Stability Risk Assessment: BF5048/SRA
December 2021

STABILITY RISK ASSESSMENT for an ENVIRONMENTAL PERMIT VARIATION APPLICATION at POPLARS LANDFILL SITE

Prepared
for
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Information Sheet

Stability Risk Assessment – ENVIRONMENTAL PERMIT VARIATION APPLICATION POPLARS LANDFILL SITE

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STABILITY RISK ASSESSMENT
ENVIRONMENTAL PERMIT VARIATION
POPLARS LANDFILL SITE

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STABILITY RISK ASSESSMENT
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1 INTRODUCTION

1.1 Report Context

Sirius Environmental Limited (Sirius) were commissioned by **Biffa Waste Services Limited** (Biffa) to prepare a Stability Risk Assessment (SRA) to support an Environmental Permit Variation Application (EPVA) for the consolidation of the PFA Landfill permit and the main landfill permit at the Poplars waste management complex, including the amendment to the pre-settlement and post settlement contours within the scramble track area of the site and allow the construction of a plastic capping system.

This Stability Risk Assessment (SRA) considers the different components of the landfill containment system and assesses how they may be affected by the proposals.

This SRA has been prepared using guidance contained within the **Environment Agency R&D Technical Report P1-385/TR2** (hereinafter referred to as 'The Guidance').

1.1.1 Outline of the Installation

Poplars Landfill Site is located approximately 1.5 km to the southeast of Cannock, Staffordshire, on the junction of the A5190 Lichfield Road and A460 Eastern Way. The National Grid Reference of the site entrance is **SJ 930 940**. The address of the site is:

Poplars Landfill Site,
Lichfield Road,
Cannock,
Staffordshire,
WS11 8NQ.

Poplars Landfill Site is an operational landfill site receiving non-hazardous wastes whilst undergoing progressive capping and restoration.

1.1.2 Summary of Previous Work

1.1.2.1 Stability Risk Assessment for Poplars Landfill Site by Golders Associates Limited in November 2003

A Stability Risk Assessment (SRA) was originally undertaken by Golder Associates (UK) Limited in **November 2003** (Document Ref: 03523506.502), as part of the PPC Permit application for Poplars Landfill Site. The SRA considered stability and integrity issues relating to existing and proposed phases of Poplars Landfill Site.

1.1.2.2 Stability Risk Assessment Review for Poplars Landfill Site by Golders Associates Limited in March 2007

A stability risk assessment review (SRAR) letter report was issued in **March 2007** by Golder Associates. This letter report reviewed the stability of the GCL capping system based on actual site-specific testing carried out for the report on three soils types from the site.

1.1.2.3 Stability Risk Assessment Review for Poplars Landfill Site by Stratus Environmental Limited in March 2014

A comprehensive Stability Risk Assessment Review (SRAR) was issued by Stratus Environmental in **March 2014** (Report Ref: BF4824/SRAR Revision 2). This SRAR considered stability and integrity issues relating to all proposed future phases of Poplars Landfill Site.

1.1.2.4 Stability Risk Assessment for Poplars Landfill Site PFA Permit Application by Stratus Environmental Limited in April 2014

A stability risk assessment was undertaken by Stratus Environmental in **April 2014**. This stability risk assessment was produced as part of PFA cell permit and assessed the integrity of the liner and the stability of the side slope lining system during construction.

1.1.2.5 Stability Risk Assessment for 2019 Capping at Poplars Landfill Site by Sirius Environmental Limited in July 2019

A stability risk assessment was undertaken in **July 2019** to accompany the capping design to ensure the stability of the proposed tapered wedge was maintained, and the integrity of the capping system was maintained from the additional loading that would occur as a result of the placement of the tapered wedge.

1.2 Conceptual Stability Site Model

The following sub-sections present a summary of the natural geological, geosynthetic, or fill materials (including engineered fill and un-engineered infill) used in the model, relating specifically to the components identified in **Form IPPC Landfill Part B**, and from the guidance contained within the *Environment Agency R&D Technical Report P1385/TR2*.

1.2.1 Geology and Ground Conditions

The ground conditions in and around Poplars Landfill Site have been described in detail in the ESID report for the site. The ESID report for the site has been reviewed along with the 2008 Geoplan site investigation and the 2013 Stratus hydrogeological site investigation. Based on the above information sources, a summary of the geology and ground conditions is provided below. The geology within the immediate vicinity of the site is summarised in **Table SRA1**.

Table SRA1 below provides a summary of the geological succession for the site.

TABLE SRA1: SUMMARY OF GEOLOGICAL SUCCESSION		
Age	**Geological Unit**	**Description**
Recent	Made Ground	**OPENCAST BACKFILL MATERIAL**/Colliery spoil: generally comprising a clay or silt with variable gravel of mudstone, siltstone, sandstone and coal;
Pleistocene	**GLACIAL DRIFT DEPOSITS**	Gravelly slay or sand and gravel;
Carboniferous	Upper Coal Measures	**ETRURIA MARL**: generally comprising stiff reddish-brown clay/mudstone with sandstone bands/beds;
	MIDDLE COAL MEASURES	Grey shales and mudstones with coal seams and sandstone bands;

Note: Capitalised words indicate the terminology used to describe the ground types in the report text

Opencast mining has taken place in the western part of the PFA cell. These area shall comprise of opencast backfill material as the subgrade. All other areas of the graveyard area, which this assessment shall focus on were not subjected to opencast mining and comprises glacial drift deposits which overlie the Etruria Marl formation. In these areas, the subgrade will comprise Etruria Marl.

1.2.2 Hydrogeology and Groundwater

Evidence from historic site investigation boreholes (and water level data from monitoring wells around the site) indicates that a body of groundwater is present within the Glacial Drift Deposits. This groundwater body is considered to be perched on top of the lower permeability Etruria Marl strata. Discrete groundwater strikes are also encountered within the bedrock unit below the site, within the Coal Measures and the Etruria Marl.

Groundwater within the drift deposits underlying, and in the vicinity of, the site generally flows from east to west. The estimated groundwater levels in the drift deposits vary due to the level of the interface between the two strata varying around the site. In the western part of the site it approximately **135mAOD** and in the eastern part of the site it is approximately **146mAOD**.

From recent monitoring data, the groundwater levels within the bedrock units are showing a potentiometric head, which mirrors the levels within superficial deposits.

During previous works groundwater seepages have been noted within the Etruria Marl which has required a groundwater collection system to be installed behind the engineered clay liner (ECL).

1.2.3 Stability Section Selection

As part of this stability risk assessment, 1 section has been selected for analysis following a review of multiple sections. The chosen section covers the worst case scenario for each assessment undertaken as part of this SRA. The section position run north-south, through the PFA waste mass, through the proposed non-hazardous containment system, and through the proposed waste tipping profile, allowing for a complete stability assessment to be undertaken. A drawing showing the section positions is presented in **Appendix SRA1**.

1.2.4 Basal Subgrade Model

The basal subgrade for the PFA cell, the existing cells and the proposed cells in the scramble track area (Phase C) at Poplars Landfill Site mainly comprise of Etruria Marl, however a small area of the site will comprise open cast backfill material underlain by middle coal measures.

1.2.5 Side-Slope Subgrade

The side-slope subgrade shall comprise in-situ glacial deposits and in-situ Etruria Marl. The glacial deposits overlie the Etruria Marl.

1.2.6 Basal Lining System Model

The proposed and constructed basal lining system at Poplars Landfill Site shall comprise the following:

- 750mm thick compacted artificially established geological barrier (AEGB) above areas of opencast backfill with a maximum permeability of $k=1x10^{-7}$ m/s for the PFA landfill;
- For the non-hazardous landfill cells an artificial sealing liner (ASL) comprising a 1000mm compacted clay liner (permeability of less than $k=1x10^{-9}$ m/s); and
- 300mm leachate drainage stone, or tyre shred/tyre bales approved alternative with preferential leachate pipework.

No AEGB has been established across the base of the eastern section of the PFA landfill. Moreover, CQA records confirm that only a very small section of the eastern section of the PFA landfill was underlain by natural occurring strata of the Middle Coal Measures. This section of landfill was therefore engineered to the same AEGB specification as that constructed over the opencast backfill deposits (see above), which comprises any greater thickness than that originally specified for this basal subgrade area (i.e. 500mm).

1.2.7 Side-Slope Lining System Model

The proposed and constructed side-slope lining at Poplars Landfill Site shall comprise the following:

- 500mm thick compacted artificially established geological barrier (AEGB) above areas of glacial drift deposits with a maximum permeability of $k=1x10^{-7}$ m/s for the PFA cell;

- The internal prepared face of the perimeter cut batter for the non-hazardous cells shall provide the subgrade formation surface for the artificial sealing liner (ASL) comprising a 1,000mm (1m) compacted clay liner with a permeability less than $k=1x10^{-9}$ m/s. In areas where drift deposits are encountered in the subgrade, an artificially established geological barrier (AEGB) will also be installed, comprising 1,000mm of compacted clay with a permeability of less than $k=1x10^{-9}$ m/s.

1.2.8 Waste Mass Model

The in-filling timeframes have been modelled in accordance with **Table SRA2** below, to represent an accurate construction sequence for the site, and outline the required time frames to ensure stability is maintained.

TABLE SRA2: CONSTRUCTION TIMEFRAME USED IN THE MODEL	
Construction Description	**Timescale (Days)**
Waste infill to Cell G4	548
Excavate Subgrade Cell 1	60
Capping Cell G4	45
Lining Cell 1	30
Inert Fill to **156.9mAOD**	40
Waste Cell 1	720
Capping Cell 1	30
Inert soils placed to final profile	180

1.2.9 Capping System Model

The proposed capping system that shall be constructed over the waste deposits within the main landfill once areas are up to the final proposed contours shall consist of the following:

Option 1

- 300mm regulating/cap bedding layer;
- Geosynthetic Clay Liner (GCL) Cap; and
- 1,000mm thick restoration soils.

Option 2

- 300mm regulating/cap bedding layer;
- 1mm (LLPDE) Geomembrane;
- 1,000mm thick restoration soils; and
- Protector Geotextile above the geomembrane.

No engineered capping system is required to be installed across the final surface of the PFA landfill.

1.2.10 Valley Infill Model

The valley between the PFA waste mass and the non-hazardous waste mass shall comprise inert soils mainly comprising the excavated material from cell construction works within the scramble track area. As PFA waste inputs at the site have ceased, the PFA cells shall be completed to the approved levels with the overburden material excavated from future cell builds at the site. It is assumed this material shall be the same as the material used within the valley fill.

2 STABILITY RISK ASSESSMENT

The six principal components of the conceptual stability site model have been considered and the various elements of that component have been assessed about stability, and integrity.

The principal components considered are the:

- basal sub-grade;
- side slope sub-grade;
- basal lining system;
- side-slope lining system;
- waste; and
- the capping system.

2.1 Risk Screening

Issues relating to stability and integrity for each principal component of the landfill have been subject to a preliminary review to determine the need to undertake further detailed geotechnical analyses. The following sections present the results of this screening exercise.

2.1.1 Basal Subgrade Screening

The basal subgrade comprises predominately in-situ Etruria Marl, however there is a small area where the basal subgrade comprises opencast backfill and middle coal measures. The key considerations for the basal subgrade, and the implications for stability (and integrity) are presented in **Table SRA3** below**.**

TABLE SRA3: STABILITY COMPONENTS FOR BASAL SUBGRADE		
Excessive Deformation	Compressible subgrade	The basal subgrade is predominately in situ Etruria marl which is very stiff (very low compressibility). An appropriate stiffness has been used in the analyses, to reflect this. A small area of subgrade comprises opencast backfill which is much softer in nature, an appropriate stiffness has been given in the models. An appropriate stiffness has been used in the analyses for the coal measures, however due to small area this is likely to not impact the modelling. The interface between the two material within the PFA Cell has been assessed in previous SRA's and is unchanged therefore it is not assessed further in this assessment.
	Basal Heave	*Groundwater:* The global groundwater level is well below the base of the proposed engineering levels; therefore, basal heave is not an issue.
	Cavities in subgrade	None anticipated.
Filling on Waste	Compressible Waste	Not applicable.
	Cavities in Waste	Not applicable.

2.1.2 Side-Slope Subgrade Screening

The side-slope subgrade comprises in-situ glacial drift deposits (sands and gravels) and Etruria Marl. The key considerations for the side-slope subgrade, and the implications for stability and integrity, are presented in **Table SRA4** below.

TABLE SRA4: STABILITY COMPONENTS FOR SIDE-SLOPE SUBGRADE		
Excessive Deformation	Compressible subgrade	The side-slope subgrade shall consist of glacial drift deposits (sand and gravel) and Etruria Marl. The side-slope materials are considered to be stiff and an appropriate stiffness has been used in the analyses, to reflect this.
	Heave	There is a local perched groundwater level on the interface between the glacial drift deposits and the underlying strata. Within the model the groundwater level has been modelled on the interface as it is on site to reflect the site groundwater conditions. Previous cell builds have had a groundwater collection system installed were groundwater has been encountered during the works and this shall continue during future cell works, therefore heave is not considered to be an issue.
	Cavities in subgrade	None anticipated.
Filling on Waste	Compressible Waste	Not applicable.
	Cavities in Waste	Not applicable.

2.1.3 Basal Lining System Screening

The controlling factors that influence the stability, and integrity, in the basal lining system are given in **Table SRA5** below.

TABLE SRA5: STABILITY COMPONENTS FOR BASAL LINING SYSTEM		
Mineral Liner	Stability and Integrity	The basal subgrade is predominately in situ Etruria marl which is very stiff (very low compressibility). An appropriate stiffness has been used in the analyses, to reflect this. A small area of subgrade comprises opencast backfill which is much softer in nature, an appropriate stiffness has been given in the models. The interface between the two material has been assessed in previous SRAs for the potential differential settlement and is unchanged therefore it is not assessed further in this assessment.
	Compressible subgrade	The basal subgrade is predominately in situ Etruria marl which is very stiff (very low compressibility). An appropriate stiffness has been used in the analyses, to reflect this. A small area of subgrade comprises opencast backfill which is less stiff in nature; an appropriate stiffness has been adopted in the models.
	Cavities	None anticipated.
	Basal Heave	As the groundwater level is below the engineering, heave (as a result of groundwater pressure uplift) is not anticipated.

2.1.4 Side-Slope Lining System Screening

The key considerations for the side-slope lining system and the implications for stability, and integrity, are presented in **Table SRA6** below.

TABLE SRA6: STABILITY COMPONENTS FOR SIDE-SLOPE LINING SYSTEM		
Un-confined	Stability	The side-slope lining systems will be least stable when the slope is un-confined, and no (confining) waste has been placed against it. As waste is placed against the side-slope the factor of safety will increase as the waste provides a passive wedge (confinement) at the base of the slope. This will be assessed in this report. The PFA side-slope stability has been assessed previously and shall not be assessed further in this assessment.
Confined	Stability	Confinement of the side-slope lining systems will increase the factor of safety from that of an un-confined slope, as the waste will provide passive resistance and added stability to the system. The confined slope will be assessed in this report.
	Integrity	An assessment will be made of the shear strains in the side-slope lining system as the liner is constructed, and during waste placement. This shall be assessed further in this report, to ensure the additional loading does not impact the integrity of the compacted clay liner. The results of these analyses will be compared with the work of the most recently published papers concerning the long-term integrity of mineral liners.

2.1.5 Waste Mass Screening

The controlling factors that influence the stability of the waste mass are presented in **Table SRA7** below.

