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ALLAWUNA FARM LANDFILL

Stability Analysis and Liner System Integrity Assessment for Landfill Development

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REPORT

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1.0 INTRODUCTION

1.1 Overview

SITA Australia Pty Ltd (SITA) has appointed Golder Associates Pty Ltd (Golder) to provide engineering design services and supporting technical advice for the Allawuna Farm Landfill (Allawuna Landfill). This includes carrying out stability analyses for the landfill and integrity assessments for the geosynthetic liner systems (basal liner).

This document summarises the stability analyses and liner integrity assessment for the Allawuna Landfill. Golder's scope of services for the work summarised in this document is outline in Golder's proposal dated 11 December 2014 (Reference [1]).

1.2 Purpose

The purpose of this document is to provide engineering information regarding the stability of the proposed landfill design and integrity of the liner system materials prior, during and subsequent to waste placement. This information has been used by Golder in the completion of the revised WAA that will be submitted by Golder to the DER for the Allawuna Landfill.

1.3 Objectives

The objectives of the stability analysis and liner system integrity assessment are the following:

- Assess the proposed final landfill slopes and potential cover materials.
- Assess the stability of the liner system and waste landform during operational stages and after closure.
- Assess the impact of a possible malfunction of leachate pumps on the stability of the landfill during operational stages.
- Assess the stability of embankments and foundation throughout the life of the facility and after closure.
- Assess the impact of a seismic event on the stability of the landfill during operational stages and after closure.
- Assess the integrity of the proposed liner system prior, during and subsequent to waste placement.
- Propose suitable materials for the construction of the basal liner system and capping system.

2.0 ABBREVIATIONS AND DEFINITIONS

The acronyms and abbreviations used in this document are defined in Table 1.

Table 1: Acronyms, Abbreviations and their Meanings.

Name/Acronym	Definition
Allawuna Landfill	Allawuna Farm Landfill located south of the Great Southern Highway, approximately 20 km west from the town of York.
AS	Australian Standard
ASTM	American Society for Testing and Materials
BA	Bowman & Associates Pty Ltd
BPEM	Best Practice Environmental Management
CIU	Isotropically consolidated undrained triaxial test
CPT	Cone Penetration Test
DER	Department of Environment and Regulation
Division Bunds	Short-term embankments that separate (divide) adjacent cells in the landfill

Name/Acronym	Definition
FLAC	Two-dimensional explicit finite difference modelling software
FoS	Factor of Safety
GCL	Geosynthetic Clay Liner
Global Stability	Stability of the basal liner system interface, waste slopes, and embankment and foundation
Golder	Golder Associates Pty Ltd
GRI	Geosynthetic Research Institute
HDPE	High Density Polyethylene
LLDPE	Linear Low Density Polyethylene
MCE	Maximum Credible Earthquake. MCE represents the largest earthquake that is expected to occur in the landfill area, for which the facility is designed to remain stable, without major damages following the event. MCE events are unlikely to occur during the design life of the facility hence it was applied to the final landform.
MDE	Maximum Design Earthquake. MDE represents the earthquake that is expected to generate the highest loading conditions in the landfill area for which the landfill is designed to remain operational, with potential damages being readily repairable following the event. MDE events are unlikely but possible to occur during the design life of the facility hence it was applied to the landform during operational stages.
MSW	Municipal Solid Waste
NEHRP	National Earthquake Hazards Reduction Program
OBE	Operating Basis Earthquake. OBE represents the earthquake for which the landfill is designed to remain operational, with minimal damages following the event. OBE events are more likely to occur during the design life of the facility hence it was applied to the landform during operational stages.
Perimeter Bunds	Long-term embankments that delineate the boundaries of the waste disposal area
PGA	Peak Ground Acceleration
SITA	SITA Australia Pty Ltd
V : H	Vertical : Horizontal
Veneer Stability	Stability of capping liner system
WAA	Works Approval Application
Waste	Municipal solid waste

3.0 PROPOSED LANDFILL DESIGN

3.1 Design Overview

The Allawuna Landfill will be developed in two phases, each with separate leachate collection and abstraction systems, and divided into a number of cells. The phases will be formed by excavating into natural ground, while fill will be placed in some areas to achieve the required grade. The material excavated will be used to construct the waste containment embankment and provide cover material.

The base of the landfill will be sloped at approximately 2-3% towards a sump located at the southern corner of the cells. A pipe network will be installed at the base of the landfill (drainage layer). Leachate will be pumped from the sumps to a leachate collection pond.

3.2 Liner System Configuration

3.2.1 Basal Liner System

Leachate generated during the landfill operation will be contained by a composite liner system overlaying the engineered subgrade (i.e. compacted sandy clay/clayey sand material, based on geotechnical field investigation and laboratory testing carried out by Golder, Reference [2]). The composite lining system will comprise the following (from bottom to top):

- Engineered subgrade including re-compaction of at least 0.5 m of clayey soil.
- Geosynthetic Clay Liner (GCL)
- 2.0 mm High Density Polyethylene (HDPE)
- Non-woven cushion geotextile
- Drainage layer consisting of 300 mm layer of aggregate
- Non-woven separation geotextile.

3.2.2 Capping Liner System

The proposed capping system will comprise the following (from bottom to top):

- Interim cover soil overlaying the waste (i.e. sandy clay/clayey sand material)
- 300 mm layer of sand (i.e. gas collection layer)
- Geosynthetic Clay Liner (GCL)
- 2.0 mm Linear Low Density Polyethylene (LLDPE) geomembrane
- Non-woven cushion geotextile, if required
- Geocomposite drainage layer
- Capping layer consisting of 700 mm capping soil and 300 mm topsoil (i.e. clayey/silty sand material).

4.0 LANDFILL STABILITY AND INTEGRITY OF LINER MATERIALS

4.1 Landfill Stability

The Allawuna landfill stability analysis was divided into three main portions as follows:

1) Liner interface stability, comprising:
   a) Assessment of the capping liner system stability (i.e. veneer stability)
   b) Analyses of the basal liner system interface stability
   c) Assessment of the basal liner system integrity
2) Waste stability
3) Embankment and foundation stability.

Section 5.0 summarises the stability assessment of the capping liner system interface (i.e. veneer stability). The stability analyses for the basal liner system interface, waste slopes, and embankment and foundation are summarised in the global stability analyses in Section 6.0.

Typically, critical conditions in MSW landfills take place when the active waste slope is not buttressed, either by an engineered embankment (i.e. bund) or by waste in adjacent cells, hence the deposition planning of waste into the landfill (i.e. sequence of deposition into each cell) was a key consideration for the stability analyses herein. Waste deposition for the Allawuna Landfill is planned to follow the cell numbers starting from Cell 1.

Based on the proposed lining systems (Refer to Section 3.2) and geotechnical properties of the embankment and foundation materials for the Allawuna Landfill (inferred from laboratory testing and CPT interpretation in the geotechnical field investigation report, Reference [2]), the most likely mechanism of failure is expected to be within the interface of the basal liner system.

Although the basal liner system interface may be considered the most likely failure plane in the landfill design, undertaking stability assessment of the landfill cover (veneer stability) is essential to verify if the proposed final landfill slopes and potential cover materials (Refer to Section 3.2.2) are appropriate.

It should be noted that the stability analyses were performed for the selected cross-sectional geometries (Refer to Section 4.3), thus providing a two-dimensional (2D) representation of the actual (3D) conditions. Although this is consistent with standard industry practice, it is known that 2D analysis of landfills, when performed for critical cross-sections, is generally a conservative approach as it does not account for beneficial 3D geometry effects.

4.2 Liner Materials' Integrity

The assessment of stresses in the basal liner system is one of the key design aspects that govern the selection of appropriate geosynthetic materials. Stresses on the liner are assessed pre-waste placement, during operation (up to final waste level deposition) and post-waste deposition.

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

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

The assessment of the integrity of the basal liner system in Section 3.2.1 prior to placement of waste, during waste deposition (operations) and post waste deposition are summarised in Section 7.0.

4.3 Critical Sections

Three design cross-sections were considered for the stability analyses and one for the basal liner system integrity assessment. These identified sections represent the highest risk of instability for the landfill slopes based on geometry, sequence of deposition of the waste into each cell, and subsurface conditions.

The critical sections used in the models are described as follows:

- Section A: This section extends north to south through the southern aspect of the final landform.
- Section B: This section extends north-east to south-west through Cells 1, 4 and 6
- Section C: This section extends north-west to south-east through Cells 1 and 2.

The location of the above sections can be seen in Figure 1. Section A was used in the veneer stability assessment as it represents the longest slope at landfill closure. Sections B and C were used in the global stability assessment as these have been identified as the critical sections for the landfill stability. Section C was also used in the liner integrity stability assessment based on the outcomes of the global stability assessment (Refer to Section 7.0).

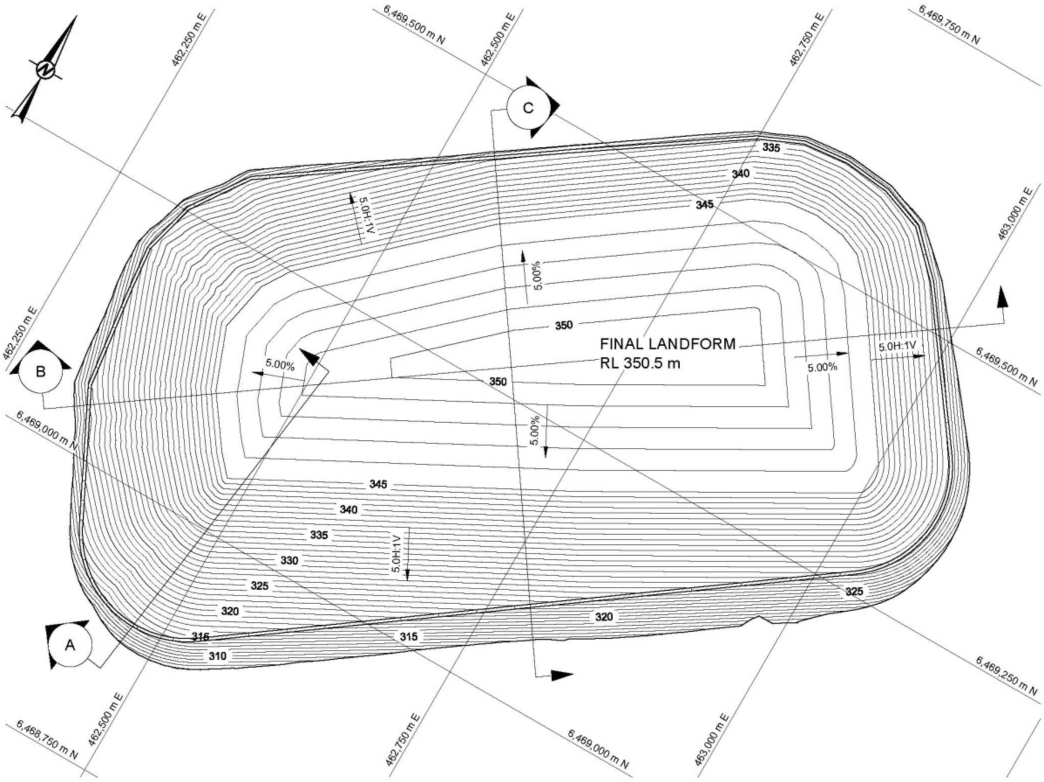

Figure 1: Approximate location of the critical sections for the stability analyses and liner integrity stability assessment

5.0 VENEER STABILITY ASSESSMENT

5.1 Approach

The critical cap slope geometry was identified based on the model of the proposed final landform, and this geometry was used as the basis of the calculations. A sensitivity analysis was performed to estimate the minimum friction angle required for geosynthetic interfaces, assuming zero adhesion. Interface shear parameters were then adopted for each interface using values from the Golder database and from literature and the factor of safety calculated for each. The calculated factor of safety was then compared to the acceptable minimum value and recommendations made for future testing.

For the purposes of the veneer stability assessment, the term ‘critical interface’ is used to refer to the shear interface with the lowest factor of safety (FoS), which will determine the minimum requirements for frictional strength. The topsoil and capping soil are collectively referred to as ‘over-liner’ soil and are treated as one layer of one meter thickness. Similarly, the interim cover soil and waste below the geosynthetic layers are collectively referred to as ‘subgrade’ and are also treated as one layer.

The three general planes of veneer instability relevant to the proposed capping system are as follows:

- Sliding of the over-liner soil along the uppermost geosynthetic
- Sliding of one geosynthetic relative to another, and
- Sliding of the combined over-liner soil and geosynthetic components along the subgrade.

Due to the typically low frictional properties (friction angle and adhesion) of the interface LLDPE against geotextile, the critical interface in terms of veneer stability is expected to be within one of these geosynthetic components of the capping liner system (i.e. between the cushion geotextile and LLDPE; or LLDPE and GCL).

5.2 Assumptions

The following assumptions have been made for the veneer stability assessment:

1) The materials beneath the liner system including the stored waste, interim cover soil and gas collection layer are stable (i.e. failure within or between these sub-cap materials do not occur).

2) The effect of landfill gas pressurisation on the capping system stability is negligible (i.e. gas collection system is fully functional).

3) A head of 0.1 m is present above the LLDPE (i.e. hypothetical operating condition simulating intense rainfall and malfunctioning drainage geocomposite).

4) Post-peak shear strengths will be mobilised at geosynthetic interfaces due to expected settlement in the waste. Post-peak adhesion for geosynthetic interfaces has been ignored (conservative approach).

5) No reinforcement strength is provided by geosynthetic materials in tension (conservative approach).

6) The GCL is unsaturated.

5.3 Veneer Stability Inputs

5.3.1 Model Section

Veneer stability assessments were carried out on one cross-section (Section A) that was considered to represent the critical slope for the final landform in regards to veneer stability. The location of this section can be seen in Figure 1 in Section 4.3.

The overall critical slope is 168 meters long at 1V:5H, and runs approximately North-South through the southern aspect of the final landform. Given the limitations on geosynthetic material roll length, a maximum slope length of 40 meters has been adopted for the veneer stability assessment, which represents the typical length of a GCL roll. Design of intermediate liner anchoring was not considered in the assessment. This should be considered in the future as part of the detailed design of the cover system.

5.3.2 Minimum Acceptable Factor of Safety

Minimum acceptable values for factor of safety (FoS) against veneer failure (static conditions) have been adopted using the qualitative method proposed by Koerner and Soong (Reference [3]), adopting a permanent duration, a noncritical concern and a non-hazardous type of waste as inputs. Using these inputs Koerner and Soong recommend a minimum acceptable FoS of 1.4. This value has therefore been adopted for use in evaluating the static veneer stability of the proposed capping system.

5.3.3 Over-liner Soil Material Properties

Material properties for over-liner soil will vary depending on borrow source. No strength testing has yet been undertaken on the proposed over-liner soil samples under the effective stresses imparted by the cover soil (i.e. up to about 50 kPa). Therefore, conservative values for sandy clay/clayey sand material have been assumed. The material properties adopted for the veneer stability assessment are summarised in Table 2.

Table 2: Cap soil properties adopted for the veneer stability assessment

Material	Property	Value (Assumed)
Cover soil	Average unit weight (g)	18.5 kN/m^3
	Internal friction angle (φ)	20° *
	Internal cohesion	0 kPa

*CIU triaxial testing at effective stresses between 100 kPa and 500 kPa indicated a friction angle value of 28°

The governing parameters for veneer stability are interface friction angle and adhesion. To identify the required minimum interface friction angle, adhesion was set to zero (Refer to Section 5.2 for assumptions).

The limit equilibrium methods outlined in Koerner and Soong (Reference [3]) have been used in this assessment to estimate the interface friction angle required to achieve a FoS of 1.4. Friction angle φ (phi) is designated as the independent variable and FoS is the dependent variable. Under the design conditions, the minimum acceptable FoS can be achieved with a friction angle of approximately 16 degrees. The adopted shear parameters used in the assessment are presented in Section 5.4.

5.4 Results of the Veneer Stability Assessment

The FoS was calculated for each interface in the proposed capping system subject to the assumptions listed in Section 5.2. A summary of the results are shown in Table 3. FoS values have been calculated using limit equilibrium (sliding-block) equations including influence of pore-pressures as outlined in Koerner and Soong (Reference [3]). In lieu of project-specific shear strength testing, values of the shear strength parameters have been obtained from the following sources:

- Golder Associates internal database of geosynthetics shear test results
- Geosynthetics Research Institute (GRI, Reference [4])

Note that for the geocomposite drain vs. geotextile interface no published data was available.

The results provided in Table 3 show that, under the adopted shear strength parameters, the critical interface is between the cushion geotextile and LLDPE geomembrane. This interface is associated with a FoS below unity, indicating failure along this plane. The implication is that the shear strength properties of a smooth LLDPE against geotextile may be insufficient and that modifications should be made in order to achieve the minimum acceptable FoS of 1.4.

Results of the sensitivity analysis show that a post-peak friction angle of 16 degrees will be required at the LLDPE/geotextile interface. Results from the Golder database and GRI (Reference [4]) indicate that this can be achieved using a textured LLDPE instead of smooth.

Table 3: Interface friction parameters and corresponding factors of safety

Interface			Adopted Shear Strength Parameters (Post-Peak)		Source	Factor of Safety[2]
No.[1]	**Upper Layer**	**Lower Layer**	**Friction Angle (°)**	**Adhesion (kPa)**		
1 (W)	Over-liner soil	Drainage Geocomposite (Non-woven geotextile)	25	0	Laboratory test results (Golder Database)	2.2
2 (W)	Drainage Geocomposite (Non-woven geotextile (heat-bonded))	Drainage Geocomposite (HDPE Geonet)	16	0	GRI handbook 2005	1.4
3 (W)	Drainage Geocomposite (Non-woven geotextile)	Cushion Geotextile	18	0	Assumption[3]	1.5
4 (W)	Cushion Geotextile	Smooth LLDPE Geomembrane	11	0	Laboratory test results (Golder Database)	*0.9*
5	Smooth LLDPE Geomembrane	GCL (upper geotextile component)	11	0	Laboratory test results (Golder Database)	*1.0*
6	GCL internal shear		8	11	Laboratory test results (Golder Database)	3.9
7	GCL (lower geotextile)	Interim cover soil	35	0	Laboratory test results (Golder Database)	2.9

Notes: [1](W) Interface subject to pore-pressures from 0.1 m head acting on LLDPE.
[2]Values in *italic* do not meet minimum acceptable FoS.
[3]Database test results not available for this interface.

6.0 GLOBAL STABILITY ANALYSES

6.1 Approach

The global stability for the Allawuna Landfill (i.e. stability analyses for the basal liner system interface, waste slopes, and embankment and foundation) was carried out using the 2D limit equilibrium slope stability analysis software SLIDE version 6.0 (Rocscience 2010). A number of scenarios, boundaries and loading conditions were considered in the analyses.

Given the Allawuna Landfill project constraints and proposed landfill boundaries, a number of stability analyses were performed with different configurations to establish the most suitable arrangement for the landfill. These analyses were carried out taking into consideration the basal liner system interface, and the waste and embankment slopes. Once the design geometry and parameters were selected, the critical sections (Refer to Section 6.3.1) were analysed to confirm the adopted values. Initial results lead to changes in the stability analyses models. These changes were integrated in the final design and cross-sections analysed.

In terms of global stability, the most likely failure plane for the Allawuna Landfill design should be within the interface of the basal liner system, possibly between the cushion geotextile and HDPE (or HDPE and GCL) due to the typically low frictional properties of HDPE (friction angle and adhesion).

6.2 Assumptions

The following assumptions have been made for the global stability analyses:

1) The effect of landfill gas on the landfill stability is negligible (i.e. gas collection system is fully functional).

2) No reinforcement strength is provided by geosynthetic materials in tension.

3) The area of refusal in the geotechnical field investigation report (Reference [2]) has been assumed to be granite bedrock, which was considered to have infinite strength.

6.3 Global Stability Inputs

6.3.1 Model Sections

Stability analyses were carried out on two cross-sections (Section B and Section C) that were considered to represent the highest risk of instability for the landfill slopes based on geometry, sequence of deposition of the waste into each cell, and subsurface conditions. The location of these sections can be seen in Figure 1 in Section 4.3.

