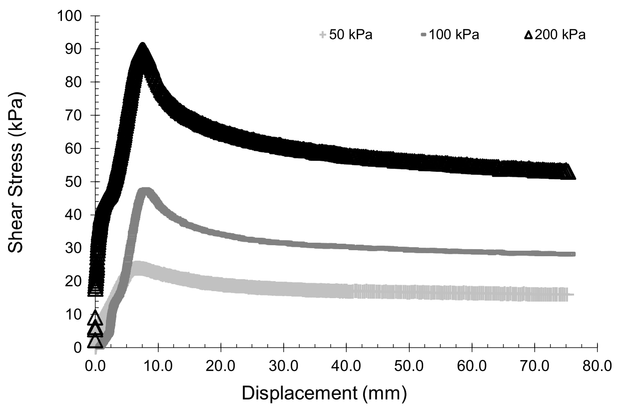
TRI Log #: A24-375
Test Method: ASTM D5321

Tested Interface: Geotextile vs Geomembrane No 2



Test Results		
	Peak	Large Displacement (@ 75 mm)
Friction Angle (degrees):	23.5	13.9
Y-intercept or Adhesion (kPa):	3	3

Shearing occurred at the GT/GM interface .



Test Conditions

Upper Box: Bidim A84 Nonwoven Geotextile
Roll No. 1743412

Lower Box: Solmax Double sided Textured LLDPE
Roll No. 0904-094695

Box Dimensions: 305 mm x 305 mm x 102 mm

Interface Conditioning: Interface soaked and loading applied for a minimum of 24 hours prior to shear.

Test Condition: Wet

Shearing Rate: 0.1 mm/minute

Test Data

Specimen No.	1	2	3
Bearing Slide Resistance (kPa)	1	1	2
Normal Stress (kPa)	50	100	200
Corrected Peak Shear Stress (kPa)	24	48	90
Corrected Large Displacement Shear Stress (kPa)	16	28	53
Peak Secant Angle (degrees)	25.6	25.4	24.1
Large Displacement Secant Angle (degrees)	17.6	15.7	14.9
Asperity (mm)	0.550	0.550	0.525

Warren Hornsey, Pr.Eng.

Director
Approved Signatory

END OF REPORT

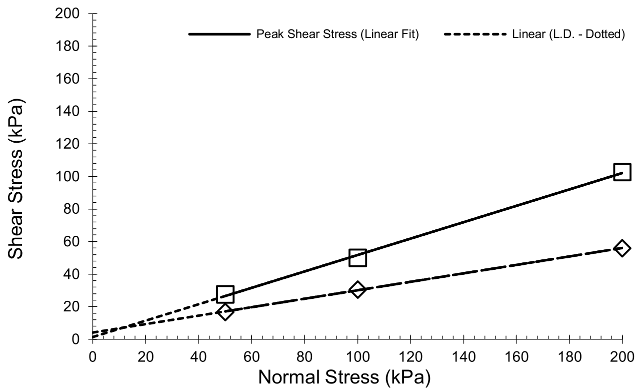
Interface Friction Test Report

Accredited for compliance with ISO/IEC 17025 & GAI-LAP Testing

Client: **Talis Consulting**
Project: **Tamala Park Cap**
Date: 30-10-2024 to 2-11-2024

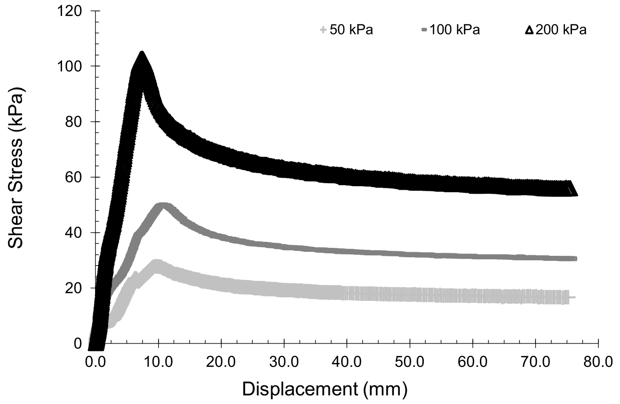
TRI Log #: A24-375
Test Method: ASTM D5321

Tested Interface: Geotextile vs Geomembrane No 3



Test Results		
	Peak	**Large Displacement (@ 75 mm)**
Friction Angle (degrees):	**26.7**	**14.6**
Y-intercept or Adhesion (kPa):	**1**	**4**

Shearing occurred at the GT/GM interface .



Test Conditions

Upper Box: Bidim A84 Nonwoven Geotextile Roll No. 1743412

Lower Box: Solmax Double sided Textured LLDPE Roll No. 0904-094695

Box Dimensions: 305 mm x 305 mm x 102 mm

Interface Conditioning: Interface soaked and loading applied for a minimum of 24 hours prior to shear.

Test Condition: Wet

Shearing Rate: 0.1 mm/minute

Test Data

Specimen No.	1	2	3
Bearing Slide Resistance (kPa)	1	1	2
Normal Stress (kPa)	50	100	200
Corrected Peak Shear Stress (kPa)	28	50	103
Corrected Large Displacement Shear Stress (kPa)	17	31	56
Peak Secant Angle (degrees)	29.0	26.6	27.2
Large Displacement Secant Angle (degrees)	18.5	17.0	15.6
Asperity (mm)	0.500	0.550	0.550

Warren Hornsey, Pr.Eng.

Director
Approved Signatory

END OF REPORT

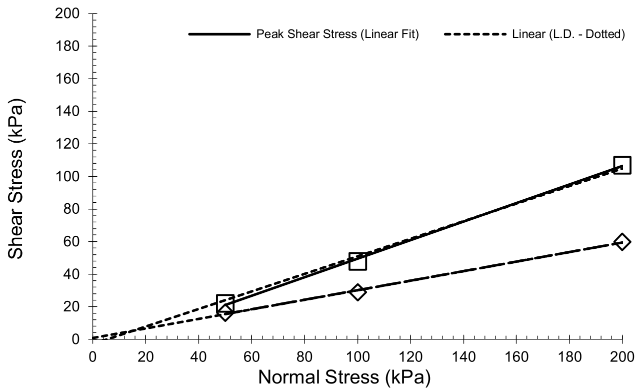
Interface Friction Test Report

Accredited for compliance with ISO/IEC 17025 & GAI-LAP Testing

Client: **Talis Consulting**
Project: **Tamala Park Cap**
Date: 29-11-2024 to 30-11-2024

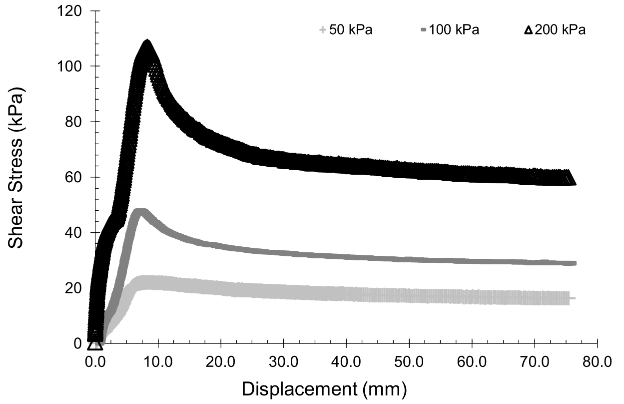
TRI Log #: A24-375
Test Method: ASTM D5321

Tested Interface: Geotextile vs Geomembrane No 4



Test Results		
	Peak	**Large Displacement (@ 75 mm)**
Friction Angle (degrees):	**27.5**	**16.4**
Y-intercept or Adhesion (kPa):	**0**	**1**

Shearing occurred at the GT/GM interface . The peak friction angle regression analysis was adjusted to fit a zero y-intercept.



Test Conditions	
Upper Box	Bidim A84 Nonwoven Geotextile Roll No. 1743412
Lower Box	Solmax Double sided Textured LLDPE Roll No. 0904-094695
Box Dimensions:	305 mm x 305 mm x 102 mm
Interface Conditioning:	Interface soaked and loading applied for a minimum of 24 hours prior to shear.
Test Condition:	Wet
Shearing Rate:	0.1 mm/minute

Test Data			
Specimen No.	1	2	3
Bearing Slide Resistance (kPa)	1	1	2
Normal Stress (kPa)	50	100	200
Corrected Peak Shear Stress (kPa)	22	48	107
Corrected Large Displacement Shear Stress (kPa)	16	29	60
Peak Secant Angle (degrees)	23.8	25.6	28.1
Large Displacement Secant Angle (degrees)	18.1	16.1	16.7
Asperity (mm)	0.625	0.675	0.700

Warren Hornsey, Pr.Eng.

Director
Approved Signatory

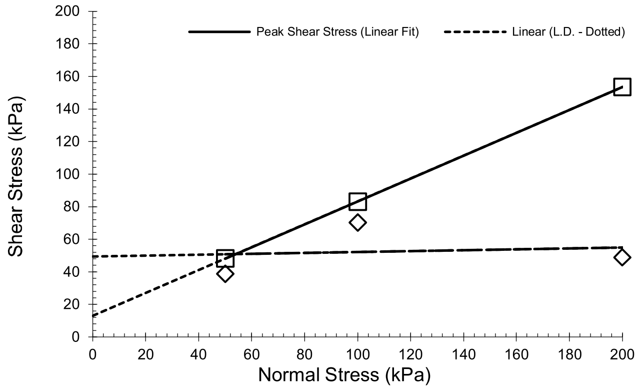
END OF REPORT

Interface Friction Test Report

Accredited for compliance with ISO/IEC 17025 & GAI-LAP Testing

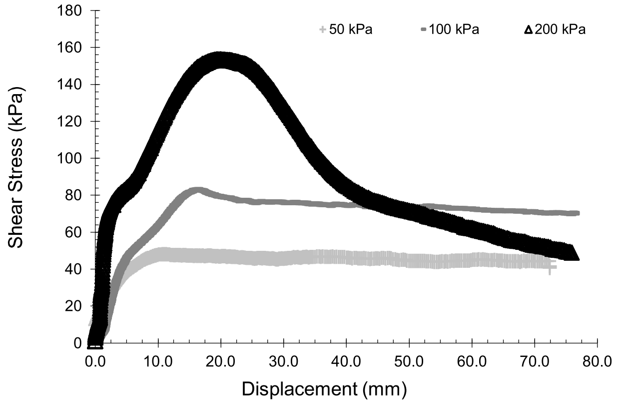
Client: **Talis** TRI Log #: A25-013
Project: **Tamala Park** Test Method: ASTM D6243
Date: 12-03-2025 to 14-03-2025

Tested Interface: Elcoseal X2000 GCL v Subgrade No 1 NO CUT



Test Results		
	Peak	Large Displacement (@ 75 mm)
Friction Angle (degrees):	35.1	1.6
Y-intercept or Adhesion (kPa):	13	49.4

Shearing occurred at the GCL/Sub interface under 50kPa, 100kPa load. Internal Shearing of the GCL occurred at the Cover/Carrier interface under 200kPa loads.



Test Conditions	
Upper Box	Elcoseal X2000 GCL Carrier Layer Facing Down
Lower Box	Subgrade, Compacted to 95% of 1790kg/m3 @13.7% moisture content
Box Dimensions: 305 mm x 305 mm x 102 mm	
Interface Conditioning:	Interface soaked and loading applied for a minimum of 24 hour prior to shear.
Test Condition:	Wet
Shearing Rate:	0.1 mm/minute

Test Data			
Specimen No.	1	2	3
Bearing Slide Resistance (kPa)	1	1	2
Normal Stress (kPa)	50	100	200
Corrected Peak Shear Stress (kPa)	48	83	153
Corrected Large Displacement Shear Stress (kPa)	39	70	49
Peak Secant Angle (degrees)	43.9	39.7	37.5
Large Displacement Secant Angle (degrees)	37.7	35.1	13.7

Warren Hornsey, Pr.Eng.

Director
Approved Signatory

200kPa

Bunching of GCL at traing edge

"necking" of GCL and Hole Elongation along clamped edg

Complete delamination in 300mm x 300mm confinement zone

END OF REPORT

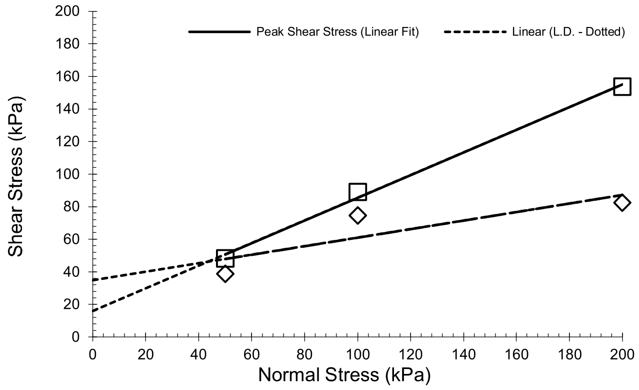
Interface Friction Test Report

Accredited for compliance with ISO/IEC 17025 & GAI-LAP Testing

Client: **Talis**
Project: **Tamala Park**
Date: 17-03-2025 to 19-03-2025

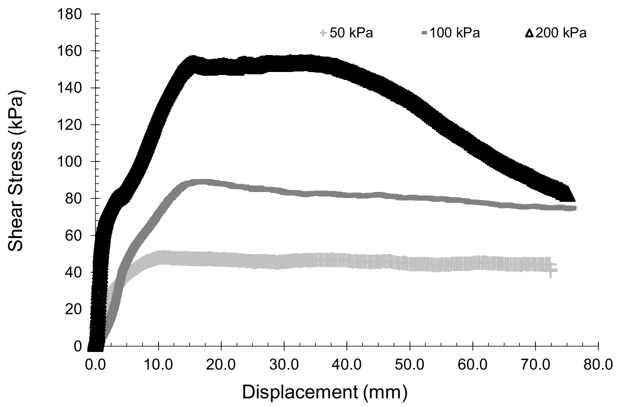
TRI Log #: A25-013
Test Method: ASTM D6243

Tested Interface: Elcoseal X2000 GCL v Subgrade No 2 NO CUT



Test Results		
	Peak	**Large Displacement (@ 75 mm)**
Friction Angle (degrees):	**34.8**	**14.7**
Y-intercept or Adhesion (kPa):	**16**	**34.7**

Shearing occurred at the GCL/Sub interface under 50kPa, 100kPa load. Internal Shearing of the GCL occurred at the Cover/Carrier interface under 200kPa loads.



Test Conditions	
Upper Box	Elcoseal X2000 GCL Carrier Layer Facing Down
Lower Box	Subgrade, Compacted to 95% of 1790kg/m3 @13.7% moisture content
Box Dimensions:	305 mm x 305 mm x 102 mm
Interface Conditioning:	Interface soaked and loading applied for a minimum of 24 hour prior to shear.
Test Condition:	Wet
Shearing Rate:	0.1 mm/minute

Test Data

Specimen No.	1	2	3
Bearing Slide Resistance (kPa)	1	1	2
Normal Stress (kPa)	50	100	200
Corrected Peak Shear Stress (kPa)	48	89	154
Corrected Large Displacement Shear Stress (kPa)	39	75	83
Peak Secant Angle (degrees)	43.9	41.7	37.5
Large Displacement Secant Angle (degrees)	37.7	36.7	22.4

Warren Hornsey, Pr.Eng.

Director
Approved Signatory

200kPa

END OF REPORT

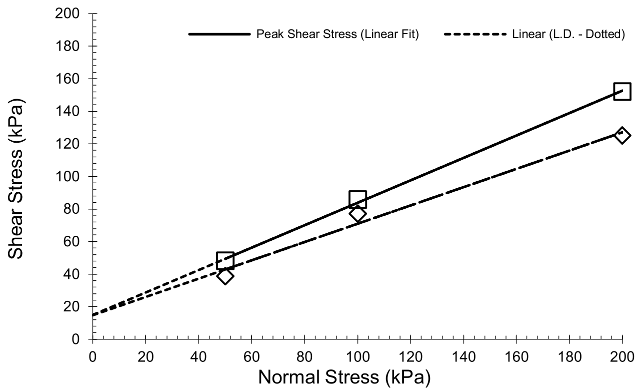
Interface Friction Test Report

Accredited for compliance with ISO/IEC 17025 & GAI-LAP Testing

Client: **Talis**
Project: **Tamala Park**
Date: 17-03-2025 to 22-03-2025

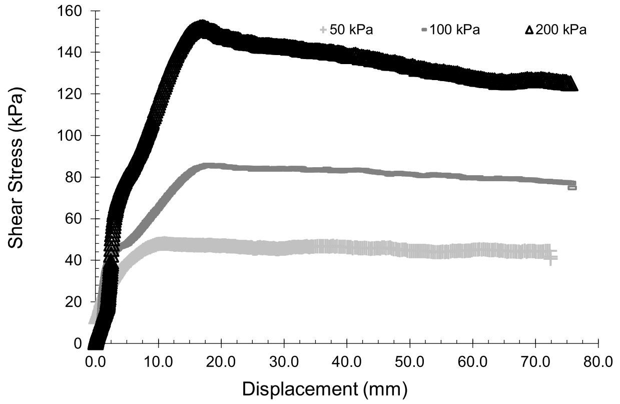
TRI Log #: A25-013
Test Method: ASTM D6243

Tested Interface: Elcoseal X2000 GCL v Subgrade No 3 NO CUT



Test Results		
	Peak	**Large Displacement (@ 75 mm)**
Friction Angle (degrees):	**34.6**	**29.3**
Y-intercept or Adhesion (kPa):	**15**	**14.7**

Shearing occurred at the GCL/Subgrade interface under 50, 100 & 200kPa loads.



Test Conditions	
Upper Box	Elcoseal X2000 GCL Carrier Layer Facing Down
Lower Box	Subgrade, Compacted to 95% of 1790kg/m3 @13.7% moisture content
Box Dimensions:	305 mm x 305 mm x 102 mm
Interface Conditioning:	Interface soaked and loading applied for a minimum of 24 hour prior to shear.
Test Condition:	Wet
Shearing Rate:	0.1 mm/minute

Test Data			
Specimen No.	1	2	3
Bearing Slide Resistance (kPa)	1	1	2
Normal Stress (kPa)	50	100	200
Corrected Peak Shear Stress (kPa)	48	86	152
Corrected Large Displacement Shear Stress (kPa)	39	77	125
Peak Secant Angle (degrees)	43.9	40.6	37.2
Large Displacement Secant Angle (degrees)	37.7	37.6	32.0

Warren Hornsey, Pr.Eng.

Director
Approved Signatory

END OF REPORT

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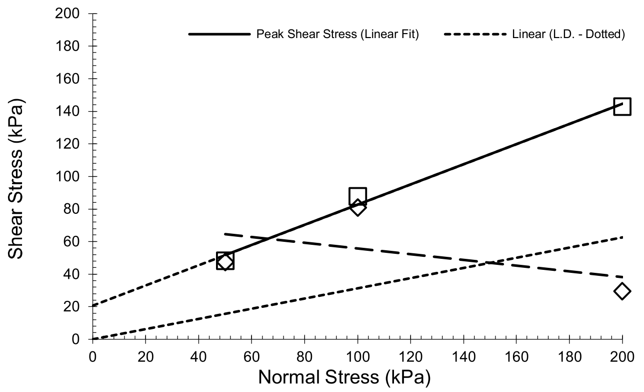
Interface Friction Test Report

Accredited for compliance with ISO/IEC 17025 & GAI-LAP Testing

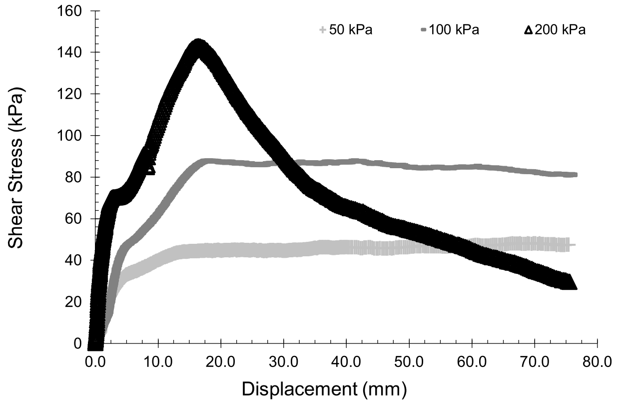
Client: **Talis** TRI Log #: A25-013
Project: **Tamala Park** Test Method: ASTM D6243
Date: 20-03-2025 to 26-03-2025

Tested Interface: Elcoseal X2000 GCL v Subgrade No 4 NO CUT



Test Results		
	Peak	Large Displacement (@ 75 mm)
Friction Angle (degrees):	31.8	17.3
Y-intercept or Adhesion (kPa):	21	0.0

Shearing occurred at the GCL/Subgrade interface under 50, 100kPa loads. Internal Shearing of the GCL occurred at the Cover/Carrier interface under 200kPa loads. The large displacement friction angle regression analysis was adjusted to fit a zero y-intercept.



Test Conditions	
Upper Box	Elcoseal X2000 GCL Carrier Layer Facing Down
Lower Box	Subgrade, Compacted to 95% of 1790kg/m^3 @13.7% moisture content
Box Dimensions:	305 mm x 305 mm x 102 mm
Interface Conditioning:	Interface soaked and loading applied for a minimum of 24 hour prior to shear.
Test Condition:	Wet
Shearing Rate:	0.1 mm/minute

Test Data			
Specimen No.	1	2	3
Bearing Slide Resistance (kPa)	1	1	2
Normal Stress (kPa)	50	100	200
Corrected Peak Shear Stress (kPa)	48	88	143
Corrected Large Displacement Shear Stress (kPa)	48	81	30
Peak Secant Angle (degrees)	43.9	41.3	35.6
Large Displacement Secant Angle (degrees)	43.5	39.0	8.4

Warren Hornsey, Pr.Eng.

Director
Approved Signatory

200kPa

END OF REPORT

TESTING, RESEARCH, CONSULTING AND FIELD SERVICES

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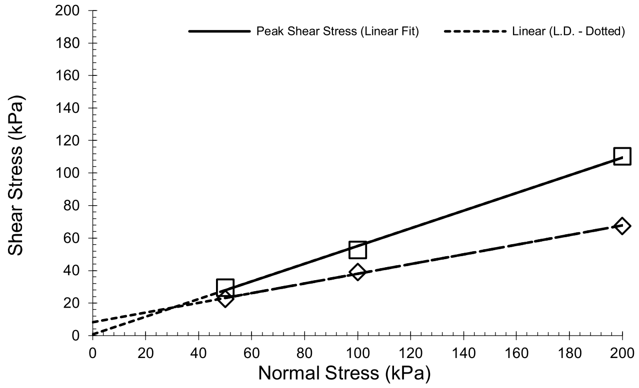
Interface Friction Test Report

Accredited for compliance with ISO/IEC 17025 & GAI-LAP Testing

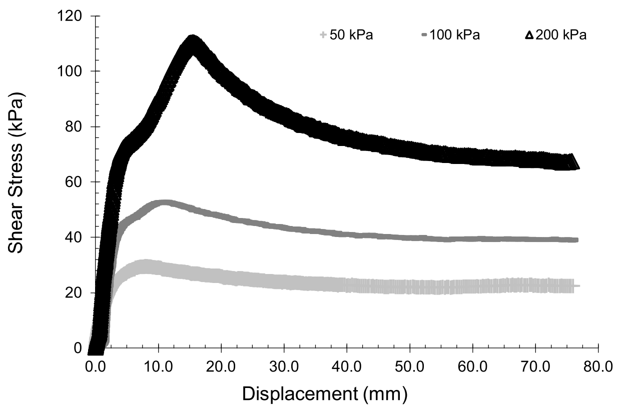
Client: **Talis** TRI Log #: A25-013
Project: **Tamala Park** Test Method: ASTM D6243
Date: 12-03-2025 to 15-03-2025

Tested Interface: Elcoseal X2000 GCL v TXGM No 1 NO CUT



Test Results		
	Peak	Large Displacement (@ 75 mm)
Friction Angle (degrees):	**28.6**	**16.6**
Y-intercept or Adhesion (kPa):	**1**	**8.2**

Shearing occurred at the GCL/TXGM interface under 50kPa, 100kPa and 200kPa load.



Test Conditions	
Upper Box	Elcoseal X2000 GCL Cover Layer Facing Down
Lower Box	Solmax Double Sided Textured LLDPE Geomembrane
Box Dimensions: 305 mm x 305 mm x 102 mm	
Interface Conditioning:	Interface soaked and loading applied for a minimum of 24 hour prior to shear.
Test Condition: Wet	
Shearing Rate: 0.1 mm/minute	

Test Data			
Specimen No.	1	2	3
Bearing Slide Resistance (kPa)	1	1	2
Normal Stress (kPa)	50	100	200
Corrected Peak Shear Stress (kPa)	30	53	110
Corrected Large Displacement Shear Stress (kPa)	23	39	68
Peak Secant Angle (degrees)	30.6	27.8	28.9
Large Displacement Secant Angle (degrees)	24.2	21.3	18.6
Asperity (mm)	0.550	0.600	0.525

Warren Hornsey, Pr.Eng.

Director
Approved Signatory

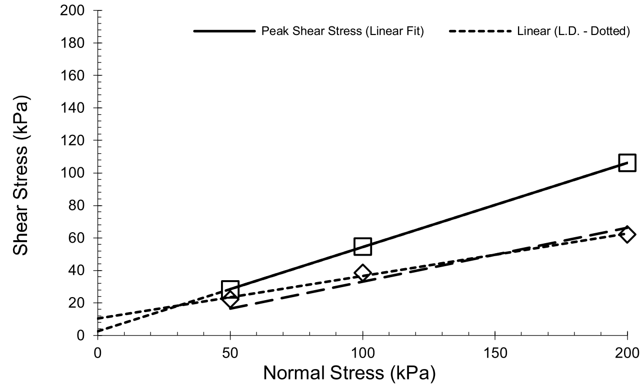
END OF REPORT

Interface Friction Test Report

Accredited for compliance with ISO/IEC 17025 & GAI-LAP Testing

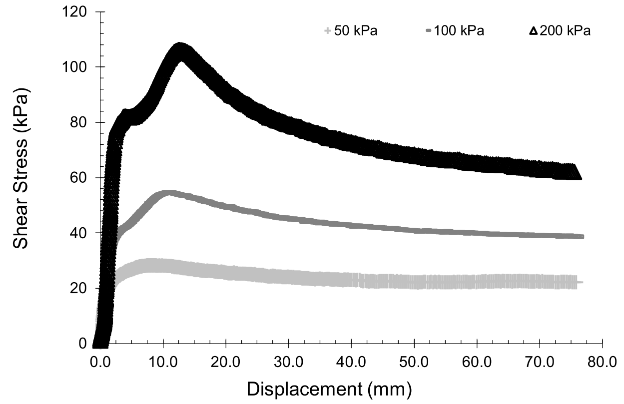
Client: **Talis**
Project: **Tamala Park**
Date: 16-02-2025 to 29-03-2025
TRI Log #: A25-013
Test Method: ASTM D6243

Tested Interface: Elcoseal X2000 GCL No Cut v Solmax Double Sided Textured LLDPE No 2



Test Results		
	Peak	**Large Displacement (@ 75 mm)**
Friction Angle (degrees):	**27.4**	**14.7**
Y-intercept or Adhesion (kPa):	**3**	**10**

Shearing occurred at the GCL/TXGM interface under 50kPa, 100kPa & 200kPa loads.