TABLE SRA7: STABILITY COMPONENTS OF WASTE SLOPES			
Failure wholly in waste	Stability		The waste will be placed in horizontal layers therefore reducing the height of slope batters/angles of the waste, improving the stability. The waste will be modelled as it will be in-filled on site, to ensure the stability is assessed within the report.
Failure involving liner and waste	Mineral Clay	Stability	Loading of the waste against the liner may cause the waste to fail along (or through) the lining system, causing damage to the liner. This will be assessed further in this report.
		Integrity	An assessment will be made of the shear strains in the side-slope lining system after each lift of waste placement, as the placement of waste may apply additional strains to the liner. The results of these analyses will be compared with the work of the most recently published papers concerning the long-term integrity of liners.

2.1.6 Capping System Screening

The key considerations for the capping system and the implications for stability are presented in **Table SRA8** below.

TABLE SRA8: STABILITY COMPONENTS OF THE WASTE CAP		
Failure of the Cap	Stability	Capping slopes will be present at the completion of the landfill. The stability of the capping system over these slopes will need to be considered further within this report. The analysis will need to consider the build-up of water above the cap, and the influence of gas pressure below the cap.
Failure of the Cap due to Plant Movements	Stability	Plant movements are likely; up, down and across the cap, during, and after, its construction. The effect of these plant movements (acceleration and deceleration in particular) on the stability of the landfill cap will be considered further in this report.
	Integrity	There with be additional loads being applied to the capping system from the plant movements. These additional loads are likely to increase the strains which occur within the capping system. These strains from the plant loading need to be assessed further in this report to ensure they are within acceptable limits to ensure the integrity of the capping system is maintained. The effect on the geomembrane from the inert fill within the valley between the PFA and the non-hazardous waste mass shall need to be assessed as this has not been assessed in previous reports. The effect of the inert fill on the GCL capping system shall not be assessed further in this report as this remains unchanged from previous stability risk assessments.

2.2 Justification for Modelling Approach and Software

In order to perform a comprehensive stability risk assessment (SRA), the components of the landfill containment systems have to be considered not only individually, but also in conjunction with one another, where relevant. Any analytical techniques adopted for such an assessment should adequately represent all of the considered scenarios for both the un-confined and confined conditions (where appropriate). The methodology and the software should also achieve the desired output parameters for the assessment. This equates to the determination of factors of safety for stability assessments, or the calculation of strains within liner components, for integrity assessments.

The analytical methods used in this stability risk assessment review include:

(a) **Finite element analyses** for the determination of **shear strains** in the mineral components of the basal and side-slope lining system, the strains within geosynthetic materials within the capping system, and the **calculation of factors of safety**;

(b) **Finite element analyses** for the determination of the stability of the landfill, for the different stages of the landfill construction and subsequent inert waste placement, and the **calculation of factors of safety**;

(c) **Limit equilibrium stability analyses** for the calculation of factors of safety for the un-confined capping system with and without a saturated zone (parallel & horizontal submergence ratio) of soil in contact with the capping interface; and

(d) **Closed-form analyses** for the capping system stability incorporating plant loading and plant movements, seepage water forces, and partial gas pressures.

2.2.1 Finite Element Analyses

The proprietary software **PLAXIS 2D** (2021) has been used for the stability assessments. This is a two-dimensional finite element programme intended for the analysis of deformation and stability in geotechnical engineering. It is equipped for the simulation of non-linear, time dependent and anisotropic behaviour of soils and rock. In addition, since soil is multi-phase material, special procedures are required to deal with hydrostatic and non-hydrostatic pore pressures in the soil. **PLAXIS 2D** was originally developed for geotechnical engineers studying river embankments on the soft soils of the lowlands of Holland. In subsequent years, **PLAXIS 2D** has been extended to cover most other areas of geotechnical engineering. It is therefore well suited for application to the assessment of stability risk associated with Poplars Landfill Site.

2.2.2 Phi-C Reduction

A safety analysis in PLAXIS is undertaken by reducing the strength parameters of the soils. This process is termed ‘Phi-C reduction’ and is carried out as a separate calculation mode. Phi-C reduction is used when it is required to calculate a factor of safety, for the situation under consideration.

In the Phi-C reduction approach, the strength parameters tanφ and c of the soils (and interface shear strengths) are incrementally reduced until failure of the system occurs. The strengths of interfaces, if used, are reduced in the same way. The strength of structural objects like plates and anchors are not influenced by the Phi-C reduction.

The total multiplier ΣMsf is used to define the value of the soil strength parameters as a given stage in the analysis:

$$\sum Msf = \frac{\tan \varphi_{input}}{\tan \varphi_{reduced}} = \frac{c_{input}}{c_{reduced}}$$

A Phi-C reduction calculation is performed using the load advancement number of steps procedure. The incremental multiplier Msf is used to specify the increment of the strength reduction of the first calculation step. The increment is by default set to 0.1, which is generally found to be a good starting value. The strength parameters are successively reduced automatically until all Additional steps have been performed. If this case, the factor of safety can be given by:

$$\text{SF} = \frac{\textit{available strength}}{\textit{strength at failure}} = \text{value of } \sum Msf \text{ at failure}$$

If a failure mechanism has not fully developed, then the calculation is repeated with a larger number of additional steps.

To capture the failure of the system accurately, the use of arc-length control in the iteration procedure is required. The use of a tolerated error of no more than 3% is also required. Both requirements are complied with when using the Standard setting of the Iterative procedure.

When using Phi-C reduction in combination with advanced soil models, these models will actually behave as a standard Mohr-Coulomb model, since stress-dependant stiffness behaviour and hardening effects are excluded. The stress-dependent stiffness modulus (where this is specified in the advanced model) at the end of the previous step is used as a constant stiffness modulus during the Phi-C reduction calculation.

For slopes, the **Phi-C reduction** approach resembles the method of calculating safety factors as conventionally adopted in traditional slip-circle analyses.

2.3 Closed Form and Limit Equilibrium Analyses

Translational slips along a combination of geosynthetic elements are analysed by modelling a material with nominal thickness along the plane under consideration. The lowest interface shear strength (interface friction angle and apparent adhesion) for a group of geosynthetic and non-geosynthetic materials can then be used as the internal interface shear strength for this nominal thickness material, to calculate the factor of safety against failure along this plane.

2.4 Selection of Appropriate Factors of Safety

The factor of safety is the numerical expression of the degree of confidence that exists for a given set of conditions, against a particular failure mechanism occurring. It is commonly expressed as the ratio of the load or action that would cause failure against the actual load or actions likely to be applied during service. This is readily determined for some types of analysis, for example limit equilibrium slope stability analyses. However, greater consideration must be given to analyses that do not report factors of safety directly. For example, a finite difference analysis of strains within a capping system would not usually indicate overall failure of the model even though the strains could be high enough to indicate a failure of the integrity of the system. In such cases, it is necessary to define an upper limit for shear strains and to express the factor of safety as the ratio of allowable strain to actual strain.

For the integrity assessment, it is proposed to present the maximum strains determined from the analyses and compare these with the conclusions of the latest research relating to this aspect of landfill design, in order to determine acceptability.

Assessing the short and long-term integrity of the composite basal lining system will be based on the work of Edelmann et al (1999), Jessberger and Stone (1991), Arch at al (1996), Needham et al (1999) and LaGatta et al (1997), as well as the Guidance. The full references for all these papers are included at the end of this report, but a summary of their conclusions (and their applicability to the situation here) will be documented in the assessment section.

The factor of safety adopted for each component of the model would be related to the consequences of a failure.

BS6031 - Code of Practice for Earthworks (Clause 6.5.1.2 Safety Factors) states that suitable safety factors in a particular case can only be arrived at after careful consideration of all the relevant factors, and the exercise of sound engineering judgement. The factors to be considered include:

a) The complexity of the soil conditions;
b) The adequacy of the site investigation;
c) The certainty with which the design parameters represent the actual in-situ conditions;
d) The length of time over which the stability has to be assured;
e) The likelihood of unfavourable changes in groundwater regime in the future;
f) The likelihood of unfavourable changes in the surface profile in the future;
g) The speed of any movement which might take place; and
h) The consequences of any failure.

A minimum factor of safety of 1.3 is considered acceptable for stability and integrity, if reasonably conservative values are used.

2.5 Justification for Geotechnical Parameters Selected for Analysis

Geotechnical data for the stability analysis has been obtained from several sources. These sources include the previous stability risk assessments, site investigation information, previous CQA validation reports, and published (conservative) data applicable to the analyses.

The parameters selected for material properties consider the analysis to be undertaken, analysis previously undertaken and existing conditions on site. Engineering properties for the waste were obtained using guidance from **Environment Agency R&D Technical Report P1385/TR1**.

The values for c' and ø' for the waste adopted throughout the modelling were 5kPa and 25 degrees. The unit weight of the waste was taken as 10.5kN/m^3.

The values for c' and ø' for the PFA were determined by back analysis. In order to prevent the model failing within the current PFA, which is standing on site currently required values of c' and ø' of 3 and 27 respectively. Therefore, these values have been adopted throughout the modelling.

The interface values used within the closed form analysis are conservative estimates based on information from previous test results utilising similar materials. Interface testing between the geosynthetic elements used in the constructions shall be carried out to validate the interface parameters used within the closed form stability analysis.

2.6 Summary of Material Parameters for Finite Element Analyses

Table SRA9 below summarises effective stress soil parameters utilised in the analyses.

TABLE SRA9: SUMMARY OF EFFECTIVE STRESS MATERIAL PARAMETERS FOR FINITE ELEMENT ANALYSES								
Material	Unit Weight	Effective Cohesion	Effective Angle of Friction	Permeability	E_{50}	E_{oed}	E_{ur}	power
	kN/m³	kN/m²	°	m/s	kN/m²	kN/m²	kN/m²	(m)
Etruria Marl	20.0-22.0	10	30.0	K=1.4E-7	75,000	75,000	225,000	1.00
Opencast Backfill	20.0-22.0	5.0	25.0	K=1E-7	4,000	4,000	12,000	0.750
Glacial Drift Deposits	18.0	0.5	33.0	K=1E-3	10,000	10,000	30,000	0.500
Engineered Fill	21.0	5.0	25.0	K=1E-9	5,000	5,000	15,000	0.750
Engineered Clay Liner	21.0	5.0	30.0	K=1E-10	7,000	7,000	21,000	1.000
Waste	10.0-10.5	5.0	25.0	K=1E-5	6,000	6,000	18,000	0.500
PFA	16.0-17.0	3.0	27.0	K=4E-7	6,000	6,000	18,000	0.500
Restoration Soils	18.0-20.0	5.0	25.0	K=1E-8	5,000	5,000	15,000	0.750
Inert Valley Fill	18.0-20.0	5.0	25.0	K=5E-9	4,000	4,000	12,000	0.750

Table SRA10 summarises the geosynthetic material parameters utilised in the finite element analyses:

TABLE SRA10: SUMMARY OF GEOMEMBRANE MATERIAL PROPERTIES	
Material	Extensional Stiffness (kN/m)
Geosynthetic Clay Liner (GCL)	120 (10% Strain corresponding to 12kN/m)
1mm (LLDPE) Geomembrane	37.5 (40% Strain corresponding to 15kN/m) 75.0 (20% Strain corresponding to 15kN/m) 150.0 (10% Strain corresponding to 15kN/m)

PLAXIS print-outs of the material parameters can be found in **Appendix SRA2** of this report.

Table SRA11 summarises the interface shear strength parameters utilised in the closed form cap stability analyses:

TABLE SRA11: SUMMARY OF INTERFACE SHEAR STRENGTH PARAMTERS USED FOR CAPPING STABILITY ANALYSES		
Material	Apparent Interface Cohesion	Apparent Interface Angle of Friction
	kN/m^2	°
Capping Interface Shear Strength GCL	2.0	23.0
Capping Interface Shear Strength GM	2.0	23.0

Table SRA12 below summarises the plant loading values utilised for the capping system integrity analysis.

TABLE SRA12: SUMMARY OF PLANT LOADINGS FOR CAPPING SYSTEM INTEGRITY ANALYSIS					
Critical Plant Scenario	Un-factored Total Plant Load (Axle Loads)	Contact Area	Un-factored Maximum Ground Bearing Pressure	Actual Width of Load	*3D Corrected UDL (Wheel or Track) Load Per Unit Length for Capping Depth
	kN	m^2	kN/m^2	m	kN/m
20 Tonne D6 Dozer/20 Tonne 360 Degree Excavator on Tracks	200kN (100+100)	2 Tracks of $1.75m^2$	$60kN/m^2$	400mm to 500mm	29kN/m over 3.5m Track
JCB 3CX Backhoe (2 Wheel Axles) Excavator on Capping System	80kN (30+50)	2 Wheels of 0.25m2 & 2 Wheels of $0.38m^2$	$60kN/m^2$ & $65kN/m^2$	300mm & 450mm	30kN/m for 500mm & 33kN/m over 750mm
Fully Laden Dumper (3 Wheel Axles) on Stone Access Road Above Capping System	380kN (100+ 140+140)	6 Wheels of $0.375m^2$	$135kN/m^2$ & $185kN/m^2$	500mm	67kN/m & 2x 93kN/m over 3 sets of 750mm

**A Boussinesq Analysis was used based on research work of Poulos & Davis (1974)*

PLAXIS print-outs showing the full set of material parameters can be found in **Appendix SRA2** of this report.

3 ANALYSIS

3.1 Introduction

The key areas of Poplars Landfill site which require analysis as part of the proposals are:

- **Landfill Construction and Waste Stability Analysis:** The stability of the landfill (including the containment system) including PFA and non-hazardous waste mass, construction of the cell lining systems, each stage of infilling with waste, and inert waste placement, using **finite element analysis**;
- **Compacted Clay Liner Integrity:** The integrity of engineered clay liner component of the proposed basal and side-slope lining systems following each engineered clay liner construction phase and each waste infilling phase using **finite element analysis**;
- **Landfill Capping Stability Analysis**: The stability of the proposed capping system, soils with a geosynthetic interface; and
- **Landfill Capping Integrity Analysis:** The integrity of the both options of proposed capping system (GCL and Geomembrane), using the proposed plant loading on the completed capping system, and the integrity from the inert valley fill using **finite element analysis**.

3.2 Landfill Construction and Waste Stability Analysis

A summary of the factors of safety from the Plaxis phi-c reduction runs for the stability models are presented in **Table SRA13** below:

TABLE SRA13: SUMMARY OF PHI-C REDUCTION ANALYSES FOR THE STABILITY MODEL (EFFECTIVE STRESS)		
Description	**Critical slope identified during analysis**	**Factor of Safety**
Waste infill to Cell G4	Circular failure of the outer edge of the Cell G4 waste mass	1.656
Excavate Subgrade Cell 1	Circular failure of the outer edge of the G4 waste mass	1.654
Lining Cell 1	Circular failure of the outer edge of the G4 waste mass	1.653
Inert Fill to **156.9mAOD**	Circular failure of the unconfined G4 waste flank	1.624
Waste Cell 1	Non-circular failure of the restoration soils overlying the PFA waste mass	1.679
Capping Cell 1	Circular failure of the outer edge of the waste mass	1.420
Inert soils placed to final profile	Non-circular failure of the inert valley fill	1.794

Graphical representations of the analyses (including failure modes) are shown in **Appendix SRA3**.

3.3 Compacted Clay Liner Integrity

The following analysis is for the integrity of the lining system, during the different stages of construction works associated with the proposed development at Poplars Landfill Site. The integrity of the liner relates to shear strains that develop in the material. Strains within the mineral clay liner can be directly analyses within the finite element model.