6.3.2 Boundary and Loading Conditions

6.3.2.1 Pore Pressure

For the stability analysis, three phreatic surface conditions (water/leachate level in waste) were considered:

- No phreatic surface (i.e. no pore pressure build-up)

- Elevated phreatic surface (i.e. low pore pressure build-up on the liner). This phreatic surface was applied to each cell simulating a head of approximately 0.3 meters above the liner at the internal toe of the embankments, representing a hypothetical ‘steady-state’ condition.

- High phreatic surface (i.e. high pore pressure build-up on the liner). This phreatic surface was applied to each cell simulating a head of approximately one meter above the liner at the internal toe of the embankments (up to crest of division bund), representing a malfunction of the leachate pumps.

6.3.2.2 Seismicity

To simulate the effect that a seismic event may have on the stability of the landfill, a pseudo-static slope stability analysis has been undertaken. A pseudo-static stability analysis is a simplified approach used in earthquake engineering to analyse the seismic response of a structure, wherein the seismic forces are modelled as an additional (destabilising) gravitational force. This force is usually assumed equal to the peak horizontal acceleration estimated for the site, with the vertical seismic acceleration component usually disregarded. Although this approach simplifies complex, dynamic seismic loading as static forces, it is commonly applied for slope stability analysis of landfill embankments, with more complex analysis generally justified only in cases where the simplified analysis indicates stability concerns.

The seismic return period intervals adopted in the pseudo-static stability analysis are as follows:

- Operating Basis Earthquake (OBE): 500 year return period
- Maximum Design Earthquake (MDE): 1000 year return period
- Maximum Credible Earthquake (MCE): 2000 year return period.

Currently, there are no specific requirements or guidelines in Australia for selecting return period intervals for the design of a MSW facility. The intervals selected are used internationally for municipal solid waste landfills and are based on experience with similar projects in Australia.

OBE and MDE events were applied to the landform during operational stages. MCE event was applied to the final landform.

Seismic forces for the stability analysis were based on information attained from the *Atlas of Seismic Hazard Maps of Australia* (Reference [5]).

A site amplification factor of 1.25 was applied to the horizontal accelerations (PGAs) and used as loading parameter in the stability analyses (i.e. horizontal seismic load coefficient). This amplification factor was used to provide for material characteristics and site conditions under seismic loading. Considering the geotechnical properties of embankment and foundation materials in the Allawuna Landfill, AS 1170.4 (Reference [6]) classifies the site sub-soil as Class C_e (shallow soil site). Per AS 1170.4, shallow soil sites have a corresponding site amplification factor of 1.25 (for period $T \leq 3$ seconds). This site amplification factor value is conservative in comparison with coefficients given by NEHRP provisions (Reference [7]) and should provide for a conservative indication of earthquake loading conditions in the Allawuna Landfill.

Based on the above considerations, the Peak Ground Acceleration (PGA) (equivalent to the spectral period of zero seconds) in proximity of the site location, and its correspondent horizontal seismic load (based on the assumed site sub-soil classification), that is used in the pseudo-static stability analyses, is as follows:

- Operating Basis Earthquake (OBE): PGA 0.075g, which corresponds to a horizontal seismic load coefficient of approximately **0.094** (based on a site amplification factor of 1.25).
- Maximum Design Earthquake (MDE): 1000 year return period, PGA 0.125g, which corresponds to a horizontal seismic load coefficient of approximately **0.156** (based on a site amplification factor of 1.25).
- Maximum Credible Earthquake (MCE): 2000 year return period, PGA 0.200g, which corresponds to a horizontal seismic load coefficient of approximately **0.250** (based on a site amplification factor of 1.25).

6.3.3 Scenarios

The scenarios listed in Table 4, representing different conditions throughout the life of the landfill, were considered as part of the global stability analyses.

Table 4: Scenarios considered in the Global Stability Analyses

Scenario		Description
1	Operational landform, Static, Low pore pressure build-up on liner	Assess the stability of the landfill during normal operational conditions.
2	Operational landform, Static, High pore pressure build-up on liner	Assess the impact of a possible malfunction of leachate pumps on the stability of the landfill during operational stages.
3	Operational landform, Earthquake (OBE seismic load), No pore pressure build-up on liner	Assess the impact of a seismic event on the stability of the landfill during operational stages. The landfill should remain operational, with minimal damages to embankments and liner system following an OBE event.
4	Operational landform, Earthquake (MDE seismic load), No pore pressure build-up on liner	Assess the impact of a seismic event on the stability of the landfill during operational stages. The landfill should remain operational, with potential damages to embankments and liner system being readily repairable following an MDE event.
5	Post Closure/Final landform, Static, Low pore pressure build-up on liner	Assess the stability of the landfill after closure for normal conditions.
6	Post Closure/Final landform, Earthquake (MCE seismic load), No pore pressure build-up on liner	Assess the impact of a seismic event on the stability of the landfill after closure. The landfill should remain stable, without major damages following an MCE event.

6.3.4 Material Parameters

The material parameters adopted for the stability analysis are summarised in Table 5.

Table 5: Summary of Material Parameters for Geotechnical Stability Analyses

Parameter		Material				
		Waste[1]	Liner System[2]	Embankment and Subgrade[3]	*In Situ* Soil[4]	Bedrock[5]
Unit Weight (kN/m^3)	Unsaturated	10	10	18.5	18.5	-
	Saturated	12		22	22	
Drained	Cohesion/Adhesion (kPa)	5	0	5	5	-
	Friction Angle (degrees)	25	16	28	28	-
Undrained	Vertical stress ratio (Su/σ'v)	-	-	0.35	0.35	-
	Minimum shear strength (kPa)	-	-	0	150	-

Notes: [1]Waste parameters based on published values for municipal solid waste (Reference [8]).

[2]Liner system parameters adopted based on previous experience with similar materials. Friction angle for liner system based on peak values for the critical interface between liner components (i.e. Double Textured HDPE and cushion geotextile, or Double Textured HDPE and GCL).

[3]Parameters for compacted Sandy Clay/Clayey Sand based on laboratory testing results.

[4]Parameters for uncompacted (i.e. *in situ*) Sandy Clay/Clayey Sand based laboratory testing results

[5]Material assumed as bedrock considered to have infinite strength.

Geosynthetic material parameters are highly dependent on material composition and manufacturing process, hence the material parameters adopted for the basal liner system were based on engineering judgement as site specific materials have not been identified at this time. As the stability of the landfill relies, to a large extent, on the liner interface shear strength, testing will be undertaken prior to construction (when actual materials are available) to determine the critical interface between liner components and verify the assumptions made in the stability analysis. Ideally, the critical interface should be between the HDPE and cushion geotextile.

Results of a sensitivity analysis for the basal liner system showed that a friction angle of 16 degrees (peak value) will be required at the HDPE/geotextile interface (or HDPE/GCL interface) to achieve minimum acceptable factors of safety (Refer to Section 6.3.5). This value has been adopted to evaluate the liner interface stability of the basal liner system. The peak friction angle of 16 degrees can be achieved using a double textured HDPE instead of single textured.

Results of a sensitivity analysis for waste parameters showed that a friction angle of 25 degrees and cohesion of 5 kPa should provide a conservative representation of the waste in the Allawuna Landfill. A friction angle of 25 degrees and cohesion of 5 kPa are in the lower range of values commonly used for municipal solid waste internationally (Reference [8]). These values have been adopted to evaluate the stability of the operational and final landform.

6.3.5 Minimum Factor of Safety

The minimum acceptable factor of safety (minimum FoS) used to assess the outcome of the stability analyses for the Allawuna Landfill were determined based on typical values used internationally for municipal solid waste landfills and experience with similar projects in Australia. Currently, there are no specific requirements or guidelines from the Western Australian regulatory authorities.

Minimum factors of safety were applied in the stability analysis as follows:

- FoS of 1.5 or greater to provide acceptable lifetime stability under static loading; this has been applied for long-term conditions that may be present for 20 years or more (i.e. perimeter bunds).
- FoS of 1.3 or greater to provide acceptable interim stability under static loading; this has been applied for short-term conditions that may be present for less than 20 years (i.e. division bunds).
- FoS of 1.1 or greater to provide acceptable stability under static loading, where the landfill is subject to high pore pressure build-up from a phreatic surface about one meter above the liner.
- FoS of 1.1 or greater to provide acceptable stability under earthquake loading, where the landfill is subject to an OBE event.
- FoS of 1.0 or greater to provide acceptable stability under earthquake loading, where the landfill is subject to MDE or MCE events.

6.4 Results of the Global Stability Analyses

6.4.1 Basal Liner System Interface Stability Analyses

The liner interface stability analyses were carried out using a polyline type of search (non-circular surface type) located within the composite liner layers (i.e. liner material interface). A summary of the results for these analyses (i.e. minimum FoS) for each scenario are shown in Table 6. Calculated FoS values were selected for the critical section with the lowest FoS (i.e. Section C). Figures showing these results (i.e. output figures) are attached in Appendix A.

Table 6: Summary of Results for the Basal Liner System Interface Stability Analyses

Scenario		Condition[1]	Minimum FoS[2] Required	Calculated FoS[3]	Output Figures
1	Operational, Static, Low pore pressure build-up on liner	Short-Term	1.3	1.76	A1
		Long-Term	1.5	1.94	A2
2	Operational, Static, High pore pressure build-up on liner	Short-Term	1.1	1.72	A3
		Long-Term	1.1	1.87	A4
3	Operational, Earthquake (OBE), No pore pressure build-up on liner	Short-Term	1.1	1.32	A5
4	Operational, Earthquake (MDE), No pore pressure build-up on liner	Short-Term	1.0	1.14	A6
5	Post Closure, Static, Low pore pressure build-up on liner	Long-Term	1.5	1.93	A7
6	Post Closure, Earthquake (MCE), No pore pressure build-up on liner	Long-Term	1.0	**0.81**	A8

Notes: [1]Maximum slope for short-term condition is 1V:3H for the embankments (i.e. division bunds) and 1V:3H for the waste. Maximum slope for long-term condition is 1V:3H for the embankments (i.e. perimeter bunds) and 1V:5H for the waste.
[2]Minimum FoS based on typical values used internationally for municipal solid waste landfills and experience with similar projects in Australia.
[3]FoS values rounded up to two decimal places. Values in **bold** do not meet minimum acceptable FoS (Refer to Deformation Analysis in Section 6.4.3).

Based on the results shown in Table 6, the minimum acceptable factor of safety (minimum FoS) are achieved for scenarios 1 to 5. Although the result for Scenario 6 is lower than the minimum FoS, the estimated permanent deformation due to earthquake action (MCE) is well below acceptable values (refer to Section 6.4.3).

6.4.2 Waste Stability Analyses

The waste stability analyses were carried out using a grid and slope type of search (circular surface failure mechanism) located through the waste landform (i.e. waste slope). A summary of the results of these analyses (i.e. minimum FoS) for each scenario are shown in Table 7. Calculated FoS values were selected for the critical section with the lowest FoS (i.e. Section C). Figures showing these results (i.e. output figures) are attached in Appendix A.

Table 7: Summary of Results for the Waste Stability Analyses

Scenario		Conditions[1]	Minimum FoS[2] Required	Calculated FoS[3]	Output Figures
1	Operational, Static, Low pore pressure build-up on liner	Short-Term	1.3	1.88	B1
		Long-Term	1.5	2.24	B2
2	Operational, Static, High pore pressure build-up on liner	Short-Term	1.1	1.88	B3
		Long-Term	1.1	2.22	B4
3	Operational, Earthquake (OBE), No pore pressure build-up on liner	Short-Term	1.1	1.39	B5
4	Operational, Earthquake (MDE), No pore pressure build-up on liner	Short-Term	1.0	1.17	B6
5	Post Closure, Static, Low pore pressure build-up on liner	Long-Term	1.5	2.25	B7
6	Post Closure, Earthquake (MCE), No pore pressure build-up on liner	Long-Term	1.0	**0.95**	B8

Notes: [1]Maximum slope for short-term condition is 1V:3H for the embankments (i.e. division bunds) and 1V:3H for the waste. Maximum slope for long-term condition is 1V:3H for the embankments (i.e. perimeter bunds) and 1V:5H for the waste.
[2]Minimum FoS based on typical values used internationally for municipal solid waste landfills and experience with similar projects in Australia.
[3]FoS values rounded up to two decimal places. Values in **bold** do not meet minimum acceptable FoS (Refer to Deformation Analysis in Section 6.4.3).

Based on the results shown in Table 7, the minimum acceptable factor of safety (minimum FoS) are achieved for scenarios 1 to 5. Although result for Scenario 6 is lower than the minimum FoS, the estimated permanent deformation due to earthquake action (MCE) is well below acceptable values (refer to Section 6.4.3).

6.4.3 Seismic Deformation Assessment

The stability of the basal liner system and waste mass during MCE events (i.e. calculated FoS below the minimum acceptable FoS of 1.0) were analysed according to the Newmark Method (1965), a commonly used approach for calculating permanent seismic deformations (Reference [9]) for materials that, while shearing, harden and are able to dissipate the pore pressure built-up during seismic loading. In this approach, the potential failure mass is treated as a rigid body on a yielding base. Permanent seismic deformations occur when the rigid body acceleration (i.e. average acceleration of the failure mass) exceeds its yield acceleration (i.e. horizontal seismic load that results in a factor of safety of 1.0).

The results of the seismic deformation analyses (i.e. calculated displacement) were compared with values obtained from Makdisi and Seed (1978) charts, which are based on Newmark analyses for slopes and embankments for earthquake induced deformations (References [9] and [10]). Makdisi and Seed permanent displacement curves are widely used to estimate the seismic displacement for the liner system in MSW landfills.

Table 8 summarises the outcome of the seismic deformation analyses for the basal liner system and waste mass during MCE events. Allowable displacements shown in Table 8 are based in conservative values used for MSW, calculated using Newmark-type seismic displacement analyses (Reference [11]). Values of allowable displacement may be lower in an advanced coupled numerical modelling. However, this more complex analysis is generally justified only in cases where the simplified assessment indicates stability concerns (i.e. very high estimated displacement).

Table 8: Summary of Results for the Seismic Deformation Analyses

Analysis	Seismic Load[1] (g)	Seismic Yield[2] (g)	Calculated Displacement[3] (mm)	Chart Value[4] (mm)	Allowable Displacement[5] (mm)	Outcome
Basal Liner System Seismic Deformation	0.25	0.17	63	± 60	150 to 300	Acceptable
Waste Mass Seismic Deformation	0.25	0.22	49	10	1000	Acceptable

Notes: [1]Horizontal seismic load coefficient corresponding to calculated FoS values that did not meet the minimum acceptable factor of safety (i.e. MCE events)
[2]Seismic yield values for MCE events where the calculated FoS equals 1.0
[3]Displacement values calculated according to Newmark method (References [9] and [10]).
[4]Displacement estimated using Makdisi and Seed (1978) permanent displacement chart (References [9])
[5]Allowable displacement for the liner system and the waste mass based on typical values used for MSW (References [9] and [11]).

According to the permanent deformation analyses, MCE events will result in a displacement of 63 mm in the liner system; well below allowable displacements reported by Seed and Bonaparte (References [9] and [11]). This result is consistent with displacement values suggested in Makdisi and Seed (1978) charts (References [9] and [10]). Similarly, permanent seismic displacement of 49 mm was calculated for the waste mass. This result is considerably lower than allowable displacements typically used for MSW (Reference [11]).

6.4.4 Foundation and Embankment Stability Analyses

The foundation and embankment stability analyses were carried out using a grid and slope type of search (circular surface failure mechanism) located through the waste landform, embankment and foundation. Drained and undrained strength parameters were used in the analyses. A summary of the results of these analyses (i.e. minimum FoS) for drained parameters for each scenario are shown in Table 9. Calculated FoS values were selected for the critical section with the lowest FoS (i.e. Section C). Figures showing these results (i.e. output figures) are attached in Appendix A. Results for undrained parameters were not included in the stability analyses herein as these parameters resulted in higher calculated FoS values in comparison with drained parameters.

Table 9: Summary of Results for the Foundation and Embankment Stability Analyses

Scenario		Conditions[1]	Minimum FoS[2]	Calculated FoS[3]	Output Figures
5	Post Closure, Static	Long-Term	1.5	2.14	C1
6	Post Closure, Earthquake (MCE)	Long-Term	1.0	1.08	C2

Notes: [1]Maximum slope for short-term condition is 1V:3H for the embankments (i.e. division bunds) and 1V:3H for the waste. Maximum slope for long-term condition is 1V:3H for the embankments (i.e. perimeter bunds) and 1V:5H for the waste.
[2]Minimum FoS based on typical values used internationally for municipal solid waste landfills and experience with similar projects in Australia.
[3]FoS values rounded up to two decimal places.

In addition to the scenarios listed in Table 9, the foundation and embankment stability immediately after construction was analysed for short-term conditions (undrained parameters). These analyses resulted in calculated FoS values for Section C ranging from 1.31 (internal slope) to 5.74 (external slopes) for perimeter bunds. Based on these results and the results summarised in Table 9, the minimum acceptable factors of safety (minimum FoS) are achieved for all scenarios and model conditions regarding foundation and embankment stability, prior to and post waste deposition.

7.0 LINER SYSTEM INTEGRITY ASSESSMENT

7.1 Approach

To assess the integrity of the side slope basal liner system pre-waste placement, the self-weight of each liner component was compared with the tensile strength of the material to establish whether any layer of geosynthetic is over-stressed.

The integrity of the basal liner system during waste deposition (operations) and post waste deposition was assessed using the explicit finite difference modelling software FLAC version 7.0 (ITASCA 2011). FLAC was used due to its capability to model large deformation associated with the waste settlement whilst avoiding numerical calculation difficulties associated with large strain continuum modelling. Additionally, FLAC allows simulation of a geosynthetic material by introducing a beam element with a moment of inertia set to zero to represent a flexible sheet, which is unable to support any bending moment. Different beams can be included in the model to simulate each of the geosynthetic layers. The model developed to assess the basal liner integrity was simplified by introducing only one beam element, representing the geosynthetic material under investigation. The beam was fixed at a node in order to simulate its anchoring. Linear elastic-perfect plastic models with Mohr-Coulomb failure envelopes were adopted for the interfaces and materials (excluding the bedrock).

A staged modelling sequence was adopted to track the stresses on the liner as waste is deposited and to model the progressive settlement of the waste during operation. The majority of the waste layers were placed in lifts of approximately 3.0 meters.

The degradation of the post-waste placement was simulated by introducing a pressure to induce a total settlement equal to 20% of the waste thickness. This value of settlement is consistent with published literature on municipal solid waste (Reference [12]). This final settlement (secondary settlement) can be attributed to reorientation of the solid matter, biological decomposition and chemical processes (Reference [13]). The aim of this assessment was not to accurately estimate the time-dependent settlement caused by the post-waste placement processes but to investigate if this additional settlement could induce tension on the liner system.

7.2 Liner Integrity Assessment Inputs

7.2.1 Model Sections

Liner integrity assessments during waste deposition (operations) and post waste deposition were carried out on one cross-section (Section C) that were considered to represent the highest risk of instability for the landfill slopes based on geometry, sequence of deposition of the waste into each cell, and subsurface conditions. The location of these sections can be seen in Figure 1 in Section 4.3.

The slope identified for the assessment of the basal liner integrity prior to waste placement is shown in Figure 2. This slope is located at the north-western corner of the facility, where Cell 3 is proposed. It represents one of the longest slopes in the proposed landfill design (approximately 16 m long).