Test Conditions	
Upper Box	Elcoseal X2000 GCL Cover Layer Facing Down
Lower Box	Solmax Double Sided Textured LLDPE Geomembrane
Box Dimensions:	305 mm x 305 mm x 102 mm
Interface Conditioning:	Interface soaked and loading applied for a minimum of 24 hour prior to shear.
Test Condition:	Wet
Shearing Rate:	0.1 mm/minute

Test Data			
Specimen No.	1	2	3
Bearing Slide Resistance (kPa)	1	1	2
Normal Stress (kPa)	50	100	200
Corrected Peak Shear Stress (kPa)	28	55	106
Corrected Large Displacement Shear Stress (kPa)	22	39	62
Peak Secant Angle (degrees)	29.5	28.7	28.0
Large Displacement Secant Angle (degrees)	23.8	21.1	17.2
Asperity (mm)	0.550	0.525	0.525

Warren Hornsey

Director
Approved Signatory

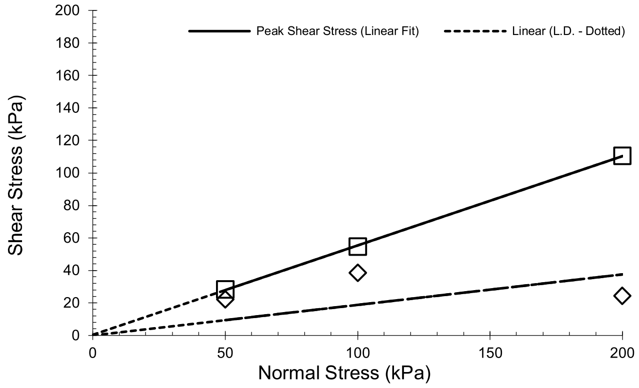
END OF REPORT

Interface Friction Test Report

Accredited for compliance with ISO/IEC 17025 & GAI-LAP Testing

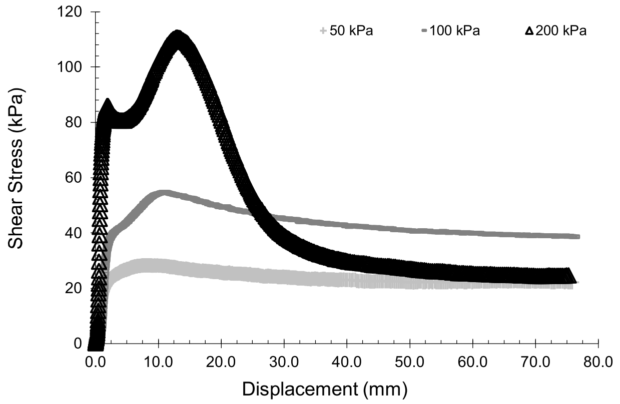
Client: **Talis**
Project: **Tamala Park**
Date: 16-02-2025 to 29-03-2025
TRI Log #: A25-013
Test Method: ASTM D6243

Tested Interface: Elcoseal X2000 GCL No Cut v Solmax Double Sided Textured LLDPE No 3



Test Results		
	Peak	Large Displacement (@ 75 mm)
Friction Angle (degrees):	28.8	10.6
Y-intercept or Adhesion (kPa):	0.3	0.0

Shearing occurred at the GCL/TXGM interface under 50kPa and 100kPa loads. GCL Internally sheared under 200kPa load. large displacement friction angle regression analyses was adjusted to fit a zero y-intercept.



Test Conditions	
Upper Box	Elcoseal X2000 GCL Cover Layer Facing Down
Lower Box	Solmax Double Sided Textured LLDPE Geomembrane
Box Dimensions:	305 mm x 305 mm x 102 mm
Interface Conditioning:	Interface soaked and loading applied for a minimum of 24 hour prior to shear.
Test Condition:	Wet
Shearing Rate:	0.1 mm/minute

Test Data

Specimen No.	1	2	3
Bearing Slide Resistance (kPa)	1	1	2
Normal Stress (kPa)	50	100	200
Corrected Peak Shear Stress (kPa)	28	55	111
Corrected Large Displacement Shear Stress (kPa)	22	39	24
Peak Secant Angle (degrees)	29.5	28.7	28.9
Large Displacement Secant Angle (degrees)	23.8	21.1	7.0
Asperity (mm)	0.550	0.525	0.700

Warren Hornsey

Director
Approved Signatory

200kPa

END OF REPORT

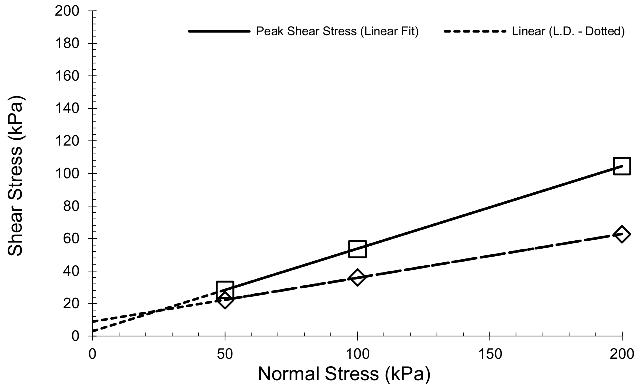
Interface Friction Test Report

Accredited for compliance with ISO/IEC 17025 & GAI-LAP Testing

Client: **Talis**
Project: **Tamala Park**
Date: 30-03-2025 to 1-04-2025

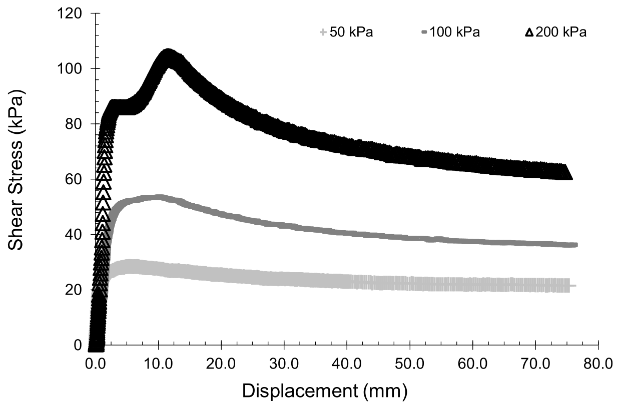
TRI Log #: A25-013
Test Method: ASTM D6243

Tested Interface: Elcoseal X2000 GCL No Cut v Solmax Double Sided Textured LLDPE No 4



Test Results	Peak	Large Displacement (@ 75 mm)
Friction Angle (degrees):	**26.9**	**15.1**
Y-intercept or Adhesion (kPa):	**3.0**	**8.8**

Shearing occurred at the GCL/TXGM interface under 50, 100 & 200kPa loads.



Test Conditions

Upper Box: Elcoseal X2000 GCL Cover Layer Facing Down

Lower Box: Solmax Double Sided Textured LLDPE Geomembrane

Box Dimensions: 305 mm x 305 mm x 102 mm

Interface Conditioning: Interface soaked and loading applied for a minimum of 24 hour prior to shear.

Test Condition: Wet

Shearing Rate: 0.1 mm/minute

Test Data

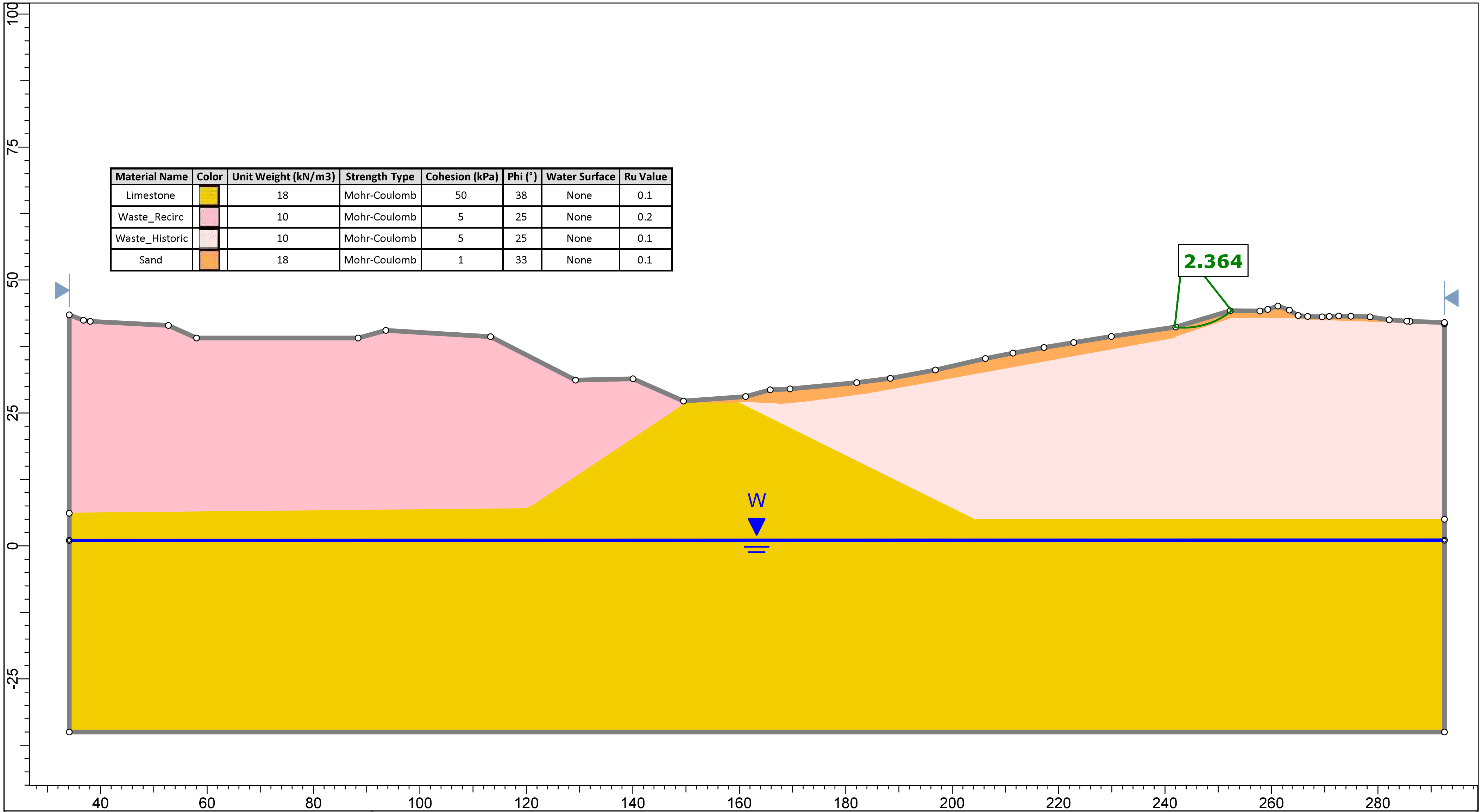
Specimen No.	1	2	3
Bearing Slide Resistance (kPa)	1	1	2
Normal Stress (kPa)	50	100	200
Corrected Peak Shear Stress (kPa)	28	54	104
Corrected Large Displacement Shear Stress (kPa)	22	36	63
Peak Secant Angle (degrees)	29.7	28.2	27.6
Large Displacement Secant Angle (degrees)	23.8	19.8	17.4
Asperity (mm)	0.475	475.000	0.525

Warren Hornsey

Director
Approved Signatory

END OF REPORT

APPENDIX C
Sideslope Stability Analysis



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Recirc		10	Mohr-Coulomb	5	25	None	0.2
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1

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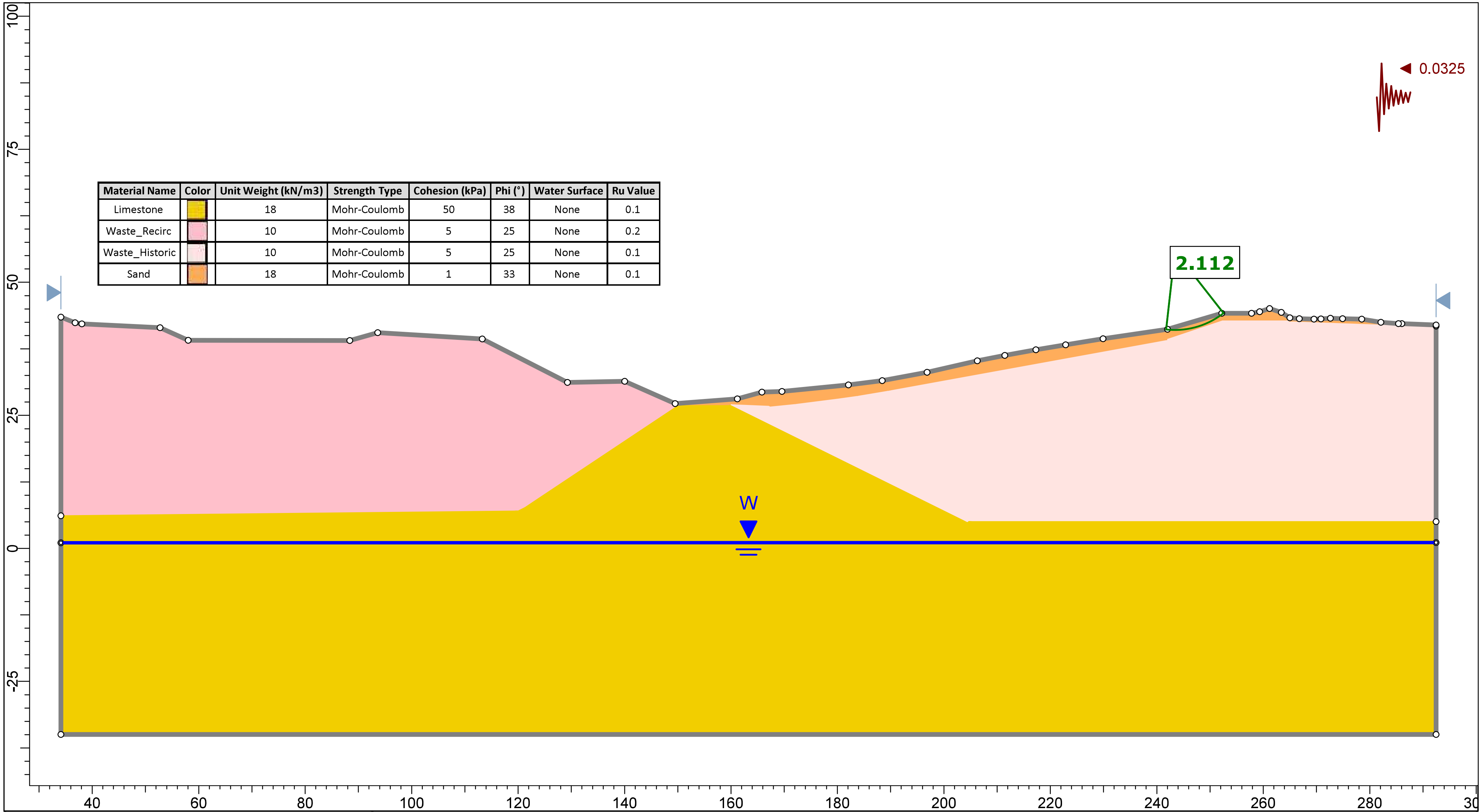
SLIDEINTERPRET 9.039

Project	MRC - Southern Piggyback Liner SRA		
Analysis Description	Circular - No Seismic Loading		
Section East-West Subgrade Circ	*Scale* 1:1000	*Company*	Talis Consultants Pty Ltd
Date	August 2025	*File Name*	Southern Piggyback Liner_East-West_Subgrade.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Recirc		10	Mohr-Coulomb	5	25	None	0.2
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1

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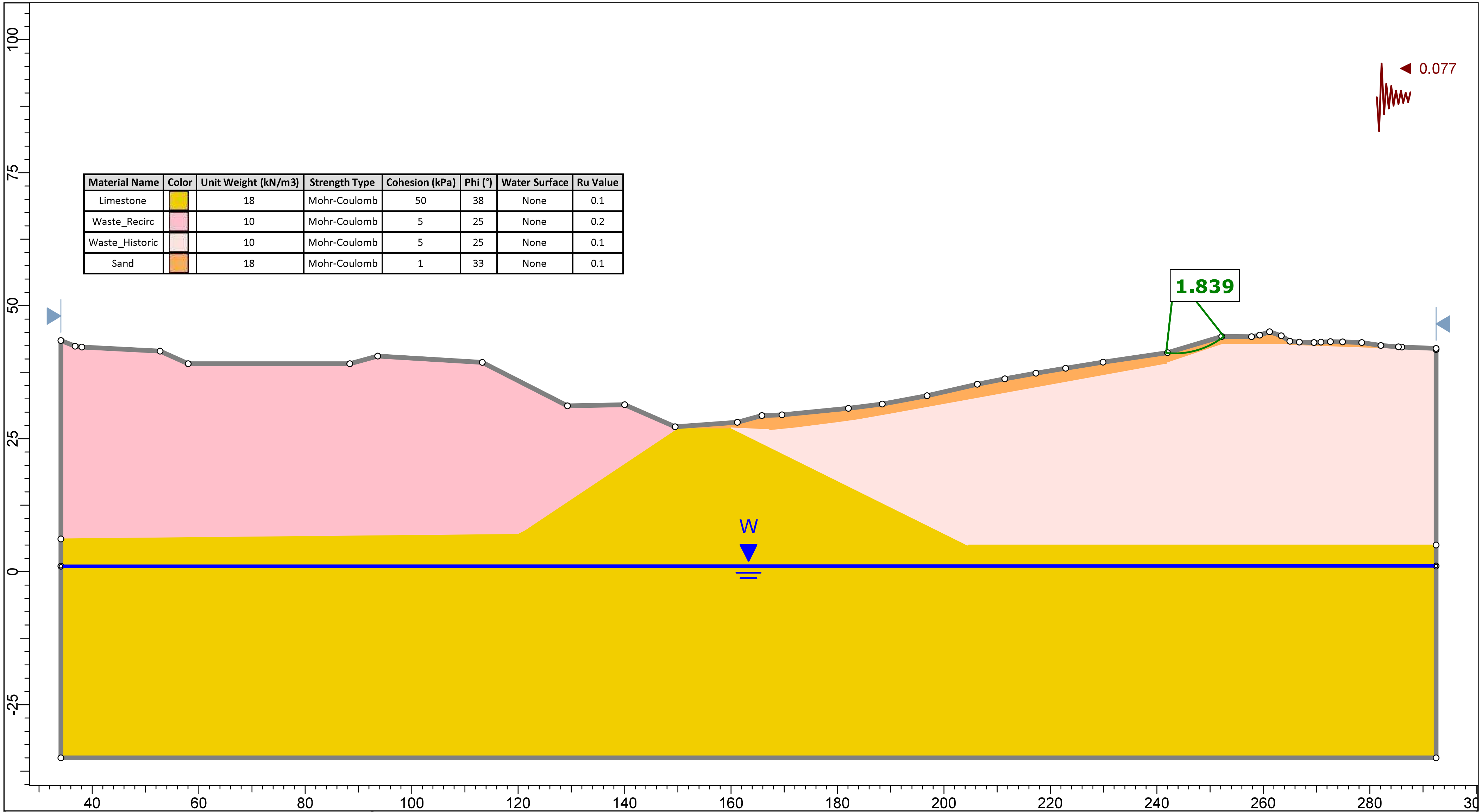
Project	MRC - Southern Piggyback Liner SRA		
Analysis Description	Circular - OBE		
Section East-West Subgrade Circ	*Scale* 1:1000	*Company*	Talis Consultants Pty Ltd
Date August 2025		*File Name*	Southern Piggyback Liner_East-West_Subgrade.slmd

SLIDEINTERPRET 9.039



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Recirc		10	Mohr-Coulomb	5	25	None	0.2
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1

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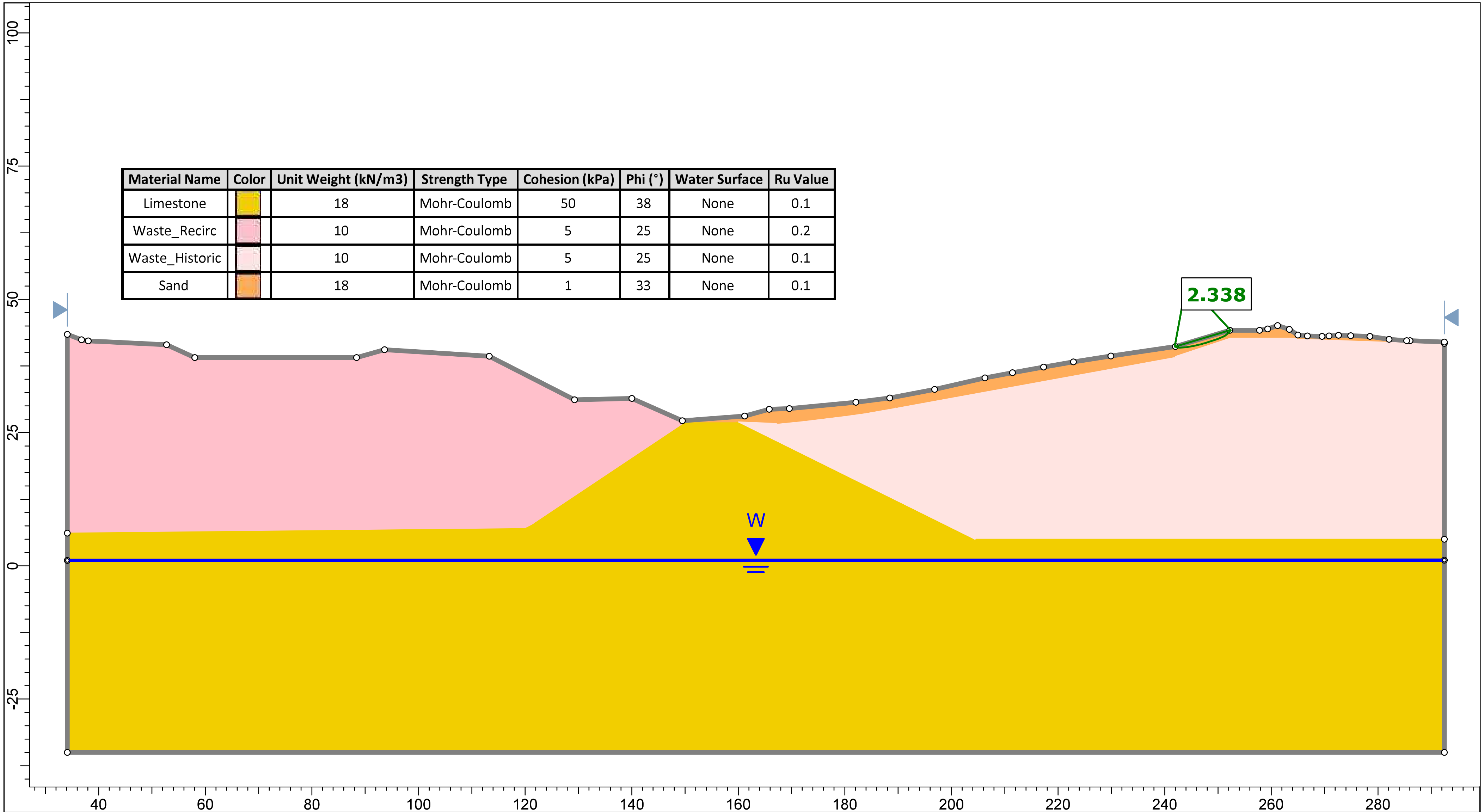
SLIDEINTERPRET 9.039

Project	MRC - Southern Piggyback Liner SRA
Analysis Description	Circular - SEE
Section	East-West Subgrade Circ
Scale	1:1000
Company	Talis Consultants Pty Ltd
Date	August 2025
File Name	Southern Piggyback Liner_East-West_Subgrade.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Recirc		10	Mohr-Coulomb	5	25	None	0.2
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1