A summary of the shear strains in the compacted clay liner, reported from the models are presented in **Table SRA14** below:

TABLE SRA14: SUMMARY OF MAXIMUM STRAINS & LOWEST FACTOR OF SAFETY FOR THE LANDFILL LINING SYSTEM (ETRURIA MARL SUBGRADE)	
Model Description	Compacted Clay Liner
	Maximum Shear Strain
Clay Liner Cell 1	0.5026%
Waste placement in Cell 1	0.8544%
Permeability and Strain Guidance Limit	**10%** (Arch et al, 1996)
Factor of Safety	**11.7**

3.4 Landfill Capping Stability Analysis

The placement of cover soil on a slope with a relatively low shear strength inclusion (such as a geomembrane or geosynthetic clay liner) should always be from the toe upward, towards the crest. This way, the gravitational forces of the cover soil and live load of the construction equipment are compacting previously placed soil and working with an ever-present passive wedge and stable lower portion beneath the active wedge. While it is prudent to specify low ground pressure equipment to place the soil, the reduction of the FOS value from no equipment load while up the slope will be seen to be minimal.

However, for soil placement down the slope, a stability analysis must add an additional dynamic stress into the solution. This stress decreases the FOS value, and in some cases, to a great extent. Unless absolutely necessary, the design must consider the dynamic force of the construction placement equipment.

<u>Static or Constant Velocity Plant Loads on the Capping System</u>

For the case of a medium-sized crawler-tractor/bulldozer (Caterpillar D6, Leibherr 734 or similar) pushing cover soil up from the toe of the slope to the crest, the calculation uses the free body diagram in the calculations, as the basis for the assessment.

This analysis adds the specific piece of plant (characterized by the weight and subsequent ground bearing pressure) and dissipates this force (or stress) through the cover soil thickness, to the interface of the geomembrane.

Upon determining the additional equipment load at the cover soil-to-geomembrane interface, the analysis proceeds as shown in the calculations, but with an additional force down (and parallel to) the slope. This additional force is equivalent to the weight of the plant load resolved parallel to the slope and adjusted to reflect the reduction of this load on the interface in question, as a result of distribution through the cover soil.

By resolving the plant load into forces parallel and perpendicular to the slope, it can be seen there is an additional load, which increases the frictional resistance to movement, or sliding. It is a well-documented proof that, if the analyses were undertaken in an infinite slope situation, the net effect of additional loads acting vertically is neutral, as far as translational slope stability is concerned. However, as the passive wedge at the base of the slope (finite slope analysis) remains the same, the factor of safety is reduced, although only slightly.

<u>Accelerating (or Decelerating) Plant Loads on the Capping System</u>

For the case of a medium-sized crawler-tractor/bulldozer (Leibherr 734, Caterpillar D6 or similar) pushing cover soil down from the crest of the slope to the toe, the analysis again uses the force diagram in the calculations. However, this time an additional force (on top of the forces from the static load) must be included, resulting from the acceleration or deceleration of the equipment.

The magnitude of this force is equipment operator dependent and related to both the equipment speed and the time to reach the speed (or time to stop). Again, this additional force from accelerating (or decelerating) must be distributed through the cover soil thickness, to determine the force per unit width at the interface in question.

<u>Water Seepage Build-up</u>

In these calculations, a saturated zone is assumed to exist within the cover soil for part or all of the thickness. The saturated boundary can be built up in the cover soil in two different ways

- A horizontal build-up from the toe;
- A parallel to slope build-up.

These two hypotheses are defined and quantified as a horizontal submergence ratio (HSR), and a parallel submergence ratio (PSR). When analysing the stability of slopes using the limit equilibrium method, free body diagrams of the passive and active wedges are taken with the appropriate forces (now including pore water pressures) are applied.

Analysis of the stability of the system has been assessed for the following scenarios:

- Without plant movements;
- With a saturated soil zone in contact with the geosynthetic interface;
- With static plant on the surface (or plant travelling at a constant velocity);
- With construction plant (accelerating, decelerating or stopping) pushing cover soil down from the crest of the slope, to the toe.

The scenario of extreme plant movements, in combination with a saturated soil zone in contact with the geosynthetic interfaces, is very unlikely to occur simultaneously, as significant plant movements on top of the final restored cap are not anticipated to take place following construction during extreme winter conditions. For this reason, we have not assessed these particular design cases as requiring a factor of safety in excess of 1.3, (or >unity where residual values are used), as long as reasonable precautions are undertaken during the soil-placement installation process.

These precautions are, primarily, the maintenance of a compacted passive wedge at the slope base, as the full soil thickness is installed from the base upwards. In addition to this, a suitable weather window has been assumed for the works, to enable the earthworks (and the restoration soil placement, in particular) to be undertaken in relatively dry conditions, reducing the risk of a translational slip in the event of excessive accelerations/decelerations from plant movements.

<u>Gas Pressure</u>

The effect of partial gas pressure on the stability of the capped slope has been considered. The calculations are based on the work of Thiel (1998 & 1999).

A summary of the factors of safety from the stability calculations for the capping system are presented in **Table SRA15** and **Table SRA16** below:

TABLE SRA15: SUMMARY OF CLOSED FORM CAP STABILITY ANALYSES - GCL		
Slope Analysed	**Parameters Description**	**Factor of Safety 1m Confining Soils**
85m Long at 1 in 3	Interface Parameters C'=2kPa, Phi'=23, Cover Soil Parameters C'=5kPa, Phi'=25, No Plant Loading Saturated Soil Loading PSR=0	1.71
85m Long at 1 in 3	Interface Parameters C'=2kPa, Phi'=23, Cover Soil Parameters C'=5kPa, Phi'=25, No Plant Loading; Saturated Soil Loading PSR=0.25	1.52
85m Long at 1 in 3	Interface Parameters C'=2kPa, Phi'=23 Cover Soil Parameters C'=5kPa, Phi'=25, No Plant Loading; Saturated Soil Loading PSR=0.5	1.34
85m Long at 1 in 3	Interface Parameters C'=2kPa, Phi'=23; Cover Soil Parameters C'=5kPa, Phi'=25; Plant Travelling Downhill Acceleration 2m/s^2	1.37
85m Long at 1 in 3	Interface Parameters C'=2kPa, Phi'=23; Cover Soil Parameters C'=5kPa, Phi'=25; Gas Pressure at 5kPa	1.64

TABLE SRA16: SUMMARY OF CLOSED FORM CAP STABILITY ANALYSES - GEOMEMBRANE		
Slope Analysed	**Parameters Description**	**Factor of Safety 1m Confining Soils**
85m Long at 1 in 3	Interface Parameters C'=2kPa, Phi'=23, Cover Soil Parameters C'=5kPa, Phi'=25, No Plant Loading Saturated Soil Loading PSR=0	1.71
85m Long at 1 in 3	Interface Parameters C'=2kPa, Phi'=23, Cover Soil Parameters C'=5kPa, Phi'=25, No Plant Loading; Saturated Soil Loading PSR=0.25	1.52
85m Long at 1 in 3	Interface Parameters C'=2kPa, Phi'=23, Cover Soil Parameters C'=5kPa, Phi'=25, No Plant Loading; Saturated Soil Loading PSR=0.5	1.34
85m Long at 1 in 3	Interface Parameters C'=2kPa, Phi'=23; Cover Soil Parameters C'=5kPa, Phi'=25; Plant Travelling Downhill Acceleration 2m/s^2	1.61
85m Long at 1 in 3	Interface Parameters C'=2kPa, Phi'=23; Cover Soil Parameters C'=5kPa, Phi'=25; Gas Pressure at 5kPa	1.34

Calculation sheets for the capping stability are presented in **Appendix SRA5**.

3.5 Landfill Cap Integrity Analysis

The integrity of a geosynthetic clay liner (GCL) cap or 1mm LLDPE geomembrane cap needs to be maintained under plant loading during construction. Finite element analyses (PLAXIS) can determine the axial forces within the capping geomembranes as a result of the construction of the landfill. However, it cannot determine strains directly. To determine the potential strains in the geo-synthetic elements, a comparison must be made with the manufacturer's stress-strain relationship (or peer reviewed published information) for the material. .

Assuming a linear relationship between stress and strain, published test data for geosynthetic clay liner's have been used to determine the likely strains that shall develop within the material. Manufacturers' data shows that for a geosynthetic clay liner, a 12kN/m axial load corresponds to a strain of 10%. Work undertaken by LaGatta (1997) quotes a figure of 4.5% tensile strain before any measurable loss of permeability needs to be considered.

Again assuming a linear relationship between stress and strain, published test data for geomembranes have been used to determine the likely uni-axial strains within the geomembranes. From **GRI-GM17**, it is not possible to determine the extensional stiffness for the in-service region of stress/strain. However, we know that the tensile strength for 1mm LLDPE at **15kN/m** axial force varies (for all manufacturers) between **~10%** (steepest gradient of stress/strain graph) and ~**40%** (shallowest gradient of stress/strain graph) uni-axial strain (assuming a conservative relationship to estimated yield point). This corresponds to extensional/tensile/axial stiffness values of between **150kN/m (at 10%), 75kN/m (at 20%)** and **37.5kN/m (at 40%).** These gradients are derived from 15kN/m divided by the different strains (10%, 20% or 40%). Work done by Peggs et al (2003) recommends limiting the maximum axial strain in LLDPE membranes to 10%.

A sensitivity analysis was undertaken as part of this assessment and the worst case factors of safety were reported for an axial stiffness of **37.5kN/m**. As a result of this sensitivity analysis 37.5kN/m, is the value that has been reported in the tables below to represent a worst case scenario.

Plaxis calculation sheets for the integrity analyses can be found in **Appendix SRA6** of this report. A summary of the maximum calculated strains in the capping system are shown in **Table SRA17, Table SRA18** and **Table SRA18A** below:

TABLE SRA17: SUMMARY OF MAXIMUM STRAINS & LOWEST FACTOR OF SAFETY FOR THE CAPPING GEOSYNTHETIC CLAY LINER (GCL) UNDER LOADING		
Activity	**Maximum Tensile Stress in GCL (kN/m)**	**Worst Case Maximum Tensile Strain (%)**
20 Tonne D6 Dozer/20 Tonne 360 Degree Excavator on Tracks	1.347	1.123%
JCB 3CX Backhoe (2 Wheel Axles) Excavator on Capping System	0.425	0.354%
Fully Laden Dumper (3 Wheel Axles) on Capping System	1.249	1.040%
Permeability & Strain Guidance Limit	-	**4.5%**
Lowest Factor of Safety	**-**	**4.00**

TABLE SRA18: SUMARY OF MAXIMUM STRAINS & LOWEST FACTOR OF SAFETY FOR THE CAPPING GEOMEMBRANE UNDER LOADING – EXTENSIONAL STIFFNESS OF 37.5kN/m		
Activity	**Maximum Tensile Stress in 1mm LLDPE Geomembrane (kN/m)**	**Worst Case Maximum Tensile Strain (%)**
20 Tonne D6 Dozer/20 Tonne 360 Degree Excavator on Tracks	0.4521	1.205%
JCB 3CX Backhoe (2 Wheel Axles) Excavator on Capping System	0.1354	0.361%
Fully Laden Dumper (3 Wheel Axles) on Capping System	0.4099	1.093%
Strain Guidance Limit (Peggs et al, 2003)	-	**10% (worst-case) for LLDPE**
Lowest Factor of Safety	**-**	**8.29**

TABLE SRA18A: SUMMARY OF MAXIMUM STRAINS & LOWEST FACTOR OF SAFETY FOR THE CAPPING GEOMEMBRANE FROM INERT FILL PLACEMENT – EXTENSIONAL STIFFNESS OF 37.5kN/m		
Activity	**Maximum Tensile Stress in 1mm LLDPE Geomembrane (kN/m)**	**Worst Case Maximum Tensile Strain (%)**
Inert Fill Placement	1.087	2.898%
Strain Guidance Limit (Peggs et al, 2003)	-	**10% (worst-case) for LLDPE**
Lowest Factor of Safety	**-**	3.45

4 ASSESSMENT

The assessments outlined above are presented in the order described.

4.1 Landfill Construction, Waste and Valley Infill Stability Assessment

Table SRA13 above outline the factors of safety for the stability of the proposed engineering works at Poplars Landfill Site, during the construction of the lining system, during the subsequent waste placement and following capping and restoration works (including the valley fill).

The lowest factors of safety are recorded during the excavation to formation within the scramble track cell. The reported factor of safety is **FOS=1.318**. The failure mode noted is through the outer edge of the waste mass on the northern flank of the landfill, rather than related to the excavation of the cell within the scramble track. The factors of safety reported for all phases of the model, are all above the minimum required 1.3.

The stability of the non-hazardous waste mass was found to be sensitive to the timeframe in which the waste is infilled. As the waste is loaded against the lining system, there is a generation of excess pore-water pressures within the clay lining material. As the lining material is proposed to comprise low permeability material (which will be further lowered by compaction of the material during the construction period) positive excess pore water pressures cannot easily dissipate. Until the excess pore-water pressures dissipate, there will be very little increase in effective stress, meaning that there will be very little increase in the shear strength of the material to improve the stability of the slope. In order for the waste mass stability to be maintained the waste must be infilled within the timeframe modelled within this SRA to ensure that acceptable factors of safety are achieved.

The modelling has found that if the valley between the PFA waste mass and the non-hazardous waste mass is filled with low permeability fill type material the factor of safety for the slope decreases below an acceptable limit due to the build-up of excess positive pore water pressures within the material, meaning there is no increase in the effective shear strength of the material until these pore water pressures have dissipated. In order to maintain the stability of the inert wedge, a band of glacial drift deposits, 3m thick, shall need be placed at the base of the unconfined slope to provide a drainage pathway for the positive pore pressures which build up to dissipate out.

4.2 Compacted Clay Liner Integrity

It is important that the permeability of the mineral liner is maintained during (and after) the in-filling process, to limit the downward migration of leachate into the surrounding ground (and potential issues with contamination). Finite element analysis cannot accurately predict changes in permeability over time, only the anticipated strains within the soils. Therefore, it is necessary to be able to assess how the strains within the clay liner affect the permeability of the material.

No site-specific data on the relationship between strain and permeability exists for this proposed clay liner material. However, research by Arch et al (1996) has shown that the permeability of compacted clay decreases for strains up to the yield point of the material (typically 6%), after which increases in permeability are exhibited. Considering the initial decrease in permeability, values above the original permeability of the compacted clay are only achieved after shear strains of around 11%. For the purposes of this report, a design value of ~10% shear strain has been adopted, since this represents a point at which permeability would remain below the as-built specification.

As part of this assessment the shear strains within the engineered clay liner have been assessed on the underlying Etruria Marl. The maximum shear strain anticipated in the compacted clay liner (CCL) is **0.8544%,** with this occurring when the waste is placed to full height (worst-case). Comparing this worst-case shear strain **(0.8544%)** against the recommended maximum strain of 10% provides a factor of safety of **FOS=11.7**.

4.3 Landfill Capping Stability Assessment

Table SRA15 and **Table SRA16** details the analysis undertaken on the stability of the landfill capping system. The cap was modelled with the steepest gradient to represent a worst-case scenario (**1 in 3, 28m** high with a slope length of approximately **85m** respectively). The cap was analysed with 1,000mm (1m) of cover soil placed above the capping system, which shall comprise either a geosynthetic clay liner (GCL) or a 1mm (LLDPE) geomembrane, to determine the effect of the stability once final capping has occurred.

The interface shear testing parameters were selected are conservative estimates based on previous experience with similar materials. Further interface shear strength testing is proposed to be undertaken on the actual materials to be used within the capping works, prior to engineering works commencing, to validate the analysis undertaken as part of this SRA.

Table SRA15 and **Table SRA16** shows the lowest factor of safety for the stability of both capping system options is **FOS=1.34**, and occurs when the restoration soils are placed to full thickness (1,000mm) and is subjected to a Parallel Submergence Ratio (PSR) of up to 0.5.