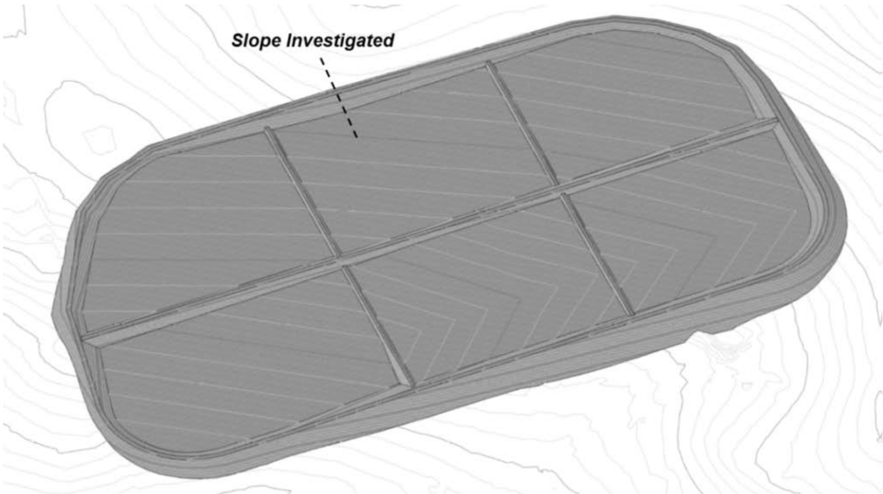

Figure 2: Slope investigated in the liner integrity assessment pre-waste placement

7.2.2 Allowable Liner Strains

The maximum allowable global design strain for geomembrane is discussed by Ian Peggs in a 2003 paper on geomembrane liner durability (Reference [14]). This paper is referenced in the BPEM guidelines (Reference [15]) as a minimum value to use for assessing the global strain stability of geomembranes. The values reported in Table 10 are applicable to the liner system integrity assessment undertaken herein.

Table 10: Maximum allowable strain for various geomembrane materials

Geomembrane Type	Maximum Allowable Strain (%)
HDPE smooth	6
HDPE randomly textured	4
HDPE structured profile	6
LLDPE density <0.935 g/cm^3	12
LLDPE density >0.935 g/cm^3	10
LLDPE randomly textured	8
LLDPE structured profile	10

The allowable global strains of GCL materials are not reported in the BPEM guidelines. However, a study undertaken to compare the performances of compacted clay liner (CCL) and GCL subjected to differential settlement (Reference [16]) have shown that GCL can resist strain greater than 10% and still maintain a low hydraulic conductivity (1 x 10^{-9} m/s or less). Therefore, for the Allawuna proposed basal liner system the structural integrity of the HDPE represents the weakest barrier in regards to allowable strain (4%). If the HDPE integrity is maintained, it is reasonable to assume that the GCL integrity would also be maintained.

Protection geotextile material can withstand global strains greater than 50% prior to reaching its break point. Therefore, a conservative value of 25% global strain could be used as allowable strain for the protection geotextile.

7.2.3 Scenarios

The following two scenarios were investigated in the liner integrity assessment during waste deposition (operation) and post-waste placement:

1) Integrity of the cushion geotextile

2) Integrity of the HDPE geomembrane.

It is important to maintain the integrity of the cushion geotextile (Scenario 1) during the operation of the facility, as this layer will protect the containment system (HDPE and GCL) from damage due to puncturing caused by the gravel particles that are part of the drainage layer. Additionally, if the weakest interface is between the cushion geotextile and the geomembrane, tensile strains in the geomembrane and the GCL will be minimised. In our assessment, we have assumed that this interface represents the weakest interface.

The integrity of the HDPE is investigated in Scenario 2 to estimate the possible tensile stresses developed in the HDPE during the placement of the waste. The GCL has been excluded from the assessment as it not represents the weakest barrier material and if the HDPE integrity is maintained, the GCL layer will therefore not be impacted (Refer section 7.2.2).

7.2.4 Geotechnical Material Properties

The interface strength parameters adopted in the FLAC modelling have been based on results of the global stability analysis in Section 6.0, and conservative assumptions based on engineering experience.

Figure 3 presents a schematic representation of the liner system interface friction angles.

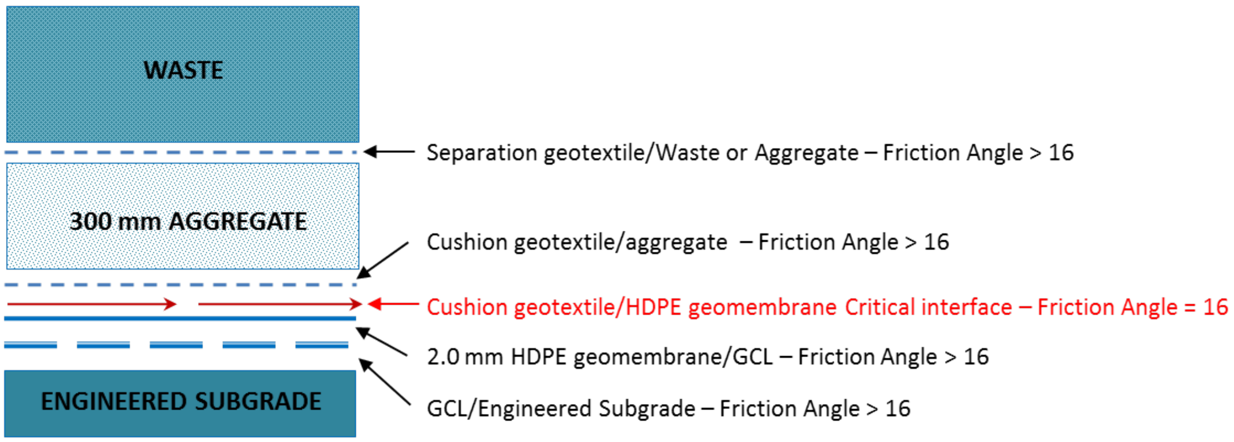

Figure 3: Schematic representation of liner system interface friction angles.

In order to minimise tensile strains on the containment system (HDPE and GCL), it is preferable to select the geosynthetic materials such that the weakest interface is located above the containment system. Therefore, the lowest required interface friction angle (16 degrees) was assumed to be located between the HDPE and the cushion geotextile. Consequently, the interface friction angle above and below the critical interface should be higher than the critical interface (i.e. greater than 16 degrees).

Interfaces were introduced at two locations: above and below the cushion geotextile or HDPE. Table 11 presents the interface properties used in the FLAC model.

Table 11: Interface properties used in the FLAC model

Interface	Friction (°)	Adhesion (kPa)	Normal Stiffness (Pa/m)	Shear Stiffness (Pa/m)
Scenario 1				
Upper: Cushion Geotextile/Aggregate	35	0	1×10^6	1×10^6
Lower: Cushion Geotextile/Textured HDPE	16	0	1×10^9	1×10^9
Scenario 2				
Upper: Cushion Geotextile/Textured HDPE	16	0	1×10^6	1×10^6
Lower: Textured HDPE/GCL (or Sub-base)	18	0	1×10^9	1×10^9

In scenario 1, a friction angle of 35° for the upper interface was assumed based on experience with similar materials. This value is considered conservative as it represents the highest typical interface friction angle and should result in high strains induced on the cushion.

In scenario 2, a friction angle of 18° for the lower interface was assumed. Typically, based on experience with similar materials, peak interface friction angles greater than 20° can be achieved for this interface. The assumed value is considered conservative as it results in high strains induced on the HDPE.

Table 12 presents the mechanical parameters used to simulate the protection geotextile and the HDPE geomembrane in the FLAC model.

Table 12: Geosynthetic materials properties used in the FLAC model

Beam Element	Young's Modulus (Pa) Secant at Yield	Thickness (mm)	Reference
Protection Geotextile	1×10^7	10	Assumed
HDPE Geomembrane	1×10^8	2	Internal database from multi-axial tensile testing

The Young's Modulus adopted in the model for the geosynthetic is the secant modulus at yield (assumed 50% of strain for the geotextile and 20% of strain for the HDPE). This is considered a simplified and conservative assumption used to model systems that could exceed a tensile strain greater than 2% (Reference [17]).

The material properties used for the numerical analyses are presented in Table 13.

Table 13: Material properties used in the FLAC model

Material	Model	Density (kg/m^3)	Friction (°)	Cohesion (kPa)	Young's Modulus (MPa)	Poisson's Ratio (-)
Granite bedrock	Elastic	2700	-	-	3700	0.22
In situ soil	Mohr-Coulomb	1850	28	5	50	0.25
Engineered Fill		1850	28	5	15	0.25
Waste		1000	25	5	0.5	0.30

The waste stiffness values between 2 MPa (high stiffness) to 0.5 MPa (low stiffness) were adopted in the liner integrity assessment based on published values for municipal solid waste (Reference [8]). A low stiffness waste parameter has been conservatively adopted in our assessment as this will induce more settlement and, consequently, a greater propensity to strain the liner.

The FLAC internal database was used to identify properties typical of the granite bedrock. The properties assigned to the engineered fill and *in situ* clayey material are based on the isotropically consolidated undrained triaxial tests (CIU) undertaken on samples of remoulded material and interpretation of the confining modulus from CPTs (Reference [2]), respectively. Based on interpretation of the CIU undertaken on the remoulded clayey material, a young's modulus of 15 MPa was estimated for this material. Based on interpretation of the confining modulus of the *in situ* clayey material, its young's modulus could significantly vary, between 50 MPa and more than 150 MPa. A Young's Modulus of 50 MPa (low range) was used in the liner integrity assessment, which should represent a conservative assumption.

7.3 Results of the Liner Integrity Assessment

7.3.1 Liner Integrity pre-waste placement

The results of the liner integrity assessment prior to waste placement are summarised in Table 14. Based on the estimated tensile stresses for the liner materials the integrity of the proposed lining system is not compromised prior to waste placement.

Table 14: Summary of Results for the Liner Integrity Assessment Pre-Waste Placement

Property	Unit	Geosynthetic Liner System Material		
		GCL	HDPE	Cushion Geotextile
Thickness	mm	Not required	2.0	Not required
Density	kg/m³		940	
Mass per unit area	g/m²	4 500	1 880	1000
Tensile strength	kN/m	8	29(*)	36(**)
Height	m	7	7	7
Slope angle	V:H	1:3	1:3	1:3
Length	m	22	22	22
Assumed interface peak friction angle between geosynthetic and underlying material	degrees	18	18	16
Tensile Stress	N/m	2	1	3
Factor of Safety (Tensile Strength/Tensile Stress)	**-**	**3 250**	**28 150**	**11 900**

Notes: *Yield tensile strength of 2.0 mm at 12% yield elongation HDPE based on GRI-GM13

**Tensile strength of 1000 g/m² cushion geotextile based on GRI-GT12b

In Table 14, the assumed peak interface friction angle for the cushion geotextile against the HDPE geomembrane is based on the slope stability assessment (Refer to Section 6.4.4). The interface friction angles assumed for the underlying materials to the GCL and HDPE geomembrane are conservative (the lower the underlying interface friction angle, the higher the stress transferred to the geosynthetic). The proposed HDPE geomembrane is double textured. Therefore, the interface friction angle between the HDPE and GCL is expected to be higher than18°. Typically the interface friction angle between the GCL and a clayey subgrade would also be higher than the assumed value of 18°.

7.3.2 Liner Integrity during operation and post-waste placement

Table 15 presents the outcome of the liner integrity assessment during operation and post-waste placement. The outcomes of the numerical modelling are illustrated in the figures presented in Appendix B.

Table 15: Integrity liner assessment FLAC model outcomes

Scenario	Maximum Tensile Strain (%)	Maximum Tensile Stress (kN/m)	Commentary
1 – Cushion geotextile	~1.4	~1.4	Maximum strain occurs at the division bund side slope during filling of Cell 1
2 – HDPE geomembrane	~0.2	~0.5	

Based on the outcomes of the FLAC modelling, the estimated maximum strain on the geomembrane during waste placement and post-waste deposition is well below its maximum acceptable value of 4% (refer to Table 10 presented in Section 7.2.2). The strain of approximately 1.4% estimated by the numerical model for the protection geotextile during waste deposition is also well below its assumed allowable strain value of 25% and is therefore considered acceptable. .

8.0 OUTCOME AND RECOMMENDATIONS

Considering the results of the stability analyses and veneer stability assessment for the Allawuna Landfill, the following conclusions and recommendations were made:

- The stability analyses undertaken for the basal liner system interface has shown acceptable factors of safety for scenarios 1 to 5 of the landfill design. Although the result for Scenario 6 was lower than the minimum FoS, the estimated permanent deformation due to earthquake action (MCE) was well below acceptable values (Refer to Section 6.4.3). For both cross-sections analysed, the critical failure mode for the stability of the waste landforms is non-circular (i.e. sliding) along the proposed basal liner system. This is the case for both short-term and long-term, static conditions and under earthquake loading. Consequently, the stability is dependent on the shear strength of the critical interface within the proposed basal liner system. Based on the stability analyses, the minimum friction angle of this liner component at the side slopes and base of the landfill shall be no less than 16 degrees. This friction angle should be achieved using a double textured HDPE instead of single textured.
- Laboratory testing (i.e. interface shear and internal shear strength testing of each liner component) should be performed to determine the critical interface between the liner components and ensure the liner shear strength parameters are satisfied as part of the geosynthetic material selection process.

  The critical interface of the basal liner system should be located between the upper side of the HDPE geomembrane and the lower side of the cushion geotextile (Refer to Figure 3). This can be achieved by using liner components with friction angles greater than 16 degrees and HDPE with slightly different textures on each side (i.e. greater friction angle on the bottom side of the HDPE).
- The stability analyses undertaken for the waste landform has shown acceptable factors of safety for scenarios 1 to 5 of the landfill design. Although the result for Scenario 6 was lower than the minimum FoS, the estimated permanent deformation due to earthquake action (MCE) was well below acceptable values (Refer to Section 6.4.3).
- Waste slopes should not be steeper than 1V:3H for the operational landform, and not steeper than 1V:5H for the final landform (post closure).
- The stability analyses undertaken for the foundation and embankment has shown acceptable factors of safety for the landfill design prior to and post waste deposition. Embankment slopes should not be steeper than 1V:2H for short term conditions (embankments that may be present for less than 20 years; i.e. internal embankments and cell division bunds), and not steeper than 1V:3H for long term conditions (embankments that may be present for 20 years or more; i.e. external perimeter bunds).
- Laboratory testing of proposed materials should be undertaken prior to construction of the capping system to verify the veneer stability assessment presented herein with project-specific shear strength parameters.
- The HDPE and LLDPE geomembranes should be textured on both sides in order to achieve the required post-peak friction angle at both interfaces. Texturing options vary according to manufacturer and independent testing of each candidate specimen should be undertaken to verify that the required increase in frictional properties will actually be attained.
- The overall slope length exceeds the average length of a geosynthetic roll. Best practice for lining on slopes typically prevents the use of cross-slope joins. In view of that, intermediate anchoring of geosynthetic panels should be provided.
- Placement of the cap soil should be carried out from the bottom of the slope upwards, retaining a large buttress at the toe to reduce the likelihood of slippage during construction. Construction vehicles should exert a low-ground pressure (e.g. tracked) and an assessment of the operational stability of the slope during construction should be undertaken once the proposed materials and equipment are confirmed.

The basal liner system integrity assessment undertaken for Allawuna Landfill prior to waste placement, during waste deposition (operations) and post-waste placement indicates the following:

- The integrity of the lining system during waste placement is satisfactory
- The settlement of the subgrade and embankment fill due to the loading imposed by the waste will not detrimentally impact the integrity of the lining system, and
- The post-waste deposition settlement will not affect the integrity of the lining system.

To support the preceding slope stability analyses and liner integrity assessment, laboratory analysis should be carried out for the following interfaces.

- Aggregate layer vs cushion geotextile
- Cushion geotextile vs textured HDPE
- Texture HDPE vs GCL
- GCL vs engineered fill.

9.0 LIMITATIONS

Your attention is drawn to the document “Limitations”, which is included as Appendix C to this report. This document is intended to assist you in ensuring that your expectations of this report are realistic, and that you understand the inherent limitations of a report of this nature. If you are uncertain as to whether this report is appropriate for any particular purpose please discuss this issue with us.

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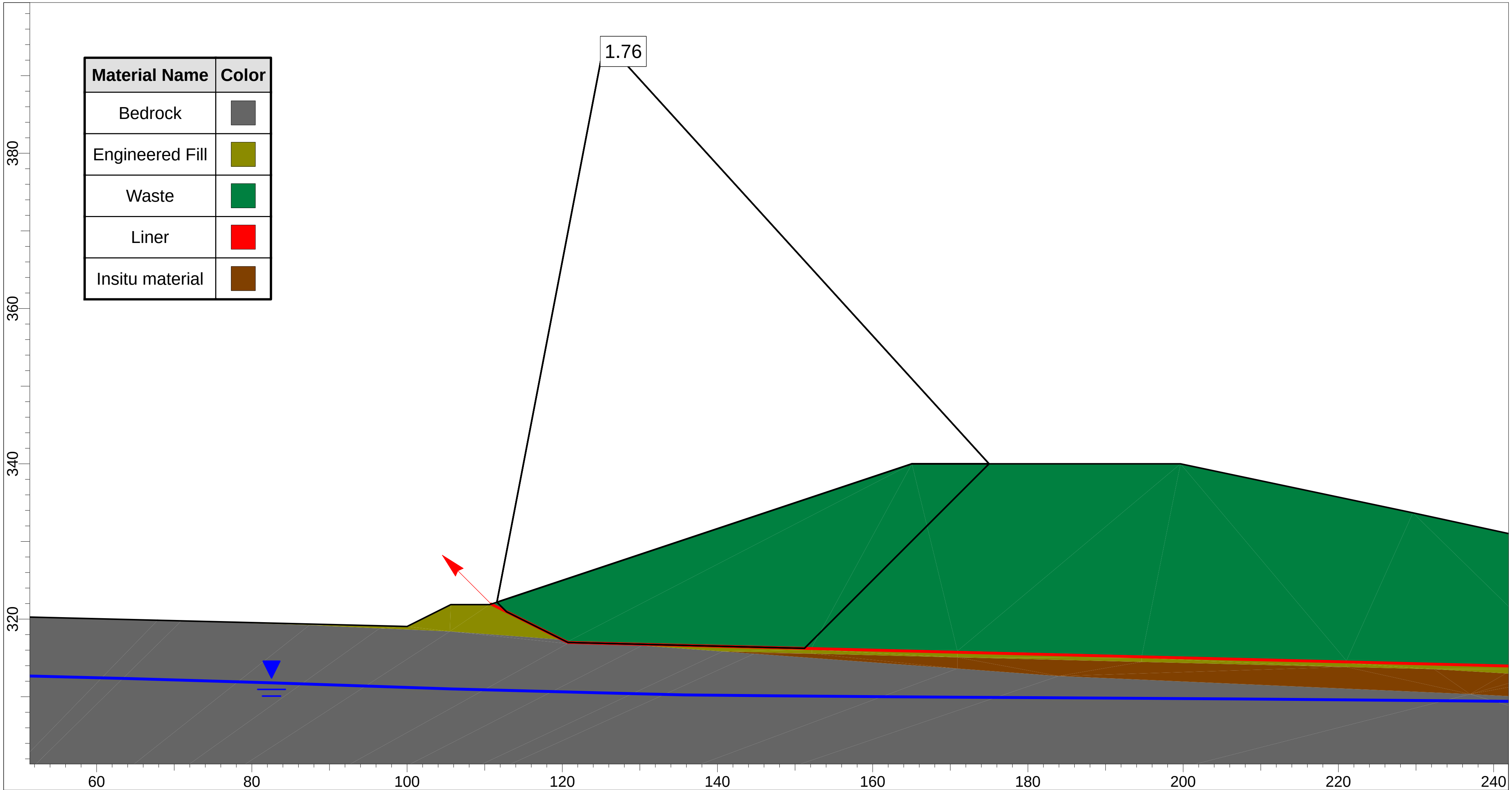
APPENDIX A