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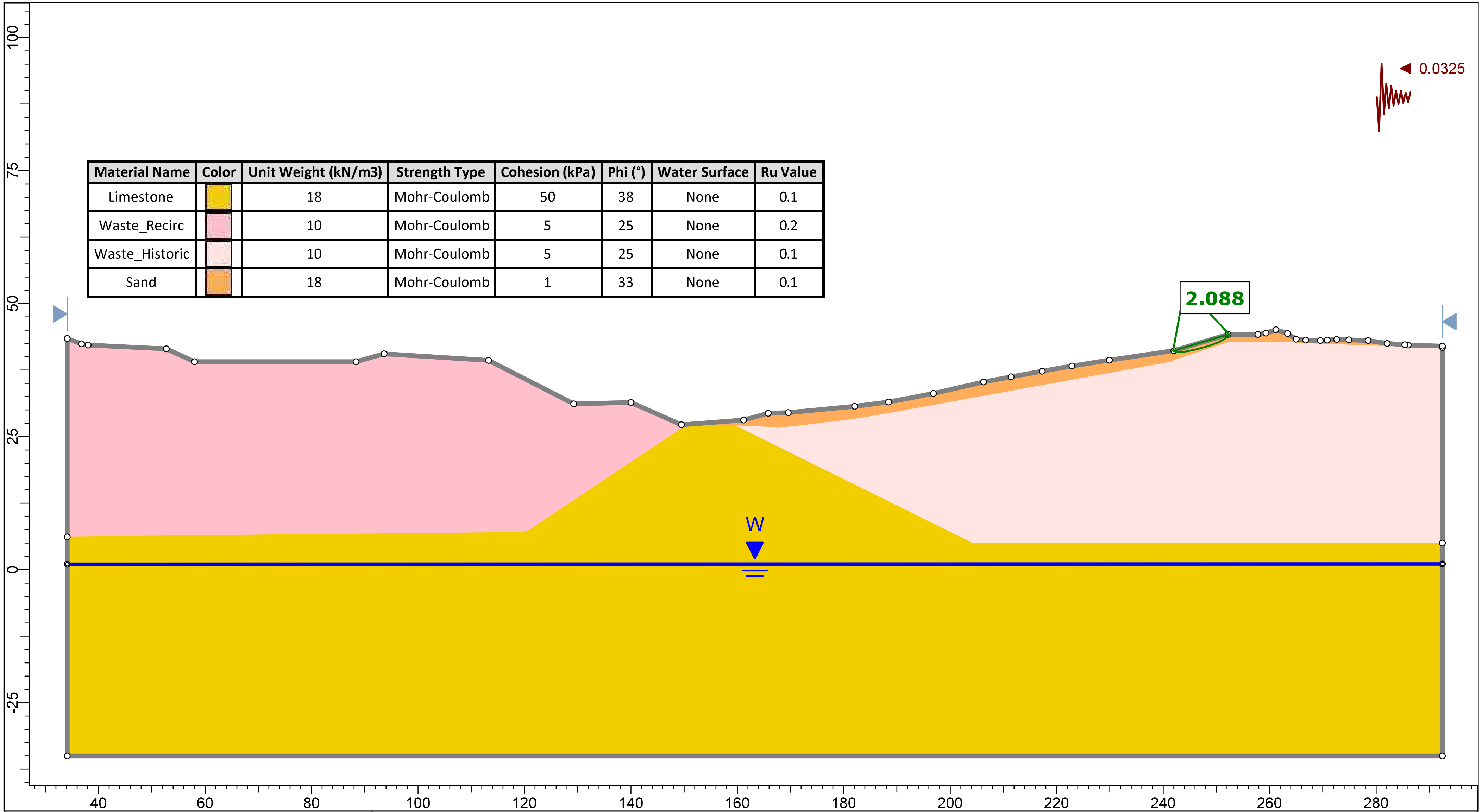
SLIDEINTERPRET 9.039

Project	MRC - Southern Piggyback Liner SRA		
Analysis Description	Non-Circular - No Seismic Loading		
Section East-West Subgrade Non Circ	*Scale* 1:1000	*Company*	Talis Consultants Pty Ltd
Date	August 2025	*File Name*	Southern Piggyback Liner_East-West_Subgrade.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Recirc		10	Mohr-Coulomb	5	25	None	0.2
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1

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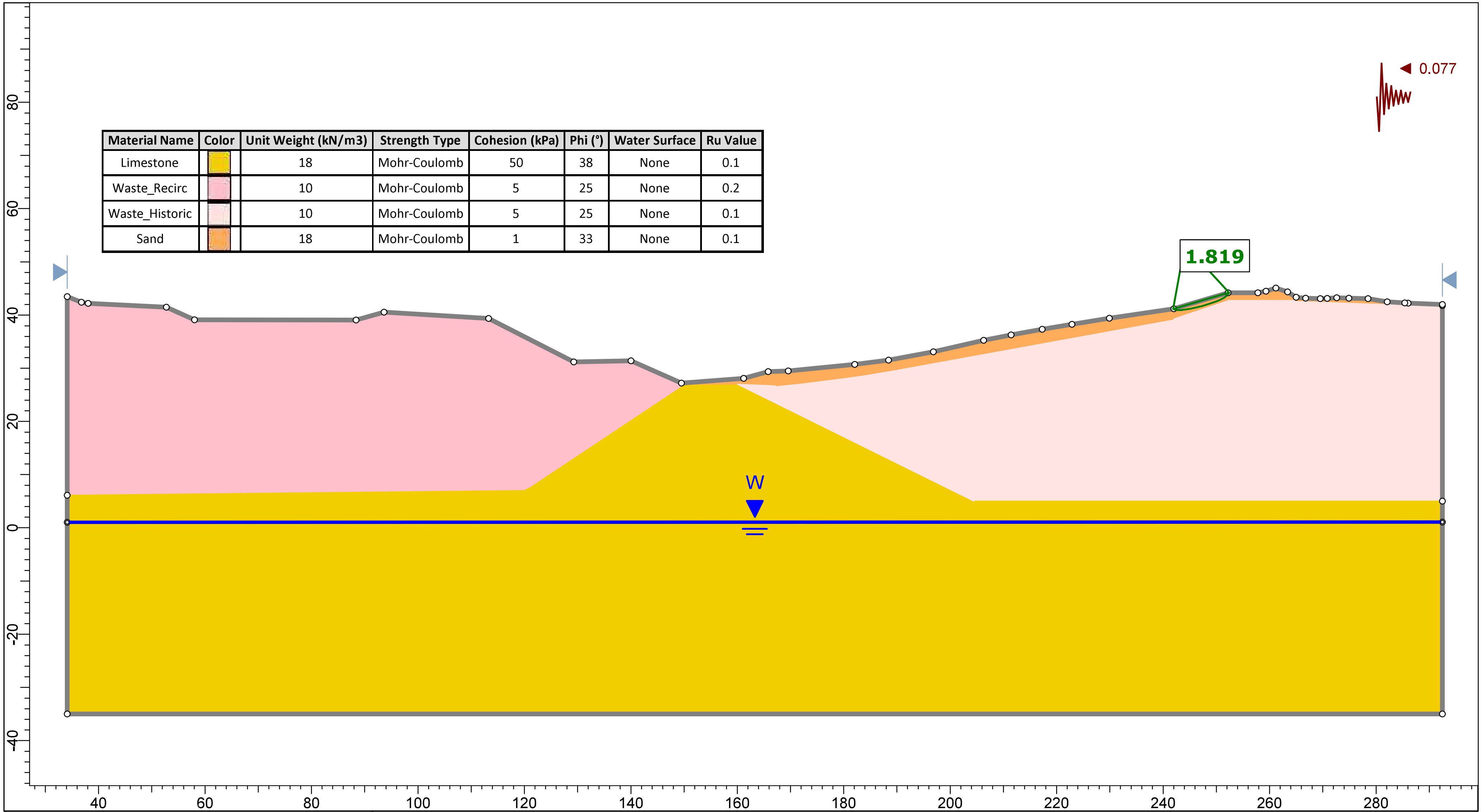
SLIDEINTERPRET 9.039

Project	MRC - Southern Piggyback Liner SRA		
Analysis Description	Non-Circular - OBE		
Section East-West Subgrade Non Circ	*Scale* 1:1000	*Company*	Talis Consultants Pty Ltd
Date	August 2025	*File Name*	Southern Piggyback Liner_East-West_Subgrade.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Recirc		10	Mohr-Coulomb	5	25	None	0.2
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1

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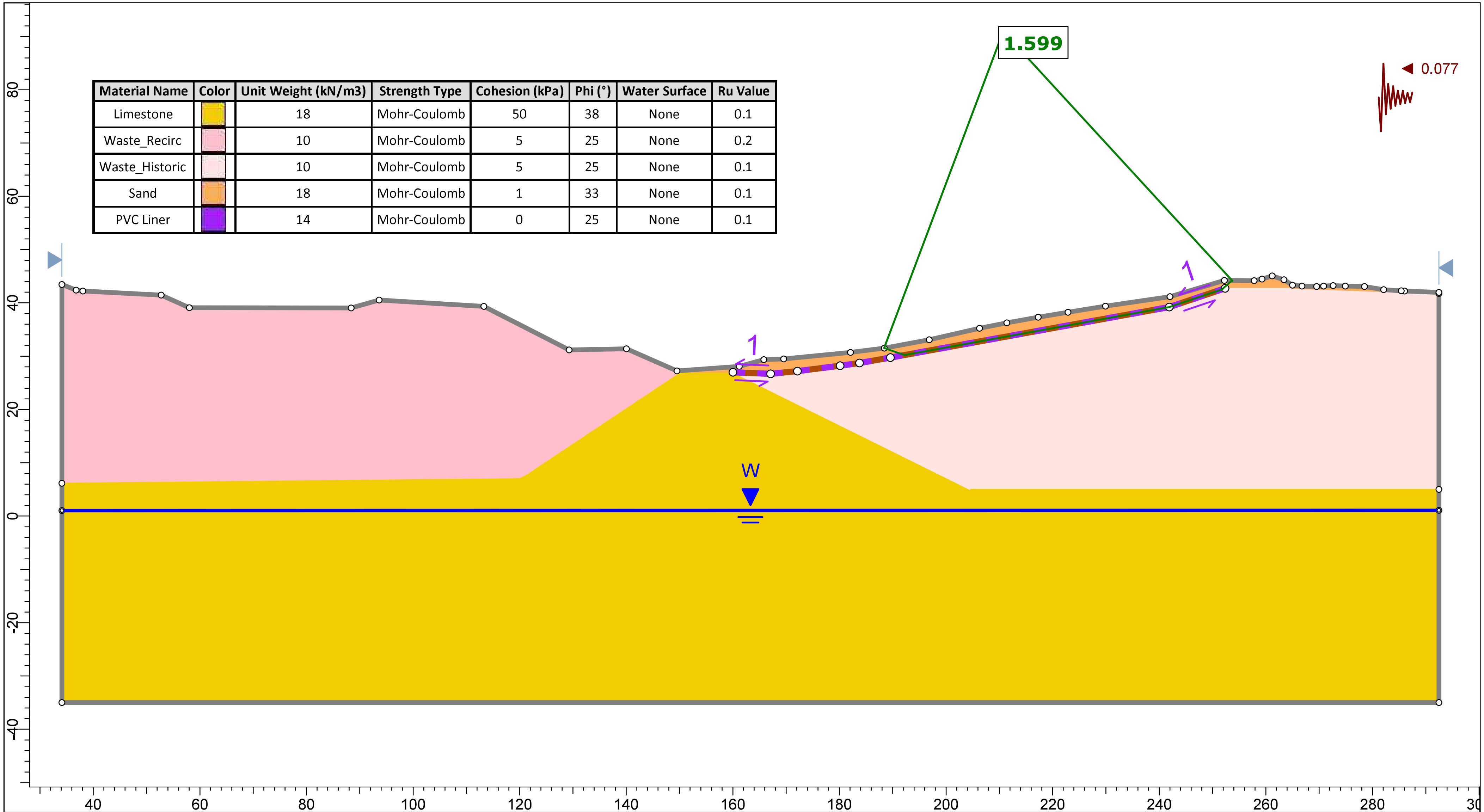
SLIDEINTERPRET 9.039

Project	MRC - Southern Piggyback Liner SRA		
Analysis Description	Non-Circular - SEE		
Section East-West Subgrade Non Circ	*Scale* 1:1000	*Company*	Talis Consultants Pty Ltd
Date	August 2025	*File Name*	Southern Piggyback Liner_East-West_Subgrade.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Recirc		10	Mohr-Coulomb	5	25	None	0.2
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1
PVC Liner		14	Mohr-Coulomb	0	25	None	0.1

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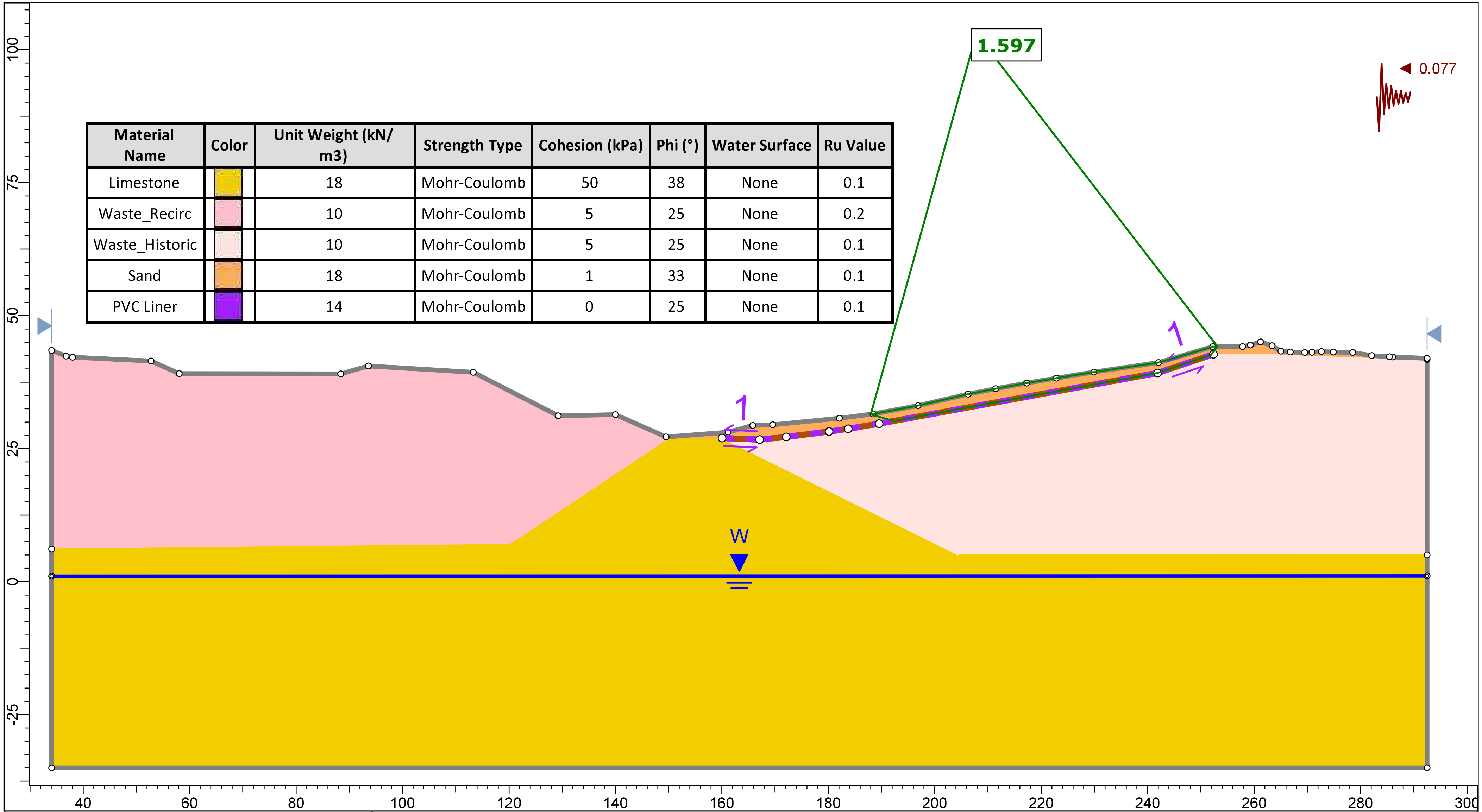
SLIDEINTERPRET 9.039

Project	MRC - Southern Piggyback Liner SRA		
Analysis Description	Circular - SEE Weak Interface		
Section East-West Subgrade Circ	*Scale* 1:1000	*Company*	Talis Consultants Pty Ltd
Date	August 2025	*File Name*	Southern Piggyback Liner_East-West_Subgrade.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Recirc		10	Mohr-Coulomb	5	25	None	0.2
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1
PVC Liner		14	Mohr-Coulomb	0	25	None	0.1

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SLIDEINTERPRET 9.039

Project MRC - Southern Piggyback Liner SRA

Analysis Description Non-Circular - SEE Weak Interface

Section East-West Subgrade Non Circ

Scale 1:1000

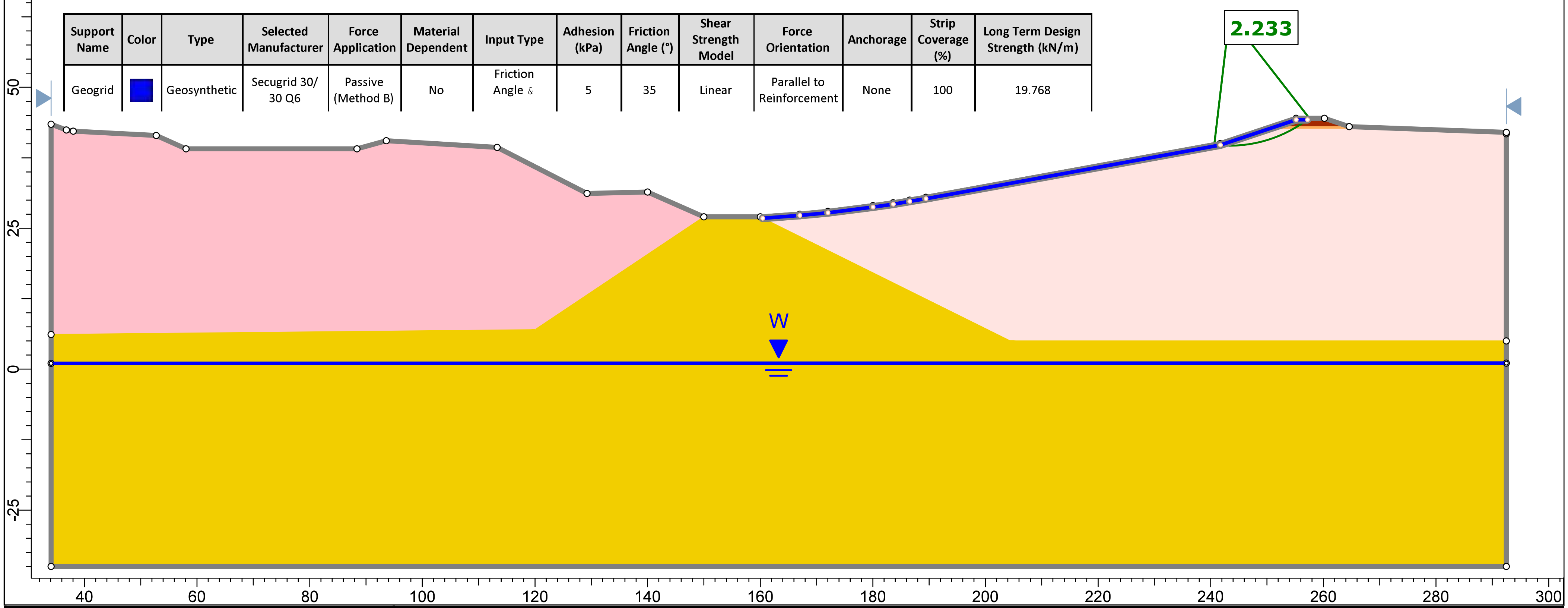
Company Talis Consultants Pty Ltd

Date August 2025

File Name Southern Piggyback Liner_East-West_Subgrade.slmd

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Recirc		10	Mohr-Coulomb	5	25	None	0.2
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1
Engineered Limestone Fill		18	Mohr-Coulomb	1	35	None	0.1

Support Name	Color	Type	Selected Manufacturer	Force Application	Material Dependent	Input Type	Adhesion (kPa)	Friction Angle (°)	Shear Strength Model	Force Orientation	Anchorage	Strip Coverage (%)	Long Term Design Strength (kN/m)
Geogrid		Geosynthetic	Secugrid 30/ 30 Q6	Passive (Method B)	No	Friction Angle &	5	35	Linear	Parallel to Reinforcement	None	100	19.768

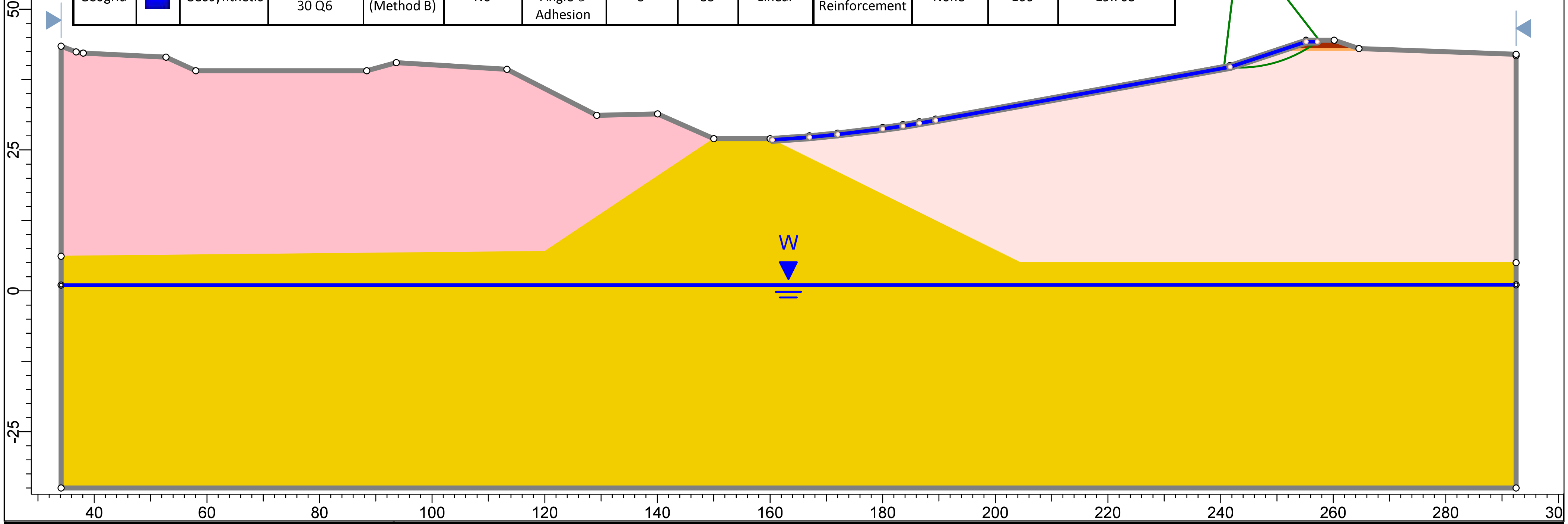


SLIDEINTERPRET 9.039

Project	MRC Southern Piggyback Liner SRA
Analysis Description	Circular_No Seismic Loading
Section	East-West Circular
Scale	1:1000
Company	Talis Consultants Pty Ltd
Date	August 2025
File Name	Southern Piggyback Liner_East-West_1V5.5H.slmd

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Recirc		10	Mohr-Coulomb	5	25	None	0.2
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1
Engineered Limestone Fill		18	Mohr-Coulomb	1	35	None	0.1

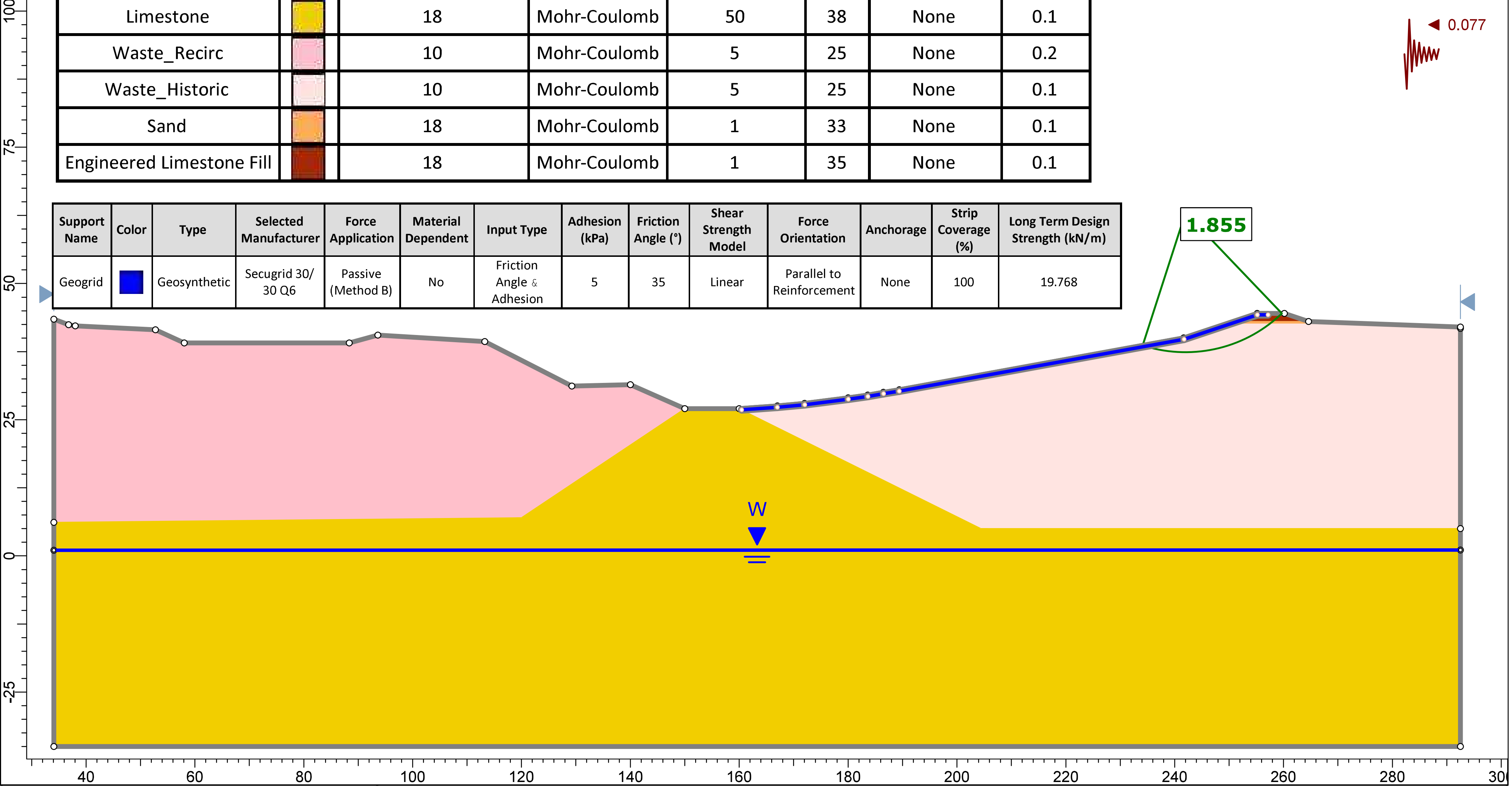
Support Name	Color	Type	Selected Manufacturer	Force Application	Material Dependent	Input Type	Adhesion (kPa)	Friction Angle (°)	Shear Strength Model	Force Orientation	Anchorage	Strip Coverage (%)	Long Term Design Strength (kN/m)
Geogrid		Geosynthetic	Secugrid 30/ 30 Q6	Passive (Method B)	No	Friction Angle & Adhesion	5	35	Linear	Parallel to Reinforcement	None	100	19.768



SLIDEINTERPRET 9.039	*Project* MRC Southern Piggyback Liner SRA		
	Analysis Description Circular_OBE		
	Section East-West Circular	*Scale* 1:1000	*Company* Talis Consultants Pty Ltd
	Date August 2025		*File Name* Southern Piggyback Liner_East-West_1V5.5H.slmd