A factor of safety of approximately FOS=1 indicates that, given the prescribed conditions, the cap would be at the point of failure. However, as the calculated factors of safety are all greater than 1.3, the prescribed conditions undertaken in this assessment indicate that, with a PSR of <0.5, the factors of safety are all in excess of 1.3. As part of the capping design, a geo-composite drainage layer shall be installed above the GCL or geomembrane and under the capping restoration soils. This means that the slope is not likely to be subject to full saturation of the cover soils meaning that a PSR of 0.5 can be considered a worst case.

It should also be noted that the steepest capping flanks considered in the models will only be a temporary feature until infilling of the valley commences, which will subsequently reduce the overall slope height and increase the factor of safety. The final gradients of the other slopes around the site once landfilling are completed shall be lower in height and slacker than that assessed, which shall result in an increased factor of safety from that shown above.

When considering the effects of extreme plant movements on the stability of the capping systems (both GCL and geomembrane options), the effect of breaking (deceleration) and turning sharply has the greatest impact on the stability of the capping soils. This has been assessed and the minimum factor of safety calculated is **FOS=1.61** which occurs when there is 1000mm of restoration soils with plant travelling in a downhill direction and decelerating to a stop. Whilst the factor of safety is in excess of 1.3, the worst-case can be avoided by preventing extreme plant movements on the cap during construction (i.e. no sharp or sudden deceleration on the cap, when traveling downhill).

The following recommendations are made, which should also be considered best practise during the construction of the engineered cap, and which should be detailed within the method statement controlling these works:

- The placement of cover soil on a slope with a relatively low shear strength inclusion (such as a geomembrane or geo-composite) should always be from the toe upward, towards the crest. This way, the gravitational forces of the cover soil and live load of the construction equipment are compacting previously placed soil, and working with an ever-present passive wedge and stable lower portion beneath the active wedge; and

- While it is prudent to specify low ground pressure equipment to place the soil, the reduction of the FOS value when comparing a ‘no equipment load scenario’ to a ‘working up-the-slope scenario’ has been shown to be minimal. However, should a crawler tractor in excess of 32 tonnes be considered for soil placement, additional assessment would be required.

The analysis of the capping system has considered the worst-case (longest-steepest) slopes across the site, with the worst-case loading conditions. By ensuring that a suitable method statement is implemented which prevents extreme plant movements on the cap, the stability of the capping design is considered acceptable.

4.4 Landfill Cap Integrity Assessment

Table SRA17, Table SRA18 and **Table SRA18A** details the maximum stress (axial forces) and resultant strains within the proposed capping geosynthetic clay liner (GCL) and the LLDPE geomembrane capping options which will be imposed by the placement of restoration soils against the proposed capping system, together with temporary plant movements on the surface of the restoration soils.

The placement of cover soils against any capped waste will induce tensile stresses in the capping systems. However, it is the size of the differential settlements and angular distortion (differential settlement/distance) that are important to the tensile stresses in the cap. Although percentage horizontal strain at yield and subsequently break is typically in excess of 20% for FMLs, Peggs et al (2003) recommended the maximum strains for different materials as follows:

- HDPE smooth SCR <1500 hr 6%
- HDPE smooth SCR >1500 hr 8%
- HDPE random texturing 4%
- HDPE structured profile 6%
- LLDPE density <0.935 g/cm^3 12%
- LLDPE density >0.935 g/cm^3 10%
- LLDPE random texture 8%
- **LLDPE structured profile 10%**
- PP un-reinforced 15%

The measurement of strain is used as an indirect measure of the stress that exists in a GCL and geomembrane. Using the strains highlighted above and the guidance limits stated in **Tables SRA17**, **Table SRA18** and **Table SRA18A**, as a value below which the integrity (and therefore long-term performance) of the material may be relied upon, allows for a design limit to be set which the calculated strains must not exceed

Option 1 - GCL

The worst-case strains which are induced in a GCL from plant loading is **1.123%**. This occurs when a tracked excavator is driven directly on 1,000mm of restoration soils, which gives a factor of safety of **FOS=4.00**. Whilst this scenario generates acceptable strains (compared to the maximum allowable), it is good practice to prevent tracked excavator from being driven directly on the capping soils, and traffic management on site should prevent this from occurring.

Option 2 - Geomembrane

The worst-case strain recorded in the 1mm LLDPE capping geomembrane is **1.205%** and occurs with an extensional stiffness of 37.5kN/m when a tracked excavator is travelling on 1000mm of restoration soils. Comparing this worst-case strain of **1.205%** to the maximum strain limit of 10% gives a factor of safety of **FOS=8.29**, which is greater than the required FOS of 1.3.

As the inert fill is placed within the valley between the PFA and the non-hazardous waste mass there is the potential for an increase in the axial forces induced and the resultant strain due to the increase in load being applied to the geomembrane of the capping system from the inert fill. This increase in loading applied to the geomembrane from the placement of the inert fill has been assessed in this SRA. The lowest factor of safety reported is **FOS=3.45**. This factor of safety is greater than the minimum requirement of 1.3 and therefore demonstrates that the placement of the inert fill within the valley between the PFA and the non-hazardous waste mass does not have a detrimental impact to the integrity of the capping system.

All the factors of safety calculated in this capping system integrity assessment, for both options, are above the minimum required 1.3. Therefore the integrity of the capping system shall be maintained during the placement of restoration soils, from temporary plant movements and following placement of the inert fill with the valley between the PFA and the non-hazardous waste mass.

It is concluded that the long-term integrity of the cap will be maintained should the proposed machinery be used on site, and during the placement of the inert soils. However, if machinery heavier that those analysed in this assessment were to be driven above the cap, further analysis is recommended to confirm that the integrity of the cap is maintained. Notwithstanding this, it must also be borne in mind that the induced strains analysed for plant loading are from temporary loads, and that a geosynthetic clay liner cap or a 1mm LLDPE geomembrane is unlikely to suffer any long-term loss of integrity.

5 CONCLUSIONS

This stability risk assessment (SRA) has addressed the stability and integrity issues which arise as a result of the proposed variations proposed for the Poplars Landfill Site.

Specifically, the following assessments have been considered:

- The stability of the proposed cell construction works, basal and side-slope lining system, non-hazardous waste infill, capping system, and inert wedge infilling, for each stage of the remaining landfill development and subsequent waste placement;
- The integrity of the compacted clay liner (CCL) during each phase of waste placement at the site;
- The stability of the proposed landfill capping system options (GCL and geomembrane) once final waste placement has taken place; and
- The integrity of the proposed landfill capping system options (GCL and geomembrane) when (initially) trafficked by plant, and (secondly) during placement of inert soils within the valley, between the PFA waste mass and the non-hazardous waste mass.

Analyses have been based on the available site investigation information, conservative materials parameters, and a worst-case interpretation.

The assessments for the stability of the landfill during construction and waste placement show that the all stages achieve a factor of safety of greater than 1.3, with the lowest factors of safety being obtained during the excavation to formation levels within the scramble track.

The assessment for the integrity of the proposed capping system shows that the anticipated tensile strains within, both the GCL and LLDPE capping geomembrane (during placement of restoration soils, inert fill, and plant movements above the cap) are below the guidance limit from published literature and provide acceptable factors of safety.

The assessments for the integrity of the Compacted Clay Liner (CCL) show that shear strains within both the side-slope and basal liner increase as the waste height increases. However, the reported shear strains are all less than 10% (our maximum adopted limit to ensure that the permeability of the lining material can be relied upon) and have a calculated factor of safety of **11.7**.

In conclusion, the stability and integrity analyses for the waste containment systems at Poplars Landfill Site report factors of safety greater than **1.3** for all the scenarios considered. This assessment is deemed acceptable, as long as the construction sequence and waste slopes are constructed (and in-filled) as modelled. Should the in-filling rate increase from that modelled in this SRA (or the fill height increases) then the models should be revisited with the new timeframe parameters, to ensure that adequate factors of safety are still achieved.

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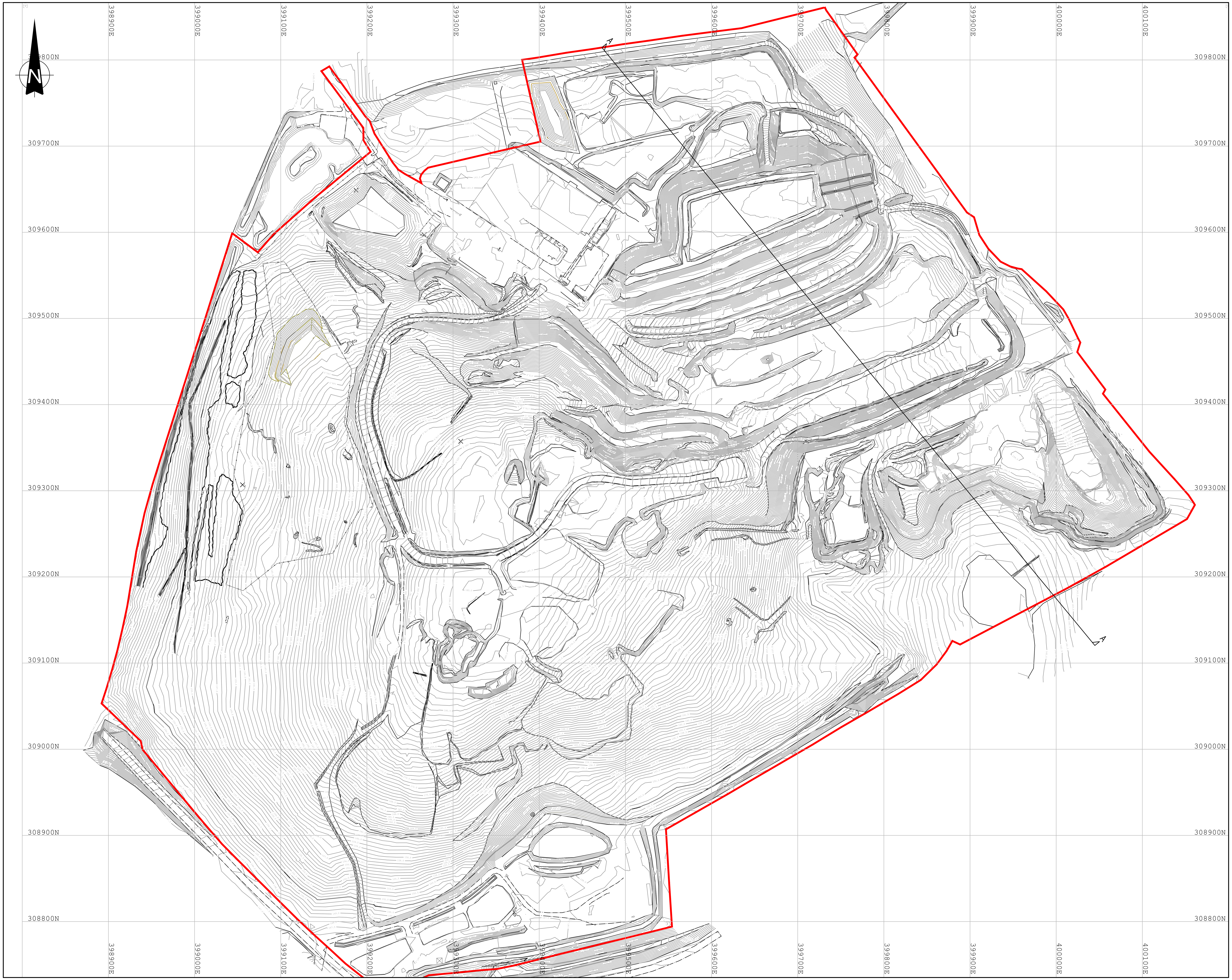
APPENDIX SRA1

DRAWINGS



NOTES

1. ALL DIMENSIONS IN MILLIMETRES AND ALL LEVELS IN METRES ABOVE ORDNANCE DATUM.
2. DO NOT SCALE FROM THIS DRAWING.
3. ANY ANOMALIES IDENTIFIED WITH THE DETAILS SHOWN ON THIS DRAWING ARE TO BE BROUGHT TO THE ATTENTION OF SIRIUS ENVIRONMENTAL PRIOR TO CONSTRUCTION WORKS COMMENCING.

REV	DESCRIPTION	DATE	BY

CLIENT

Biffa

sirius Environmental

Suite 5a, Rossett Business Village, Llyndir Lane, Rossett, Wrexham. LL12 0AY. 01244 571 358

JOB TITLE

POPLARS LANDFILL SITE
Permit Consolidation

DRAWING TITLE

SRA SECTION LOCATION

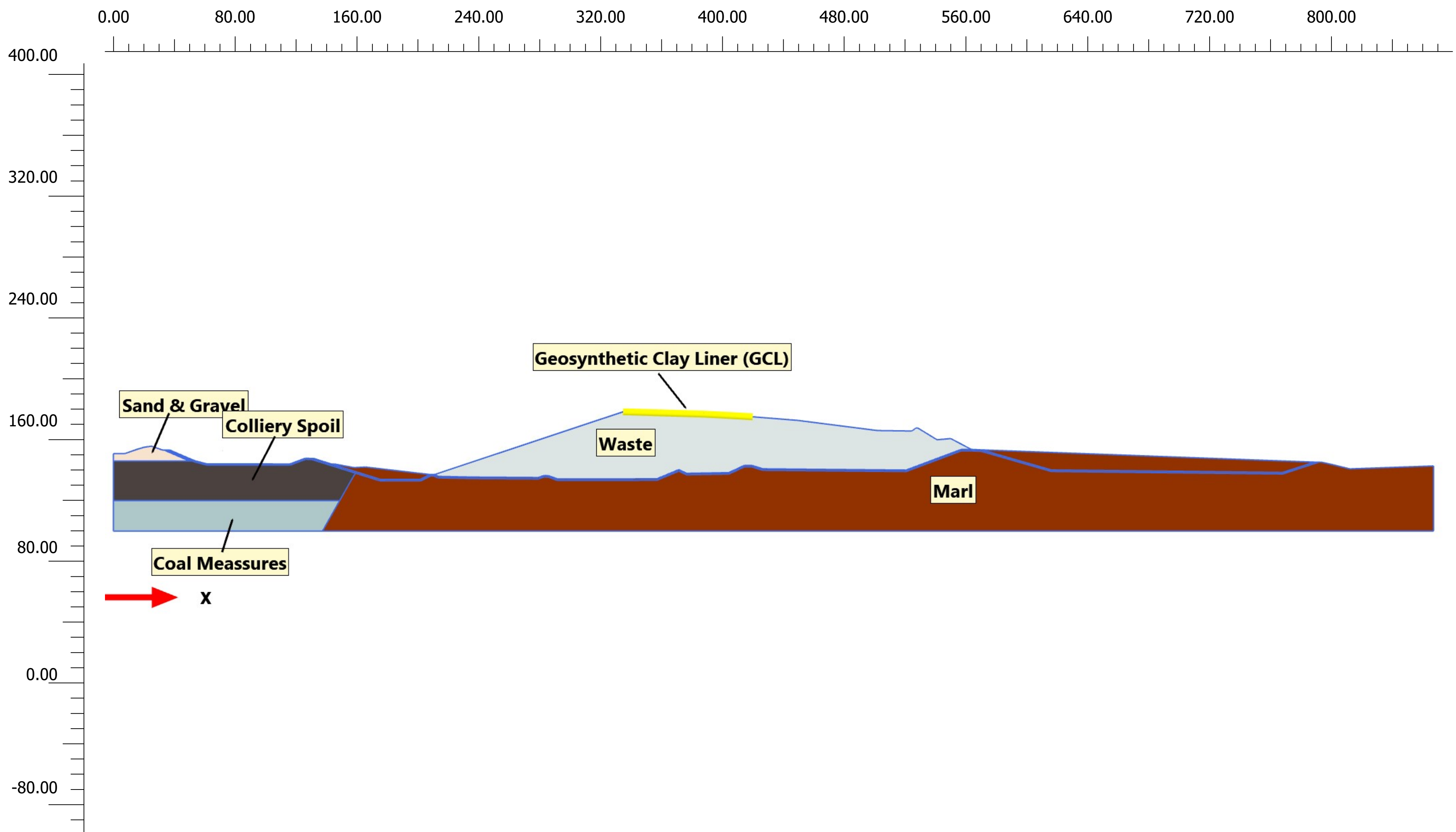
DRAWN	DATE	APPROVED	DATE
M.C.	10/11/2021	J.D.	10/11/2021