Global Stability Analyses Figures



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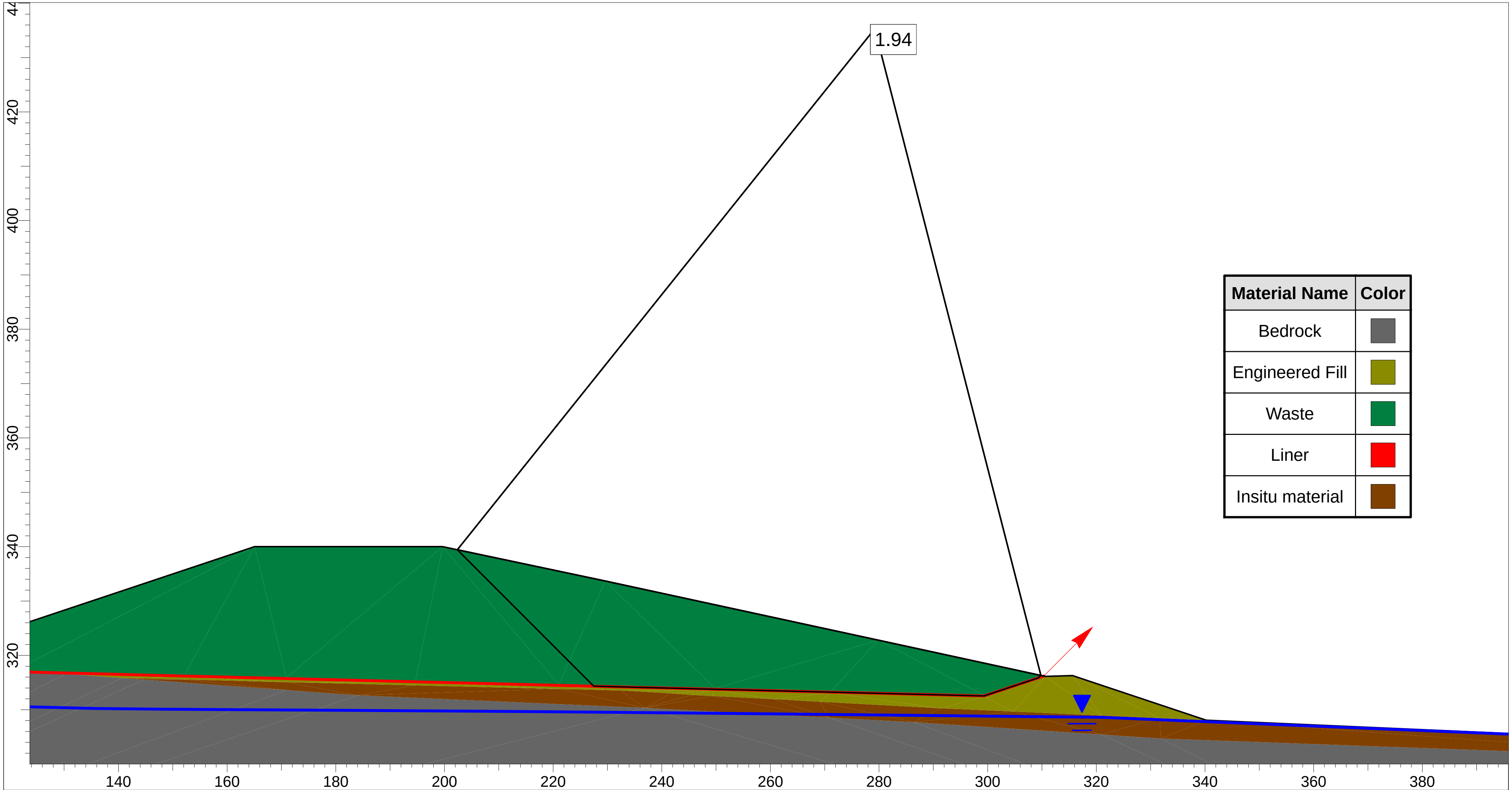
ALLAWUNA FARM LANDFILL

STABILITY ANALYSIS

LINER INTERFACE MINIMUM FACTOR OF SAFETY
OPERATIONAL, STATIC, LOW PORE PRESSURE
SECTION C

SCALE 1:700 REVISION 0 **FIGURE A1**



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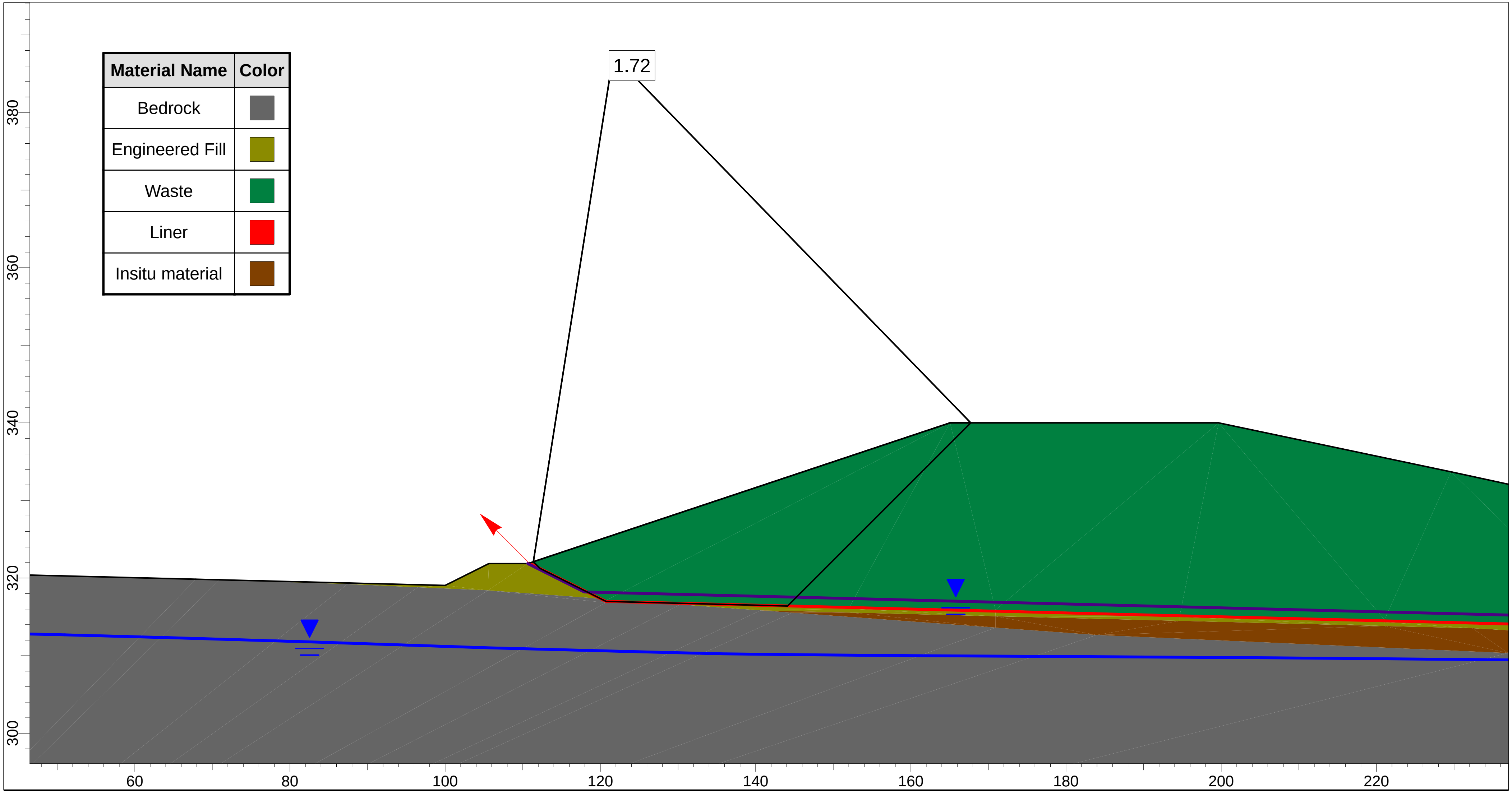
LINER INTERFACE MINIMUM FACTOR OF SAFETY
OPERATIONAL, STATIC, LOW PORE PRESSURE
SECTION C

SCALE 1:1000 REVISION 0 **FIGURE A2**



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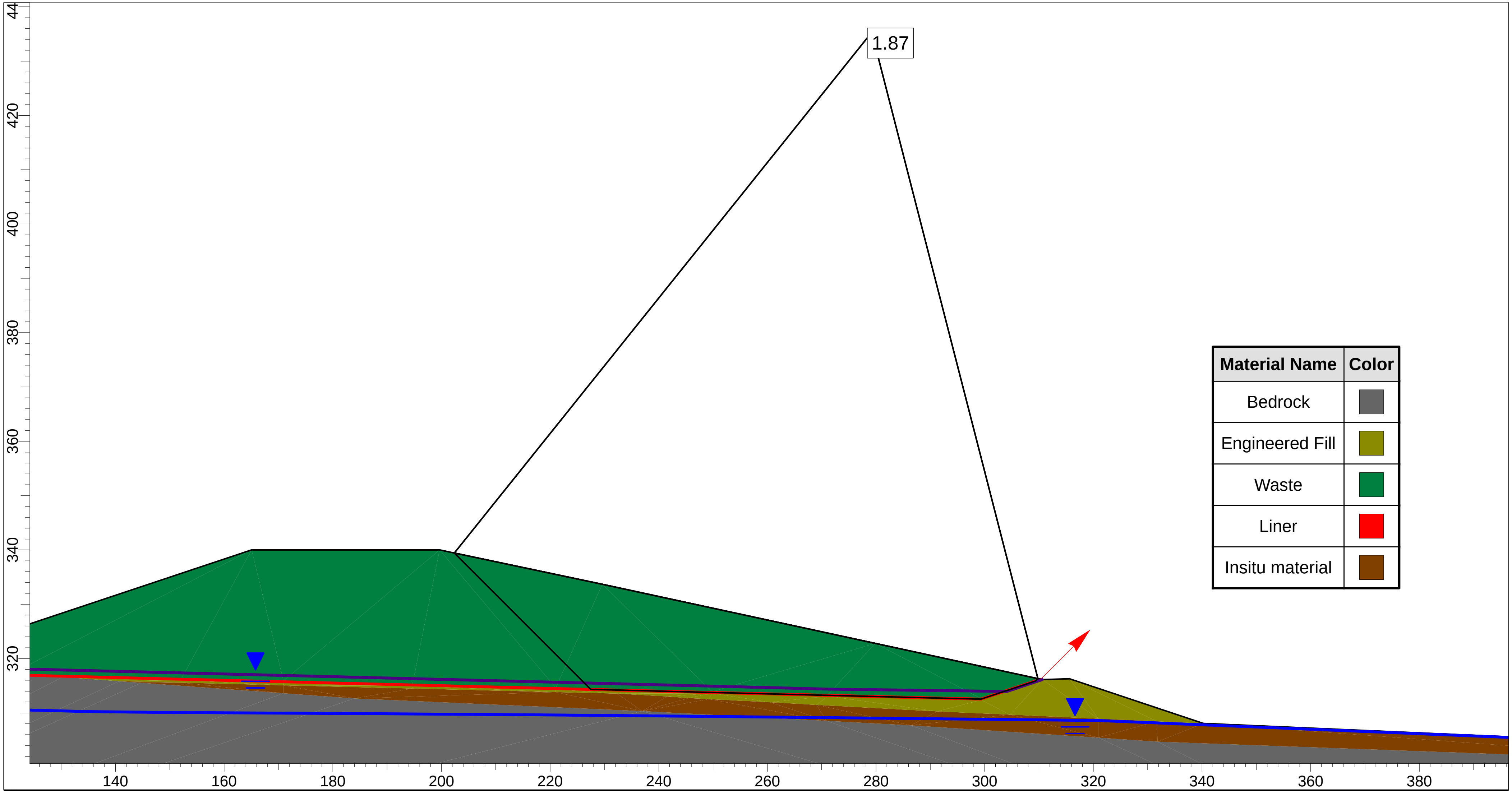
LINER INTERFACE MINIMUM FACTOR OF SAFETY
OPERATIONAL, STATIC, HIGH PORE PRESSURE
SECTION C

SCALE 1:700 REVISION 0 **FIGURE A3**



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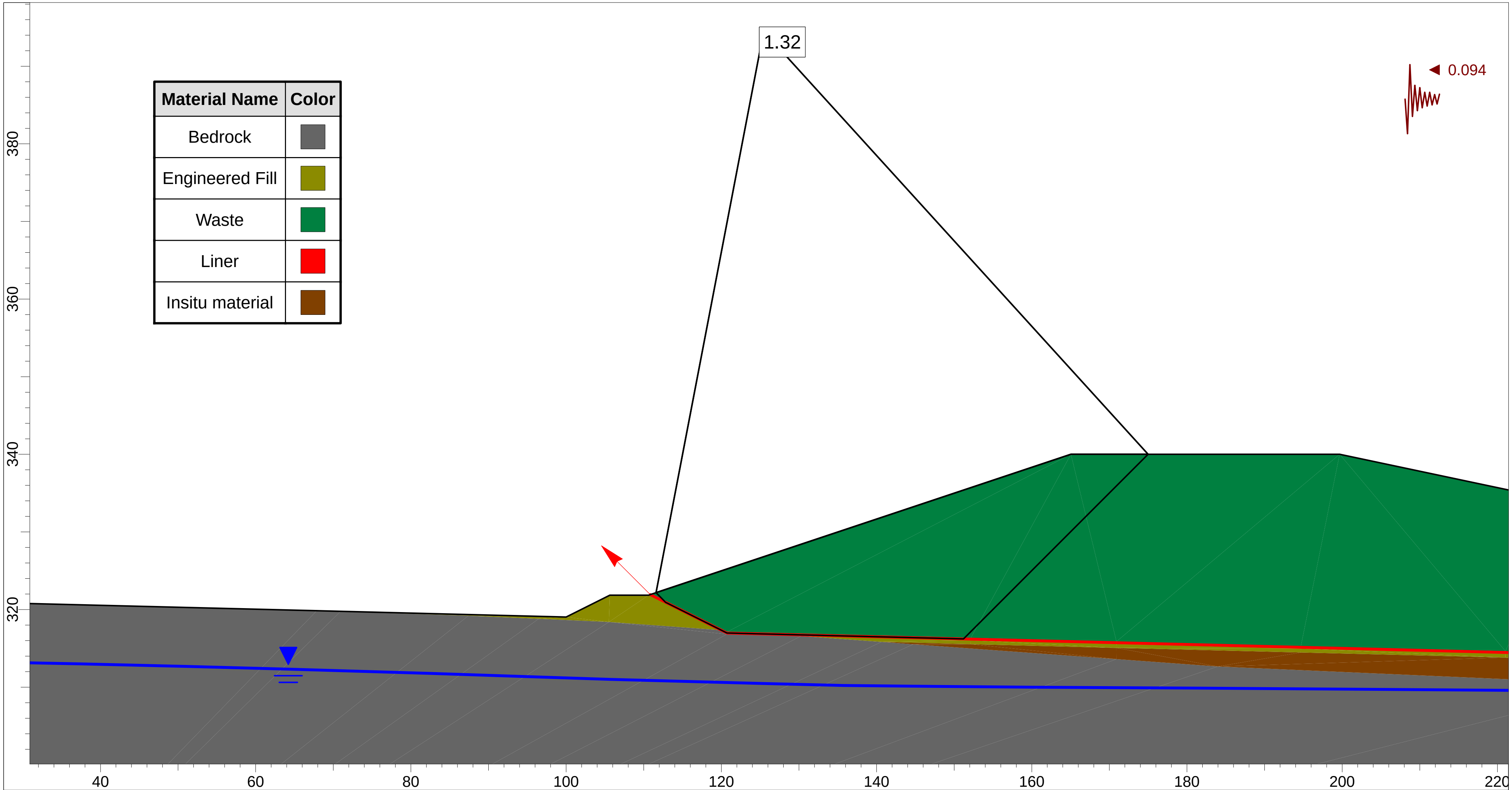
LINER INTERFACE MINIMUM FACTOR OF SAFETY
OPERATIONAL, STATIC, HIGH PORE PRESSURE
SECTION C

SCALE 1:1000 REVISION 0 **FIGURE A4**



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STABILITY ANALYSIS

LINER INTERFACE MINIMUM FACTOR OF SAFETY
OPERATIONAL, PSEUDO-STATIC, OBE
SECTION C

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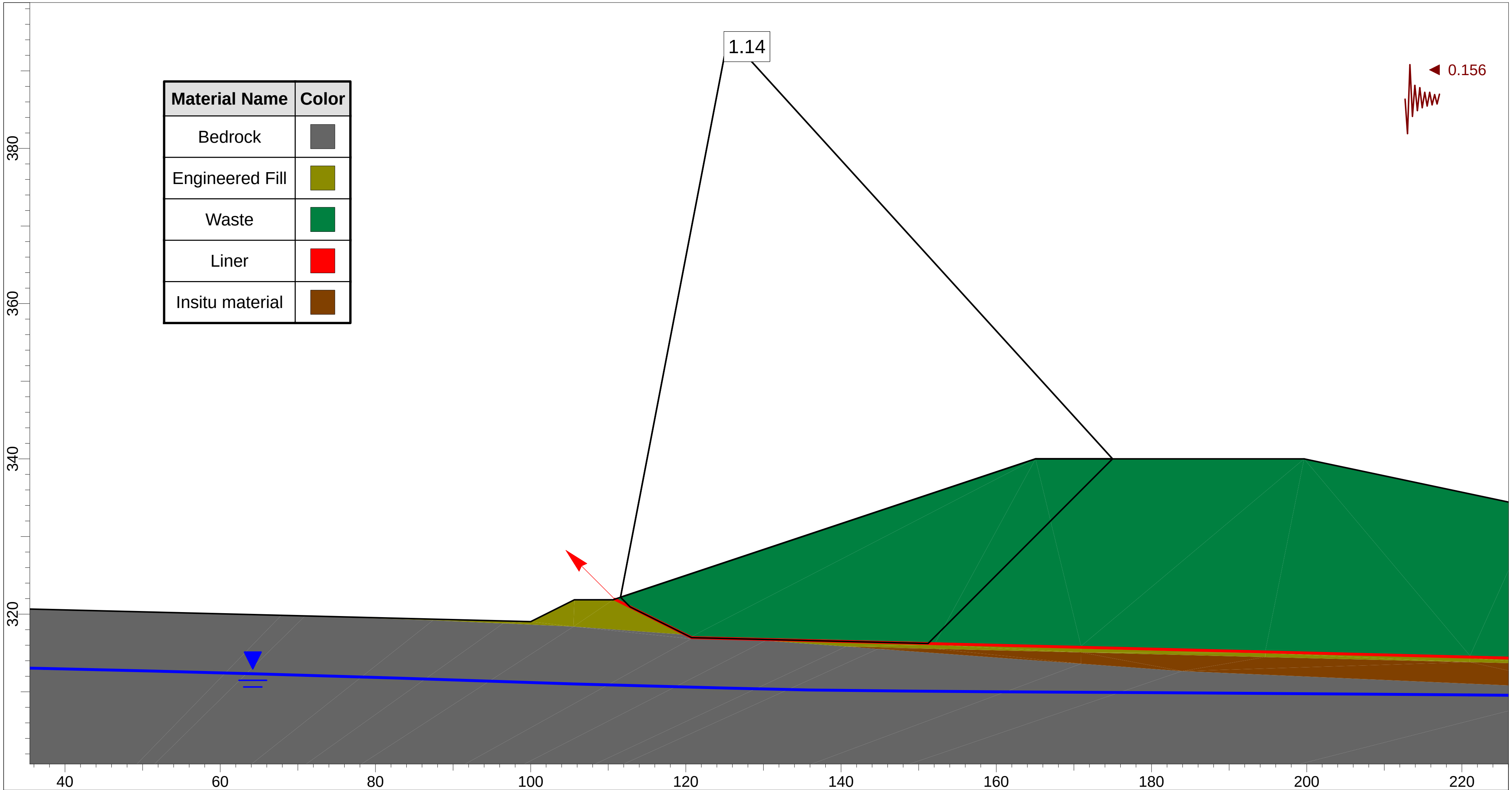
SCALE 1:700 REVISION 0 **FIGURE A5**