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Recirc		10	Mohr-Coulomb	5	25	None	0.2
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1
Engineered Limestone Fill		18	Mohr-Coulomb	1	35	None	0.1

Support Name	Color	Type	Selected Manufacturer	Force Application	Material Dependent	Input Type	Adhesion (kPa)	Friction Angle (°)	Shear Strength Model	Force Orientation	Anchorage	Strip Coverage (%)	Long Term Design Strength (kN/m)
Geogrid		Geosynthetic	Secugrid 30/ 30 Q6	Passive (Method B)	No	Friction Angle & Adhesion	5	35	Linear	Parallel to Reinforcement	None	100	19.768



Project	MRC Southern Piggyback Liner SRA		
Analysis Description	Circular_SEE		
Section	East-West Circular	Scale	1:1000
Company	Talis Consultants Pty Ltd		
Date	August 2025	File Name	Southern Piggyback Liner_East-West_1V5.5H.slmd

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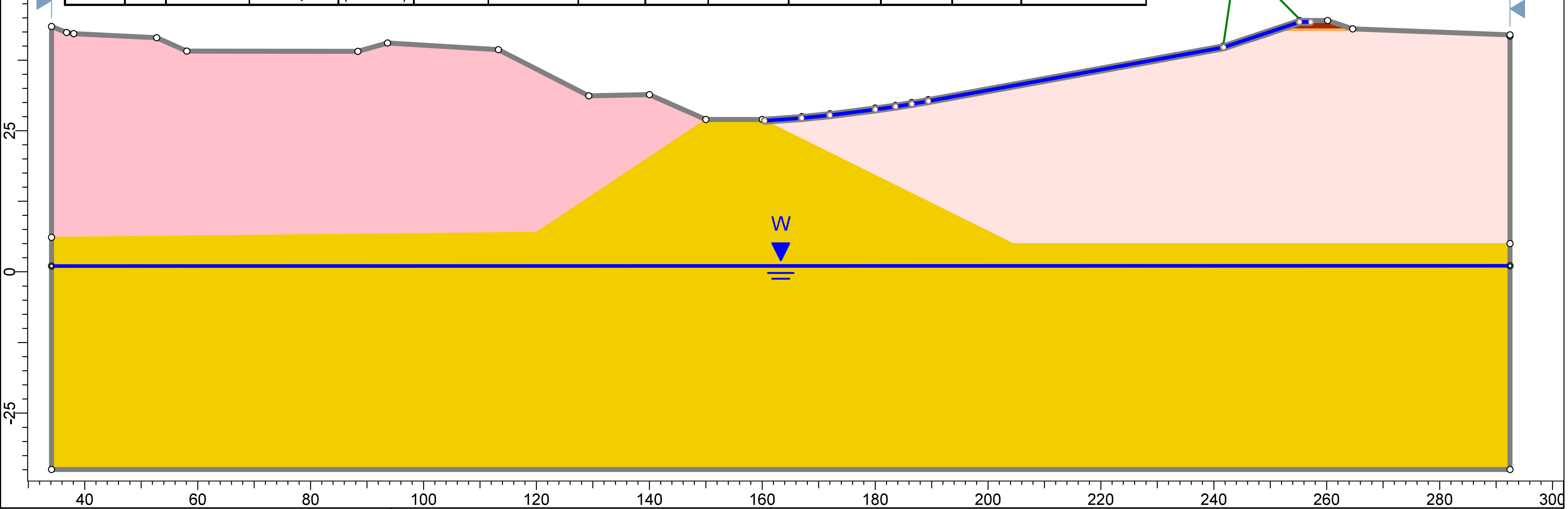
SLIDEINTERPRET 9.039

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Recirc		10	Mohr-Coulomb	5	25	None	0.2
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1
Engineered Limestone Fill		18	Mohr-Coulomb	1	35	None	0.1

Support Name	Color	Type	Selected Manufacturer	Force Application	Material Dependent	Input Type	Adhesion (kPa)	Friction Angle (°)	Shear Strength Model	Force Orientation	Anchorage	Strip Coverage (%)	Long Term Design Strength (kN/m)
Geogrid		Geosynthetic	Secugrid 30/ 30 Q6	Passive (Method B)	No	Friction Angle & Adhesion	5	35	Linear	Parallel to Reinforcement	None	100	19.768



talis delivering solutions	*Project* MRC Southern Piggyback Liner SRA
	Analysis Description NonCircular_No Seismic Loading
	Section East-West NonCircular · *Scale* 1:1000 · *Company* Talis Consultants Pty Ltd
SLIDEINTERPRET 9.039	*Date* August 2025 · *File Name* Southern Piggyback Liner_East-West_1V5.5H.slmd

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Recirc		10	Mohr-Coulomb	5	25	None	0.2
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1
Engineered Limestone Fill		18	Mohr-Coulomb	1	35	None	0.1

Support Name	Color	Type	Selected Manufacturer	Force Application	Material Dependent	Input Type	Adhesion (kPa)	Friction Angle (°)	Shear Strength Model	Force Orientation	Anchorage	Strip Coverage (%)	Long Term Design Strength (kN/m)
Geogrid		Geosynthetic	Secugrid 30/ 30 Q6	Passive (Method B)	No	Friction Angle & Adhesion	5	35	Linear	Parallel to Reinforcement	None	100	19.768

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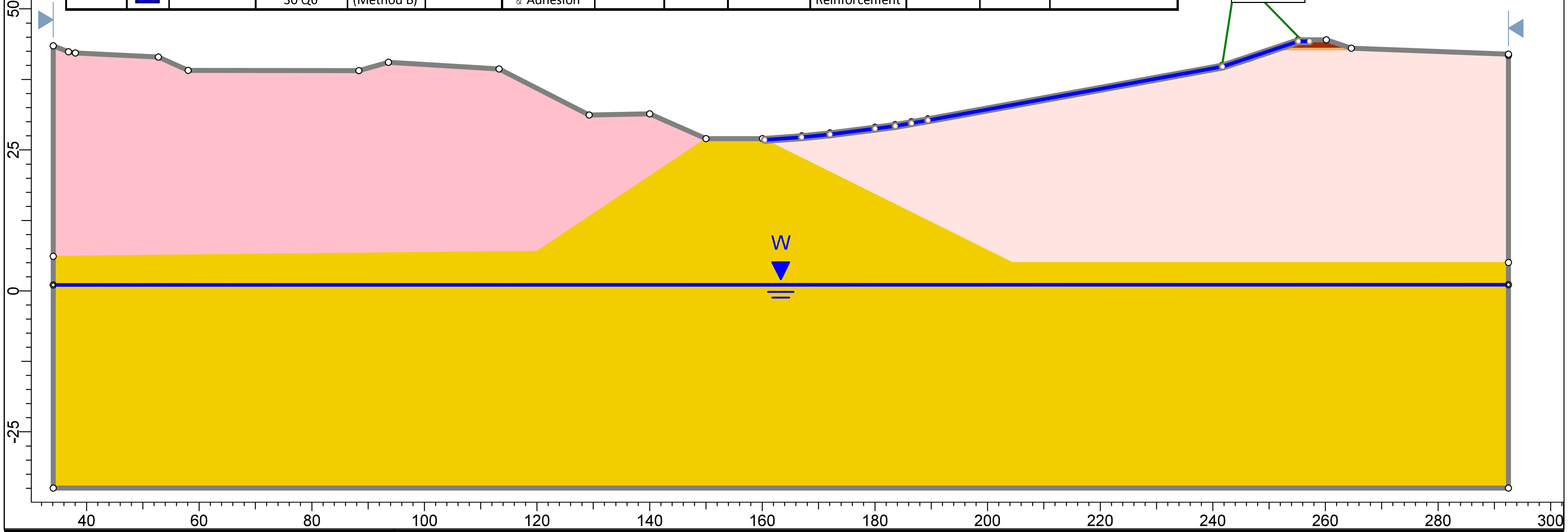

Project	MRC Southern Piggyback Liner SRA		
Analysis Description	NonCircular_OBE		
Section	East-West NonCircular	Scale	1:1000
Company	Talis Consultants Pty Ltd		
Date	August 2025	File Name	Southern Piggyback Liner_East-West_1V5.5H.slmd

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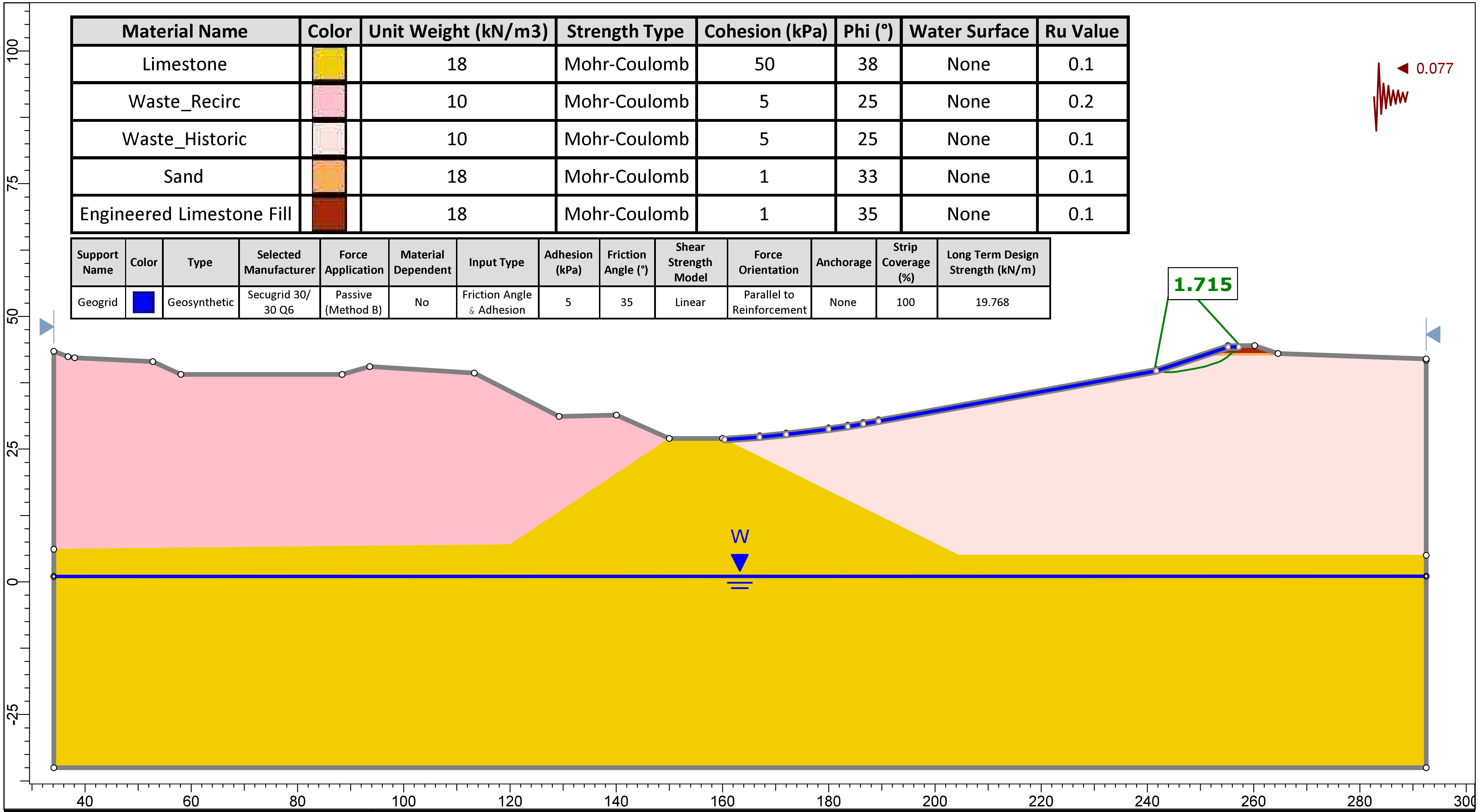
SLIDEINTERPRET 9.039

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Recirc		10	Mohr-Coulomb	5	25	None	0.2
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1
Engineered Limestone Fill		18	Mohr-Coulomb	1	35	None	0.1

Support Name	Color	Type	Selected Manufacturer	Force Application	Material Dependent	Input Type	Adhesion (kPa)	Friction Angle (°)	Shear Strength Model	Force Orientation	Anchorage	Strip Coverage (%)	Long Term Design Strength (kN/m)
Geogrid		Geosynthetic	Secugrid 30/ 30 Q6	Passive (Method B)	No	Friction Angle & Adhesion	5	35	Linear	Parallel to Reinforcement	None	100	19.768



Project	MRC Southern Piggyback Liner SRA		
Analysis Description	NonCircular_SEE		
Section East-West NonCircular	*Scale* 1:1000	*Company*	Talis Consultants Pty Ltd
Date August 2025		*File Name*	Southern Piggyback Liner_East-West_1V5.5H.slmd

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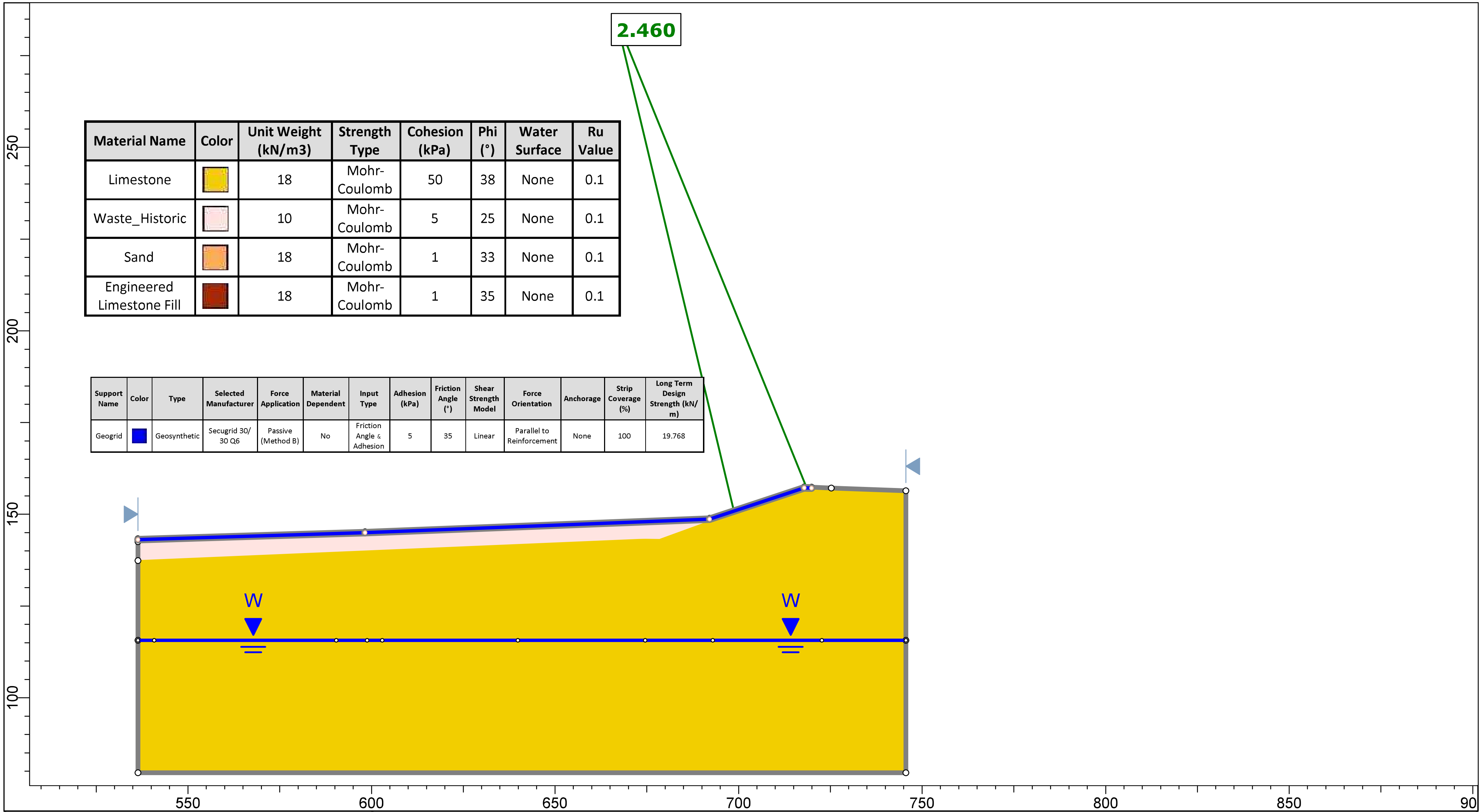
SLIDEINTERPRET 9.039



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1
Engineered Limestone Fill		18	Mohr-Coulomb	1	35	None	0.1

Support Name	Color	Type	Selected Manufacturer	Force Application	Material Dependent	Input Type	Adhesion (kPa)	Friction Angle (°)	Shear Strength Model	Force Orientation	Anchorage	Strip Coverage (%)	Long Term Design Strength (kN/m)
Geogrid		Geosynthetic	Secugrid 30/30 Q6	Passive (Method B)	No	Friction Angle & Adhesion	5	35	Linear	Parallel to Reinforcement	None	100	19.768

Project	MRC - Southern Piggyback Liner SRA
Analysis Description	Circular_No Seismic
Section	Group 2 - Circular
Scale	1:1450
Company	Talis Consultants Pty Ltd
Date	April 2025
File Name	Southern Piggyback Liner_North-South.slmd

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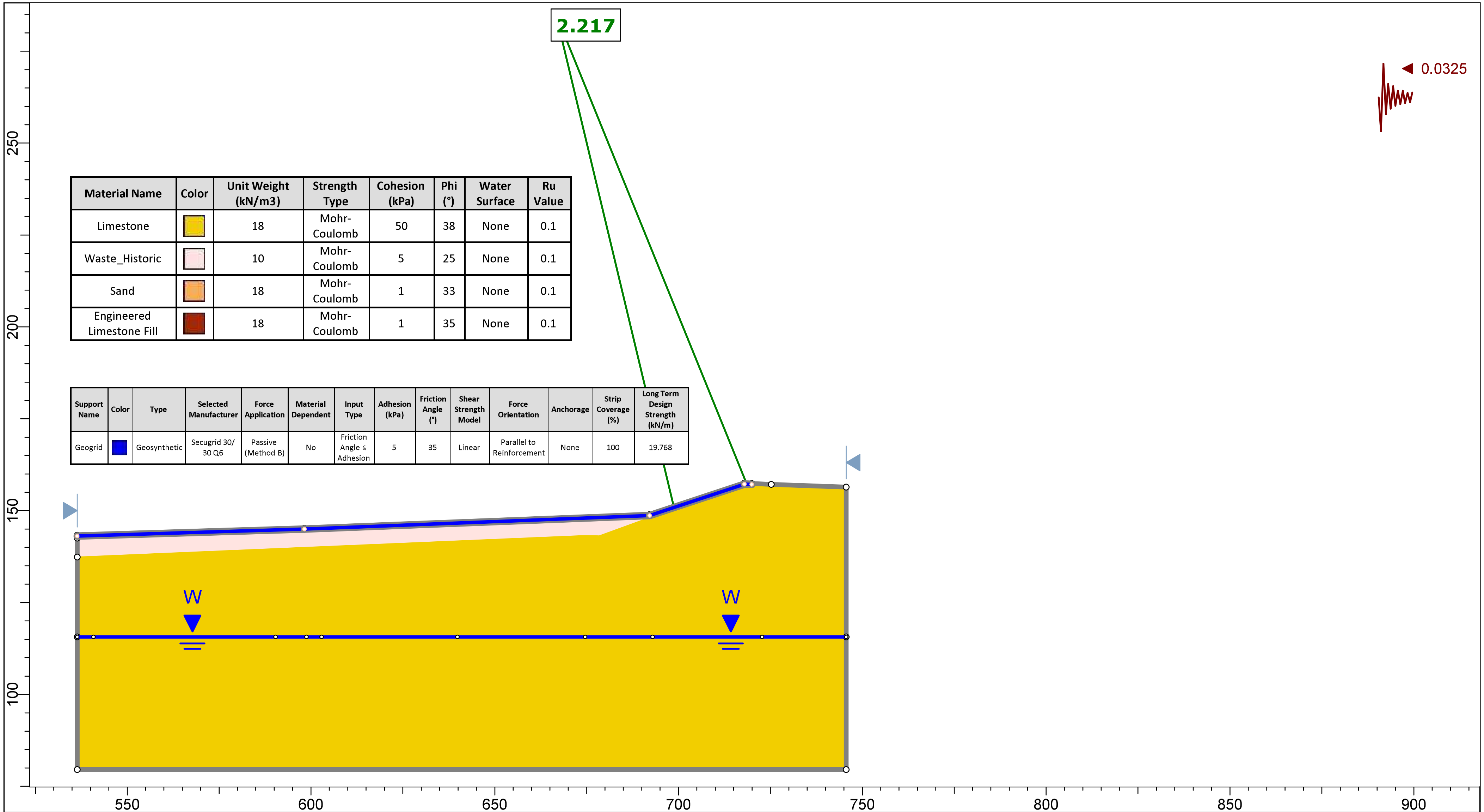
SLIDEINTERPRET 9.038

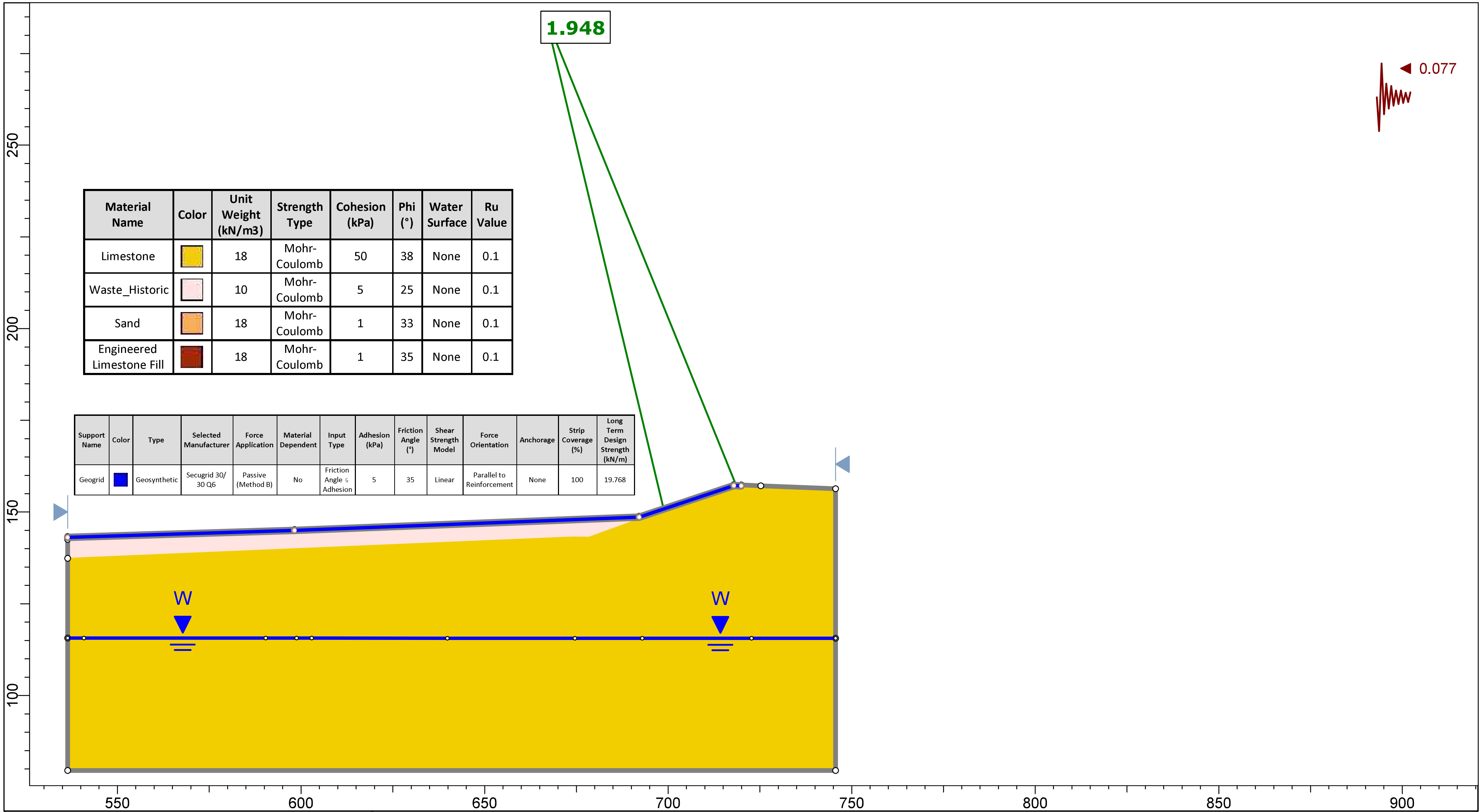


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1
Engineered Limestone Fill		18	Mohr-Coulomb	1	35	None	0.1

Support Name	Color	Type	Selected Manufacturer	Force Application	Material Dependent	Input Type	Adhesion (kPa)	Friction Angle (°)	Shear Strength Model	Force Orientation	Anchorage	Strip Coverage (%)	Long Term Design Strength (kN/m)
Geogrid		Geosynthetic	Secugrid 30/30 Q6	Passive (Method B)	No	Friction Angle & Adhesion	5	35	Linear	Parallel to Reinforcement	None	100	19.768

SLIDEINTERPRET 9.038

Project	MRC - Southern Piggyback Liner SRA
Analysis Description	Circular_OBE
Section	Group 2 - Circular
Scale	1:1450
Company	Talis Consultants Pty Ltd
Date	April 2025
File Name	Southern Piggyback Liner_North-South.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1
Engineered Limestone Fill		18	Mohr-Coulomb	1	35	None	0.1

Support Name	Color	Type	Selected Manufacturer	Force Application	Material Dependent	Input Type	Adhesion (kPa)	Friction Angle (°)	Shear Strength Model	Force Orientation	Anchorage	Strip Coverage (%)	Long Term Design Strength (kN/m)
Geogrid		Geosynthetic	Secugrid 30/ 30 Q6	Passive (Method B)	No	Friction Angle & Adhesion	5	35	Linear	Parallel to Reinforcement	None	100	19.768