SCALE	SHEET	DRAWING NUMBER	REVISION
1:2000	A1	BF5048/SRA/01	0

APPENDIX SRA2

MODEL GEOMETRY AND MATERIAL PARAMETERS

Output Version 21.1.0.479



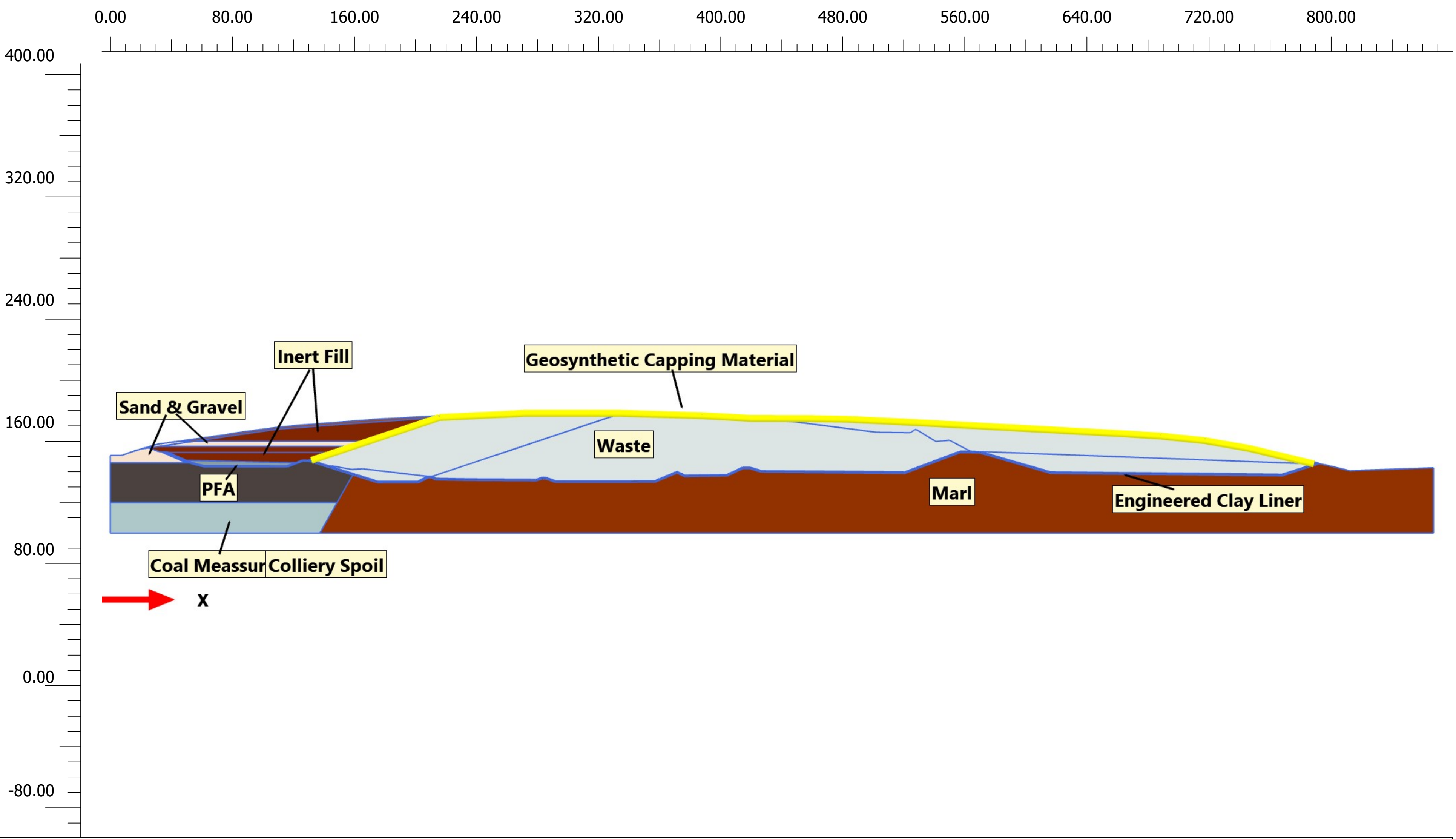
Connectivity plot

PLAXIS® 2D
CONNECT Edition

Project description	Date
Poplars EP Consolidation	10/11/2021

Project filename	Step	Company
Poplars EPVA Revised S ...	1	Sirius Environmental Ltd

Output Version 21.1.0.479



Connectivity plot

PLAXIS® 2D
CONNECT Edition

Project description			Date
Poplars EP Consolidation			10/11/2021
Project filename	Step	Company	
Poplars EPVA Revised S ...	976	Sirius Environmental Ltd	

Material set				
Identification number		1	2	3
Identification		Waste	Colliery Spoil	Sands and Gravels
Material model		Hardening soil	Hardening soil	Hardening soil
Drainage type		Undrained (A)	Undrained (A)	Undrained (A)
Colour		RGB 213, 220, 220	RGB 72, 63, 61	RGB 240, 219, 199
Comments				
General properties				
γ_{unsat}	kN/m³	10.00	20.00	18.00
γ_{sat}	kN/m³	10.50	22.00	18.00
Advanced				
Void ratio				
Dilatancy cut-off		No	No	No
e_{init}		0.5000	0.5000	0.5000
e_{min}		0.000	0.000	0.000
e_{max}		999.0	999.0	999.0
Damping				
Rayleigh α		0.000	0.000	0.000
Rayleigh β		0.000	0.000	0.000
Stiffness				
E_{50}^{ref}	kN/m²	6000	4000	10.00E3
E_{oed}^{ref}	kN/m²	6000	4000	10.00E3
E_{ur}^{ref}	kN/m²	18.00E3	12.00E3	30.00E3
power (m)		0.5000	0.7500	0.5000
Alternatives				
Use alternatives		No	No	No
C_c		0.05750	0.08625	0.03450
C_s		0.01725	0.02587	0.01035
e_{init}		0.5000	0.5000	0.5000
Strength				
c_{ref}	kN/m²	5.000	5.000	0.5000
φ (phi)	°	25.00	25.00	33.00
ψ (psi)	°	0.000	0.000	0.000

Identification		Waste	Colliery Spoil	Sands and Gravels
Advanced				
Set to default values		Yes	Yes	Yes
Stiffness				
ν_{ur}		0.2000	0.2000	0.2000
p_{ref}	kN/m²	100.0	100.0	100.0
K_0^{nc}		0.5774	0.5774	0.4554
Strength				
c_{inc}	kN/m²/m	0.000	0.000	0.000
y_{ref}	m	0.000	0.000	0.000
R_f		0.9000	0.9000	0.9000
Tension cut-off		Yes	Yes	Yes
Tensile strength	kN/m²	0.000	0.000	0.000
Undrained behaviour				
Undrained behaviour		Standard	Standard	Standard
Skempton-B		0.9866	0.9866	0.9866
ν_u		0.4950	0.4950	0.4950
$K_{w,ref}$ / n	kN/m²	737.5E3	491.7E3	1.229E6
Stiffness				
Stiffness		Standard	Standard	Standard
Strength				
Strength		Rigid	Rigid	Rigid
R_{inter}		1.000	1.000	1.000
Consider gap closure		Yes	Yes	Yes
Real interface thickness				
δ_{inter}		0.000	0.000	0.000
Groundwater				
Cross permeability		Impermeable	Impermeable	Impermeable
Drainage conductivity, dk	m³/day/m	0.000	0.000	0.000
Thermal				
R	m² K/kW	0.000	0.000	0.000

Identification		Waste	Colliery Spoil	Sands and Gravels
K0 settings				
K_0 determination		Automatic	Automatic	Automatic
$K_{0,x} = K_{0,z}$		Yes	Yes	Yes
$K_{0,x}$		0.5774	0.5774	0.4554
$K_{0,z}$		0.5774	0.5774	0.4554
Overconsolidation				
OCR		1.000	1.000	1.000
POP	kN/m²	0.000	0.000	0.000
Model				
Data set		Standard	Standard	Standard
Soil				
Type		Coarse	Coarse	Coarse
< 2 μm	%	10.00	10.00	10.00
2 μm - 50 μm	%	13.00	13.00	13.00
50 μm - 2 mm	%	77.00	77.00	77.00
Flow parameters				
Use defaults		None	None	None
k_x	m/day	0.8640	8.640E-3	86.40
k_y	m/day	0.8640	8.640E-3	86.40
$-\psi_{unsat}$	m	10.00E3	10.00E3	10.00E3
e_{init}		0.5000	0.5000	0.5000
S_s	1/m	0.000	0.000	0.000
Change of permeability				
c_k		1000E12	1000E12	1000E12

Identification		Waste	Colliery Spoil	Sands and Gravels
Parameters				
c_s	kJ/t/K	0.000	0.000	0.000
λ_s	kW/m/K	0.000	0.000	0.000
ρ_s	t/m³	0.000	0.000	0.000
Solid thermal expansion		Volumetric	Volumetric	Volumetric
α_s	1/K	0.000	0.000	0.000
D_v	m²/day	0.000	0.000	0.000
f_{Tv}		0.000	0.000	0.000
Unfrozen water content		None	None	None

Material set				
Identification number		4	6	7
Identification		PFA	Engineered Fill	Inert Fill
Material model		Hardening soil	Hardening soil	Hardening soil
Drainage type		Undrained (A)	Undrained (A)	Undrained (A)
Colour		RGB 135, 154, 155	RGB 103, 55, 25	RGB 126, 37, 1
Comments				
General properties				
γ_{unsat}	kN/m³	16.00	21.00	18.00
γ_{sat}	kN/m³	17.00	21.00	20.00
Advanced				
Void ratio				
Dilatancy cut-off		No	No	No
e_{init}		0.5000	0.5000	0.5000
e_{min}		0.000	0.000	0.000
e_{max}		999.0	999.0	999.0
Damping				
Rayleigh α		0.000	0.000	0.000
Rayleigh β		0.000	0.000	0.000
Stiffness				
E_{50}^{ref}	kN/m²	6000	5000	4000
E_{oed}^{ref}	kN/m²	6000	5000	4000
E_{ur}^{ref}	kN/m²	18.00E3	15.00E3	12.00E3
power (m)		0.5000	0.7500	0.7500
Alternatives				
Use alternatives		No	No	No
C_c		0.05750	0.06900	0.08625
C_s		0.01725	0.02070	0.02587
e_{init}		0.5000	0.5000	0.5000
Strength				
c_{ref}	kN/m²	3.000	5.000	5.000
φ (phi)	°	27.00	25.00	25.00
ψ (psi)	°	0.000	0.000	0.000

Identification		PFA	Engineered Fill	Inert Fill
Advanced				
Set to default values		Yes	Yes	Yes
Stiffness				
ν_{ur}		0.2000	0.2000	0.2000
p_{ref}	kN/m²	100.0	100.0	100.0
K_0^{nc}		0.5460	0.5774	0.5774
Strength				
c_{inc}	kN/m²/m	0.000	0.000	0.000
y_{ref}	m	0.000	0.000	0.000
R_f		0.9000	0.9000	0.9000
Tension cut-off		Yes	Yes	Yes
Tensile strength	kN/m²	0.000	0.000	0.000
Undrained behaviour				
Undrained behaviour		Standard	Standard	Standard
Skempton-B		0.9866	0.9866	0.9866
ν_u		0.4950	0.4950	0.4950
$K_{w,ref}$ / n	kN/m²	737.5E3	614.6E3	491.7E3
Stiffness				
Stiffness		Standard	Standard	Standard
Strength				
Strength		Rigid	Rigid	Rigid
R_{inter}		1.000	1.000	1.000
Consider gap closure		Yes	Yes	Yes
Real interface thickness				
δ_{inter}		0.000	0.000	0.000
Groundwater				
Cross permeability		Impermeable	Impermeable	Impermeable
Drainage conductivity, dk	m³/day/m	0.000	0.000	0.000
Thermal				
R	m² K/kW	0.000	0.000	0.000

Identification		PFA	Engineered Fill	Inert Fill
K0 settings				
K_0 determination		Automatic	Automatic	Automatic
$K_{0,x} = K_{0,z}$		Yes	Yes	Yes
$K_{0,x}$		0.5460	0.5774	0.5774
$K_{0,z}$		0.5460	0.5774	0.5774
Overconsolidation				
OCR		1.000	1.000	1.000
POP	kN/m²	0.000	0.000	0.000
Model				
Data set		Standard	Standard	Standard
Soil				
Type		Coarse	Coarse	Coarse
< 2 µm	%	10.00	10.00	10.00
2 µm - 50 µm	%	13.00	13.00	13.00
50 µm - 2 mm	%	77.00	77.00	77.00
Flow parameters				
Use defaults		None	None	None
k_x	m/day	0.03456	0.08640E-3	0.4320E-3
k_y	m/day	0.03456	0.08640E-3	0.4320E-3
$-\psi_{unsat}$	m	10.00E3	10.00E3	10.00E3
e_{init}		0.5000	0.5000	0.5000
S_s	1/m	0.000	0.000	0.000
Change of permeability				
c_k		1000E12	1000E12	1000E12

Identification		PFA	Engineered Fill	Inert Fill
Parameters				
c_s	kJ/t/K	0.000	0.000	0.000
λ_s	kW/m/K	0.000	0.000	0.000
ρ_s	t/m³	0.000	0.000	0.000
Solid thermal expansion		Volumetric	Volumetric	Volumetric
α_s	1/K	0.000	0.000	0.000
D_v	m²/day	0.000	0.000	0.000
f_{Tv}		0.000	0.000	0.000
Unfrozen water content		None	None	None

Material set		
Identification number		8
Identification		Restoration Soils
Material model		Hardening soil
Drainage type		Undrained (A)
Colour		RGB 166, 71, 8
Comments		
General properties		
γ_{unsat}	kN/m³	18.00
γ_{sat}	kN/m³	20.00
Advanced		
Void ratio		
Dilatancy cut-off		No
e_{init}		0.5000
e_{min}		0.000
e_{max}		999.0
Damping		
Rayleigh α		0.000
Rayleigh β		0.000
Stiffness		
E_{50}^{ref}	kN/m²	5000
E_{oed}^{ref}	kN/m²	5000
E_{ur}^{ref}	kN/m²	15.00E3
power (m)		0.7500
Alternatives		
Use alternatives		No
C_c		0.06900
C_s		0.02070
e_{init}		0.5000
Strength		
c_{ref}	kN/m²	5.000
φ (phi)	°	25.00
ψ (psi)	°	0.000