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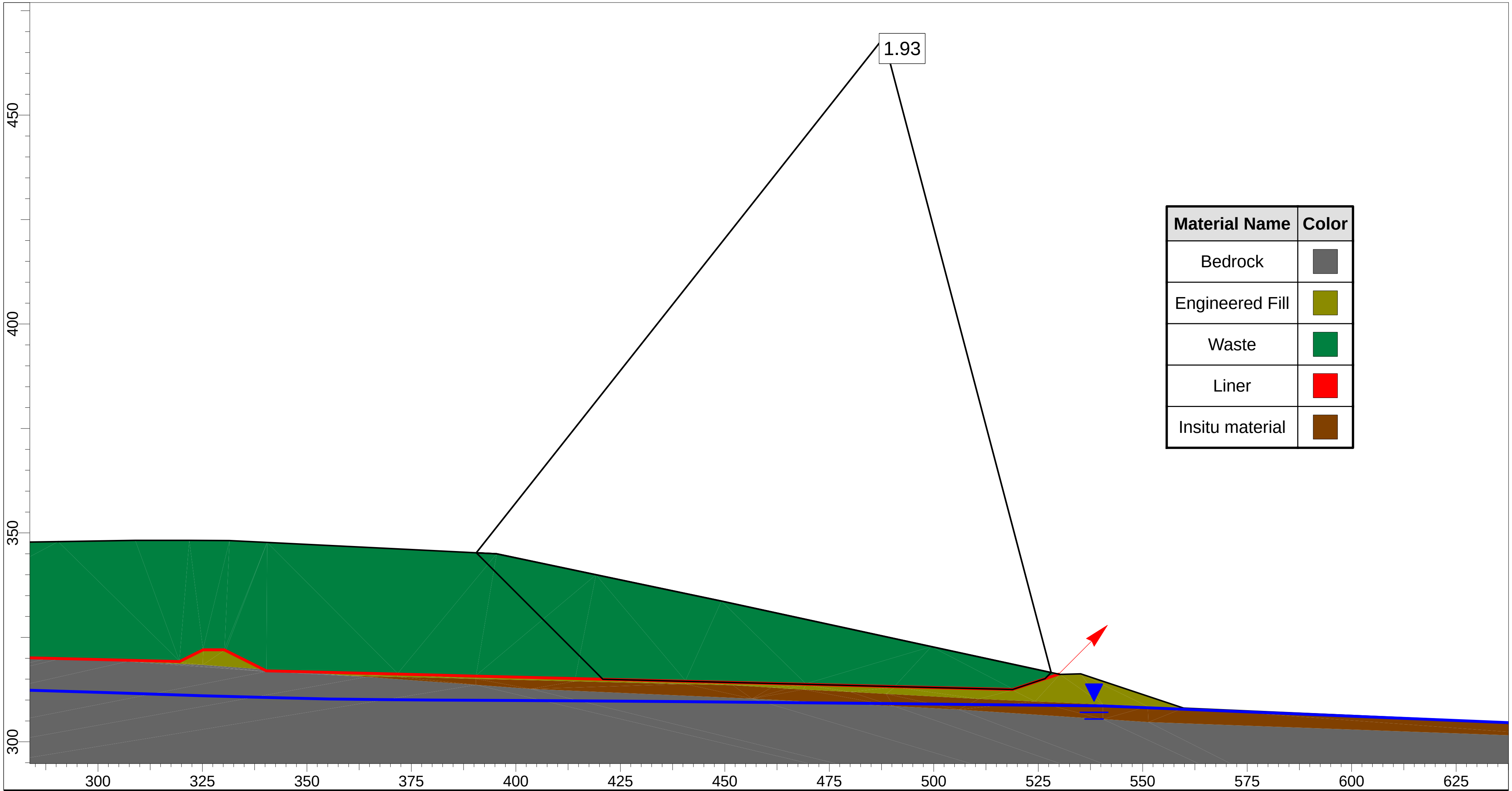
LINER INTERFACE MINIMUM FACTOR OF SAFETY
OPERATIONAL, PSEUDO-STATIC, MDE
SECTION C

SCALE 1:700 REVISION 0 **FIGURE A6**



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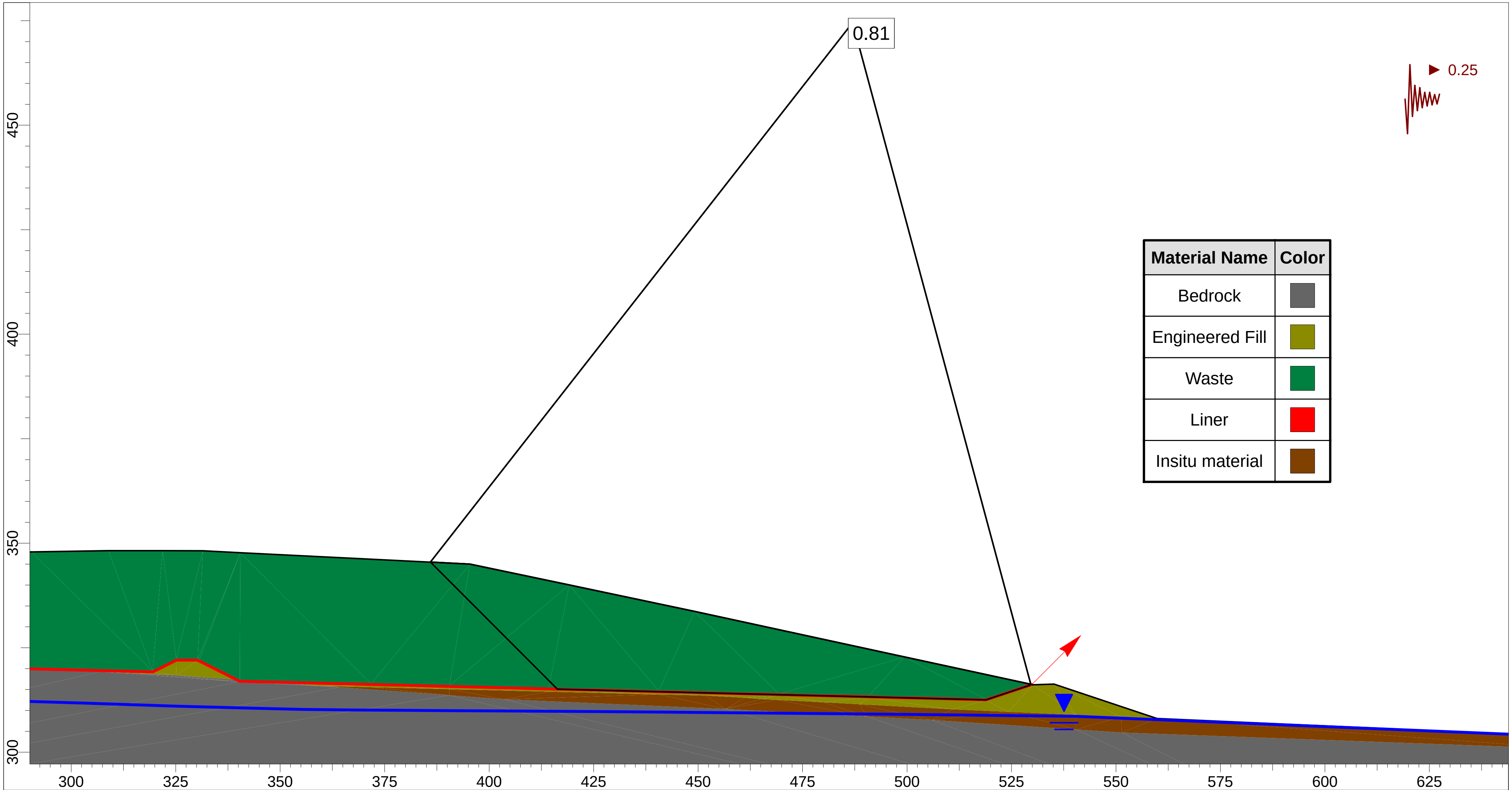
STABILITY ANALYSIS

LINER INTERFACE MINIMUM FACTOR OF SAFETY
POST CLOSURE, STATIC, LOW PORE PRESSURE
SECTION C

SCALE 1:1300 REVISION 0 **FIGURE A7**



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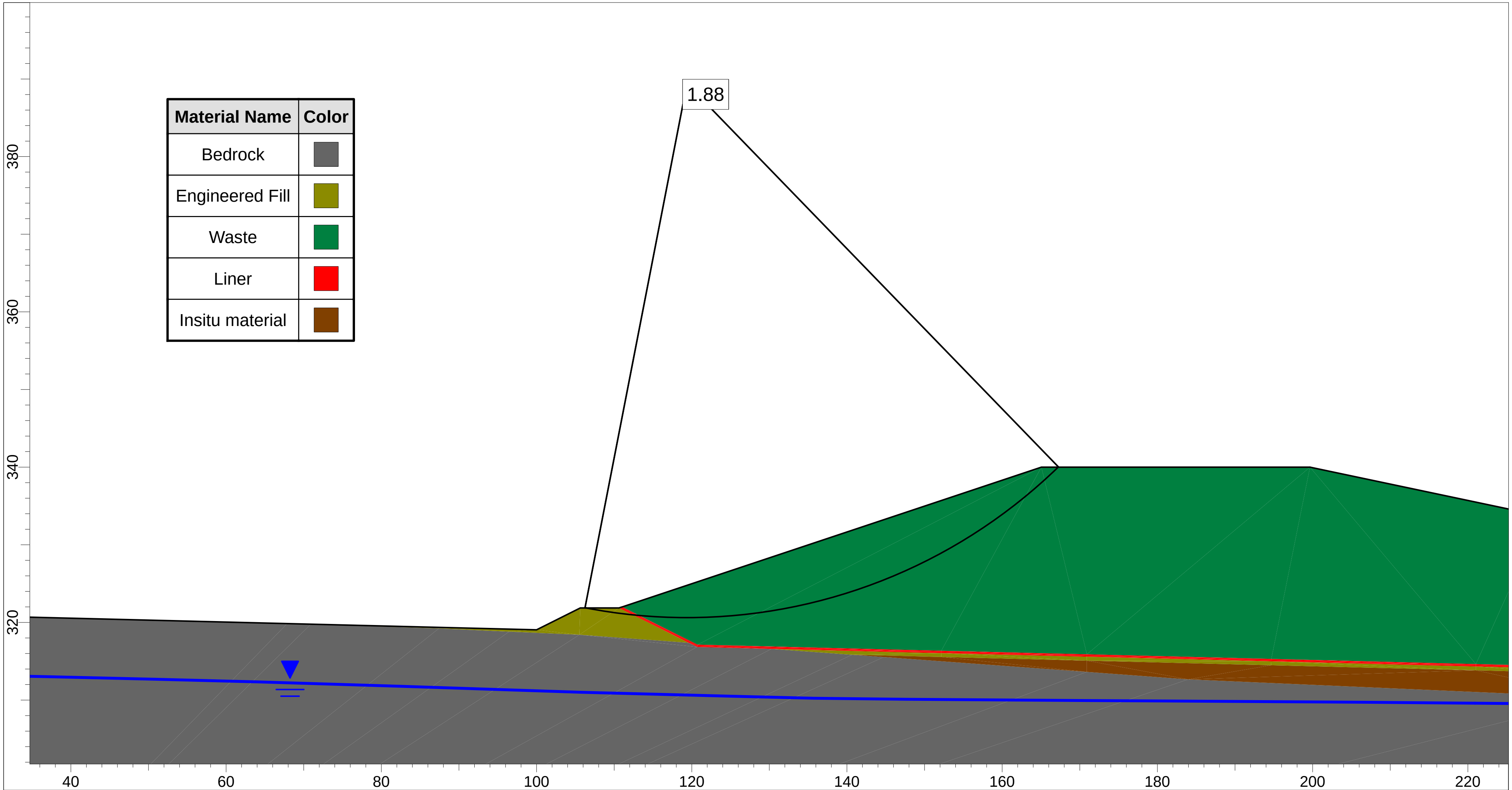
STABILITY ANALYSIS

LINER INTERFACE MINIMUM FACTOR OF SAFETY
POST CLOSURE, PSEUDO-STATIC, MCE
SECTION C

SCALE 1:1300 REVISION 0 **FIGURE A8**



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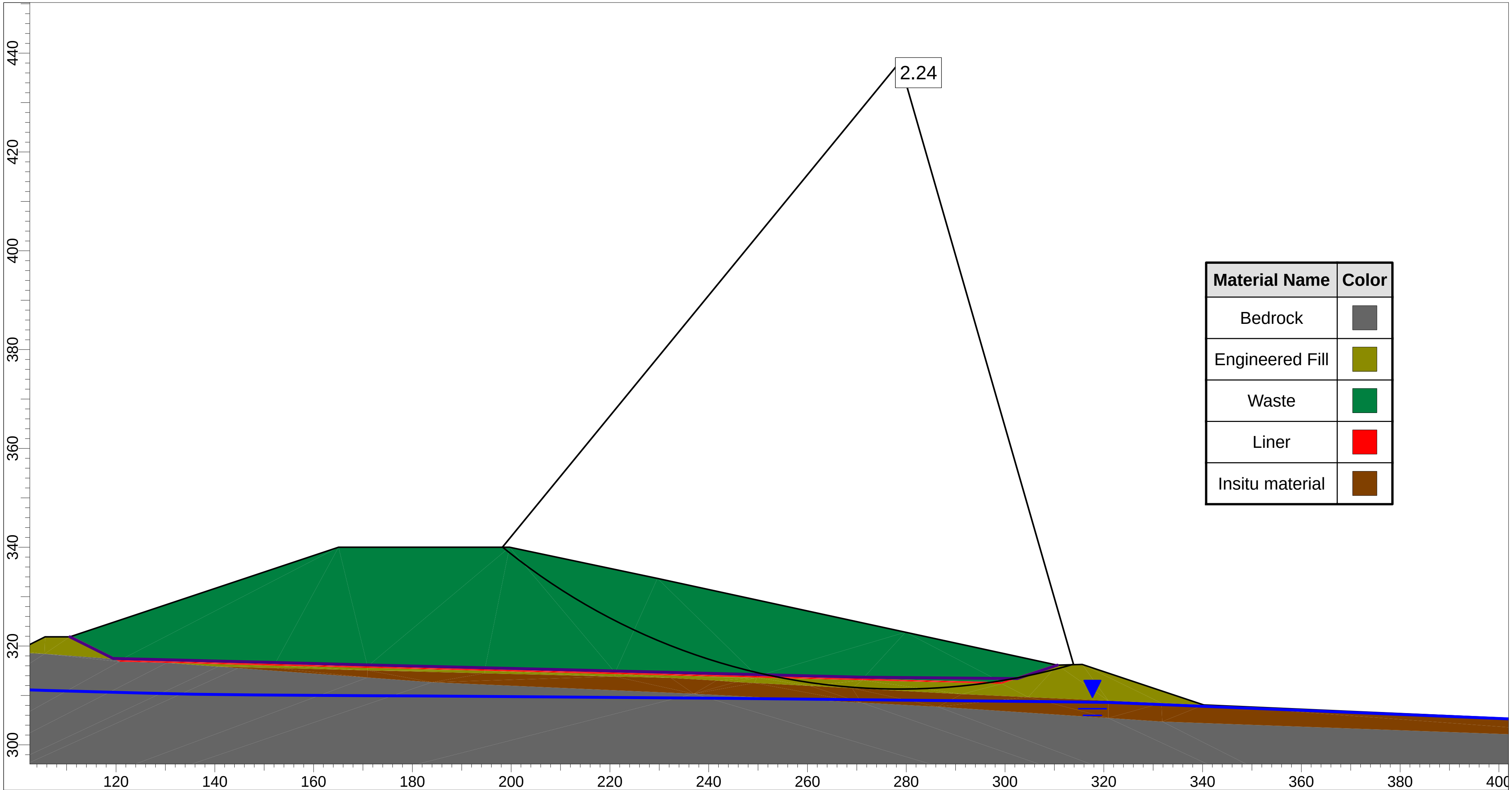
STABILITY ANALYSIS

WASTE SLOPE MINIMUM FACTOR OF SAFETY
OPERATIONAL, STATIC, LOW PORE PRESSURE
SECTION C

SCALE 1:700 REVISION 0 **FIGURE B1**



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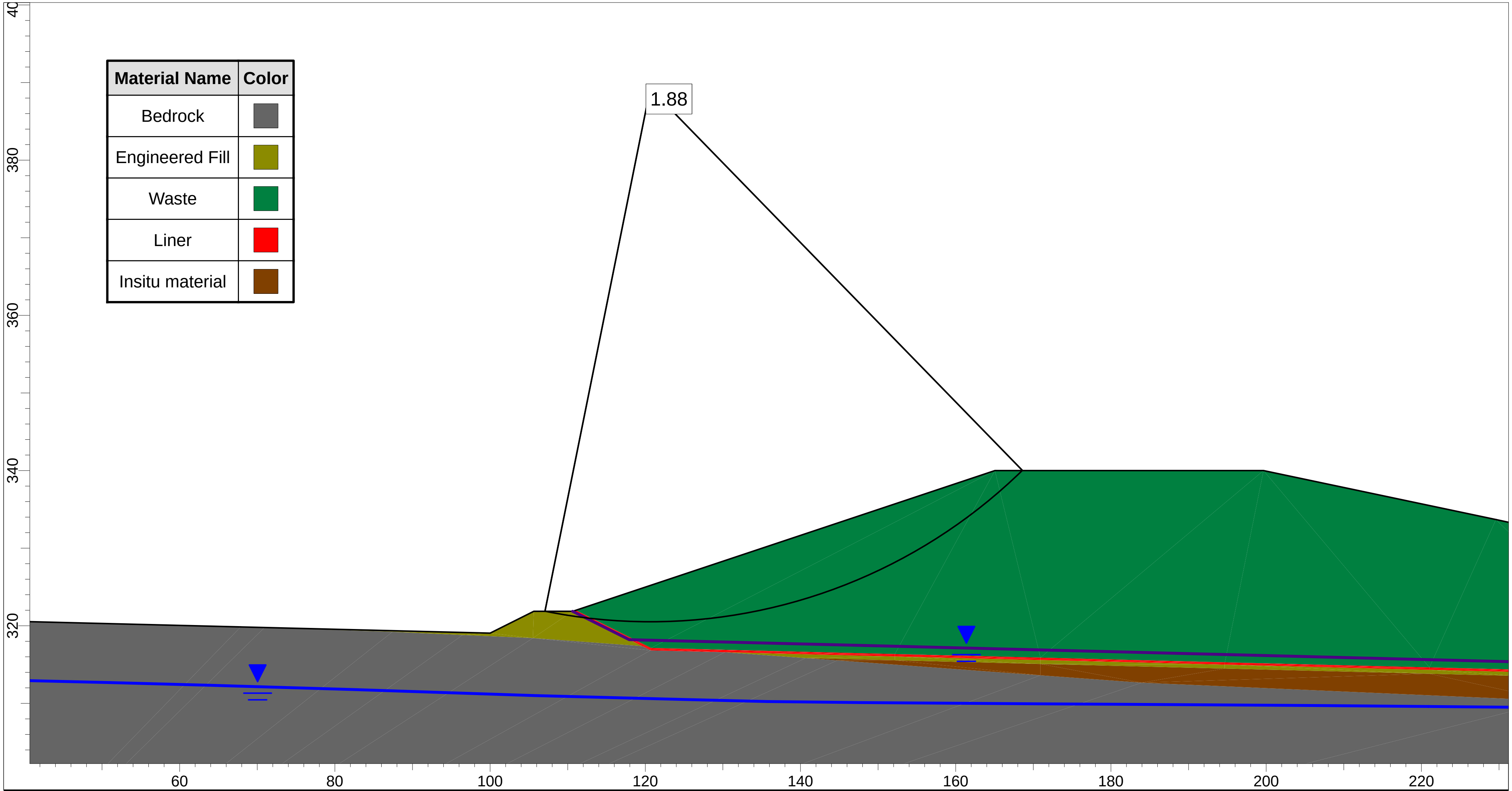
WASTE SLOPE MINIMUM FACTOR OF SAFETY
OPERATIONAL, STATIC, LOW PORE PRESSURE
SECTION C