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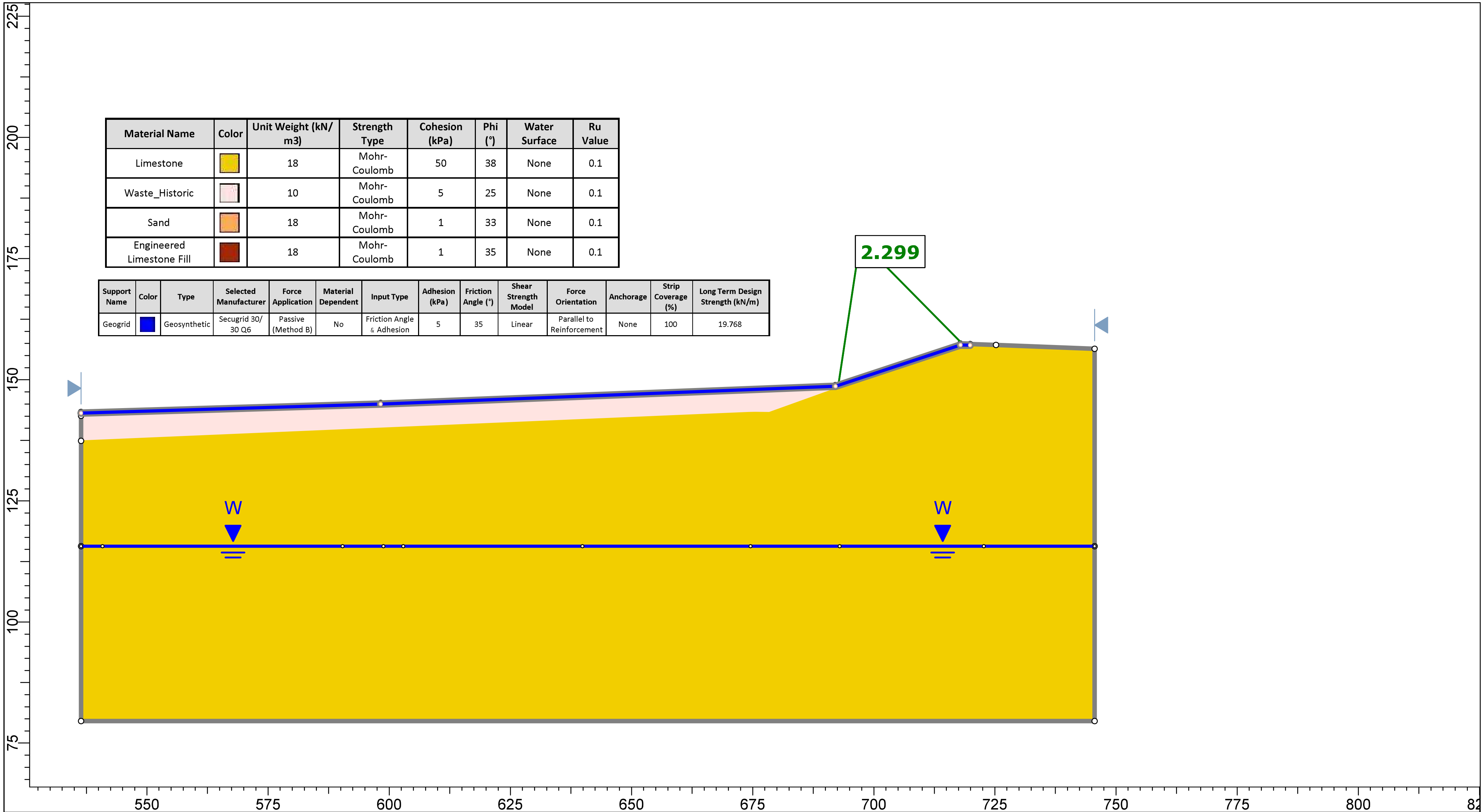
SLIDEINTERPRET 9.038

Project	MRC - Southern Piggyback Liner SRA
Analysis Description	Circular_SEE
Section	Group 2 - Circular
Scale	1:1450
Company	Talis Consultants Pty Ltd
Date	April 2025
File Name	Southern Piggyback Liner_North-South.slmd

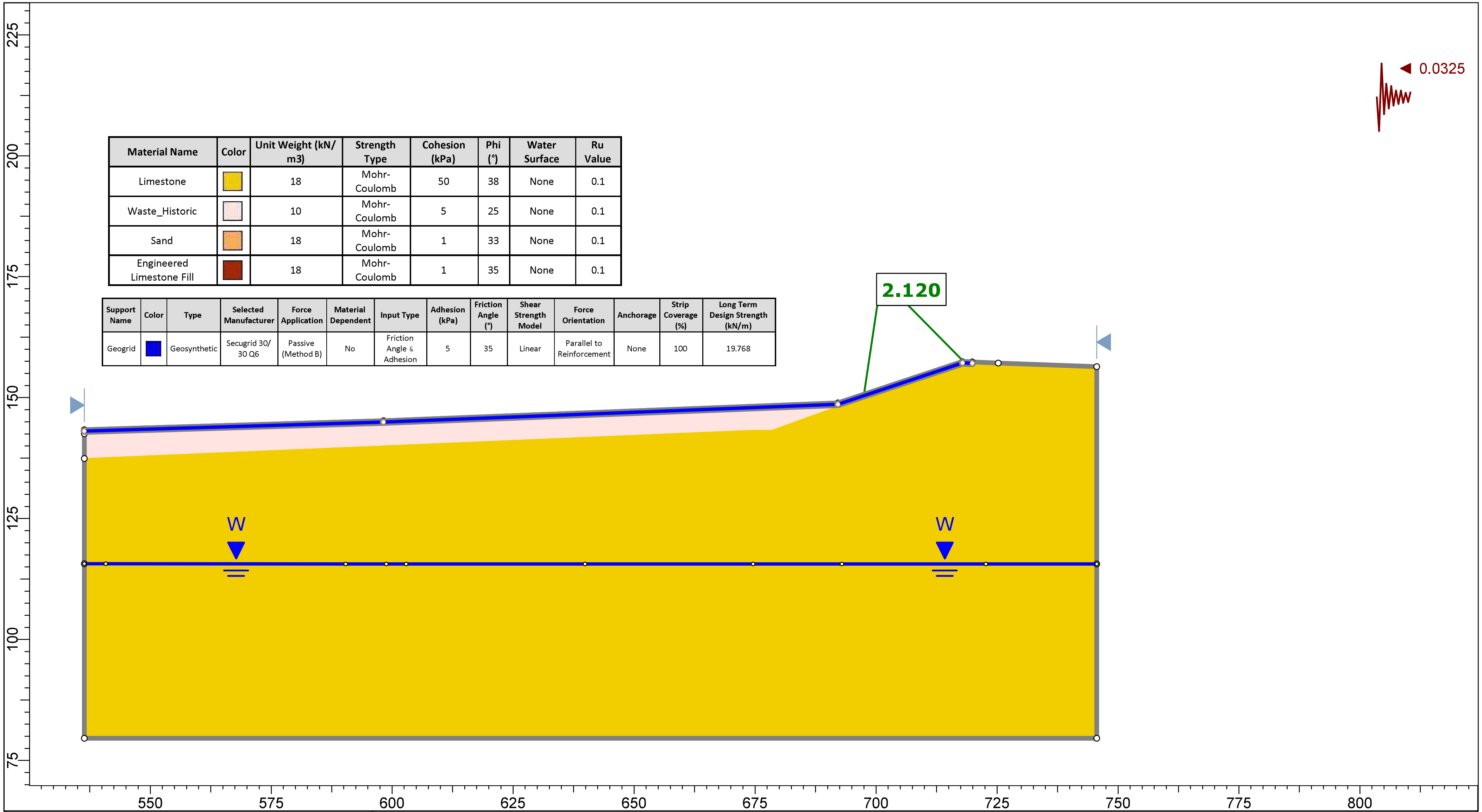


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1
Engineered Limestone Fill		18	Mohr-Coulomb	1	35	None	0.1

Support Name	Color	Type	Selected Manufacturer	Force Application	Material Dependent	Input Type	Adhesion (kPa)	Friction Angle (°)	Shear Strength Model	Force Orientation	Anchorage	Strip Coverage (%)	Long Term Design Strength (kN/m)
Geogrid		Geosynthetic	Secugrid 30/30 Q6	Passive (Method B)	No	Friction Angle & Adhesion	5	35	Linear	Parallel to Reinforcement	None	100	19.768

SLIDEINTERPRET 9.038

Project	MRC - Southern Piggyback Liner SRA
Analysis Description	Non-Circular_No Seismic
Section	Group 1 - Non-Circular
Scale	1:1100
Company	Talis Consultants Pty Ltd
Date	April 2025
File Name	Southern Piggyback Liner_North-South.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1
Engineered Limestone Fill		18	Mohr-Coulomb	1	35	None	0.1

Support Name	Color	Type	Selected Manufacturer	Force Application	Material Dependent	Input Type	Adhesion (kPa)	Friction Angle (°)	Shear Strength Model	Force Orientation	Anchorage	Strip Coverage (%)	Long Term Design Strength (kN/m)
Geogrid		Geosynthetic	Secugrid 30/30 Q6	Passive (Method B)	No	Friction Angle & Adhesion	5	35	Linear	Parallel to Reinforcement	None	100	19.768

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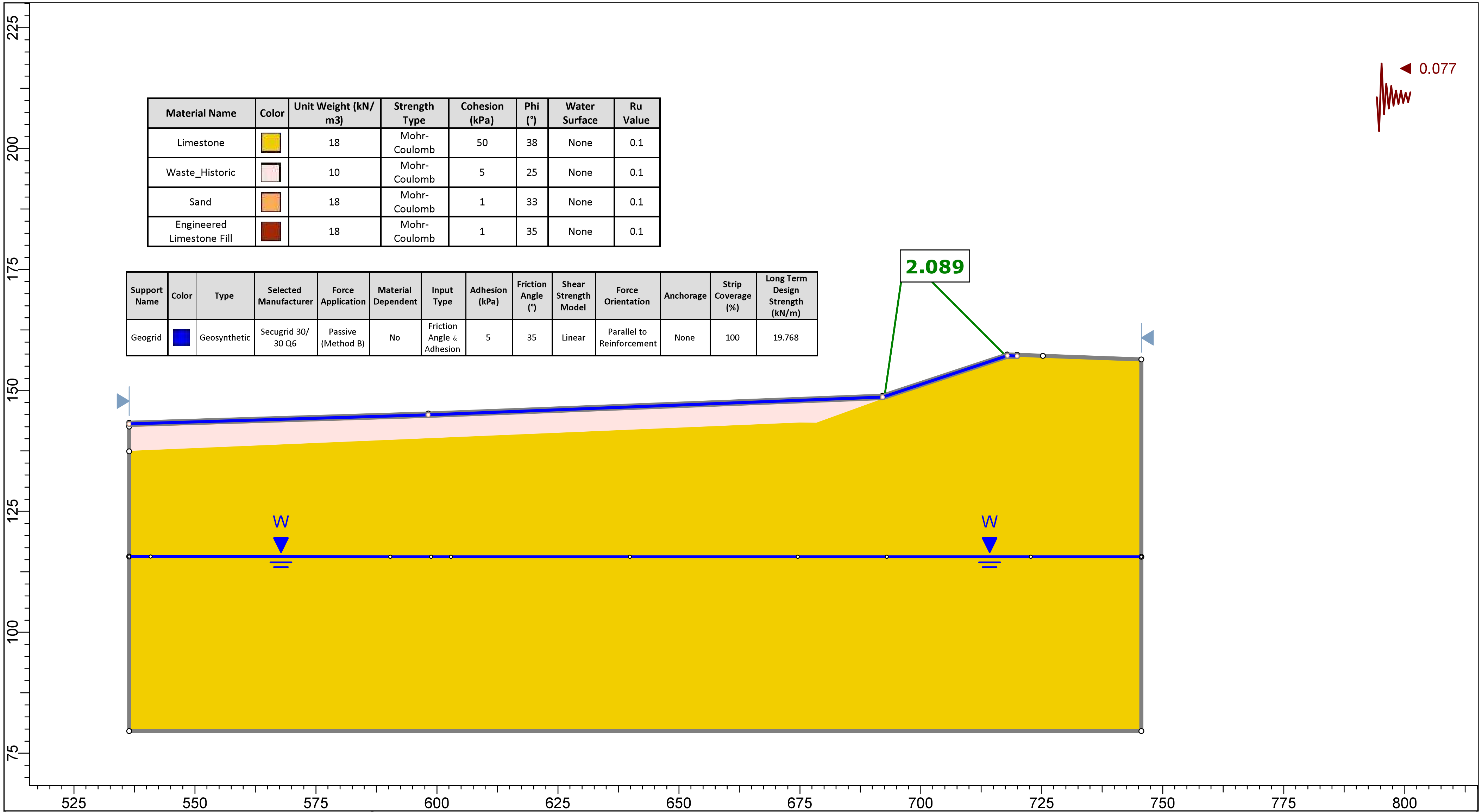
SLIDEINTERPRET 9.038

Project	MRC - Southern Piggyback Liner SRA
Analysis Description	Non-Circular_OBE
Section	Group 1 - Non-Circular
Scale	1:1100
Company	Talis Consultants Pty Ltd
Date	April 2025
File Name	Southern Piggyback Liner_North-South.slmd

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1
Engineered Limestone Fill		18	Mohr-Coulomb	1	35	None	0.1

Support Name	Color	Type	Selected Manufacturer	Force Application	Material Dependent	Input Type	Adhesion (kPa)	Friction Angle (°)	Shear Strength Model	Force Orientation	Anchorage	Strip Coverage (%)	Long Term Design Strength (kN/m)
Geogrid		Geosynthetic	Secugrid 30/30 Q6	Passive (Method B)	No	Friction Angle & Adhesion	5	35	Linear	Parallel to Reinforcement	None	100	19.768



SLIDEINTERPRET 9.038

Project	MRC - Southern Piggyback Liner SRA		
Analysis Description	Non-Circular_SEE		
Section	Group 1 - Non-Circular	*Scale* 1:1100	*Company* Talis Consultants Pty Ltd
Date	April 2025	*File Name*	Southern Piggyback Liner_North-South.slmd

APPENDIX D
Unconfined Liner Interface Assessment

Unconfined Liner Interface Stability Assessment

			Peak				Post Peak			
Input Parameters			**1V:3H**	**1V:3H**	**1V5.5H**	**1V:5.5H**	**1V:3H**	**1V:3H**	**1V5.5H**	**1V:5.5H**
b	Slope Angle	°	18.43	18.43	10.30	10.30	18.43	18.43	10.30	10.30
H	Slope height	m	2.00	2.00	13.00	13.00	2.00	2.00	13.00	13.00
h	Thickness of Aggregate Layer	m	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30
f	Friction angle of Aggregate Layer	°	33.00	33.00	33.00	33.00	26.40	26.40	26.40	26.40
c	Cohesion of Aggregate Layer	kPa	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
dct	Interface friction angle Aggregate Layer/Geotextile	°	30.00	30.00	30.00	30.00	24.00	24.00	24.00	24.00
act	Apparent cohesion of Aggregate Layer/Geotextile	kPa	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
dtg	Interface friction angle of Geotextile/LLDPE	°	23.00	23.00	23.00	23.00	14.00	14.00	14.00	14.00
atg	Apparent cohesion of Geotextile/LLDPE interface	kPa	1.00	1.00	1.00	1.00	2.00	2.00	2.00	2.00
dgs	Interface friction angle LLDPE/GCL	°	26.00	26.00	26.00	26.00	13.00	13.00	13.00	13.00
ags	Apparent cohesion of LLDPE/GCL	kPa	2.00	2.00	2.00	2.00	5.00	5.00	5.00	5.00
dgf	Interface friction angle GCL/Subgrade	°	30.00	30.00	30.00	30.00	14.00	14.00	14.00	14.00
agf	Apparent cohesion of GCL/Subgrade	kPa	5.00	5.00	5.00	5.00	1.00	1.00	1.00	1.00
PRS	Parallel Submerged Ratio		**0.00**	**0.50**	**0.00**	**0.50**	**0.00**	**0.50**	**0.00**	**0.50**
g_d	Dry unit weight of cover soil	kN	18.00	18.00	18.00	18.00	18.00	18.00	18.00	18.00
g_{sat}	Saturated weight of cover soil	kN	18.50	18.50	18.50	18.50	18.50	18.50	18.50	18.50
h_w	Thickness of saturated cover soil	m	**0.00**	**0.15**	**0.00**	**0.15**	**0.00**	**0.15**	**0.00**	**0.15**
W_A	Weight of active wedge	kN	**31.46**	**31.92**	**388.01**	**393.43**	**31.46**	**31.92**	**388.01**	**393.43**
W_P	Weight of passive wedge	kN	**2.70**	**2.72**	**4.60**	**4.64**	**2.70**	**2.72**	**4.60**	**4.64**
U_n	Resultant pore water pressure perpendicular to slope	kN	**0.00**	**8.65**	**0.00**	**106.67**	**0.00**	**8.65**	**0.00**	**106.67**
U_h	Resultant pore water pressure on interwedge surface	kN	**0.00**	**0.11**	**0.00**	**0.11**	**0.00**	**0.11**	**0.00**	**0.11**
N_{Aab}	Effective force normal to failure plane of active wedge above impermeable layer	kN	**29.85**	**21.67**	**381.76**	**280.44**	**29.85**	**21.67**	**381.76**	**280.44**
N_{Abb}	Effective force normal to failure plane of active wedge below impermeable layer	kN	**29.85**	**30.32**	**381.76**	**387.11**	**29.85**	**30.32**	**381.76**	**387.11**
U_v	Resultant vertical pore water pressure acting on passive wedge	kN	**0.00**	**0.34**	**0.00**	**0.62**	**0.00**	**0.34**	**0.00**	**0.62**
L	Slope Length	m	**6.33**	**6.33**	**72.71**	**72.71**	**6.33**	**6.33**	**72.71**	**72.71**
Soils/Geotextile Interface										
	Quadratic Equation Parameters	a	9.44	9.58	68.26	69.22	9.44	9.58	68.26	69.22
		b	-27.09	-22.42	-301.11	-243.28	-22.46	-18.85	-248.89	-204.29
		c	4.84	3.87	34.03	27.24	3.08	2.51	21.54	17.54
	Factor of Safety Against Failure		**2.68**	**2.15**	**4.30**	**3.40**	**2.23**	**1.82**	**3.56**	**2.86**
	Tension	**kN**	**-10.74**	**-8.51**	**-167.82**	**-151.04**	**-11.44**	**-9.35**	**-172.94**	**-156.76**
			No Tension	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**
Geotextile/LLDPE Interface										
	Quadratic Equation Parameters	a	9.44	9.58	68.26	69.22	9.44	9.58	68.26	69.22
		b	-22.77	-19.27	-243.69	-201.10	-22.91	-20.83	-246.84	-221.77
		c	3.90	3.19	27.26	22.26	3.15	2.83	21.35	19.11
	Factor of Safety Against Failure		**2.23**	**1.83**	**3.45**	**2.79**	**2.28**	**2.03**	**3.53**	**3.12**
	Tension	**kN**	**-19.44**	**-17.60**	**-265.30**	**-250.79**	**-30.03**	**-28.93**	**-384.19**	**-375.93**
			No Tension	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**
LLDPE/GCL Interface										
	Quadratic Equation Parameters	a	9.44	9.58	68.26	69.22	9.44	9.58	68.26	69.22
		b	-30.56	-30.58	-338.99	-341.27	-40.40	-40.35	-454.51	-455.51
		c	5.59	5.63	38.50	38.81	6.05	6.06	40.09	40.20
	Factor of Safety Against Failure		**3.04**	**2.99**	**4.85**	**4.81**	**4.13**	**4.05**	**6.57**	**6.49**
	Tension	**kN**	**-40.99**	**-41.02**	**-517.72**	**-519.76**	**-4.93**	**-4.84**	**-100.10**	**-100.43**
			No Tension	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**	**No Tension**
GCL/Subgrade Interface										
	Quadratic Equation Parameters	a	9.44	9.58	68.26	69.22	9.44	9.58	68.26	69.22
		b	-51.10	-51.16	-587.25	-590.01	-16.91	-16.87	-175.30	-176.40
		c	10.03	10.09	67.80	68.16	2.16	2.18	14.90	15.02
	Factor of Safety Against Failure		**5.21**	**5.13**	**8.49**	**8.41**	**1.65**	**1.62**	**2.48**	**2.46**

N.B. This calculation assumes friction angles and cohesion as published in R&D TECHNICAL REPORT P1-385/TR1, and Talis Shear box data.
Interface Friction tests to be undertaken on proposed geosynthetic products prior to any construction works

APPENDIX E

Gas Pressure Interface Assessment

Lining System Interface Stability Analysis - Landfill Gas

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

$$FoS = \frac{\alpha' + (h\ y \cos ß - Ug) \tan \delta'}{h\ y \sin ß'}$$

where:

	Peak	Post Peak	
α is the cohesion intercept of the lower geomembrane interface	2	2	kPa
δ is the angle of shearing resistance of the lower geomembrane interface	26	13	degrees
h is the thickness of the cover soil above the geomembrane	0.3	0.3	m
β is the slope angle	10.3	10.3	degrees
Ug is the gas pressure beneath the geomembrane	2	2	kPa
y is the average unit weight of the cover soil	18	18	kN/m^3

Note: The underliner gas collection 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.

Using peak shear strengths:

$$FoS = \frac{2 + (0.3 * 18 * \cos 10.3 - 2) \tan 26}{0.3 * 18 * \sin 10.3} = \mathbf{3.74}$$

Peak

Using residual (post peak) shear strengths:

$$FoS = \frac{2 + (0.3 * 18 * \cos 10.3 - 2) \tan 13}{0.3 * 18 * \sin 10.3} = \mathbf{2.86}$$

Post Peak

Conclusion:

Since the factor of safety for the both peak and post peak conditions is >1.3 and >1 respectively, the geosynthetic liner system is considered to be stable with respect to landfill gas pressure

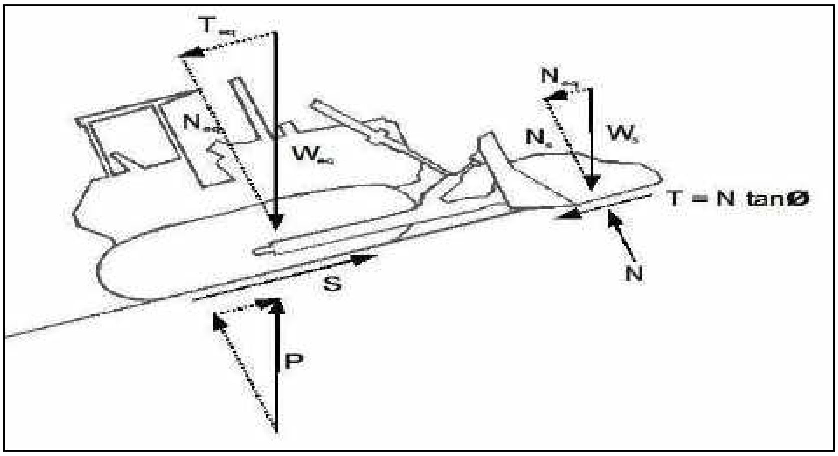
Typical Ug Values* (kPa)			
	0	LCV	Lowest Conceivable Value
	4	HCV	Highest Conceivable Value
	1	MLV	Most Likely Value
	0.67	σ	Standard Deviation 'Three sigma rule'

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.