Identification		Restoration Soils
Advanced		
Set to default values		Yes
Stiffness		
ν_{ur}		0.2000
p_{ref}	kN/m²	100.0
K_0^{nc}		0.5774
Strength		
c_{inc}	kN/m²/m	0.000
y_{ref}	m	0.000
R_f		0.9000
Tension cut-off		Yes
Tensile strength	kN/m²	0.000
Undrained behaviour		
Undrained behaviour		Standard
Skempton-B		0.9866
ν_u		0.4950
$K_{w,ref}$ / n	kN/m²	614.6E3
Stiffness		
Stiffness		Standard
Strength		
Strength		Rigid
R_{inter}		1.000
Consider gap closure		Yes
Real interface thickness		
δ_{inter}		0.000
Groundwater		
Cross permeability		Impermeable
Drainage conductivity, dk	m³/day/m	0.000
Thermal		
R	m² K/kW	0.000

Identification		Restoration Soils
K0 settings		
K_0 determination		Automatic
$K_{0,x} = K_{0,z}$		Yes
$K_{0,x}$		0.5774
$K_{0,z}$		0.5774
Overconsolidation		
OCR		1.000
POP	kN/m²	0.000
Model		
Data set		Standard
Soil		
Type		Coarse
< 2 μm	%	10.00
2 μm - 50 μm	%	13.00
50 μm - 2 mm	%	77.00
Flow parameters		
Use defaults		None
k_x	m/day	0.8640E-3
k_y	m/day	0.8640E-3
$-\psi_{unsat}$	m	10.00E3
e_{init}		0.5000
S_s	1/m	0.000
Change of permeability		
c_k		1000E12

Identification		Restoration Soils
Parameters		
c_s	kJ/t/K	0.000
λ_s	kW/m/K	0.000
ρ_s	t/m³	0.000
Solid thermal expansion		Volumetric
α_s	1/K	0.000
D_v	m²/day	0.000
f_{Tv}		0.000
Unfrozen water content		None

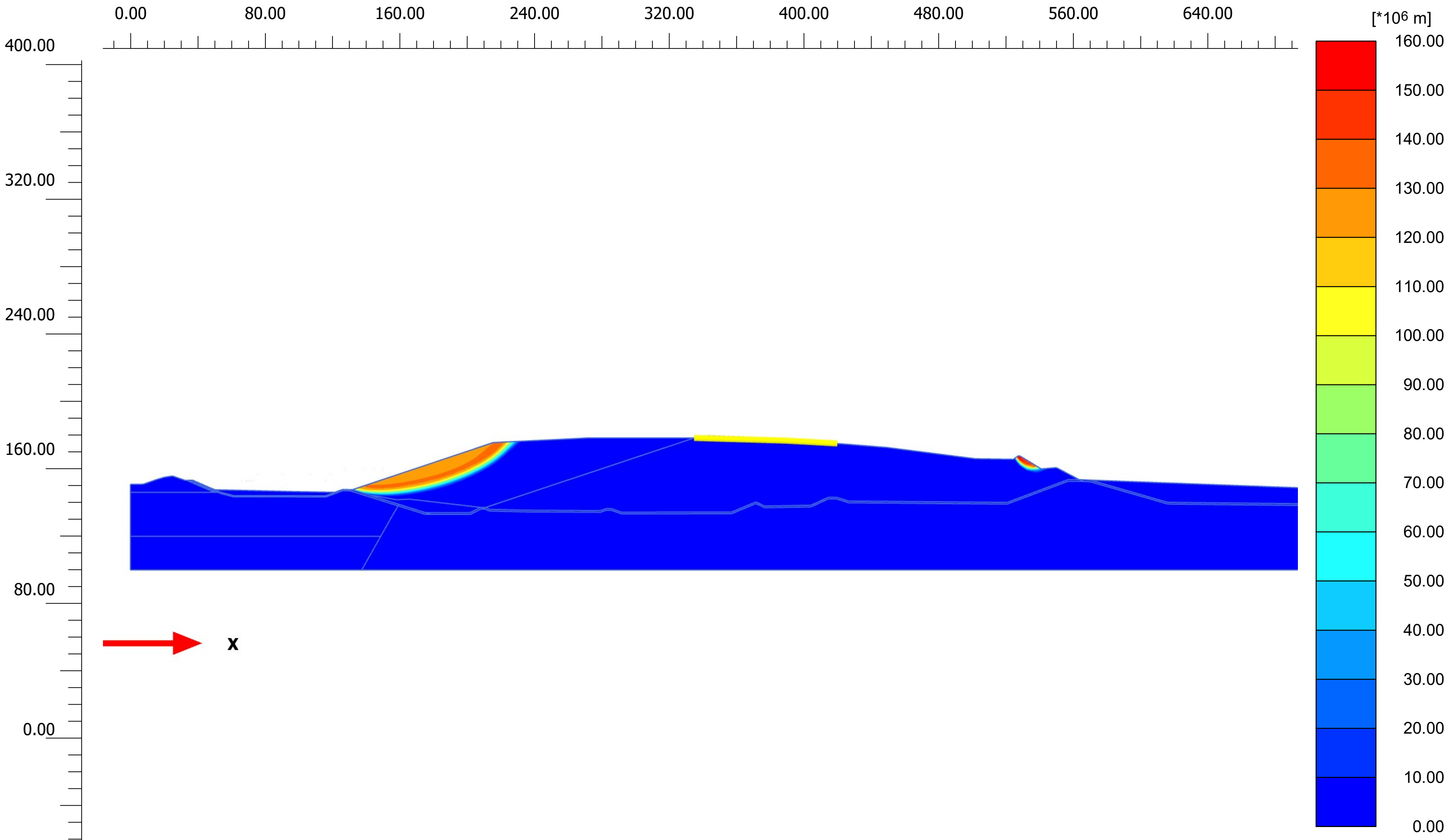
Material set		
Identification number		1
Identification		Geosynthetic Clay Liners (GCL)
Comments		
Colour		RGB 255, 255, 0
Material type		Elastic
Properties		
Isotropic		Yes
EA_1	kN/m	120.0
EA_2	kN/m	120.0
Parameters		
Identification number		1
c	kJ/t/K	0.000
λ	kW/m/K	0.000
ρ	t/m^3	0.000
α	1/K	0.000
Parameters		
Identification number		1
c	kJ/t/K	0.000
λ	kW/m/K	0.000
ρ	t/m^3	0.000
α	1/K	0.000
A	m^2	0.000

Material set		
Identification number		1
Identification		Geomembrane
Comments		
Colour		RGB 255, 255, 0
Material type		Elastic
Properties		
Isotropic		Yes
EA_1	kN/m	37.50
EA_2	kN/m	37.50
Parameters		
Identification number		1
c	kJ/t/K	0.000
λ	kW/m/K	0.000
ρ	t/m³	0.000
α	1/K	0.000
Parameters		
Identification number		1
c	kJ/t/K	0.000
λ	kW/m/K	0.000
ρ	t/m³	0.000
α	1/K	0.000
A	m²	0.000

APPENDIX SRA3

PLAXIS STABILITY PRINTOUTS

Output Version 21.1.0.479



Incremental displacements |Δu| (scaled up 0.200*10^{-6} times)

Maximum value = 157.9*10^{6} m (Element 2375 at Node 9360)

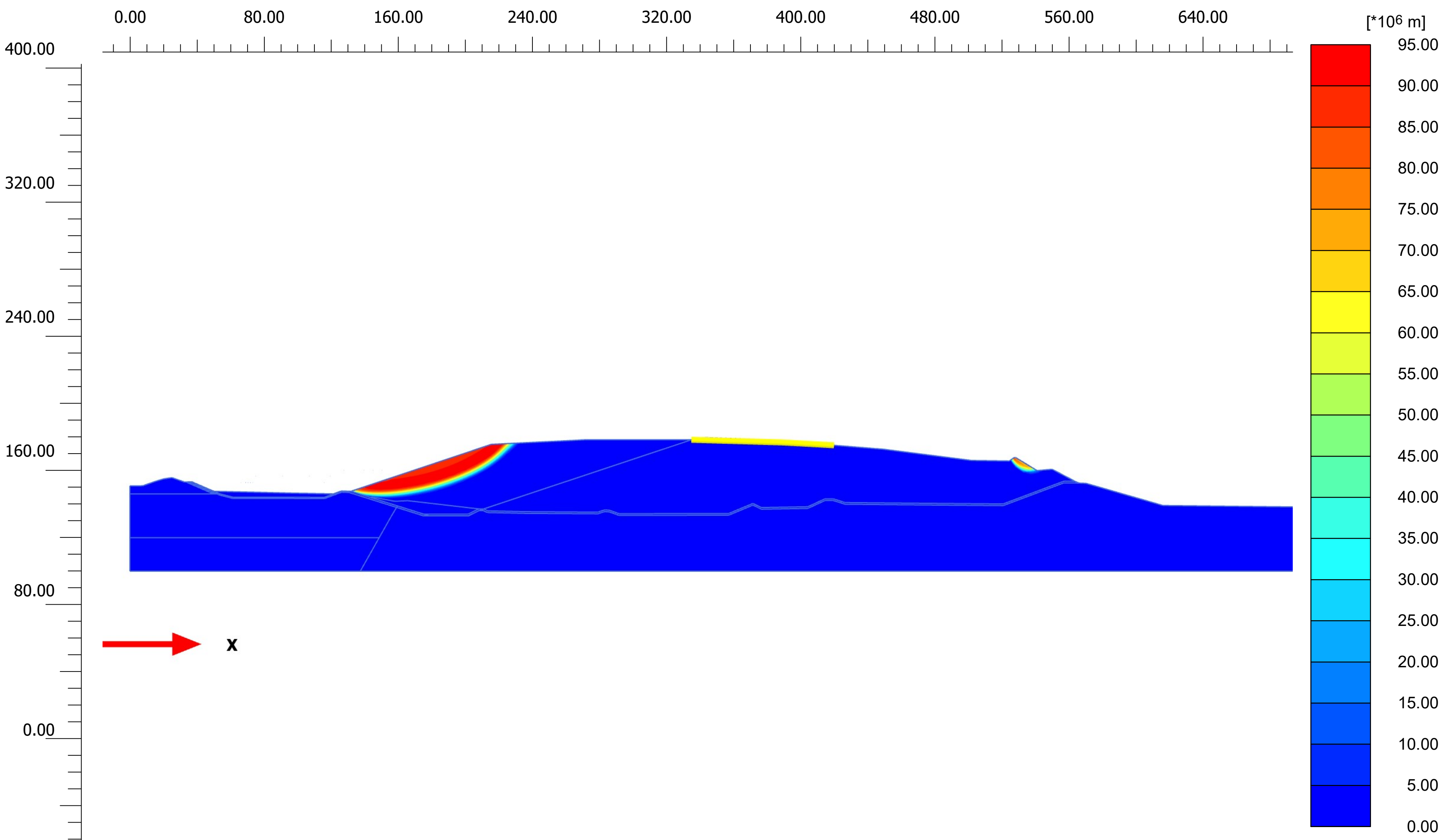
PLAXIS® 2D CONNECT Edition

Project description		Date
Poplars EP Consolidation		27/10/2021

Project filename	Step	Company
Poplars EPVA Revised S ...	439	Sirius Environmental Ltd

Step info				
Phase	Waste Safety [Phase_7]			
Step	Initial			
Calulation mode	Classical mode			
Step type	Safety			
Updated mesh	False			
Solver type	Picos			
Kernel type	64 bit			
Extrapolation factor	2.000			
Relative stiffness	0.06492E-15			
Multipliers				
Soil weight			ΣM_{Weight}	1.000
Strength reduction factor	M_{sf}	0.1101E-3	ΣM_{sf}	1.656
Time	Increment	0.000	End time	2418
Staged construction				
Active proportion total area	M_{Area}	0.000	ΣM_{Area}	0.8768
Active proportion of stage	M_{Stage}	0.000	ΣM_{Stage}	0.000
Forces				
F_X	0.000 kN/m			
F_Y	0.000 kN/m			
Consolidation				
Realised $P_{Excess,Max}$	1116 kN/m^2			

Output Version 21.1.0.479



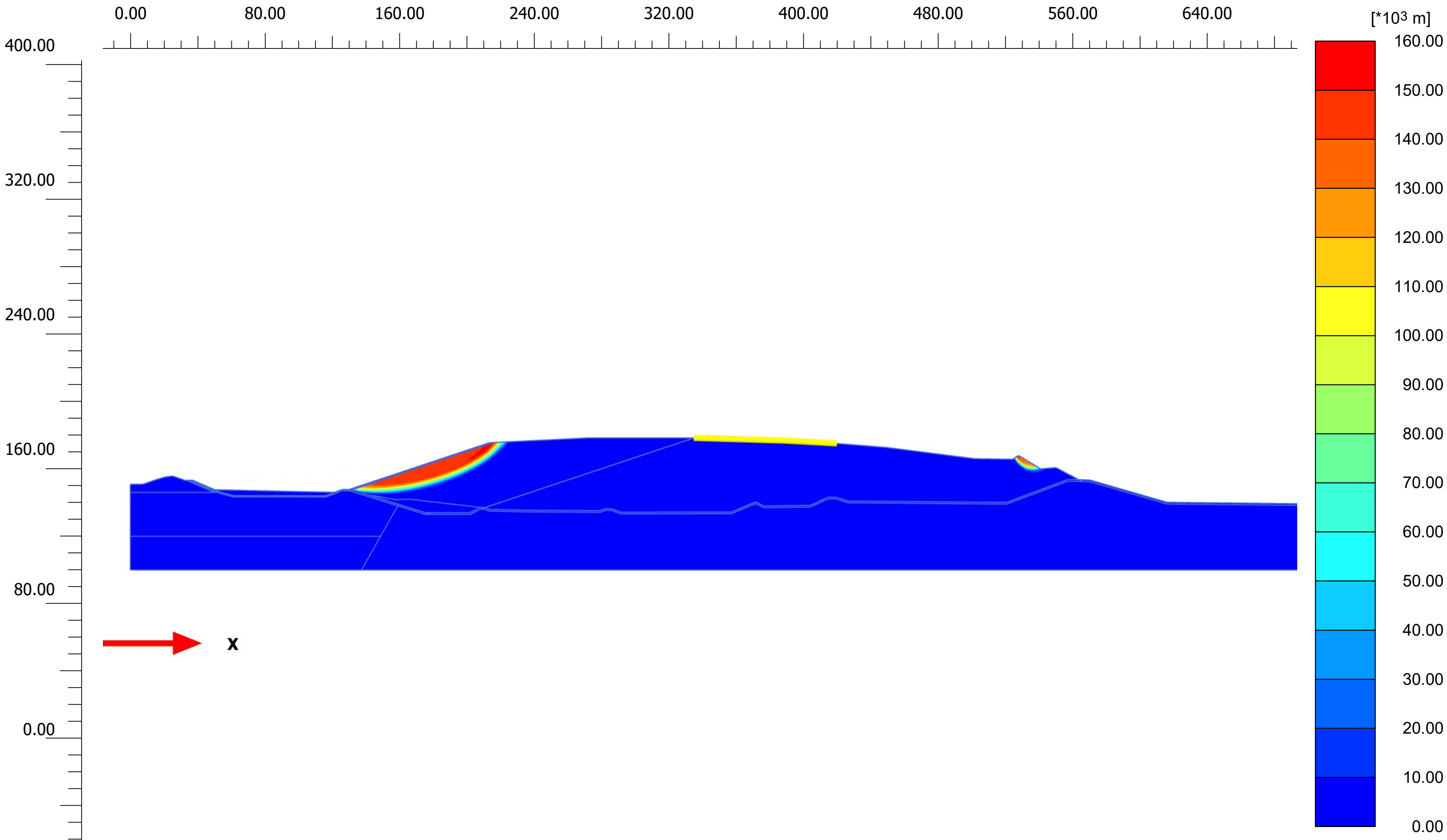
Incremental displacements |Δu| (scaled up $0.200*10^{-6}$ times)

Maximum value = $94.84*10^{6}$ m (Element 1878 at Node 33266)