SCALE 1:1100 REVISION 0 **FIGURE B2**



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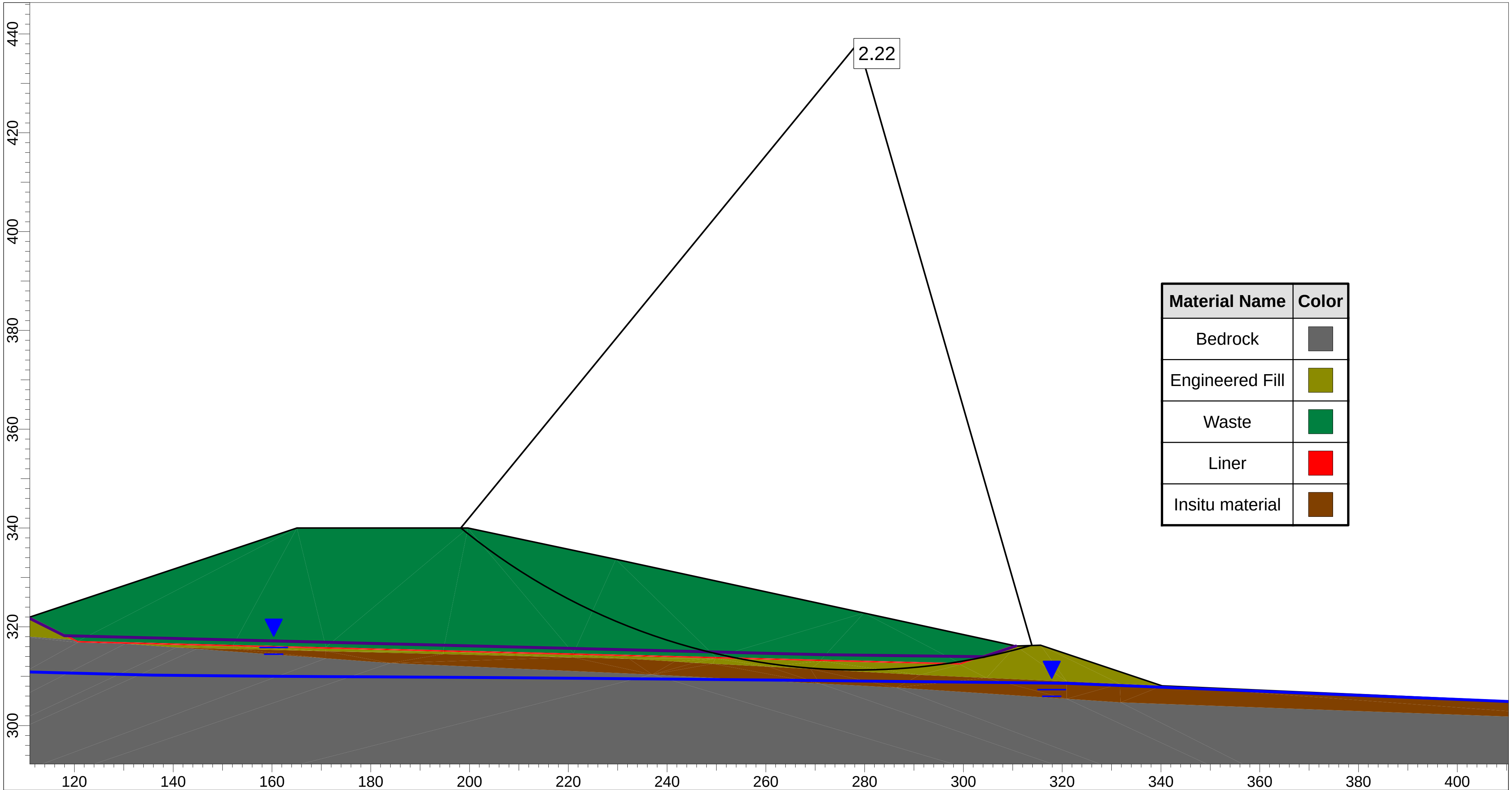
WASTE SLOPE MINIMUM FACTOR OF SAFETY
OPERATIONAL, STATIC, HIGH PORE PRESSURE
SECTION C

SCALE 1:700 REVISION 0 **FIGURE B3**



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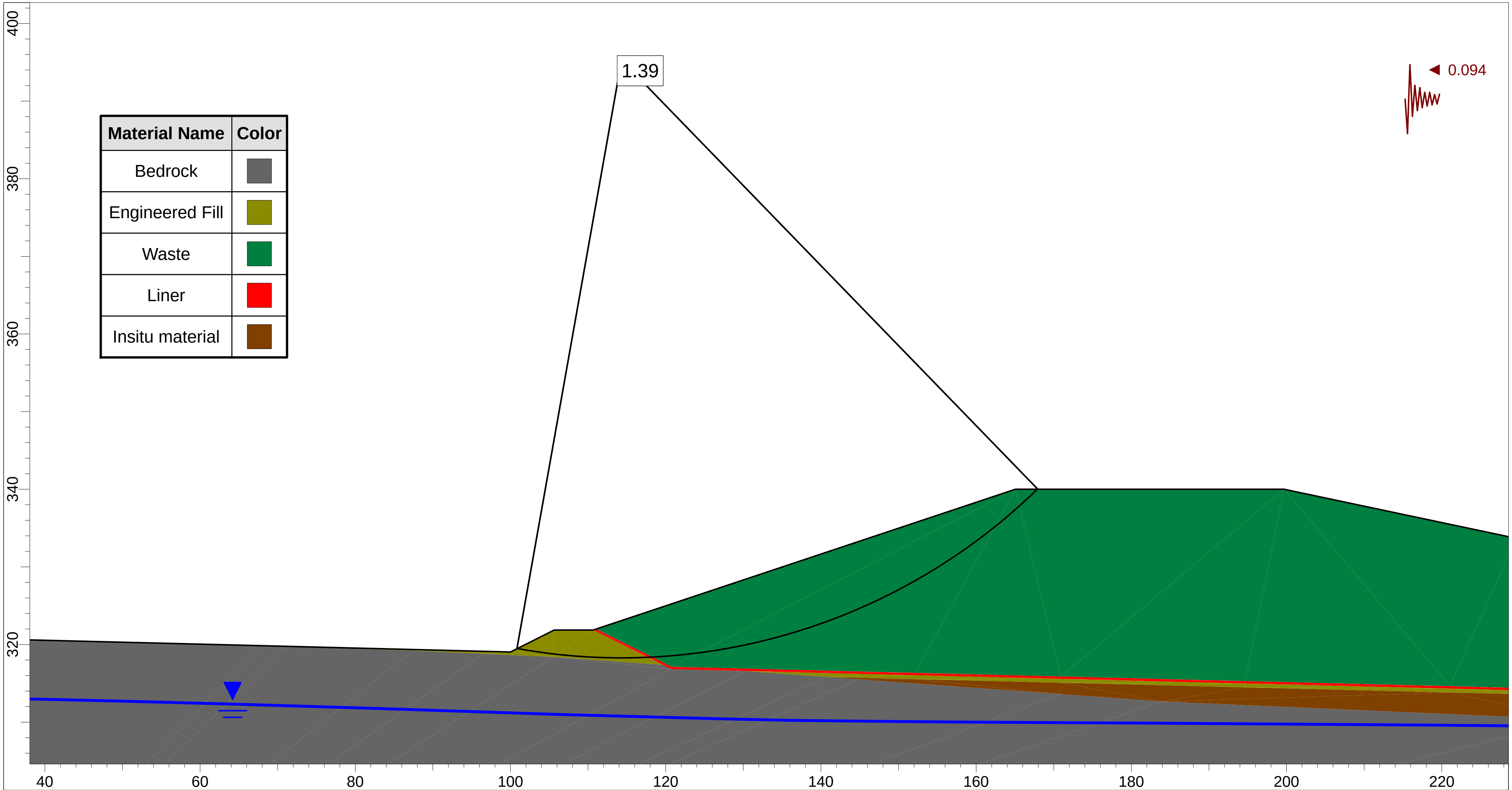
WASTE SLOPE MINIMUM FACTOR OF SAFETY
OPERATIONAL, STATIC, HIGH PORE PRESSURE
SECTION C

SCALE 1:1100 REVISION 0 **FIGURE B4**



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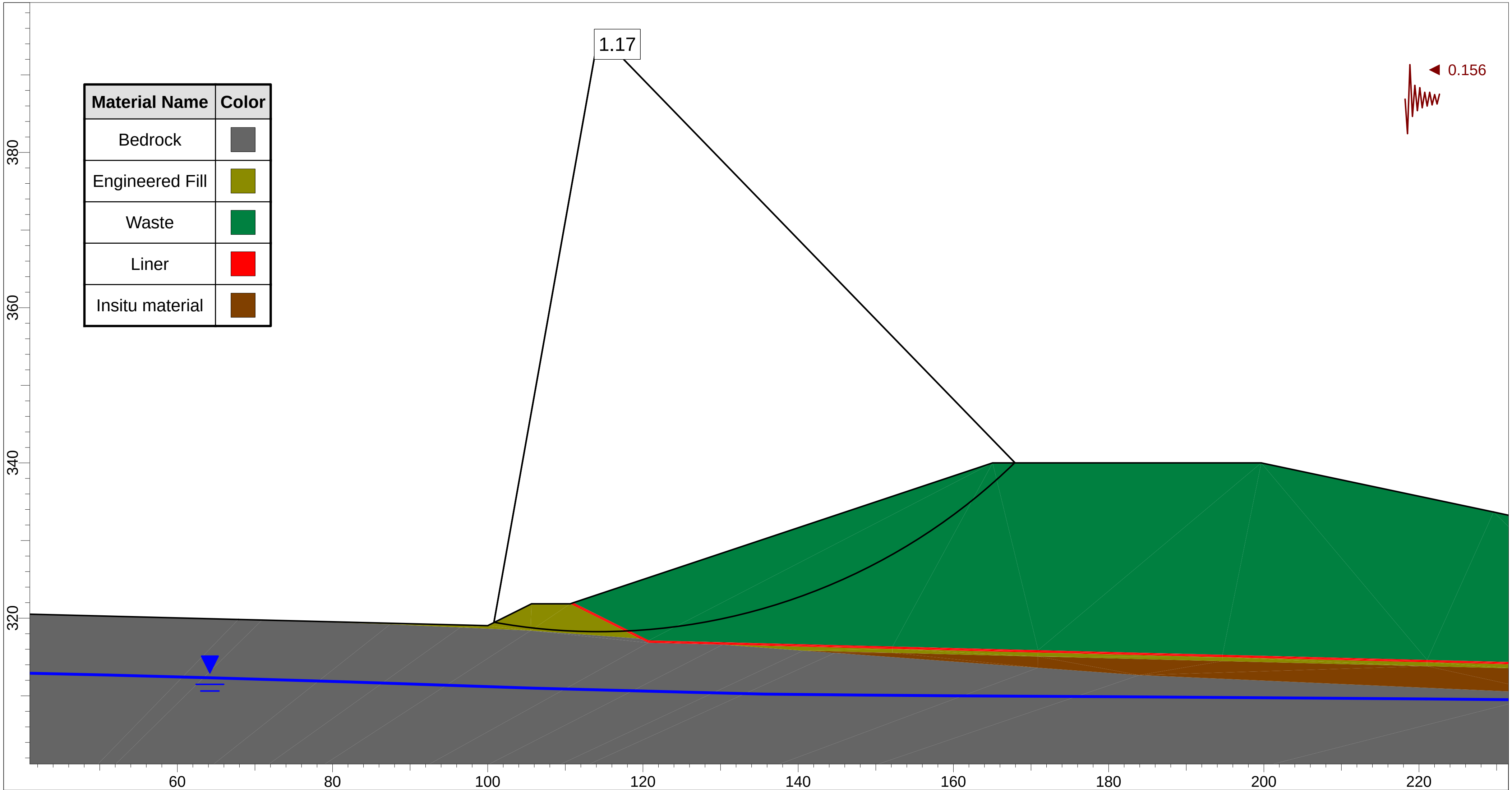
STABILITY ANALYSIS

WASTE SLOPE MINIMUM FACTOR OF SAFETY
OPERATIONAL, PSEUDO-STATIC, OBE
SECTION C

SCALE 1:700 REVISION 0 **FIGURE B5**



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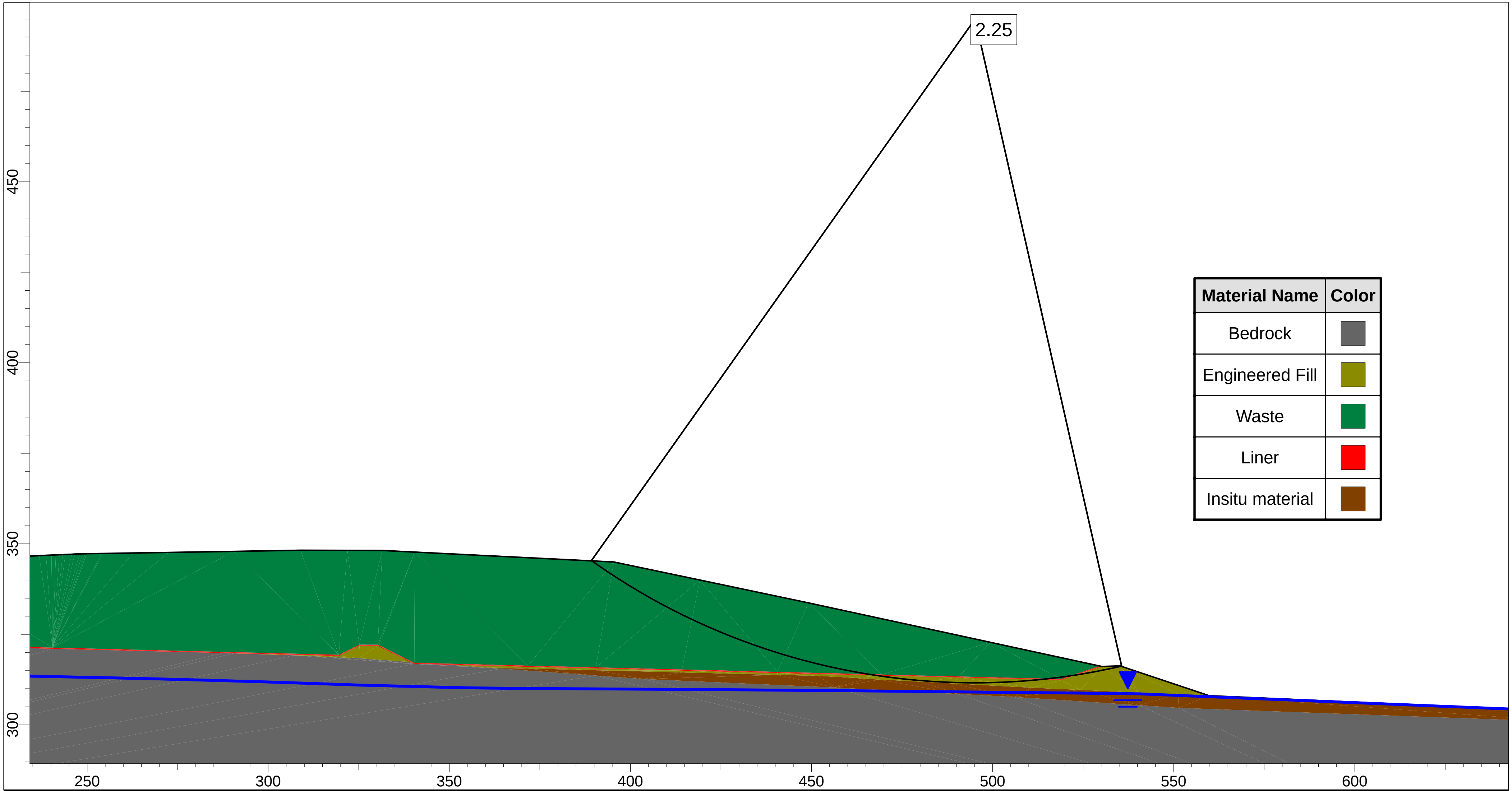
STABILITY ANALYSIS

WASTE SLOPE MINIMUM FACTOR OF SAFETY
OPERATIONAL, PSEUDO-STATIC, MDE
SECTION C

SCALE 1:700 REVISION 0 **FIGURE B6**



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STABILITY ANALYSIS

WASTE SLOPE MINIMUM FACTOR OF SAFETY
POST CLOSURE, STATIC, LOW PORE PRESSURE
SECTION C

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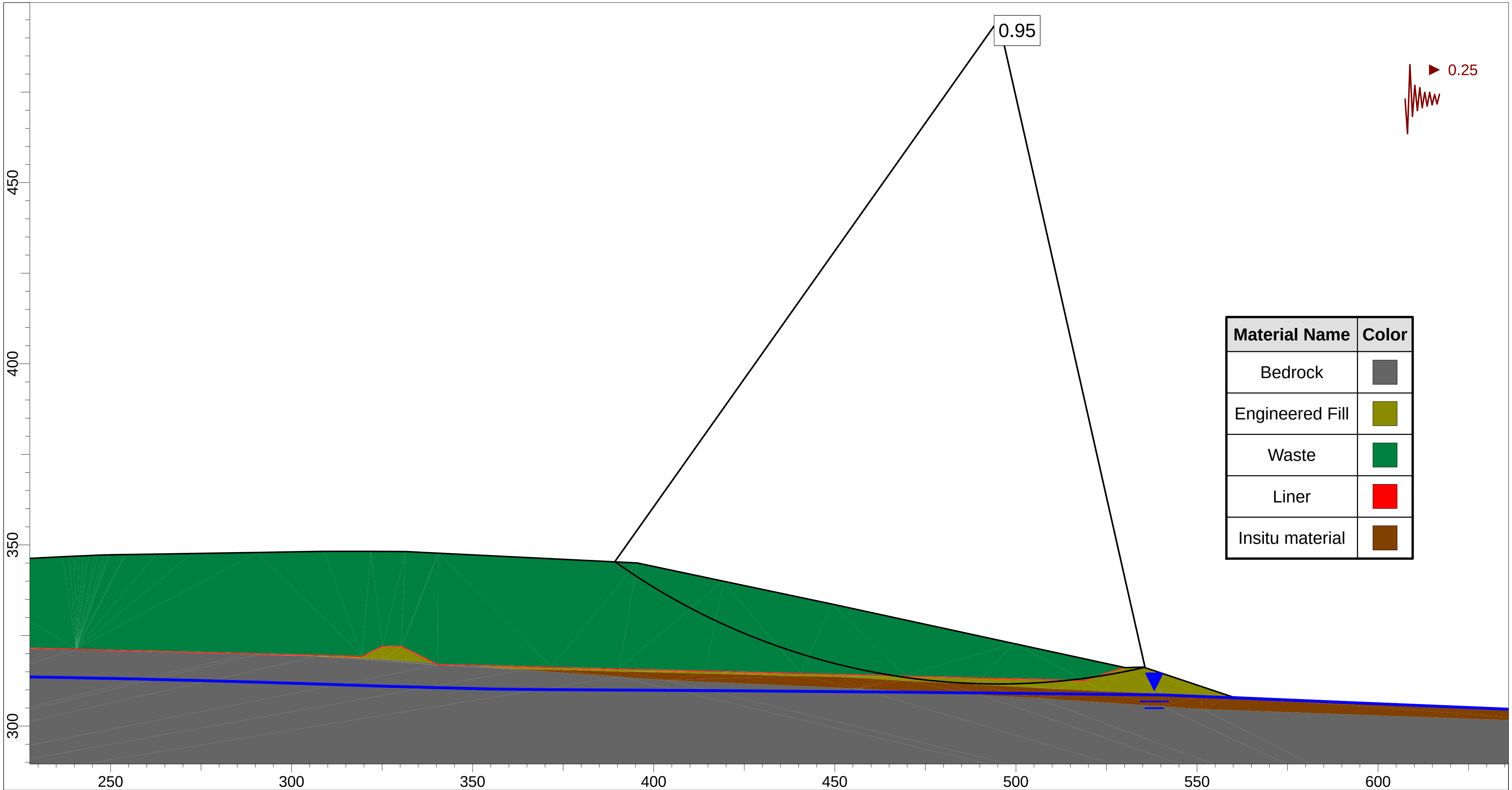
SCALE 1:1500 REVISION 0 **FIGURE B7**