APPENDIX F
Construction Plant Operations on Geosynthetics

STABILITY ASSESSMENT FOR PLANT OPERATIONS ON GEOSYNTHETIC



Unit weight of soil cover	18.00	kN/cu.m
Depth of soil cover (1st lift, D)	0.30	m
Dozer type	CAT D6R LGP	
Total dozer weight	205.00	kN
Track length (L)	3.25	metres
Track width (W)	0.92	metres
Width of dozer blade (Wb)	3.99	metres
Height of soil pile (Hb)	1.20	metres
Length in front of blade (Lb)	1.00	metres
Weight of soil being spread	86.18	kN
Slope angle, alpha	10.30	degrees
Soil cover friction angle	30.00	degrees
Interface friction angle	14.00	degrees
Interface adhesion	1.00	kN/sq.m
Unit tension (geosynthetic)	0.00	kN/m
Factor of safety	**0.79**	

Forces

N(1) = 13.28 kN

$$N_1 = \frac{W_1}{\cos\beta - \tan\phi_m \sin\beta - (\sin\beta + \tan\phi_n \cos\beta)\tan\phi_m}$$

N(2) = 115.18 kN

$$N_2 = \frac{W_2 + 0.5P - (\delta_n \rho_2 + \tau_G - 0.5S)(\sin\alpha - \cos\alpha\tan\phi_m)}{\cos\alpha + \tan\delta_n \sin\alpha + (\sin\alpha - \tan\delta_n \cos\alpha)\tan\phi_m}$$

N(3) = 0.69 kN

$$N_3 = \frac{W_3}{\cos\theta + \tan\phi_n \sin\theta + (\sin\theta - \tan\phi_m \cos\theta)\tan\phi_m}$$

N(4) = 11.85 kN

$$N_{4.} = N_1(\sin\beta + \tan\phi_m \cos\beta)$$

N(5)CB = 0.20 kN

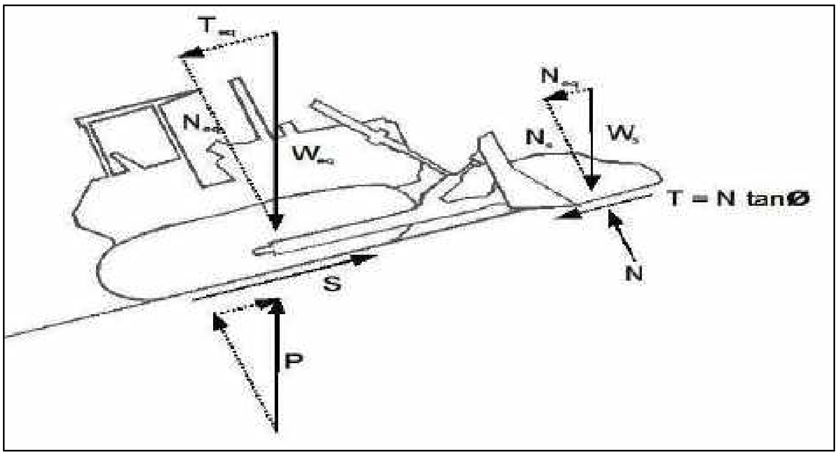
$$N_{5CB} = N_4 + N_2(\tan\delta_m \cos\alpha - \sin\alpha) + (\theta_m A_2 + T_\alpha - 0.5S)\cos\alpha$$

N(5)AB = 0.20 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

STABILITY ASSESSMENT FOR PLANT OPERATIONS ON GEOSYNTHETIC



Unit weight of soil cover	18.00	kN/cu.m
Depth of soil cover (1st lift, D)	0.72	m
Dozer type	CAT D6R LGP	
Total dozer weight	205.00	kN
Track length (L)	3.25	metres
Track width (W)	0.92	metres
Width of dozer blade (Wb)	3.99	metres
Height of soil pile (Hb)	1.20	metres
Length in front of blade (Lb)	1.00	metres
Weight of soil being spread	86.18	kN
Slope angle, alpha	10.30	degrees
Soil cover friction angle	30.00	degrees
Interface friction angle	14.00	degrees
Interface adhesion	1.00	kN/sq.m
Unit tension (geosynthetic)	0.00	kN/m
Factor of safety	**1.30**	

Forces

N(1) = 33.69 kN

$$N_1 = \frac{W_1}{\cos\beta - \tan\phi_m \sin\beta - (\sin\beta + \tan\phi_n \cos\beta)\tan\phi_m}$$

N(2) = 160.67 kN

$$N_2 = \frac{W_2 + 0.5P - (\delta_n \rho_2 + \tau_G - 0.5S)(\sin\alpha - \cos\alpha\tan\phi_m)}{\cos\alpha + \tan\delta_n \sin\alpha + (\sin\alpha - \tan\delta_n \cos\alpha)\tan\phi_m}$$

N(3) = 16.92 kN

$$N_3 = \frac{W_3}{\cos\theta + \tan\phi_n \sin\theta + (\sin\theta - \tan\phi_m \cos\theta)\tan\phi_m}$$

N(4) = 27.85 kN

$$N_4 = N_1(\sin\beta + \tan\phi_m \cos\beta)$$

N(5)CB = 1.93 kN

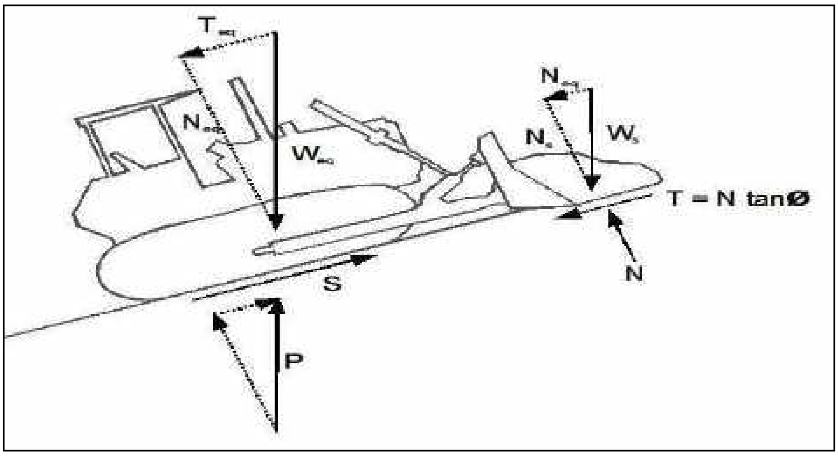
$$N_{5CB} = N_4 + N_2(\tan\delta_m \cos\alpha - \sin\alpha) + (\theta_m A_2 + T_\alpha - 0.5S)\cos\alpha$$

N(5)AB = 1.93 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

STABILITY ASSESSMENT FOR PLANT OPERATIONS ON GEOSYNTHETIC



Unit weight of soil cover	18.00	kN/cu.m
Depth of soil cover (1st lift, D)	1.00	m
Dozer type	CAT D6R LGP	
Total dozer weight	205.00	kN
Track length (L)	3.25	metres
Track width (W)	0.92	metres
Width of dozer blade (Wb)	3.99	metres
Height of soil pile (Hb)	1.20	metres
Length in front of blade (Lb)	1.00	metres
Weight of soil being spread	86.18	kN
Slope angle, alpha	10.30	degrees
Soil cover friction angle	30.00	degrees
Interface friction angle	14.00	degrees
Interface adhesion	1.00	kN/sq.m
Unit tension (geosynthetic)	0.00	kN/m
Factor of safety	**1.48**	

Forces

N(1) = 68.68 kN

$$N_1 = \frac{W_1}{\cos\beta - \tan\phi_m \sin\beta - (\sin\beta + \tan\phi_n \cos\beta)\tan\phi_m}$$

N(2) = 202.95 kN

$$N_2 = \frac{W_2 + 0.5P - (\delta_n \rho_2 + \tau_G - 0.5S)(\sin\alpha - \cos\alpha\tan\phi_m)}{\cos\alpha + \tan\delta_n \sin\alpha + (\sin\alpha - \tan\delta_n \cos\alpha)\tan\phi_m}$$

N(3) = 20.83 kN

$$N_3 = \frac{W_3}{\cos\theta + \tan\phi_n \sin\theta + (\sin\theta - \tan\phi_m \cos\theta)\tan\phi_m}$$

N(4) = 38.22 kN

$$N_4 = N_1(\sin\beta + \tan\phi_m \cos\beta)$$

N(5)CB = 7.92 kN

$$N_{5CB} = N_4 + N_2(\tan\delta_m \cos\alpha - \sin\alpha) + (\theta_m A_2 + T_\alpha - 0.5S)\cos\alpha$$

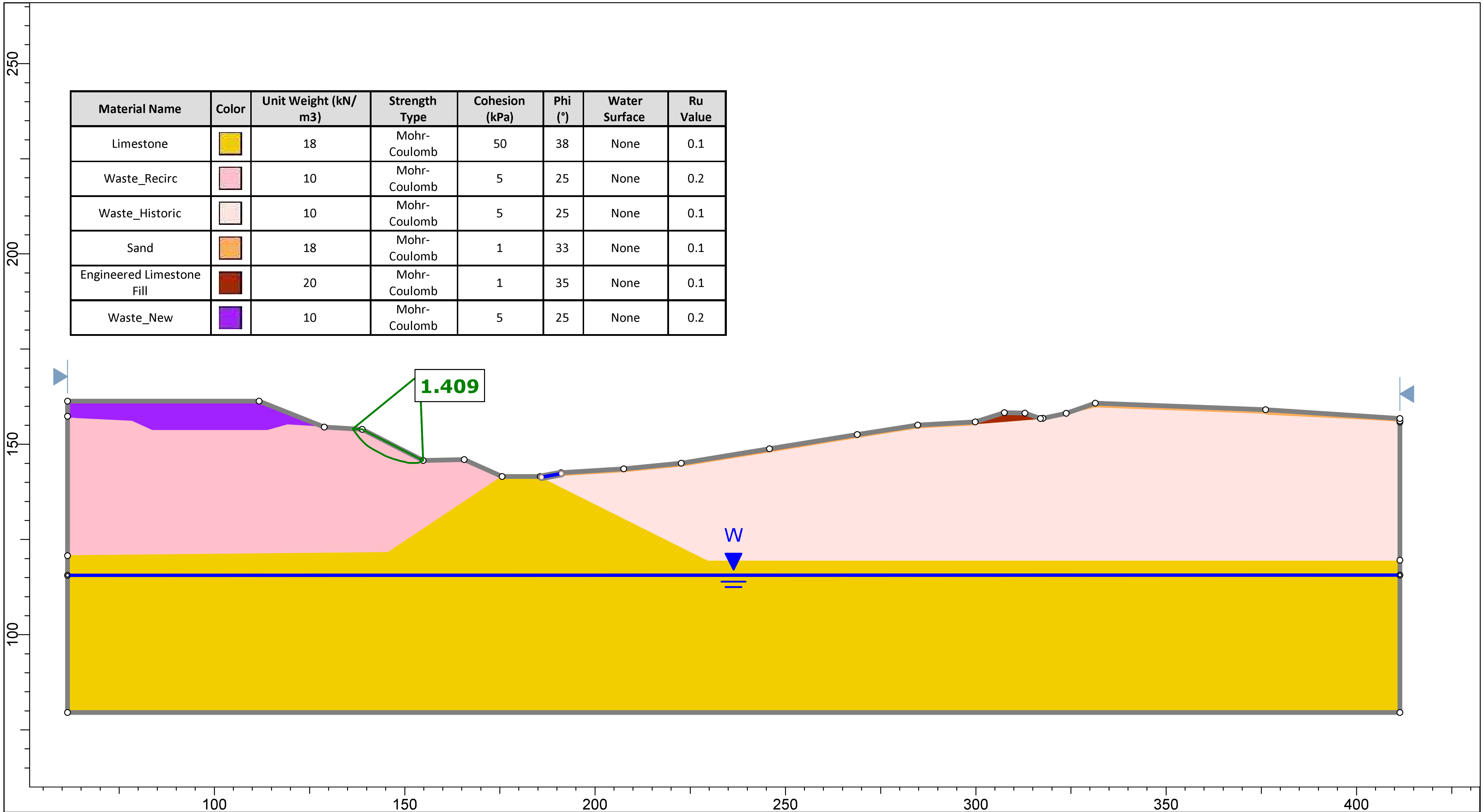
N(5)AB = 7.92 kN

$$N_{5AB} = N_3(\sin\theta - \tan\phi_m \cos\theta)$$

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

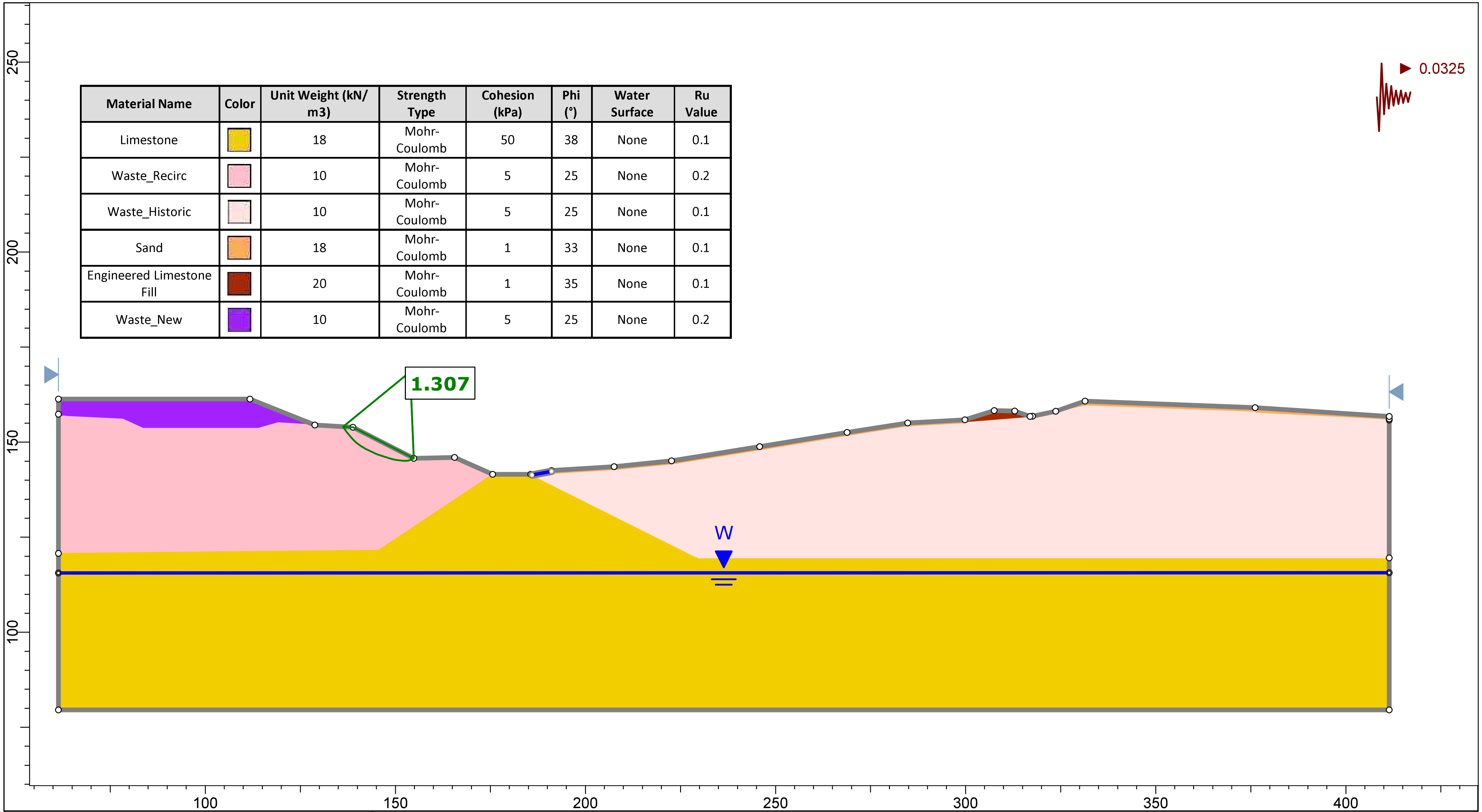
APPENDIX G
Waste Mass Stability Analysis

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Recirc		10	Mohr-Coulomb	5	25	None	0.2
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1
Engineered Limestone Fill		20	Mohr-Coulomb	1	35	None	0.1
Waste_New		10	Mohr-Coulomb	5	25	None	0.2



SLIDEINTERPRET 9.038

Project	MRC - Southern Piggyback Liner SRA
Analysis Description	Temp Waste - No Seismic Loading
Section	Group 1 - NonCirc Temp Waste Benched
Scale	1:1400
Company	Talis Consultants Pty Ltd
Date	April 2025
File Name	Southern Piggyback Liner_TempWaste_Benched.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Recirc		10	Mohr-Coulomb	5	25	None	0.2
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1
Engineered Limestone Fill		20	Mohr-Coulomb	1	35	None	0.1
Waste_New		10	Mohr-Coulomb	5	25	None	0.2

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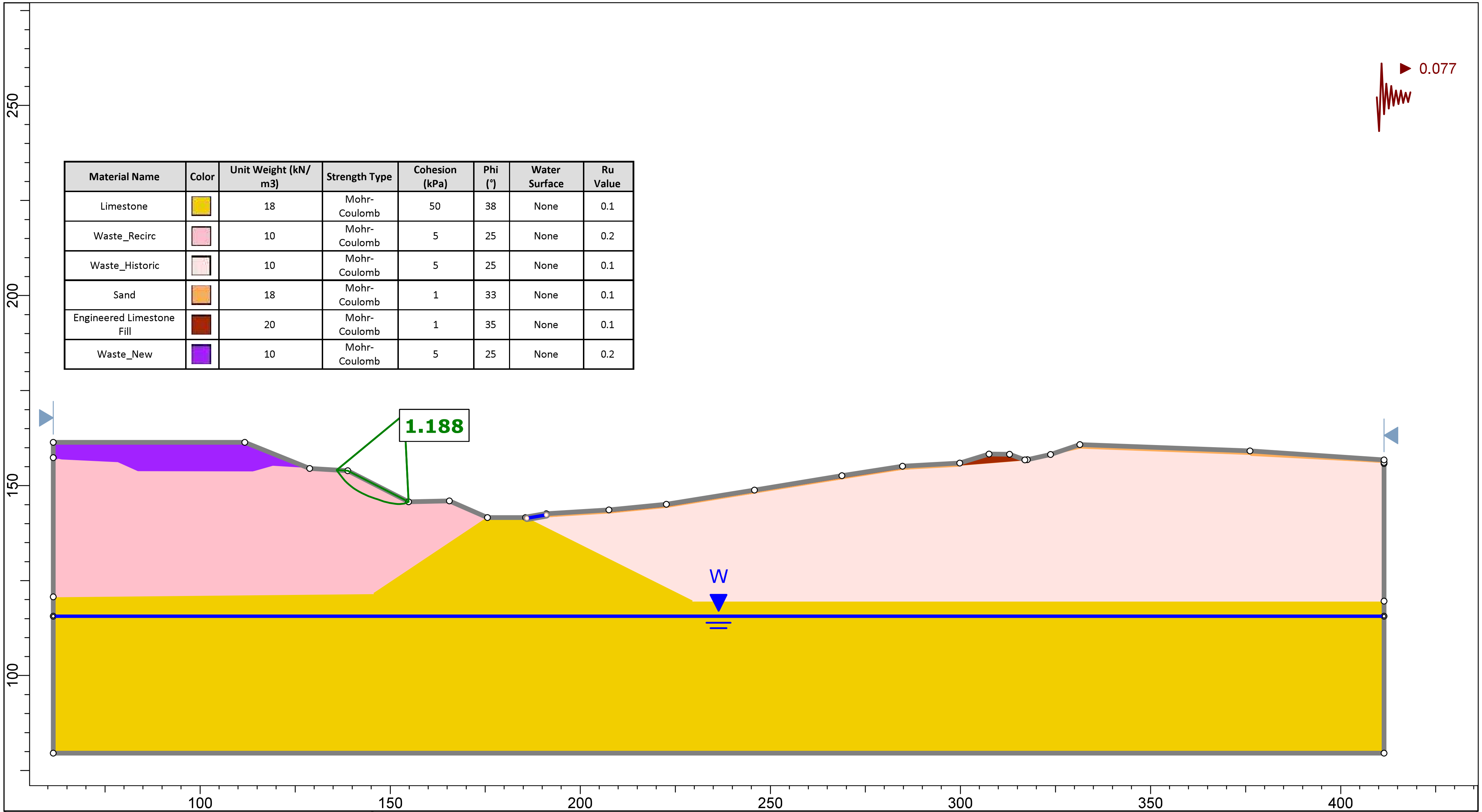
SLIDEINTERPRET 9.038

Project	MRC - Southern Piggyback Liner SRA
Analysis Description	Temp Waste - OBE
Section	Group 1 - NonCirc Temp Waste Benched
Scale	1:1400
Company	Talis Consultants Pty Ltd
Date	April 2025
File Name	Southern Piggyback Liner_TempWaste_Benched.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Recirc		10	Mohr-Coulomb	5	25	None	0.2
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1
Engineered Limestone Fill		20	Mohr-Coulomb	1	35	None	0.1
Waste_New		10	Mohr-Coulomb	5	25	None	0.2

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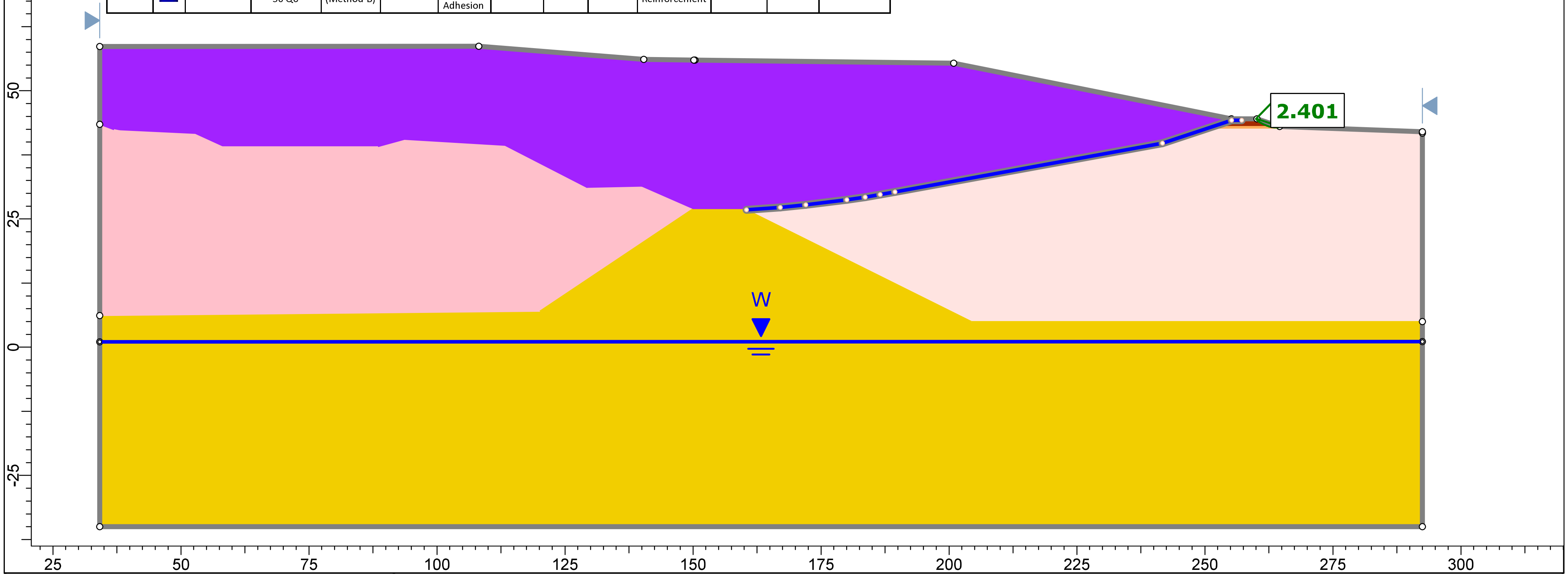
SLIDEINTERPRET 9.038

Project	MRC - Southern Piggyback Liner SRA
Analysis Description	Temp Waste - SEE
Section	Group 1 - NonCirc Temp Waste Benched
Scale	1:1400
Company	Talis Consultants Pty Ltd
Date	April 2025
File Name	Southern Piggyback Liner_TempWaste_Benched.slmd

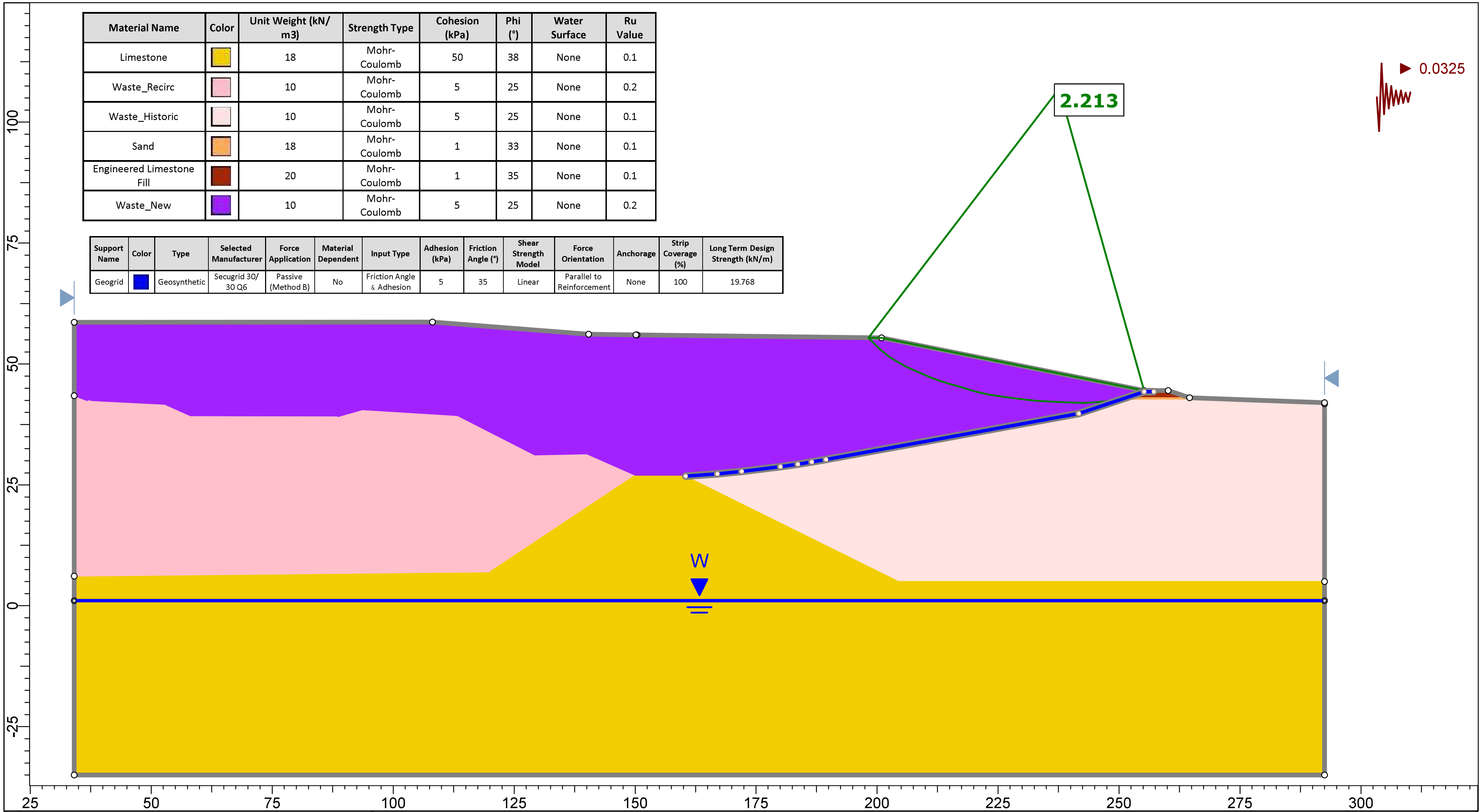
Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Recirc		10	Mohr-Coulomb	5	25	None	0.2
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1
Engineered Limestone Fill		20	Mohr-Coulomb	1	35	None	0.1
Waste_New		10	Mohr-Coulomb	5	25	None	0.2

Support Name	Color	Type	Selected Manufacturer	Force Application	Material Dependent	Input Type	Adhesion (kPa)	Friction Angle (°)	Shear Strength Model	Force Orientation	Anchorage	Strip Coverage (%)	Long Term Design Strength (kN/m)
Geogrid		Geosynthetic	Secugrid 30/30 Q6	Passive (Method B)	No	Friction Angle & Adhesion	5	35	Linear	Parallel to Reinforcement	None	100	19.768



SLIDEINTERPRET 9.039

Project	MRC - Southern Piggyback Liner SRA		
Analysis Description	Rest Waste Profile_No Seismic Loading		
Section Restoration Waste Profile	*Scale* 1:1100	*Company*	Talis Consultants Pty Ltd
Date	August 2025	*File Name*	Southern Piggyback Liner_RestorationWaste.slmd



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Recirc		10	Mohr-Coulomb	5	25	None	0.2
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1
Engineered Limestone Fill		20	Mohr-Coulomb	1	35	None	0.1
Waste_New		10	Mohr-Coulomb	5	25	None	0.2