PLAXIS® 2D CONNECT Edition	Project description: Poplars EP Consolidation		Date: 27/10/2021
	Project filename: Poplars EPVA Revised S ...	Step: 542	Company: Sirius Environmental Ltd

Step info				
Phase	Excavate Subgrade C1 Safety [Phase_9]			
Step	Initial			
Calulation mode	Classical mode			
Step type	Safety			
Updated mesh	False			
Solver type	Picos			
Kernel type	64 bit			
Extrapolation factor	0.5000			
Relative stiffness	0.03866E-15			
Multipliers				
Soil weight			ΣM_{Weight}	1.000
Strength reduction factor	M_{sf}	-0.1502E-3	ΣM_{sf}	1.654
Time	Increment	0.000	End time	2478
Staged construction				
Active proportion total area	M_{Area}	0.000	ΣM_{Area}	0.8426
Active proportion of stage	M_{Stage}	0.000	ΣM_{Stage}	0.000
Forces				
F_X	0.000 kN/m			
F_Y	0.000 kN/m			
Consolidation				
Realised $P_{Excess,Max}$	1181 kN/m^2			

Output Version 21.1.0.479



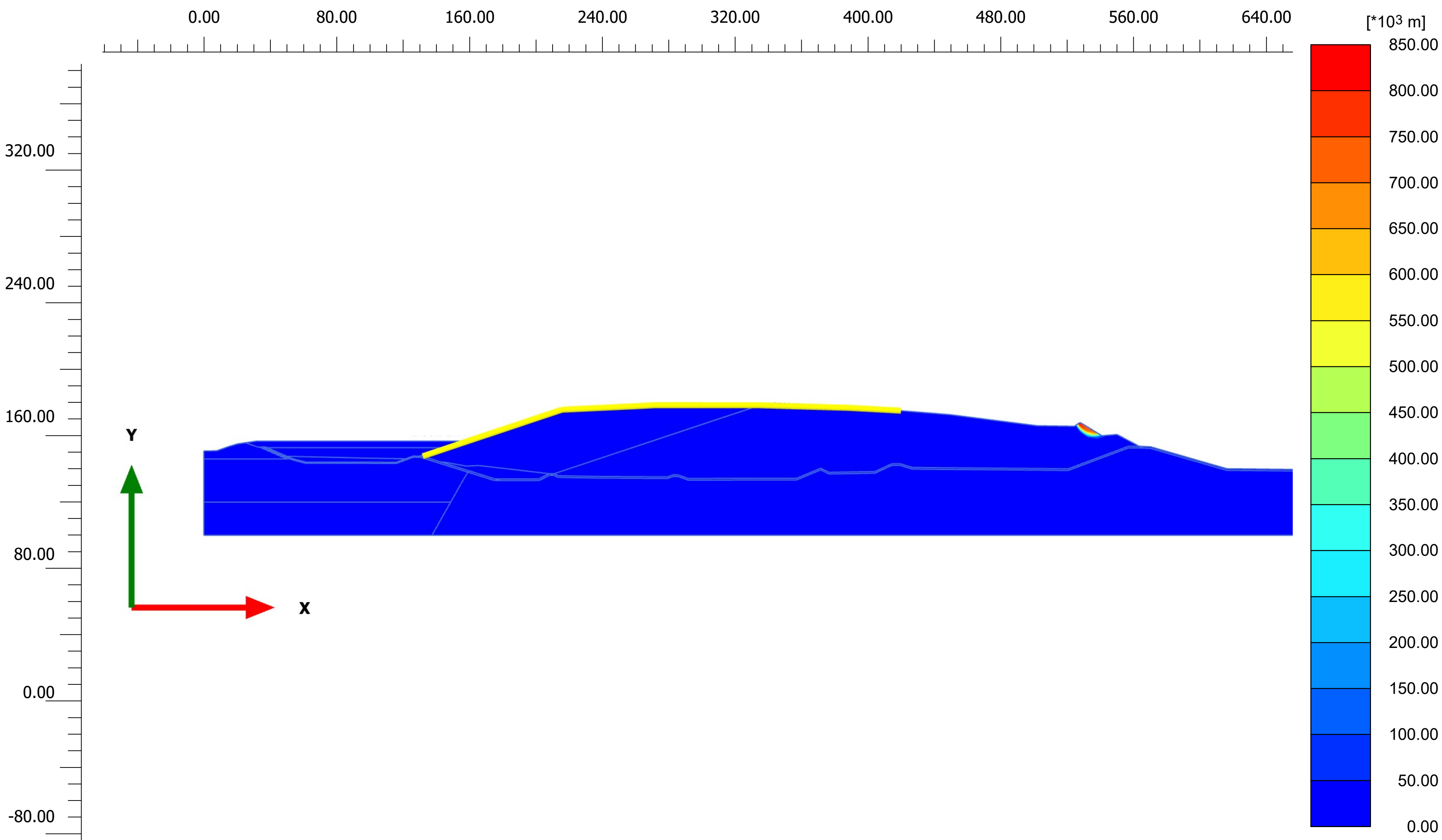
Incremental displacements |Δu| (scaled up 0.200*10^-3 times)

Maximum value = 154.9*10^3 m (Element 181 at Node 33018)

PLAXIS® 2D CONNECT Edition	*Project description* Poplars EP Consolidation		*Date* 27/10/2021
	Project filename Poplars EPVA Revised S ...	*Step* 654	*Company* Sirius Environmental Ltd

Step info				
Phase	Clay Liner C1 Safety [Phase_15]			
Step	Initial			
Calulation mode	Classical mode			
Step type	Safety			
Updated mesh	False			
Solver type	Picos			
Kernel type	64 bit			
Extrapolation factor	0.5000			
Relative stiffness	0.02022E-12			
Multipliers				
Soil weight			ΣM_{Weight}	1.000
Strength reduction factor	M_{sf}	0.04623E-3	ΣM_{sf}	1.653
Time	Increment	0.000	End time	2553
Staged construction				
Active proportion total area	M_{Area}	0.000	ΣM_{Area}	0.8474
Active proportion of stage	M_{Stage}	0.000	ΣM_{Stage}	0.000
Forces				
F_X	0.000 kN/m			
F_Y	0.000 kN/m			
Consolidation				
Realised $P_{Excess,Max}$	630.9 kN/m^2			

Output Version 21.1.0.479



Incremental displacements |Δu| (scaled up 0.0500×10^{-3} times)

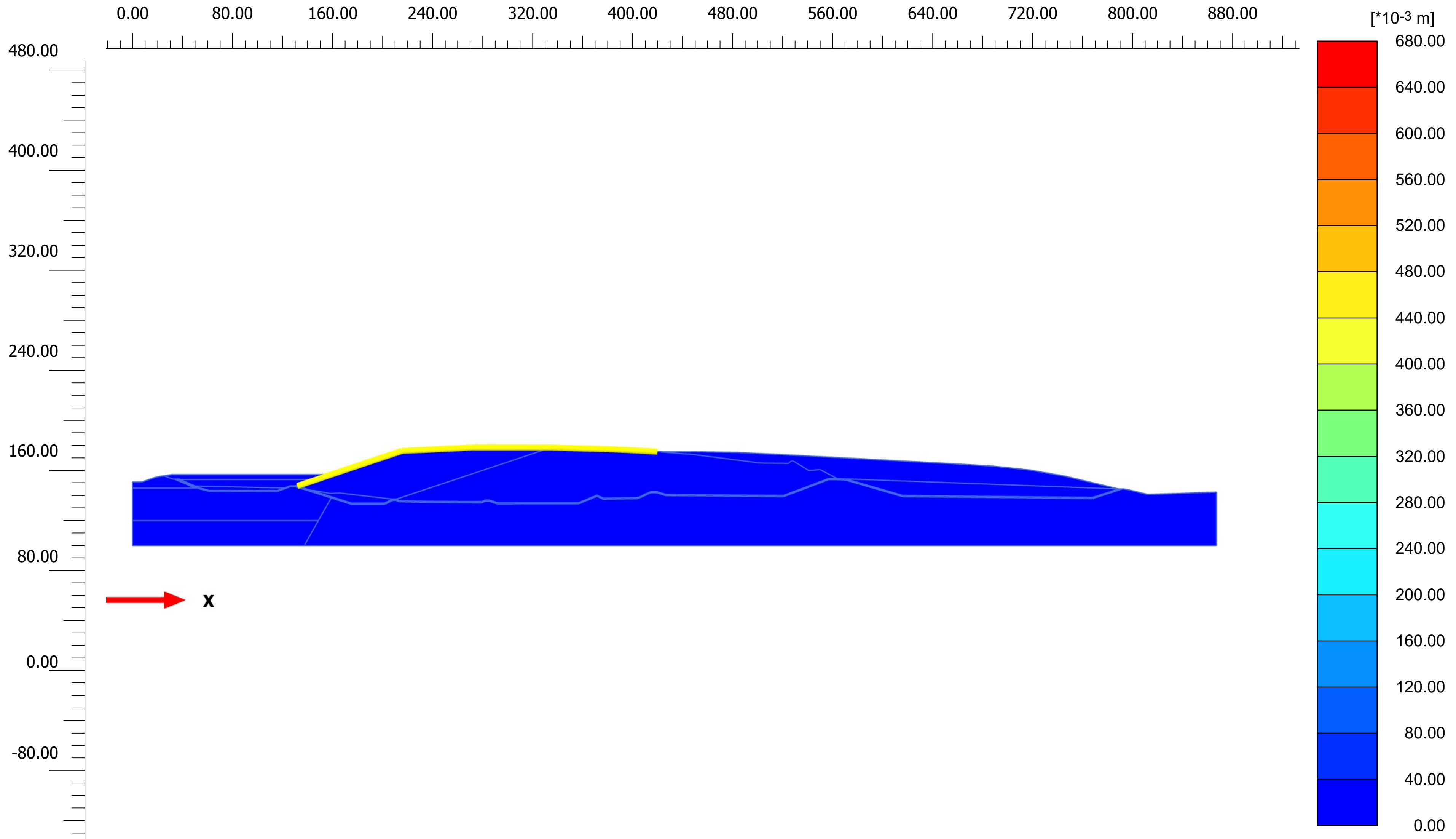
Maximum value = 808.0×10^{3} m (Element 2375 at Node 9359)

PLAXIS® 2D CONNECT Edition

Project description Poplars EP Consolidation			*Date* 27/10/2021
Project filename Poplars EPVA Revised S ...	*Step* 757	*Company* Sirius Environmental Ltd	

Step info				
Phase	Inert Wedge Safety [Phase_17]			
Step	Initial			
Calulation mode	Classical mode			
Step type	Safety			
Updated mesh	False			
Solver type	Picos			
Kernel type	64 bit			
Extrapolation factor	0.5000			
Relative stiffness	-0.07053E-15			
Multipliers				
Soil weight			ΣM_{Weight}	1.000
Strength reduction factor	M_{sf}	0.02303E-3	ΣM_{sf}	1.624
Time	Increment	0.000	End time	2593
Staged construction				
Active proportion total area	M_{Area}	0.000	ΣM_{Area}	0.8647
Active proportion of stage	M_{Stage}	0.000	ΣM_{Stage}	0.000
Forces				
F_X	0.000 kN/m			
F_Y	0.000 kN/m			
Consolidation				
Realised $P_{Excess,Max}$	393.6 kN/m^2			

Output Version 21.1.0.479

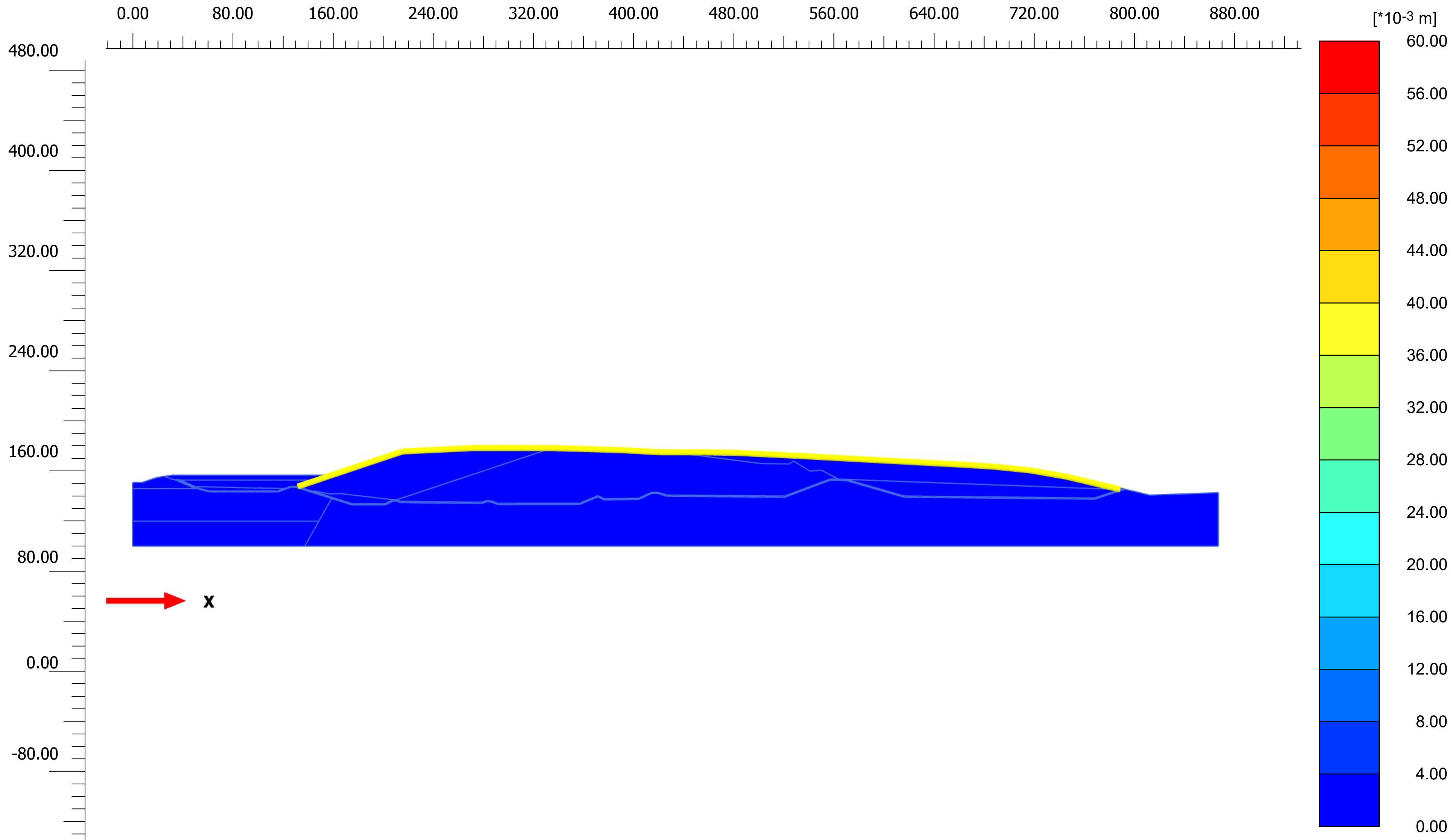


Incremental displacements |Δu| (scaled up 50.0 times)

Maximum value = 0.6651 m (Element 181 at Node 33010)

PLAXIS® 2D CONNECT Edition	*Project description* Poplars EP Consolidation		*Date* 27/10/2021
	Project filename Poplars EPVA Revised S ...	*Step* 865	*Company* Sirius Environmental Ltd