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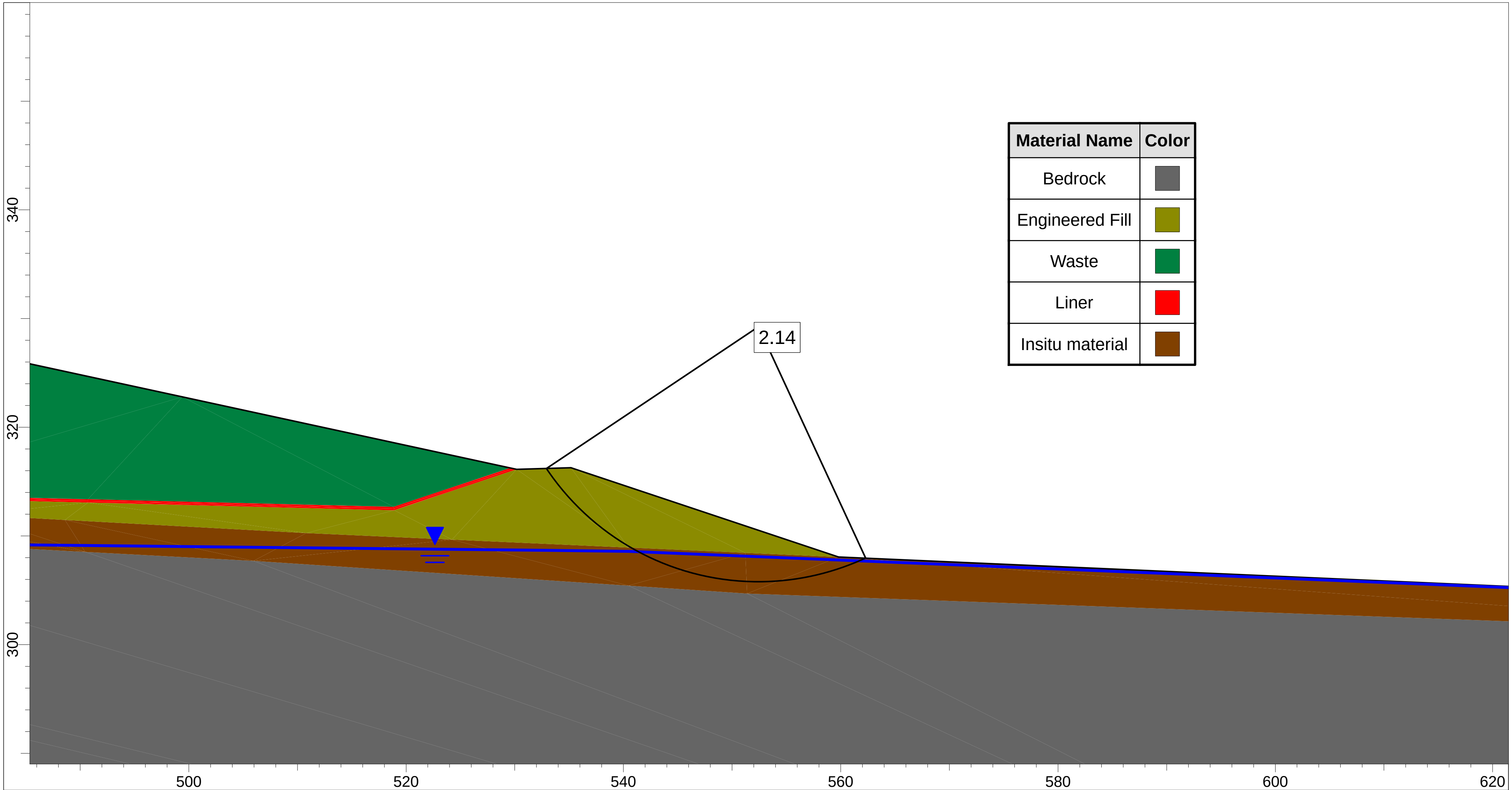
STABILITY ANALYSIS

WASTE SLOPE MINIMUM FACTOR OF SAFETY
POST CLOSURE, PSEUDO-STATIC, MCE
SECTION C

SCALE 1:1500 REVISION 0 **FIGURE B8**



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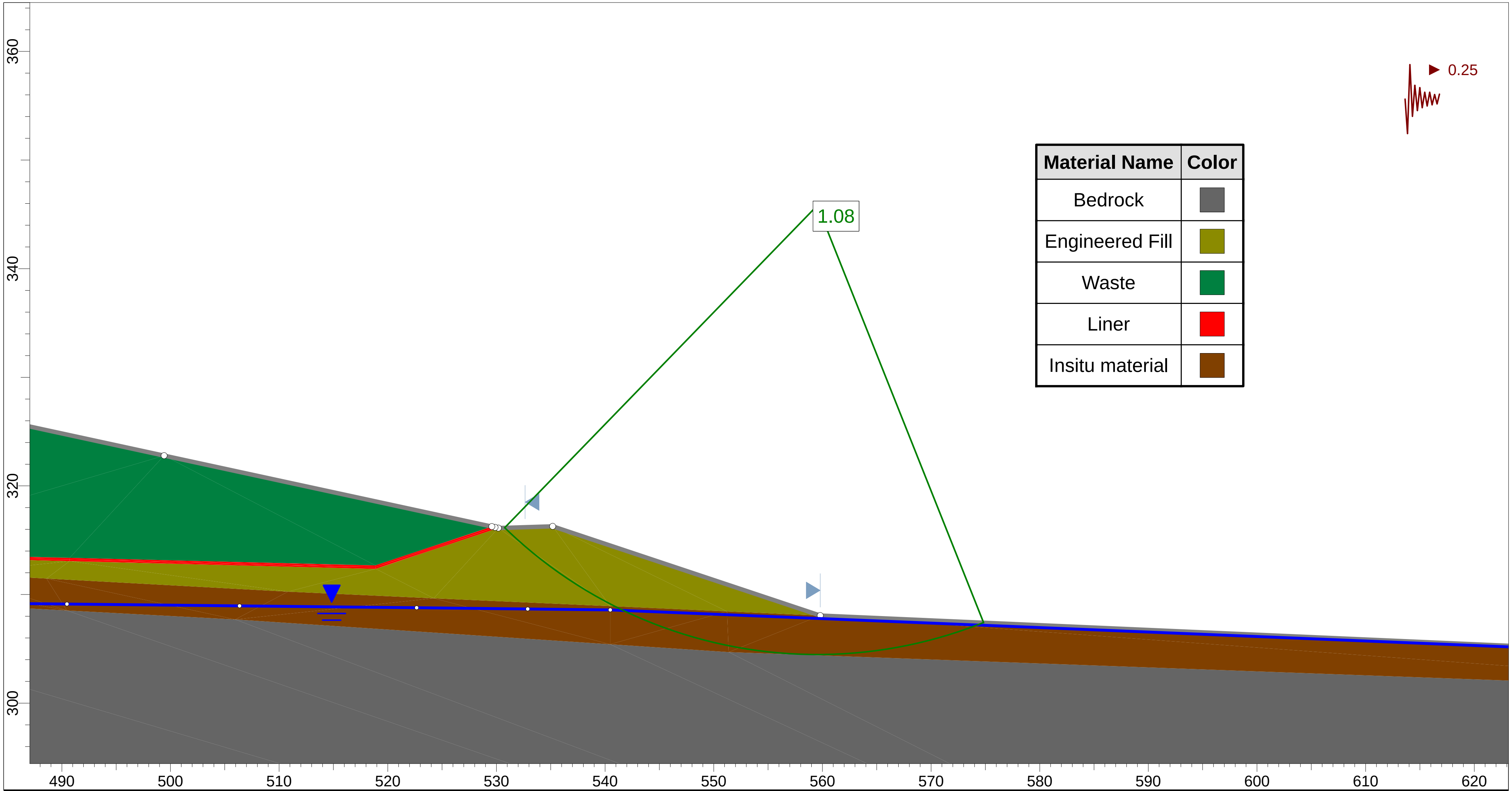
STABILITY ANALYSIS

FOUNDATION AND EMBANKMENT STABILITY
MINIMUM FACTOR OF SAFETY (DRAINED PARAMETERS)
POST CLOSURE, STATIC, LOW PORE PRESSURE
SECTION C

SCALE 1:500 REVISION 0 **FIGURE C1**



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FOUNDATION AND EMBANKMENT STABILITY
MINIMUM FACTOR OF SAFETY (DRAINED PARAMETERS)
POST CLOSURE, PSEUDO STATIC, MCE
SECTION C

SCALE 1:500 REVISION 0 **FIGURE C2**

APPENDIX B

Basal Liner System Integrity Assessment Figures

Scenarios investigated

Table 1A: Scenarios.

Scenario	Scenario
1	Integrity of the cushion geotextile
2	Integrity of the HDPE geomembrane

Material distribution and grid

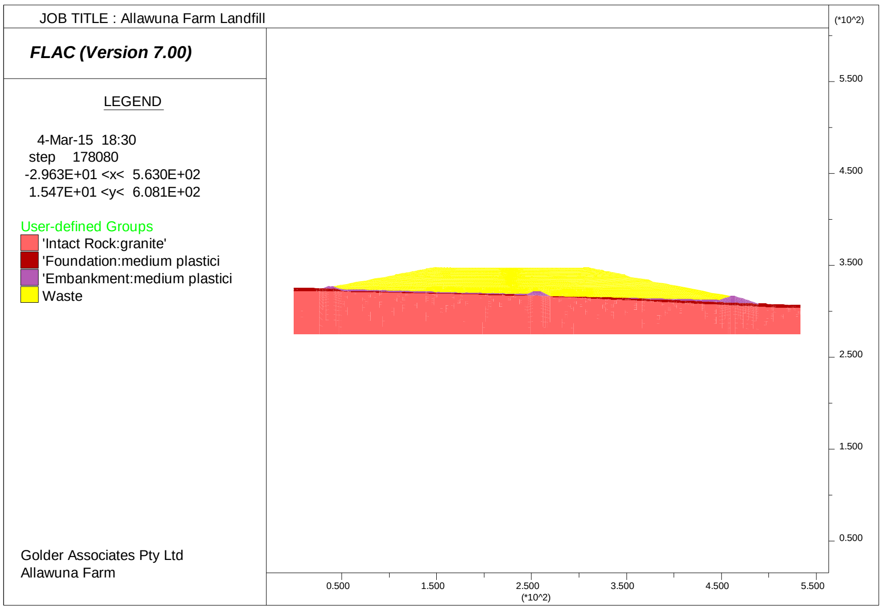

Figure 1: Finite difference modelling material distribution.

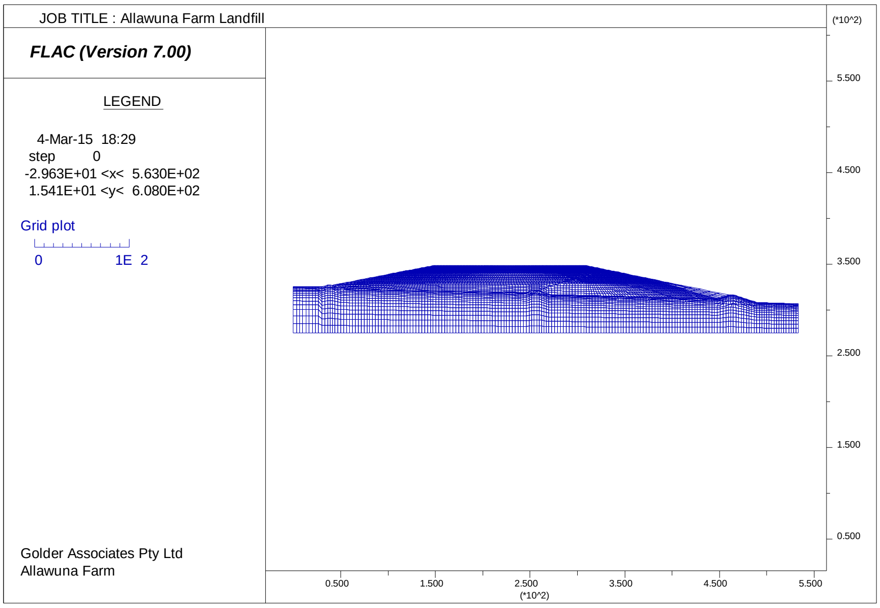

Figure 2: Finite difference modelling grid.

Scenario 1

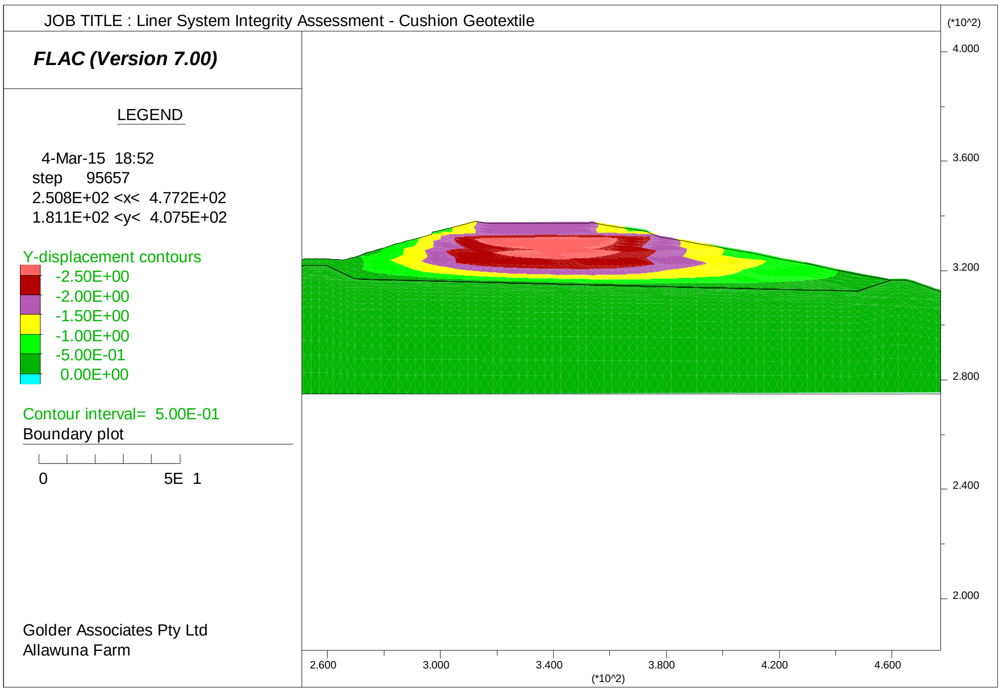

Figure 3: Vertical displacement distribution for Scenario 1 – Cell1 Filled.

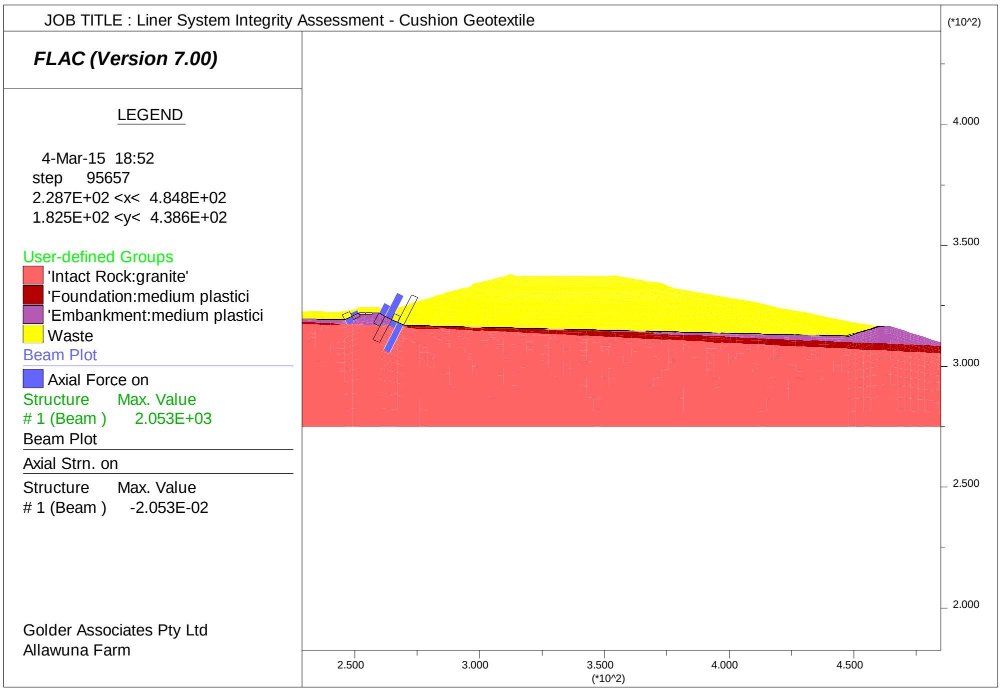

Figure 4: Axial strains and forces on cushion geotextile for Scenario 1 – Cell1 Filled.

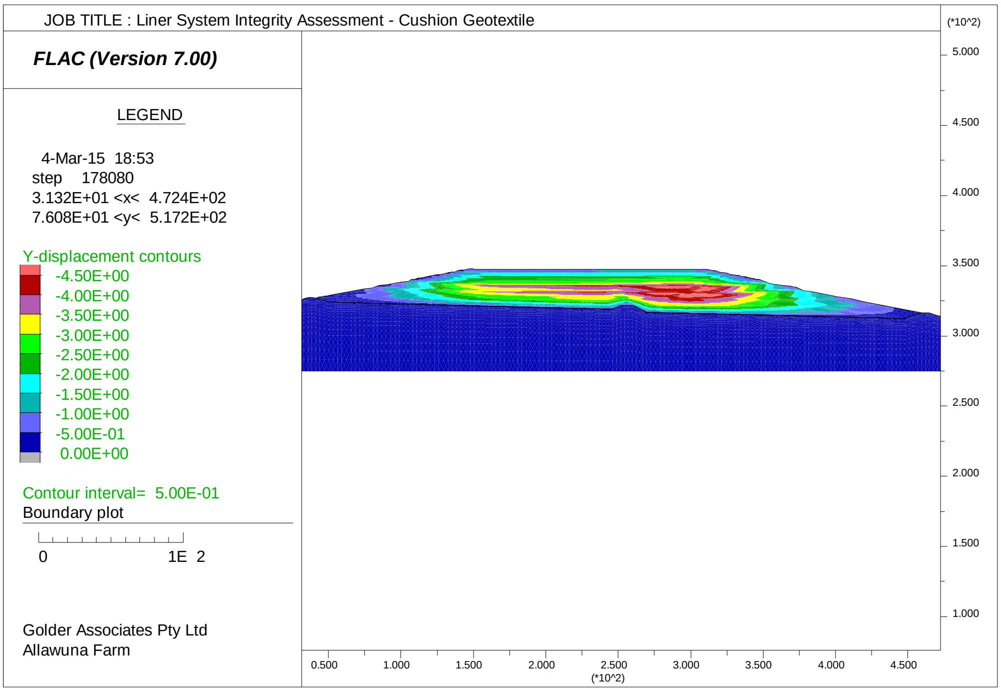

Figure 5: Vertical displacement distribution for Scenario 1 – Cell1+Cell2 Filled.