Support Name	Color	Type	Selected Manufacturer	Force Application	Material Dependent	Input Type	Adhesion (kPa)	Friction Angle (°)	Shear Strength Model	Force Orientation	Anchorage	Strip Coverage (%)	Long Term Design Strength (kN/m)
Geogrid		Geosynthetic	Secugrid 30/30 Q6	Passive (Method B)	No	Friction Angle & Adhesion	5	35	Linear	Parallel to Reinforcement	None	100	19.768

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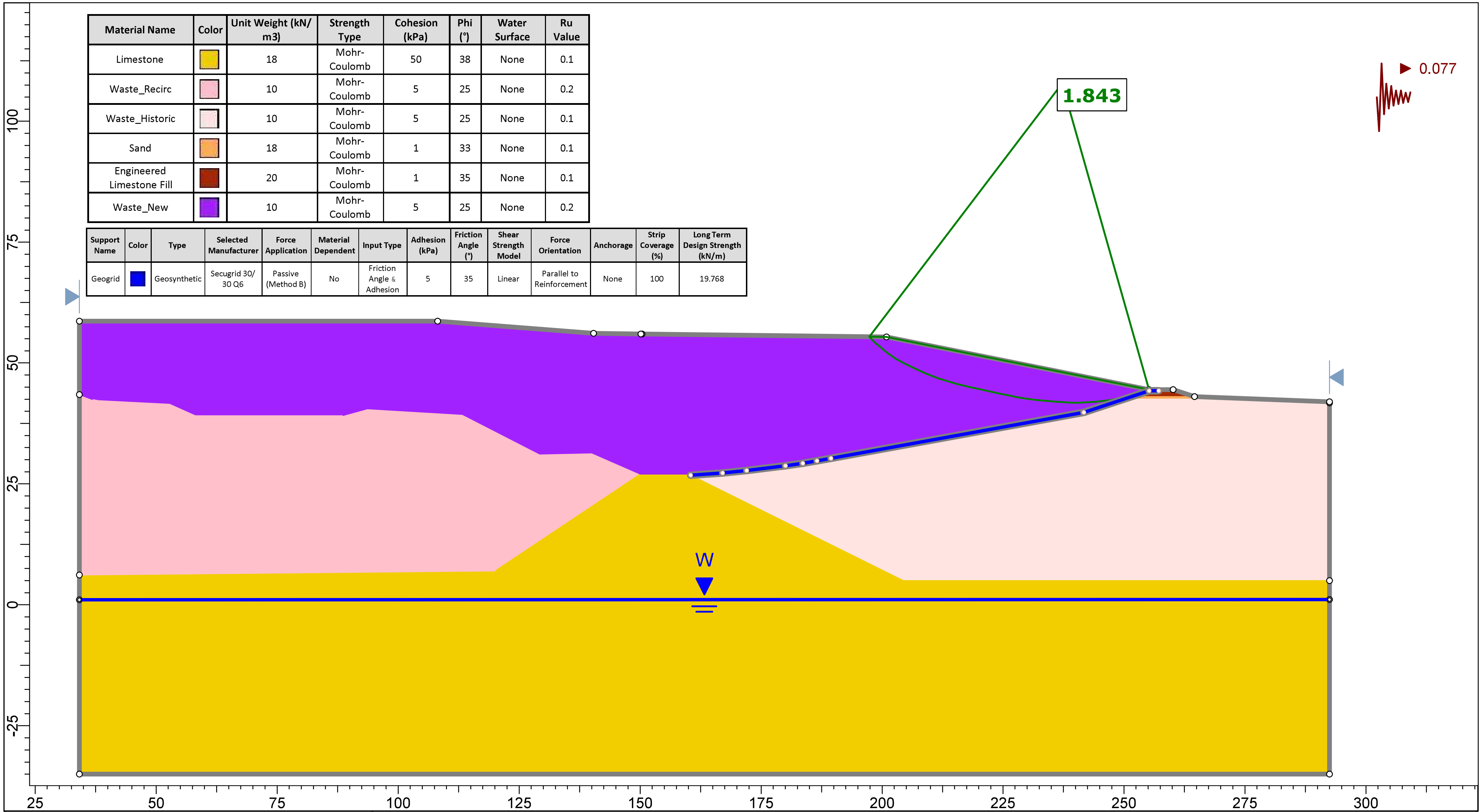
SLIDEINTERPRET 9.039

Project	MRC - Southern Piggyback Liner SRA		
Analysis Description	Rest Waste Profile_OBE		
Section	Restoration Waste Profile	*Scale*	1:1100
Company	Talis Consultants Pty Ltd		
Date	August 2025	*File Name*	Southern Piggyback Liner_RestorationWaste.slmd

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		18	Mohr-Coulomb	50	38	None	0.1
Waste_Recirc		10	Mohr-Coulomb	5	25	None	0.2
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	33	None	0.1
Engineered Limestone Fill		20	Mohr-Coulomb	1	35	None	0.1
Waste_New		10	Mohr-Coulomb	5	25	None	0.2

Support Name	Color	Type	Selected Manufacturer	Force Application	Material Dependent	Input Type	Adhesion (kPa)	Friction Angle (°)	Shear Strength Model	Force Orientation	Anchorage	Strip Coverage (%)	Long Term Design Strength (kN/m)
Geogrid		Geosynthetic	Secugrid 30/30 Q6	Passive (Method B)	No	Friction Angle & Adhesion	5	35	Linear	Parallel to Reinforcement	None	100	19.768



Project	MRC - Southern Piggyback Liner SRA		
Analysis Description	Rest Waste Profile_SEE		
Section	Restoration Waste Profile	*Scale* 1:1100	*Company* Talis Consultants Pty Ltd
Date	August 2025		*File Name* Southern Piggyback Liner_RestorationWaste.slmd

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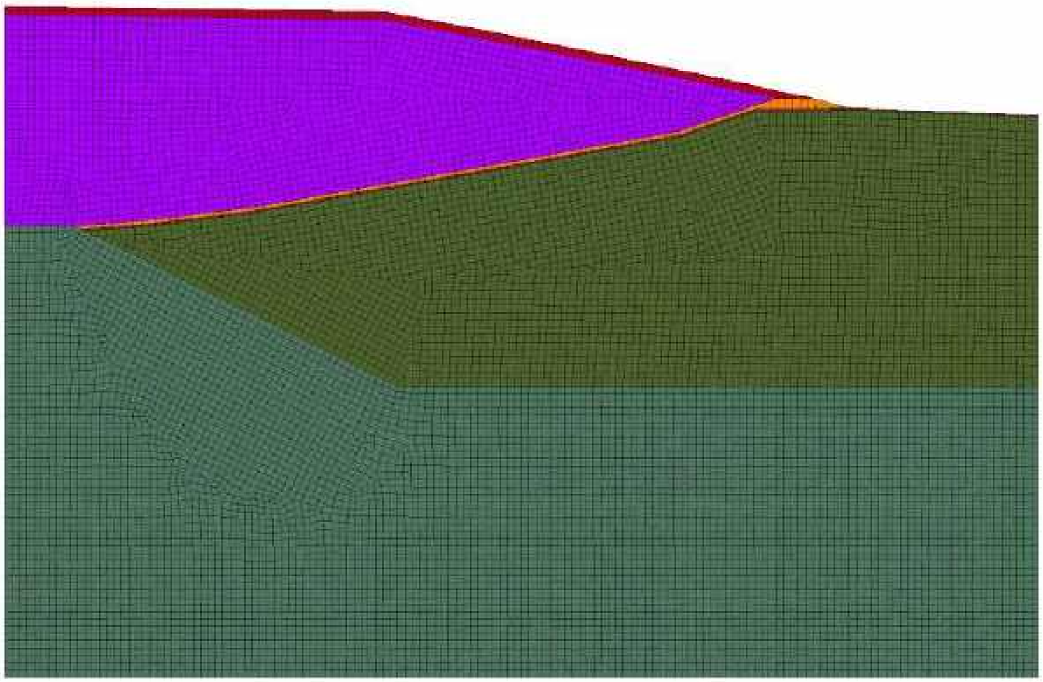
SLIDEINTERPRET 9.039

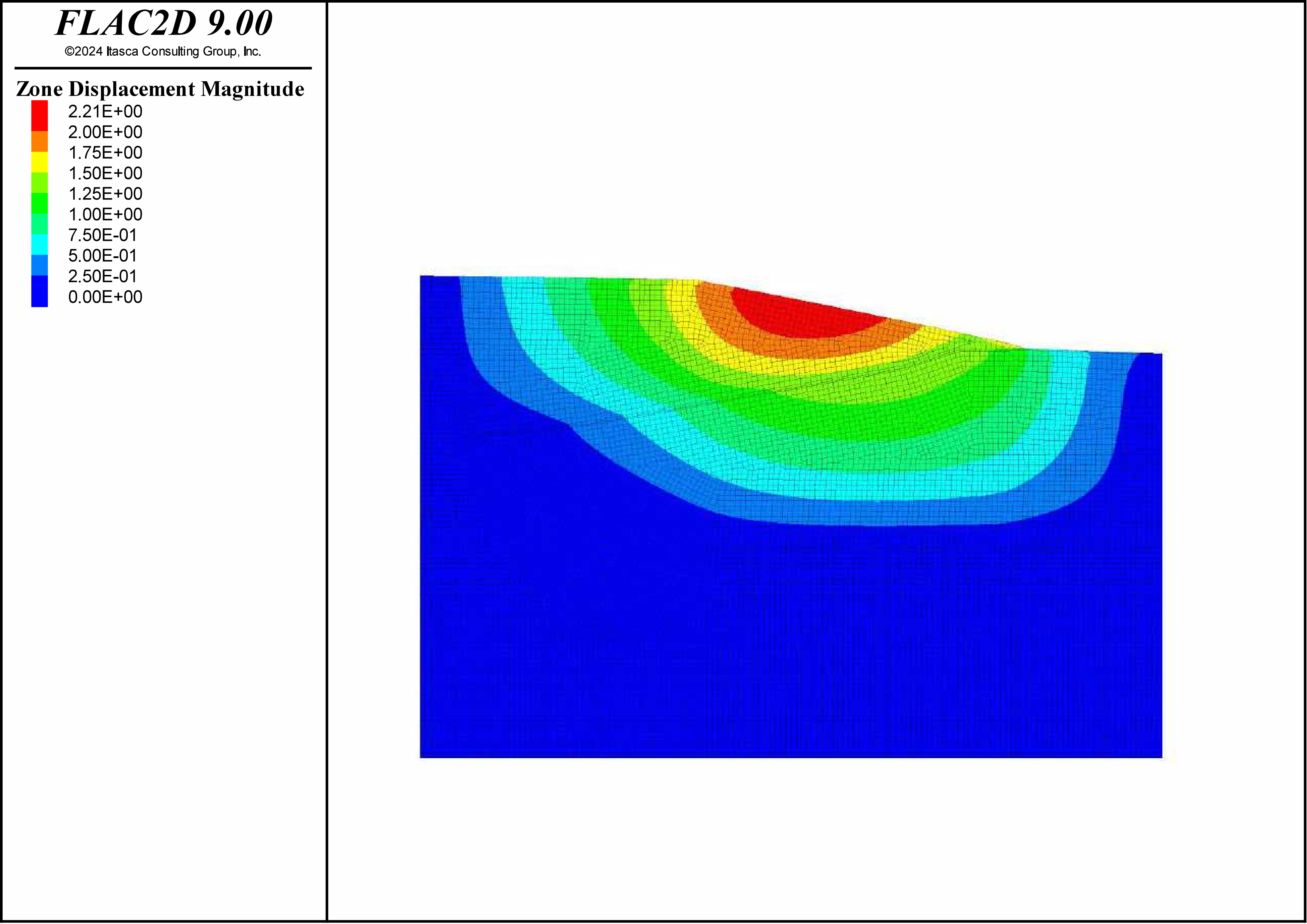
APPENDIX H
FLAC2D Liner Integrity Assessment

FLAC2D 9.00

Zone Group

- Default=Eng-Lst-Fill,Block=Block 5
- Default=Limestone,Block=Block 1
- Default=Restoration Soils,Block=Block 3
- Default=Sand,Block=Block 6
- Default=Waste-Historic,Block=Block 2
- Default=Waste-New,Block=Block 4





FLAC2D 9.00
©2024 Itasca Consulting Group, Inc.
Zone Displacement Magnitude
2.21E+00
2.00E+00
1.75E+00
1.50E+00
1.25E+00
1.00E+00
7.50E-01
5.00E-01
2.50E-01
0.00E+00

FLAC2D 9.00

Geometry

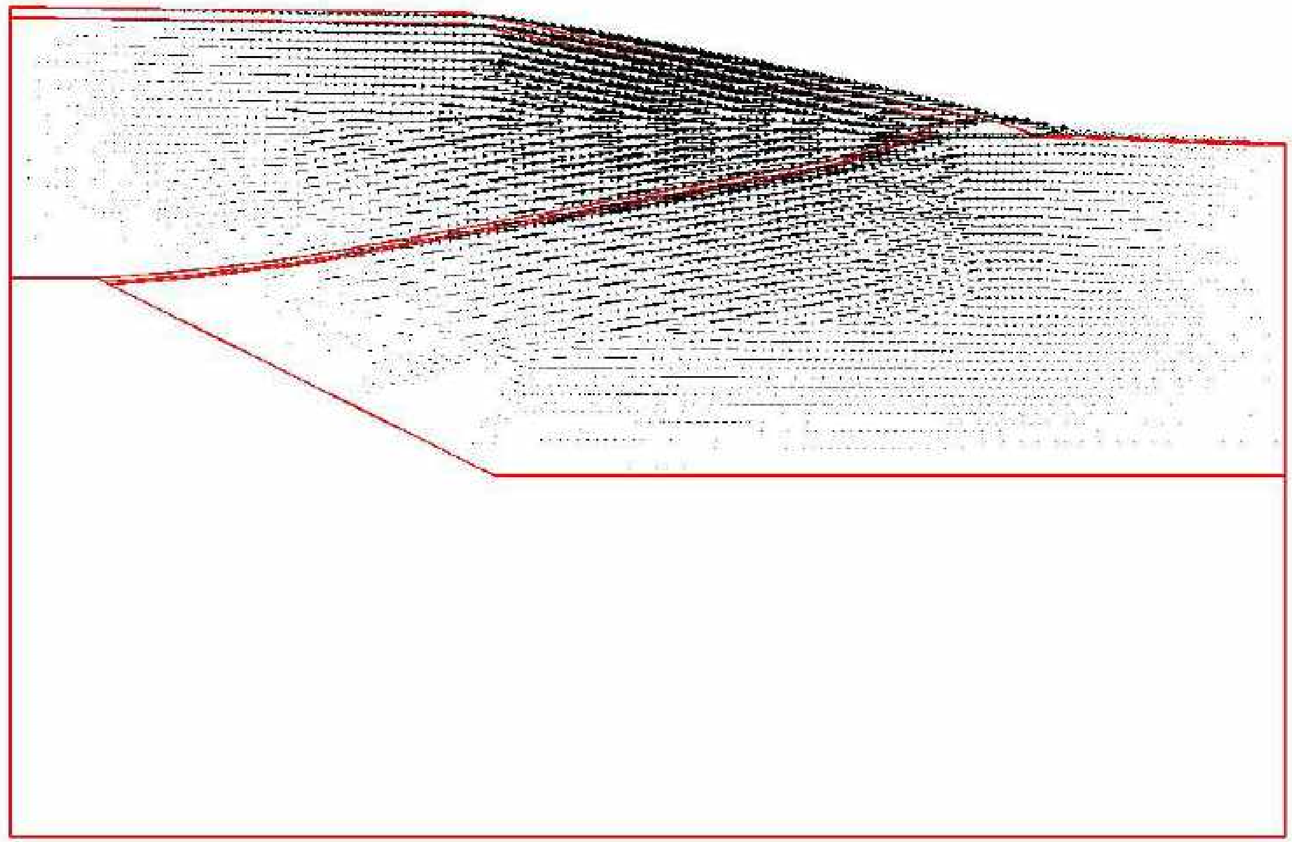
Geometry Set Name

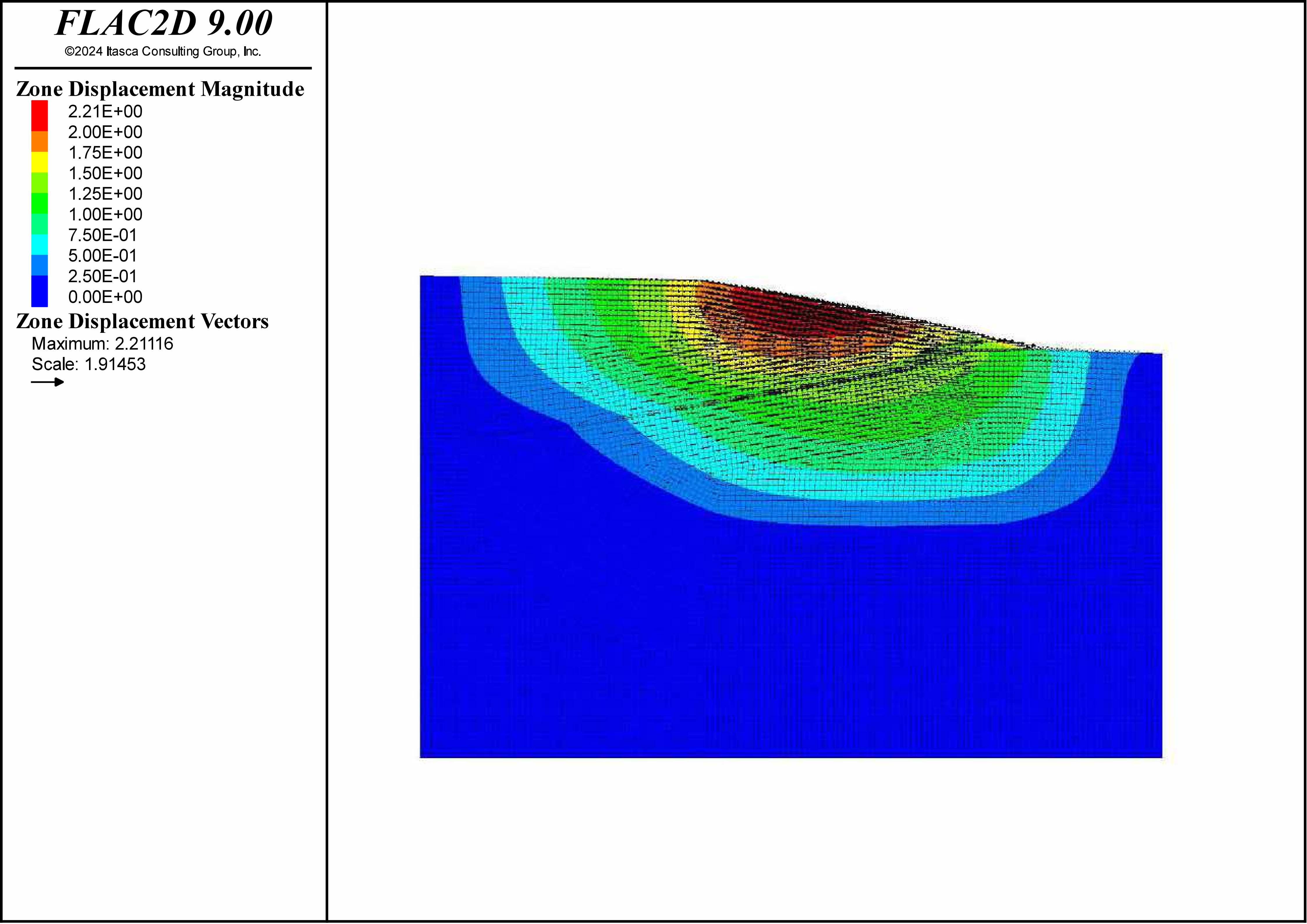
East-West_Restoration_1V5

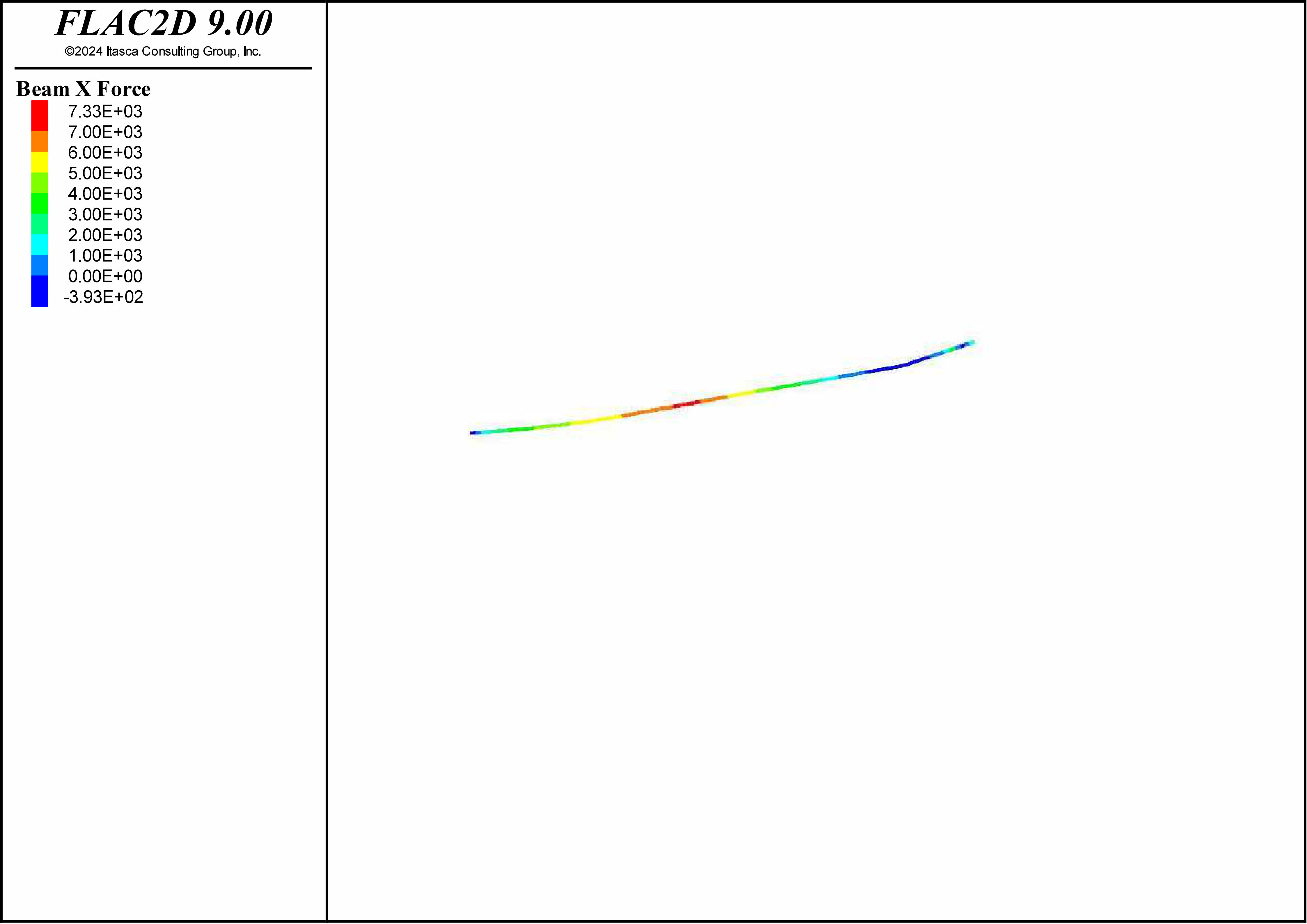
Zone Displacement Vectors

Maximum: 2.21116

Scale: 1.91488




FLAC2D 9.00
©2024 Itasca Consulting Group, Inc.
Zone Displacement Magnitude
2.21E+00
2.00E+00
1.75E+00
1.50E+00
1.25E+00
1.00E+00
7.50E-01
5.00E-01
2.50E-01
0.00E+00
Zone Displacement Vectors
Maximum: 2.21116
Scale: 1.91453


FLAC2D 9.00
©2024 Itasca Consulting Group, Inc.
Beam X Force
7.33E+03
7.00E+03
6.00E+03
5.00E+03
4.00E+03
3.00E+03
2.00E+03
1.00E+03
0.00E+00
-3.93E+02