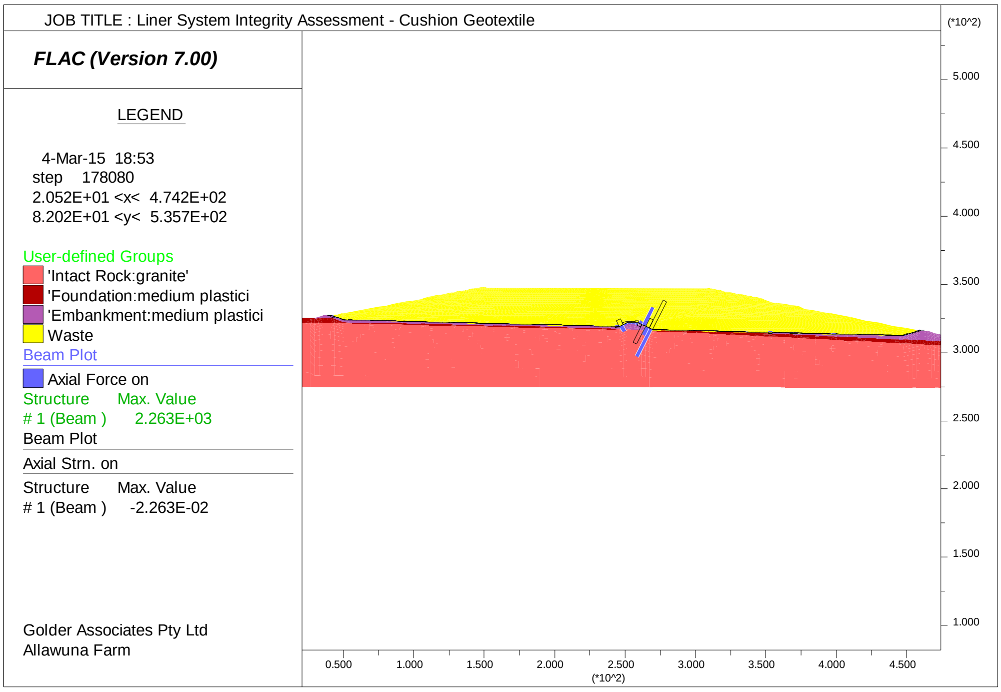

Figure 6: Axial strains and forces on cushion geotextile for Scenario 1 – Cell1+Cell2 Filled.

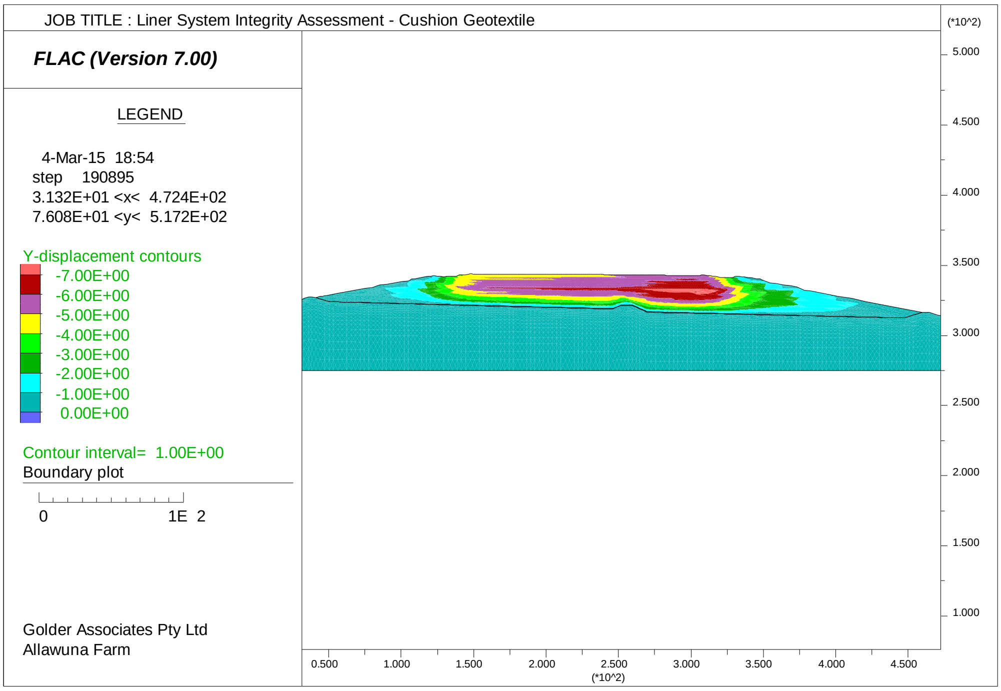

Figure 7: Vertical displacement distribution for Scenario 1 – Post-waste deposition.

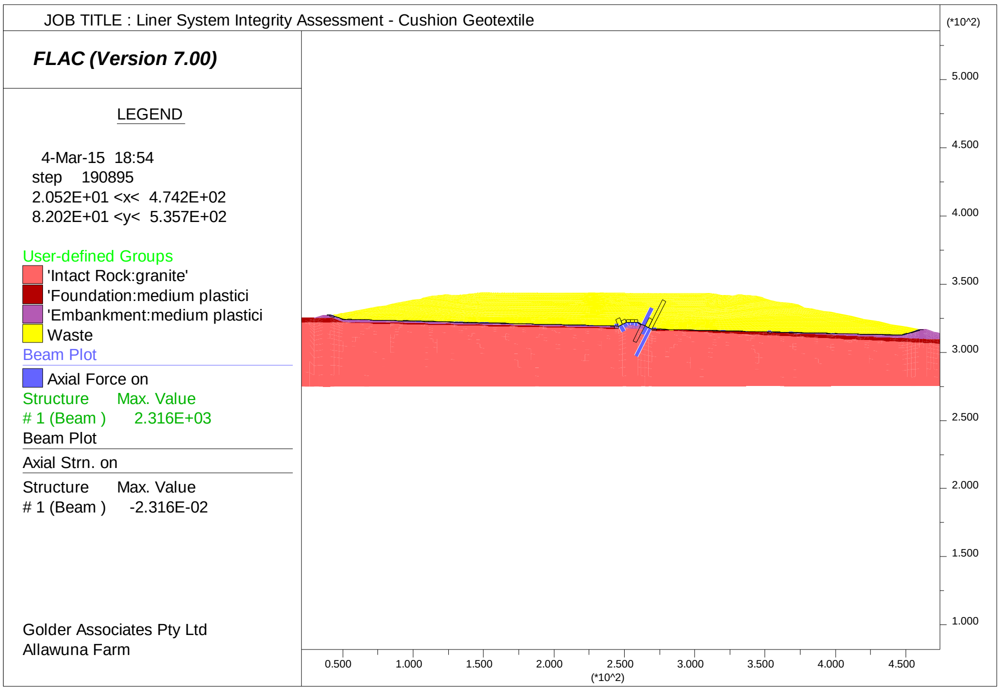

Figure 8: Axial strains and forces on cushion geotextile for Scenario 1 – Post-waste deposition.

Scenario 2

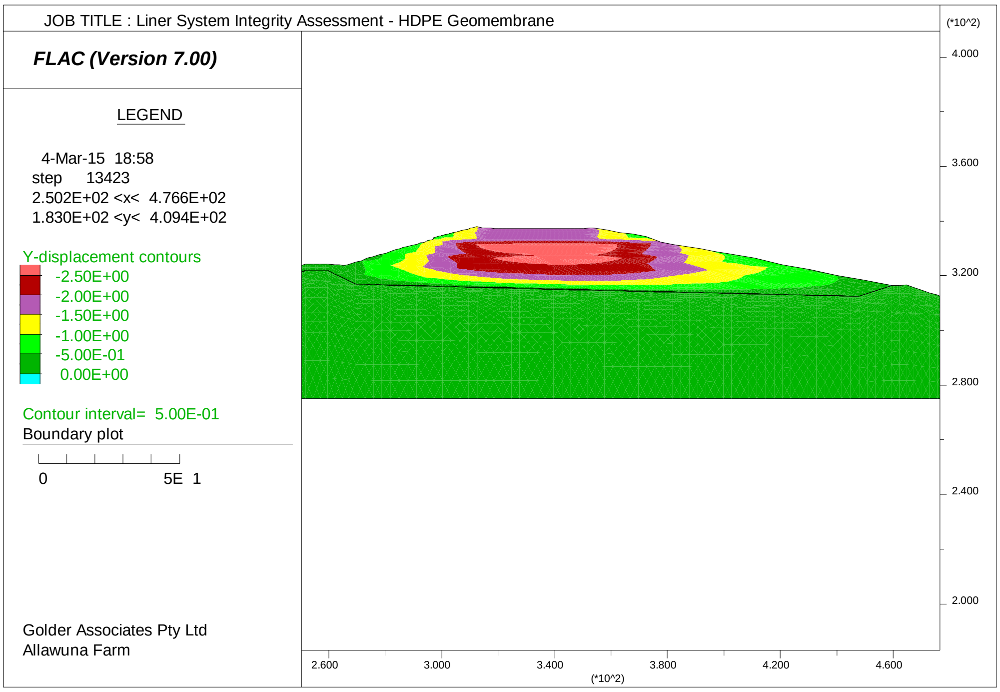

Figure 9: Vertical displacement distribution for Scenario 2 – Cell1 Filled.

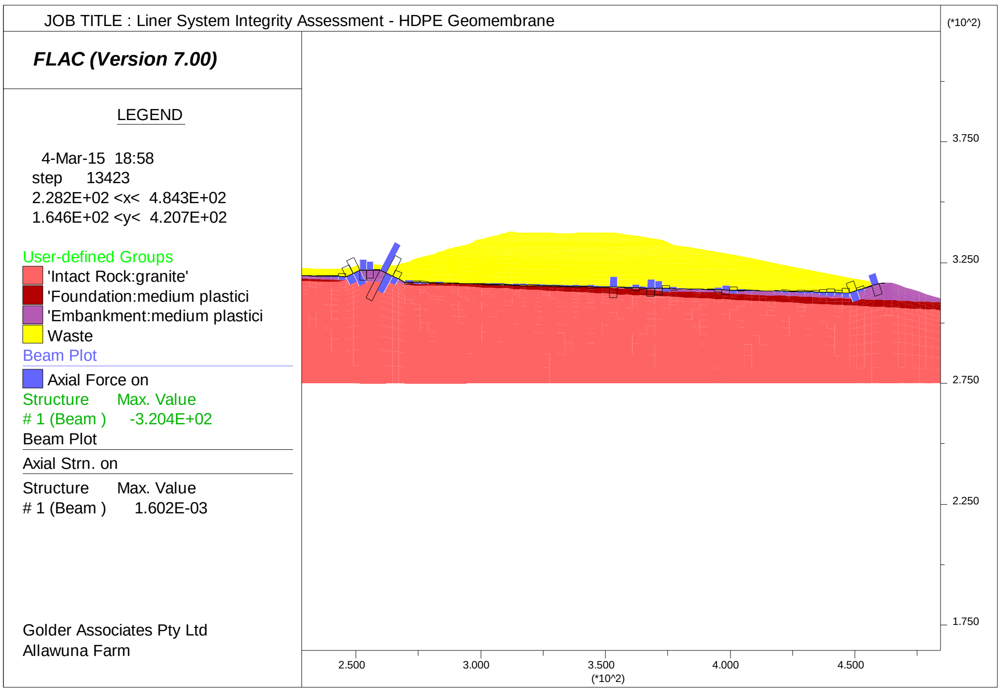

Figure 10: Axial strains and forces on HDPE geomembrane for Scenario 2 – Cell1 Filled.

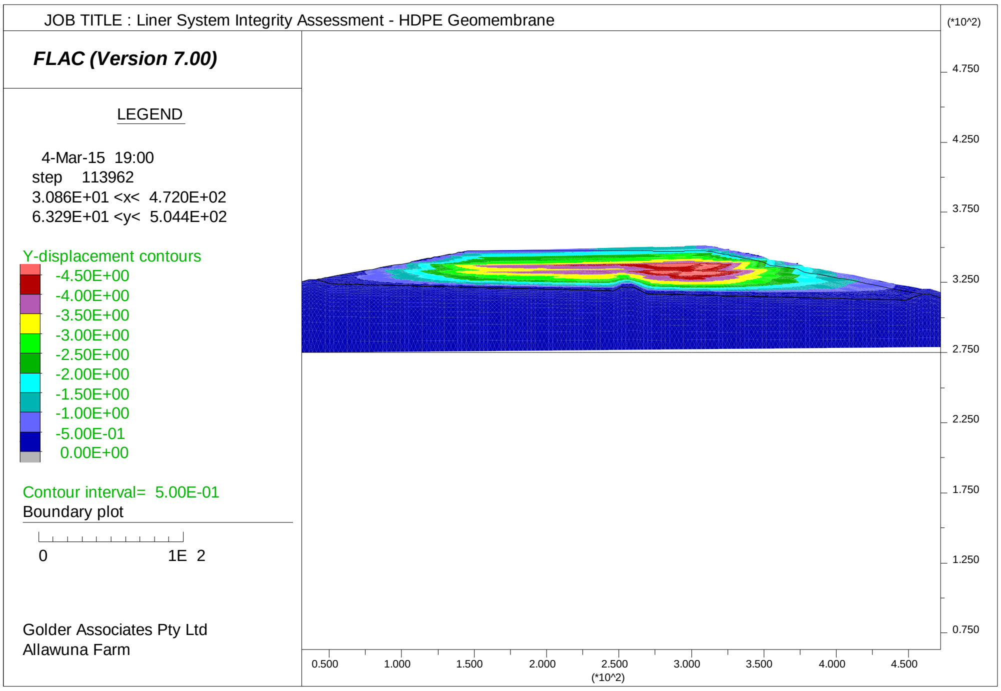

Figure 11: Vertical displacement distribution for Scenario 2 – Cell1+Cell2 Filled.

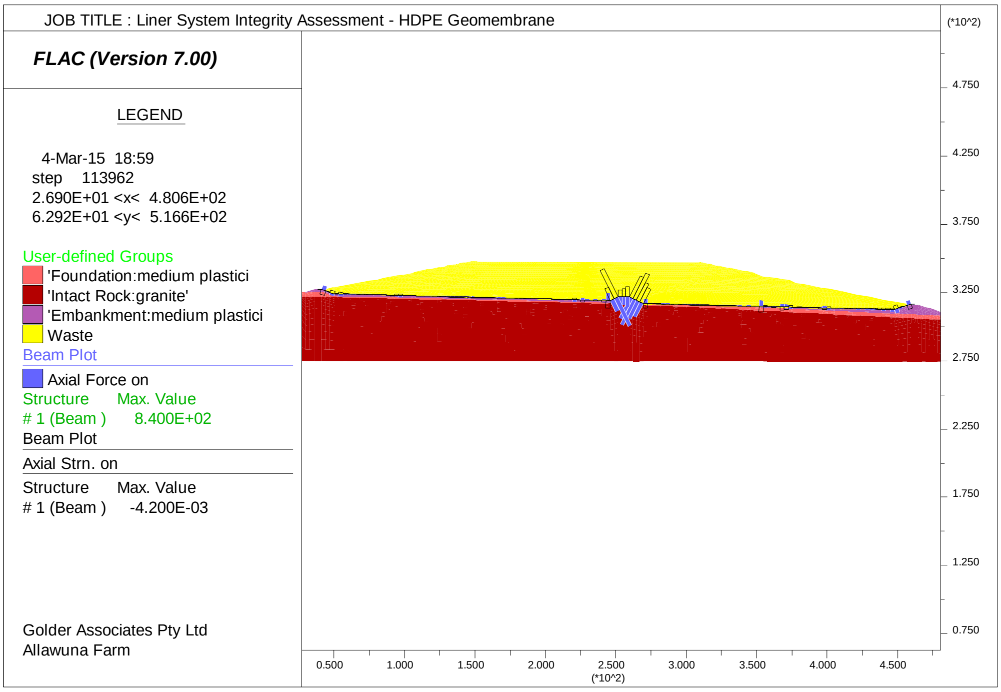

Figure 12: Axial strains and forces on HDPE geomembrane for Scenario 2 – Cell1+Cell2 Filled.

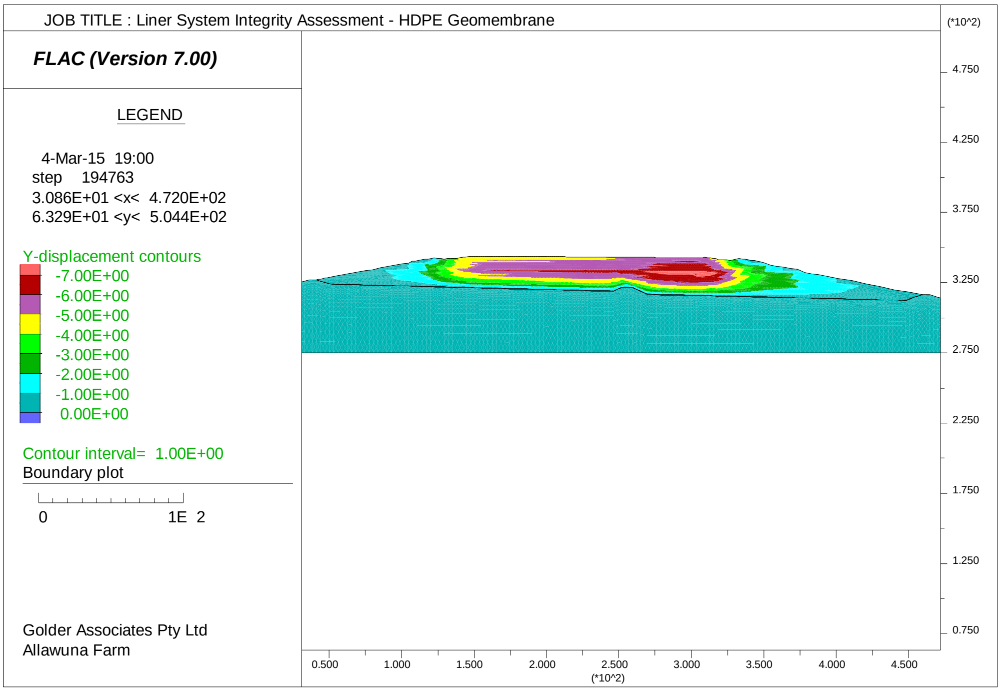

Figure 13: Vertical displacement distribution for Scenario 2 – Post-waste deposition.

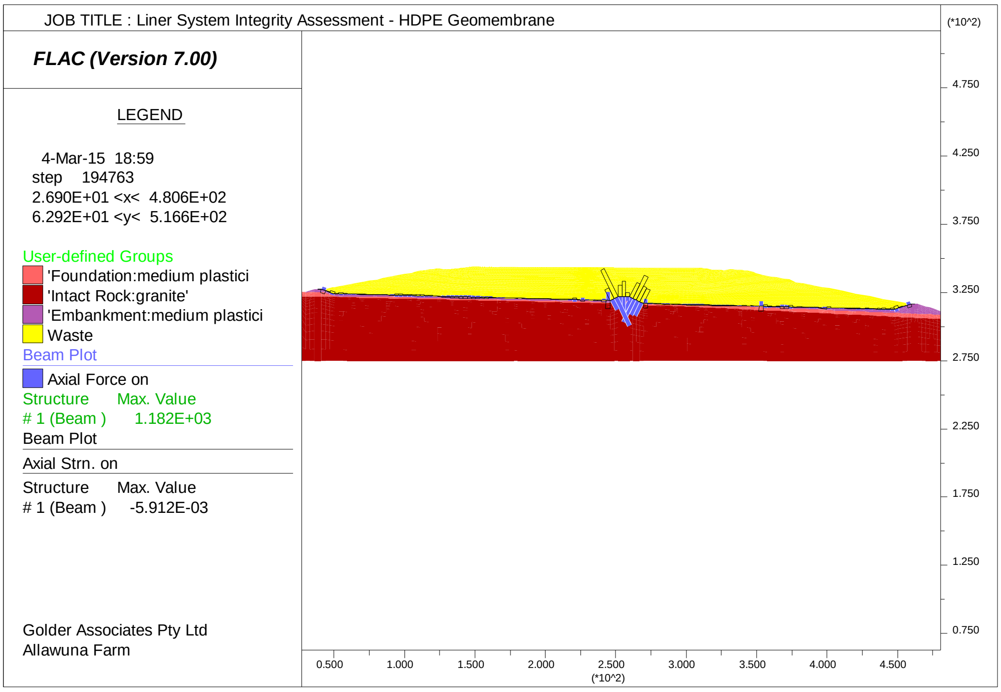

Figure 14: Axial strains and forces on HDPE geomembrane for Scenario 2 – Post-waste deposition.

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APPENDIX C

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