FLAC2D 9.00

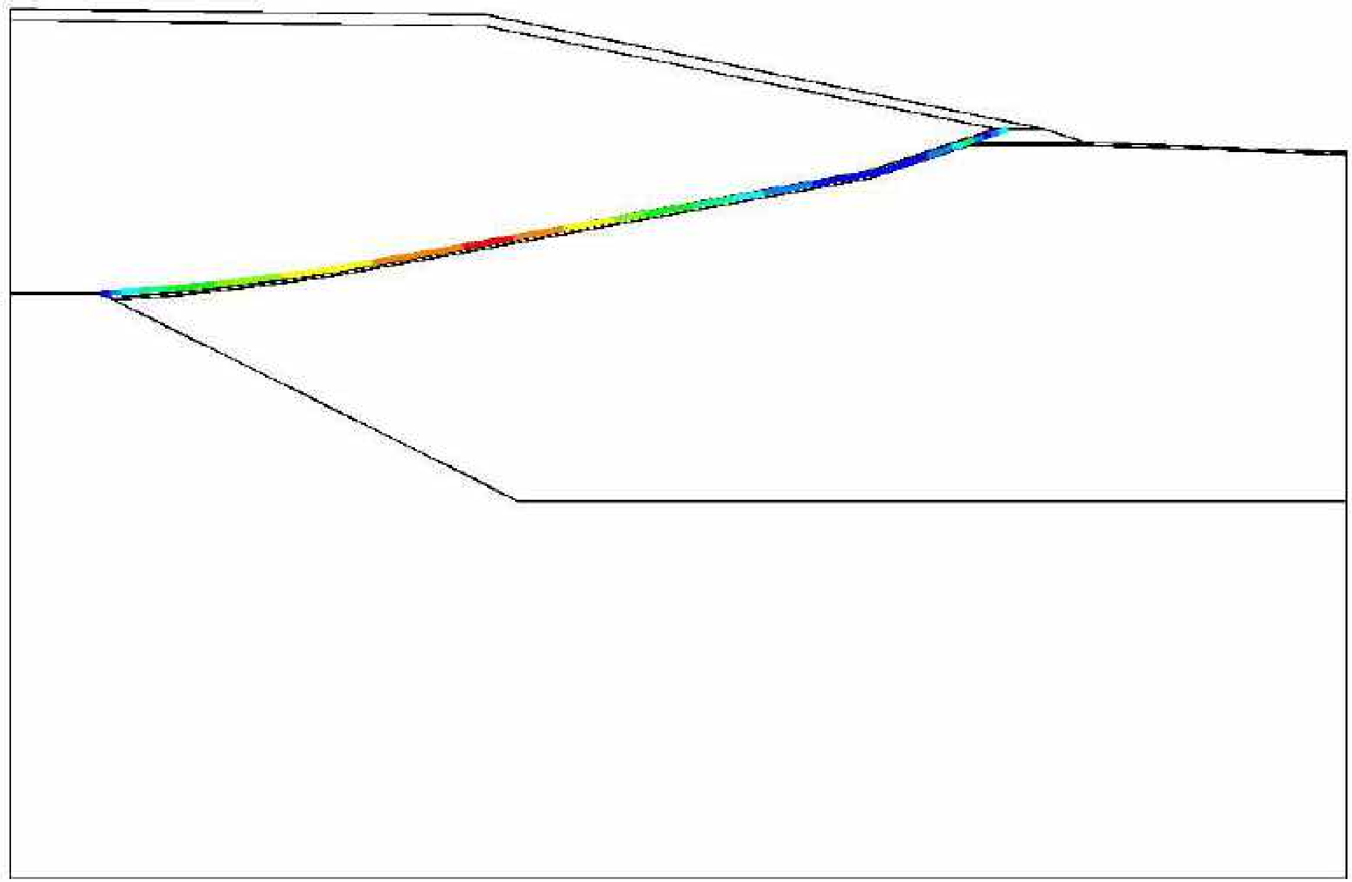
Geometry

Geometry Set Name

East-West_Restoration_1V5

Beam X Force

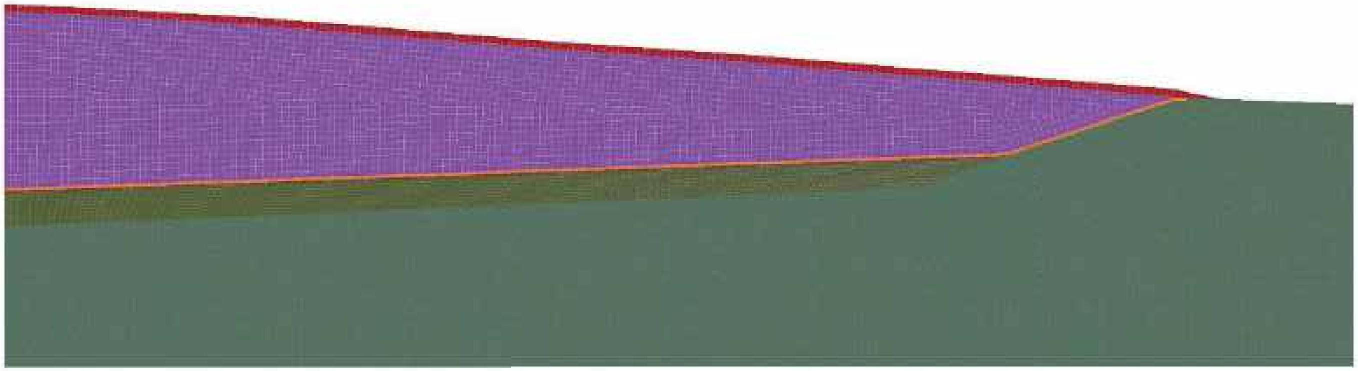
7.33E+03
7.00E+03
6.00E+03
5.00E+03
4.00E+03
3.00E+03
2.00E+03
1.00E+03
0.00E+00
-3.93E+02

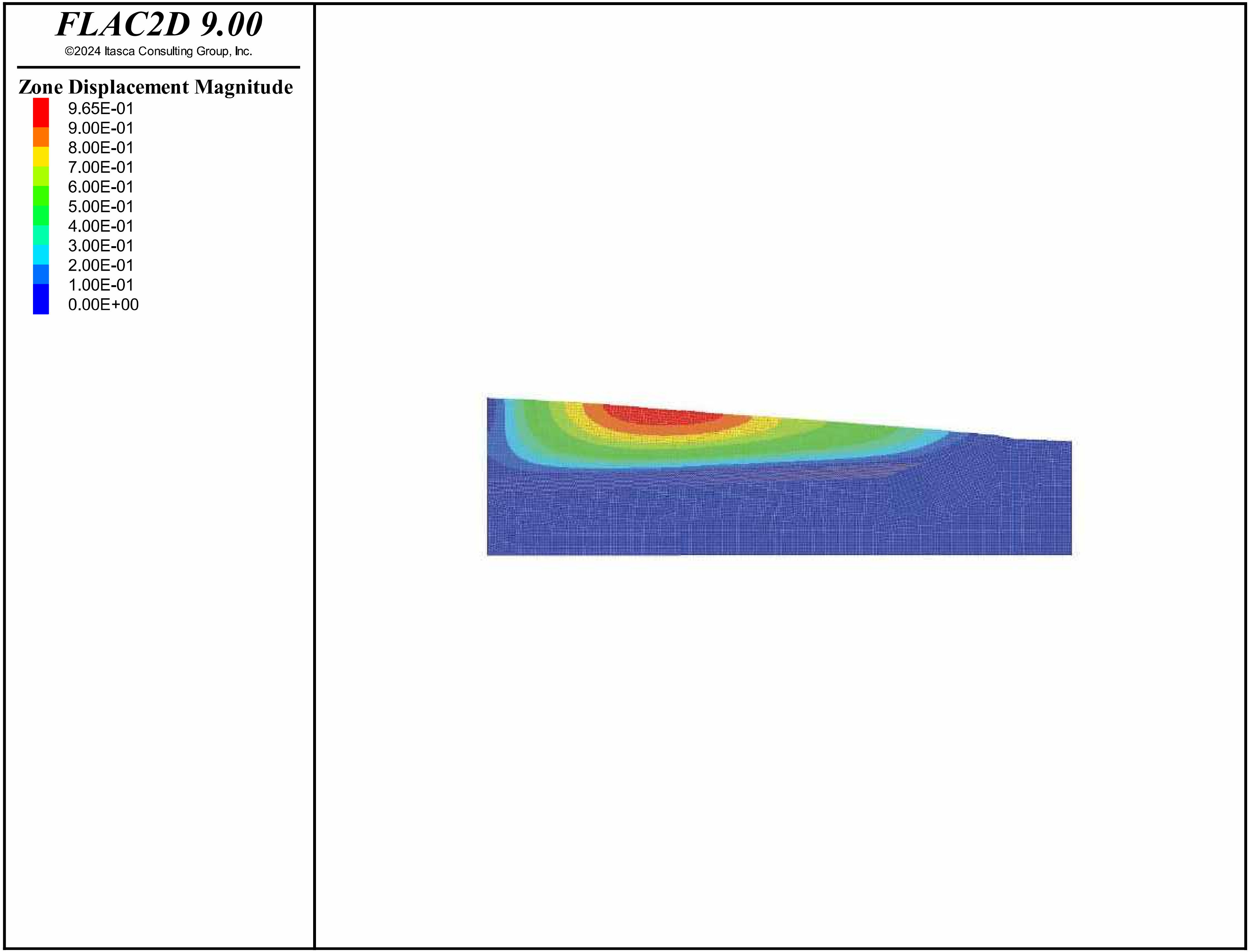


FLAC2D 9.00

Zone Group

- Default=Eng-Fill,Block=Block 8
- Default=Limestone,Block=Block 4
- Default=Reg Sand,Block=Block 7
- Default=Rest Sand,Block=Block 9
- Default=Waste-Historic,Block=Block 2
- Default=Waste-New,Block=Block 6





FLAC2D 9.00
©2024 Itasca Consulting Group, Inc.
Zone Displacement Magnitude
9.65E-01
9.00E-01
8.00E-01
7.00E-01
6.00E-01
5.00E-01
4.00E-01
3.00E-01
2.00E-01
1.00E-01
0.00E+00

FLAC2D 9.00

Zone Displacement Vectors

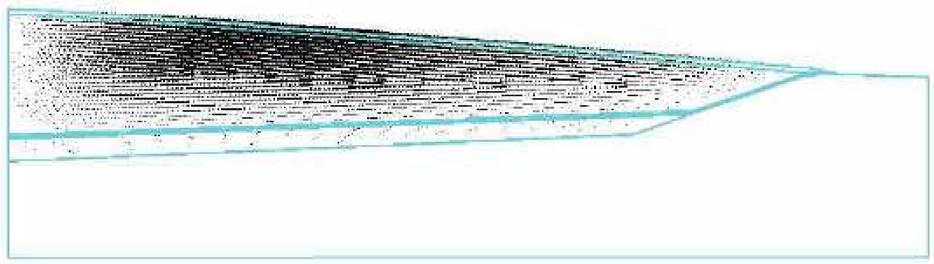
Maximum: 0.964917

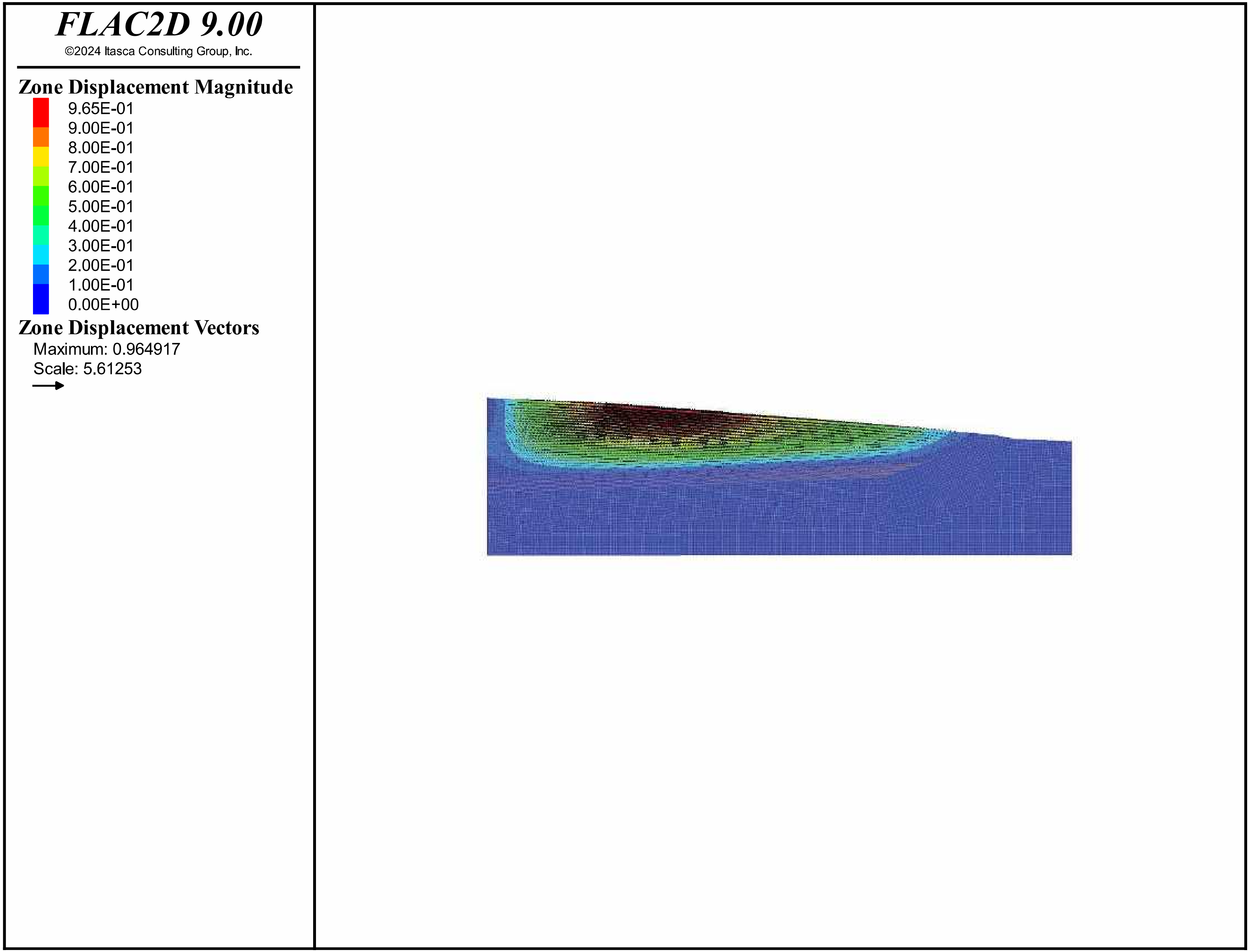
Scale: 5.6129

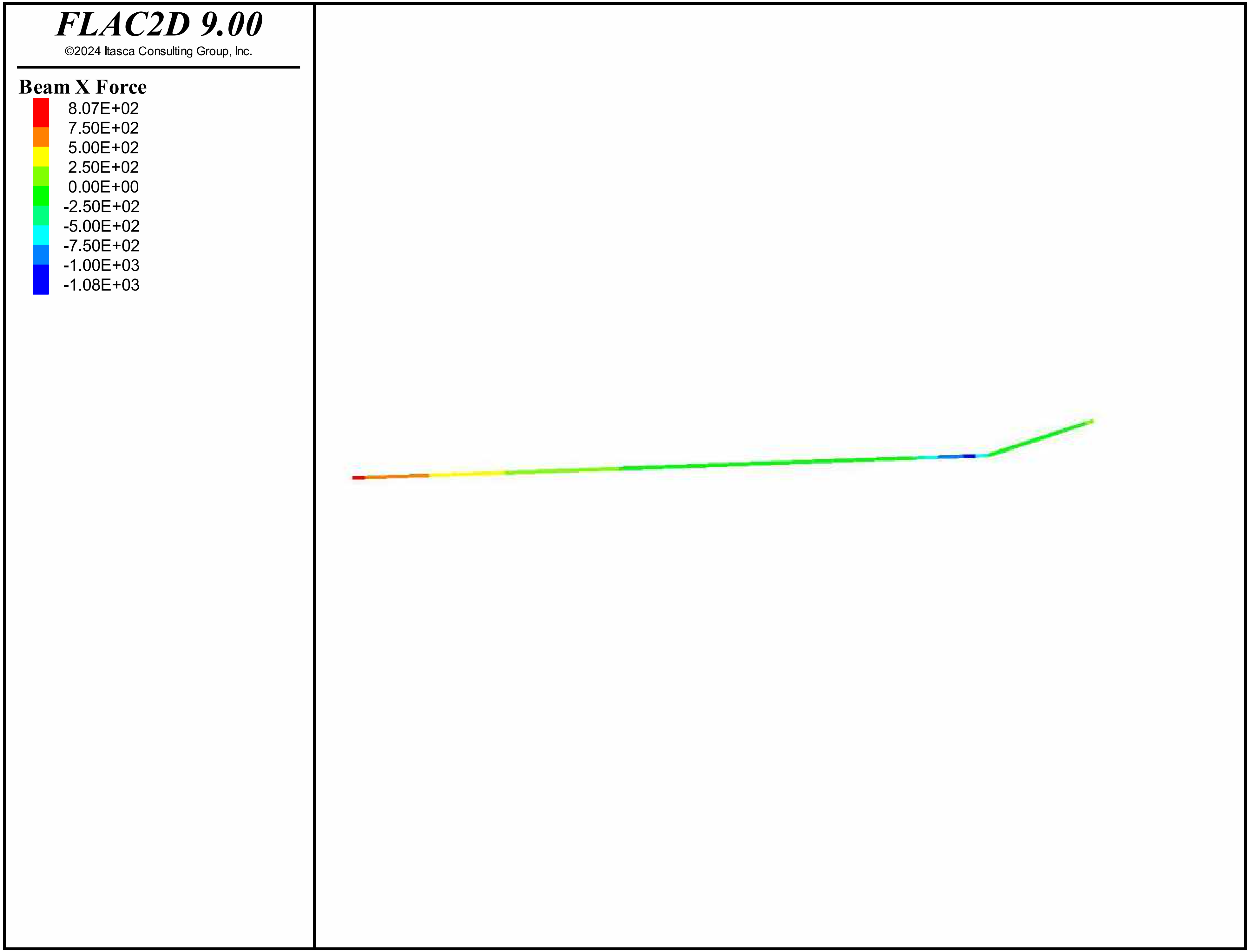
Geometry

Geometry Set Name

North-South_Restoration_FLAC_1V3H_no_bdy




FLAC2D 9.00
©2024 Itasca Consulting Group, Inc.
Zone Displacement Magnitude
9.65E-01
9.00E-01
8.00E-01
7.00E-01
6.00E-01
5.00E-01
4.00E-01
3.00E-01
2.00E-01
1.00E-01
0.00E+00
Zone Displacement Vectors
Maximum: 0.964917
Scale: 5.61253



FLAC2D 9.00
©2024 Itasca Consulting Group, Inc.
Beam X Force
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7.50E+02
5.00E+02
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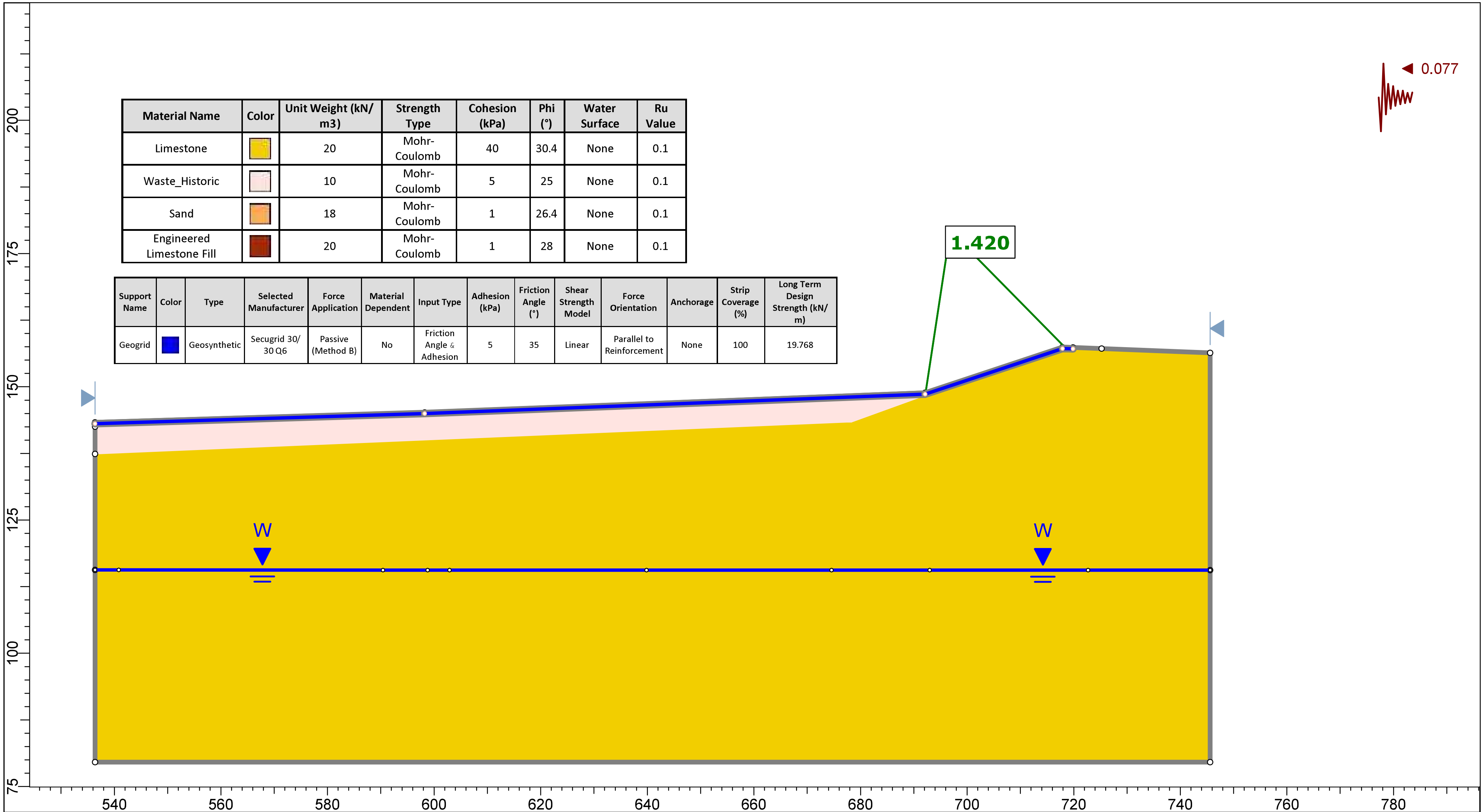
APPENDIX I
Sensitivity Analysis



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		20	Mohr-Coulomb	40	30.4	None	0.1
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	26.4	None	0.1
Engineered Limestone Fill		20	Mohr-Coulomb	1	28	None	0.1

Support Name	Color	Type	Selected Manufacturer	Force Application	Material Dependent	Input Type	Adhesion (kPa)	Friction Angle (°)	Shear Strength Model	Force Orientation	Anchorage	Strip Coverage (%)	Long Term Design Strength (kN/m)
Geogrid		Geosynthetic	Secugrid 30/30 Q6	Passive (Method B)	No	Friction Angle & Adhesion	5	35	Linear	Parallel to Reinforcement	None	100	19.768

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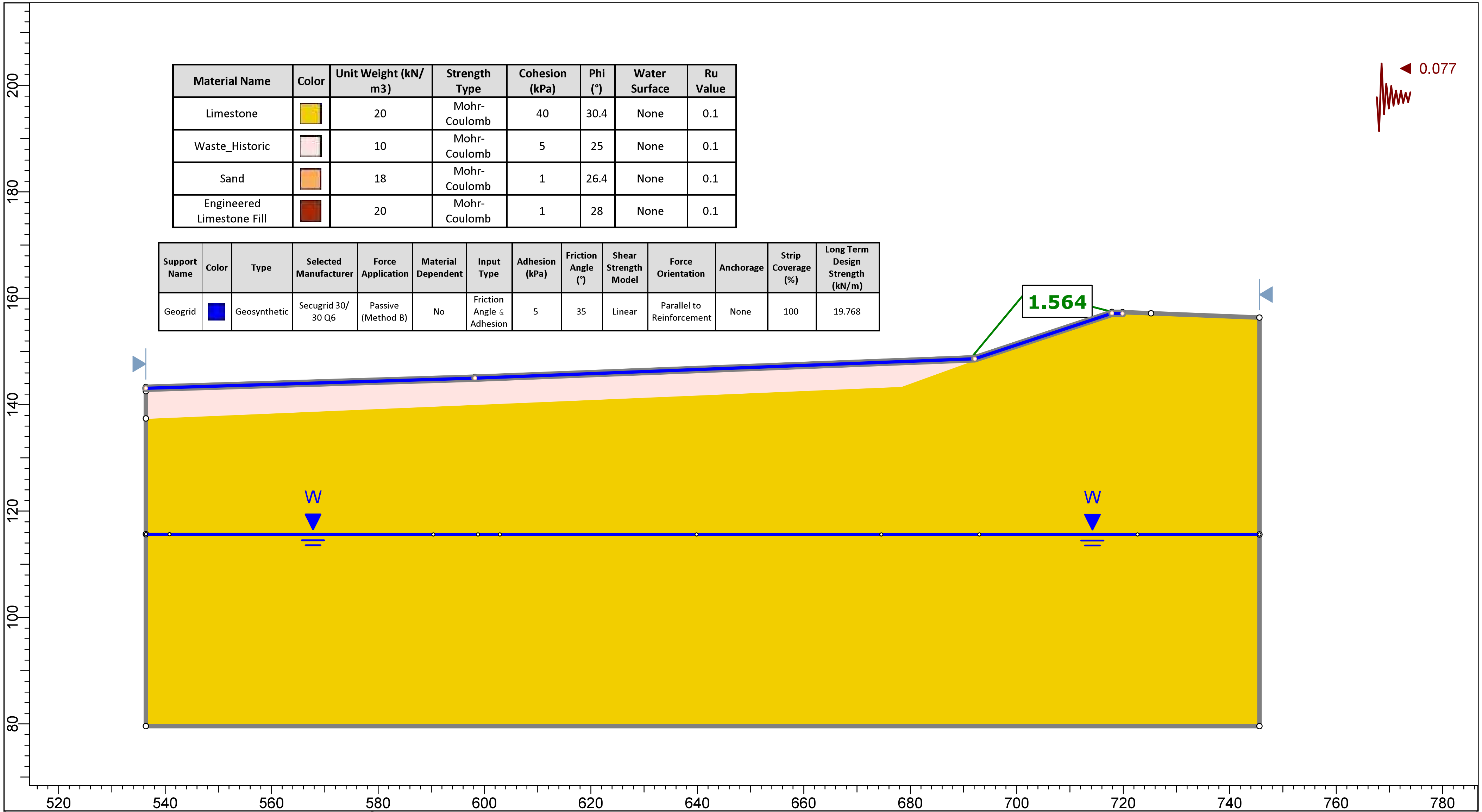
SLIDEINTERPRET 9.034

Project	MRC - Southern Piggyback Liner SRA
Analysis Description	Non-Circular_SEE
Section	Group 1 - Non-Circular
Scale	1:1000
Company	Talis Consultants Pty Ltd
Date	December 2024
File Name	Southern Piggyback Liner_North-South_sensitivity.slmd

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Water Surface	Ru Value
Limestone		20	Mohr-Coulomb	40	30.4	None	0.1
Waste_Historic		10	Mohr-Coulomb	5	25	None	0.1
Sand		18	Mohr-Coulomb	1	26.4	None	0.1
Engineered Limestone Fill		20	Mohr-Coulomb	1	28	None	0.1

Support Name	Color	Type	Selected Manufacturer	Force Application	Material Dependent	Input Type	Adhesion (kPa)	Friction Angle (°)	Shear Strength Model	Force Orientation	Anchorage	Strip Coverage (%)	Long Term Design Strength (kN/m)
Geogrid		Geosynthetic	Secugrid 30/30 Q6	Passive (Method B)	No	Friction Angle & Adhesion	5	35	Linear	Parallel to Reinforcement	None	100	19.768



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SLIDEINTERPRET 9.034

Project	MRC - Southern Piggyback Liner SRA
Analysis Description	Circular_SEE
Section	Group 2 - Circular
Scale	1:1000
Company	Talis Consultants Pty Ltd
Date	December 2024
File Name	Southern Piggyback Liner_North-South_sensitivity.slmd

Assets | **Engineering** | **Environment** | **Noise** | **Spatial** | **Waste**

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