Step info				
Phase	Waste C1 Safety [Phase_19]			
Step	Initial			
Calulation mode	Classical mode			
Step type	Safety			
Updated mesh	False			
Solver type	Picos			
Kernel type	64 bit			
Extrapolation factor	0.5000			
Relative stiffness	0.07850E-3			
Multipliers				
Soil weight			ΣM_{Weight}	1.000
Strength reduction factor	M_{sf}	7.677E-3	ΣM_{sf}	1.679
Time	Increment	0.000	End time	3313
Staged construction				
Active proportion total area	M_{Area}	0.000	ΣM_{Area}	0.9582
Active proportion of stage	M_{Stage}	0.000	ΣM_{Stage}	0.000
Forces				
F_X	0.000 kN/m			
F_Y	0.000 kN/m			
Consolidation				
Realised $P_{Excess,Max}$	149.7 kN/m^2			



Incremental displacements |Δu| (scaled up 500 times)

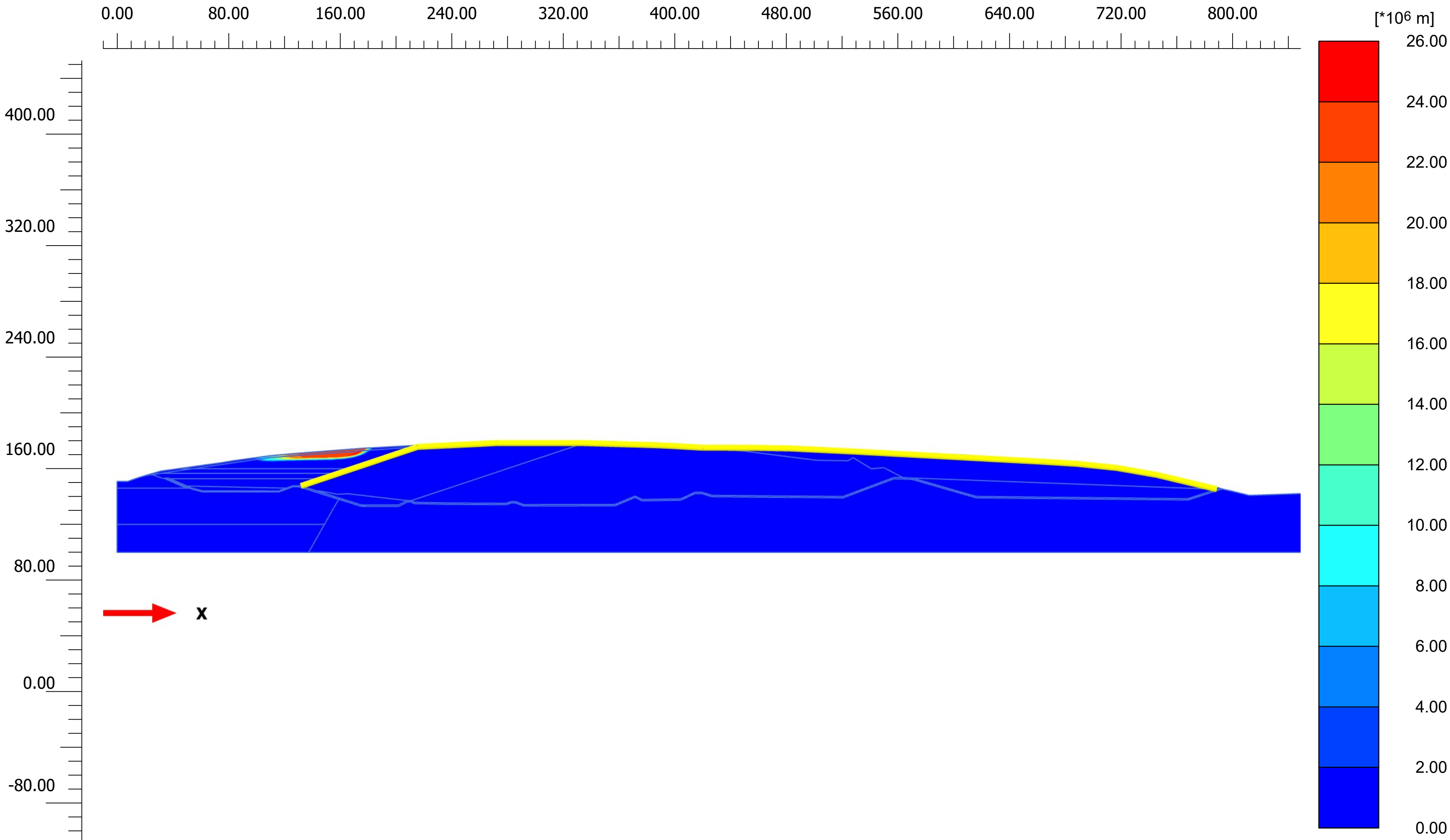
Maximum value = 0.05724 m (Element 95 at Node 33010)

PLAXIS® 2D CONNECT Edition

Project description		Date
Poplars EP Consolidation		27/10/2021
Project filename	Step	Company
Poplars EPVA Revised S ...	968	Sirius Environmental Ltd

Step info				
Phase	Capping C1 Safety [Phase_22]			
Step	Initial			
Calulation mode	Classical mode			
Step type	Safety			
Updated mesh	False			
Solver type	Picos			
Kernel type	64 bit			
Extrapolation factor	1.000			
Relative stiffness	0.1594E-3			
Multipliers				
Soil weight			ΣM_{Weight}	1.000
Strength reduction factor	M_{sf}	5.000E-3	ΣM_{sf}	1.420
Time	Increment	0.000	End time	3343
Staged construction				
Active proportion total area	M_{Area}	0.000	ΣM_{Area}	0.9702
Active proportion of stage	M_{Stage}	0.000	ΣM_{Stage}	0.000
Forces				
F_X	0.000 kN/m			
F_Y	0.000 kN/m			
Consolidation				
Realised $P_{Excess,Max}$	142.3 kN/m^2			

Output Version 21.1.0.479



Incremental displacements |Δu| (scaled up $1.00*10^{-6}$ times)

Maximum value = $24.29*10^{6}$ m (Element 788 at Node 44082)

PLAXIS® 2D
CONNECT Edition

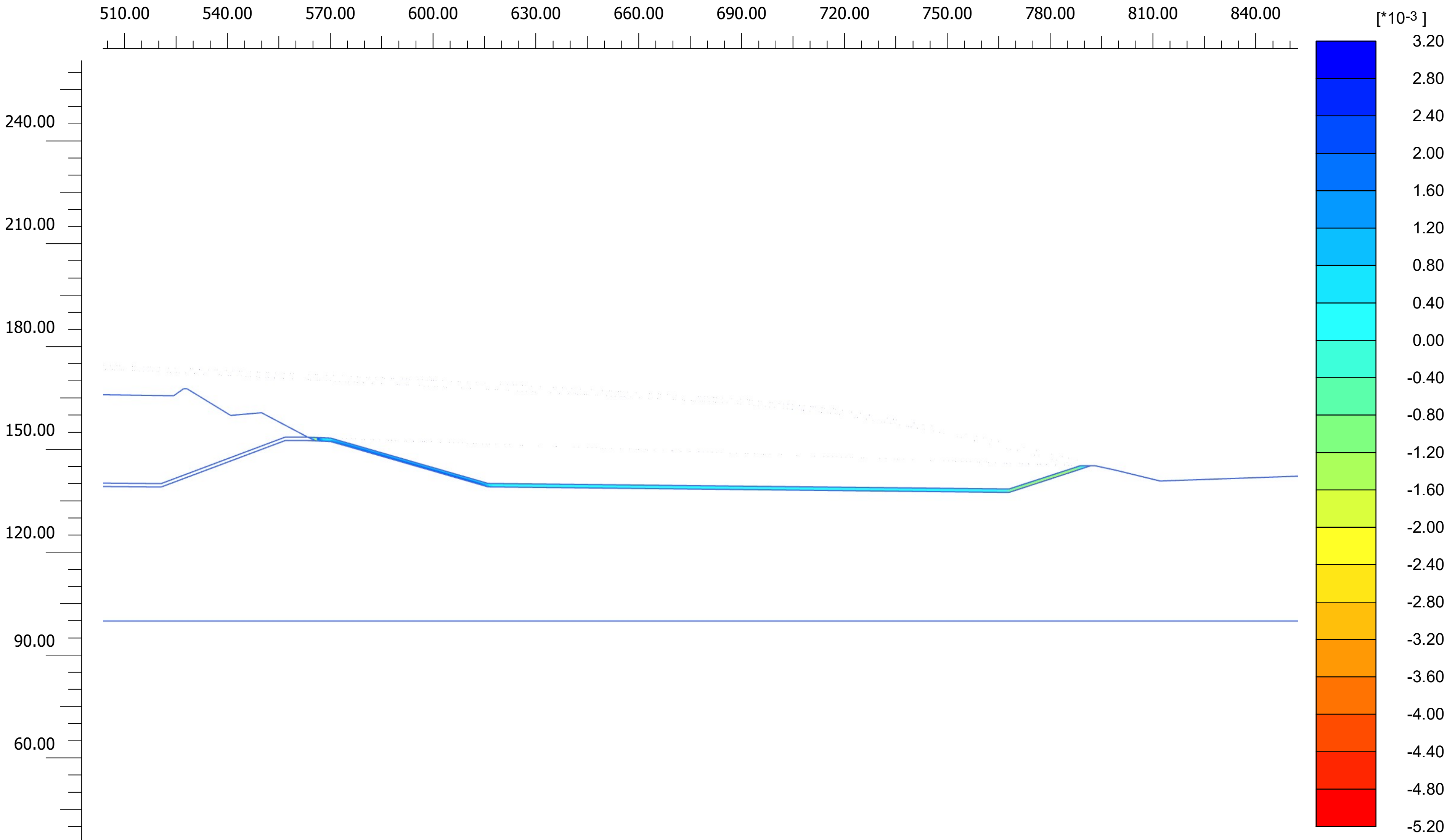
Project description		*Date*
Poplars EP Consolidation		27/10/2021
Project filename	*Step*	*Company*
Poplars EPVA Revised S ...	1076	Sirius Environmental Ltd

Step info				
Phase	Inert Wedge C2 Safety [Phase_24]			
Step	Initial			
Calulation mode	Classical mode			
Step type	Safety			
Updated mesh	False			
Solver type	Picos			
Kernel type	64 bit			
Extrapolation factor	0.5000			
Relative stiffness	0.02201E-15			
Multipliers				
Soil weight			ΣM_{Weight}	1.000
Strength reduction factor	M_{sf}	0.01344E-3	ΣM_{sf}	1.794
Time	Increment	0.000	End time	3523
Staged construction				
Active proportion total area	M_{Area}	0.000	ΣM_{Area}	1.000
Active proportion of stage	M_{Stage}	0.000	ΣM_{Stage}	0.000
Forces				
F_X	0.000 kN/m			
F_Y	0.000 kN/m			
Consolidation				
Realised $P_{Excess,Max}$	270.2 kN/m^2			

APPENDIX SRA4

PLAXIS BASAL AND SIDE SLOPE LINER INTEGRITY PRINTOUT

Output Version 21.1.0.479



Total cartesian strain γ_{xy} (scaled up 5.00×10^{3} times) (Time 2553 day)

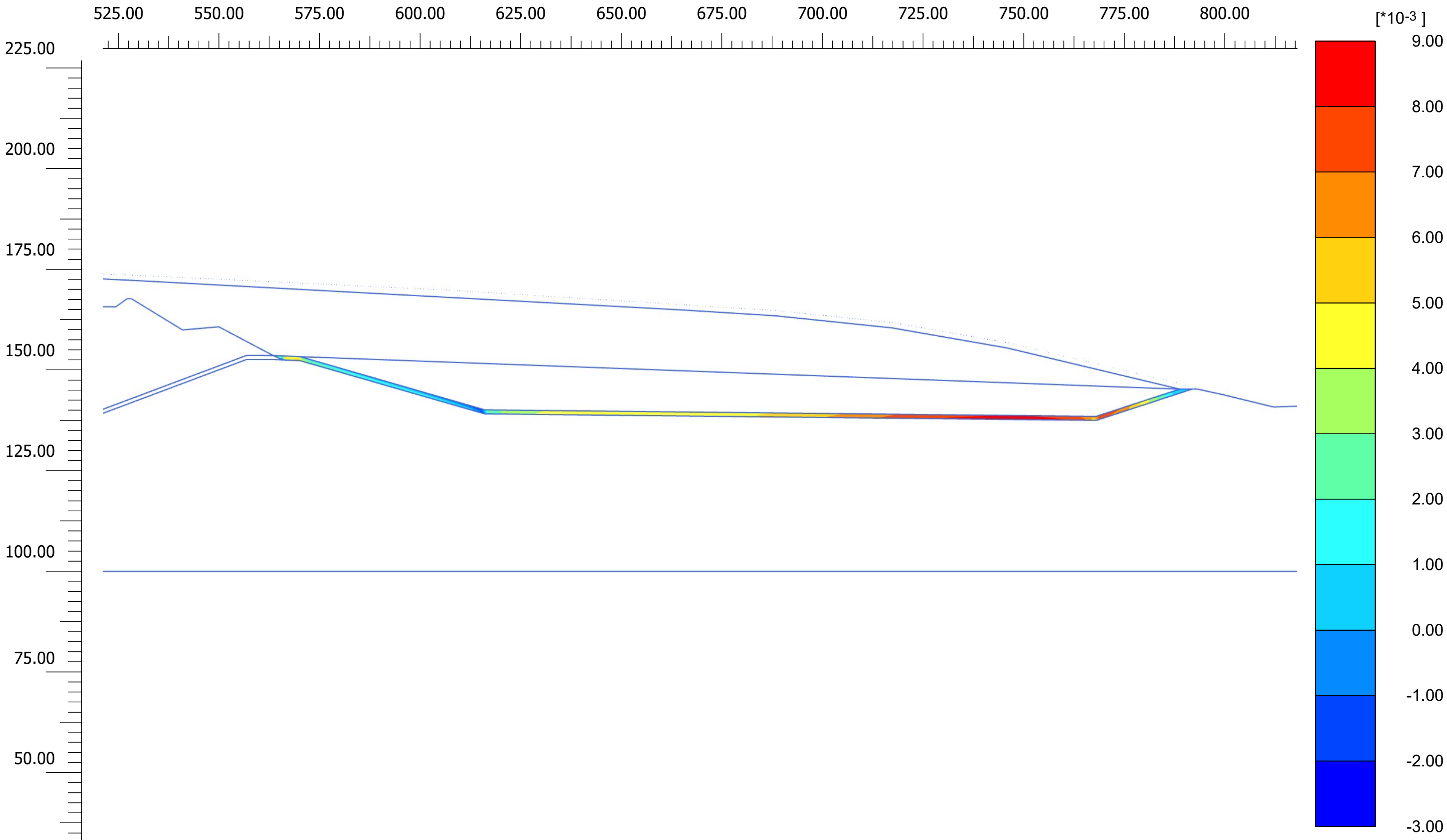
Maximum value = 2.849×10^{-3} (Element 4409 at Node 19778)

Minimum value = -5.026×10^{-3} (Element 4407 at Node 21754)

PLAXIS® 2D CONNECT Edition

Project description		Date
Poplars EP Consolidation		02/11/2021

Project filename	Step	Company
Poplars EPVA Revised S ...	554	Sirius Environmental Ltd



Total cartesian strain γ_{xy} (scaled up 2.00*10^3 times) (Time 3313 day)

Maximum value = 8.544*10^{-3} (Element 4361 at Node 15146)

Minimum value = -2.240*10^{-3} (Element 4407 at Node 21754)

PLAXIS® 2D CONNECT Edition	*Project description* Poplars EP Consolidation		*Date* 02/11/2021
	Project filename Poplars EPVA Revised S ...	*Step* 765	*Company* Sirius Environmental Ltd

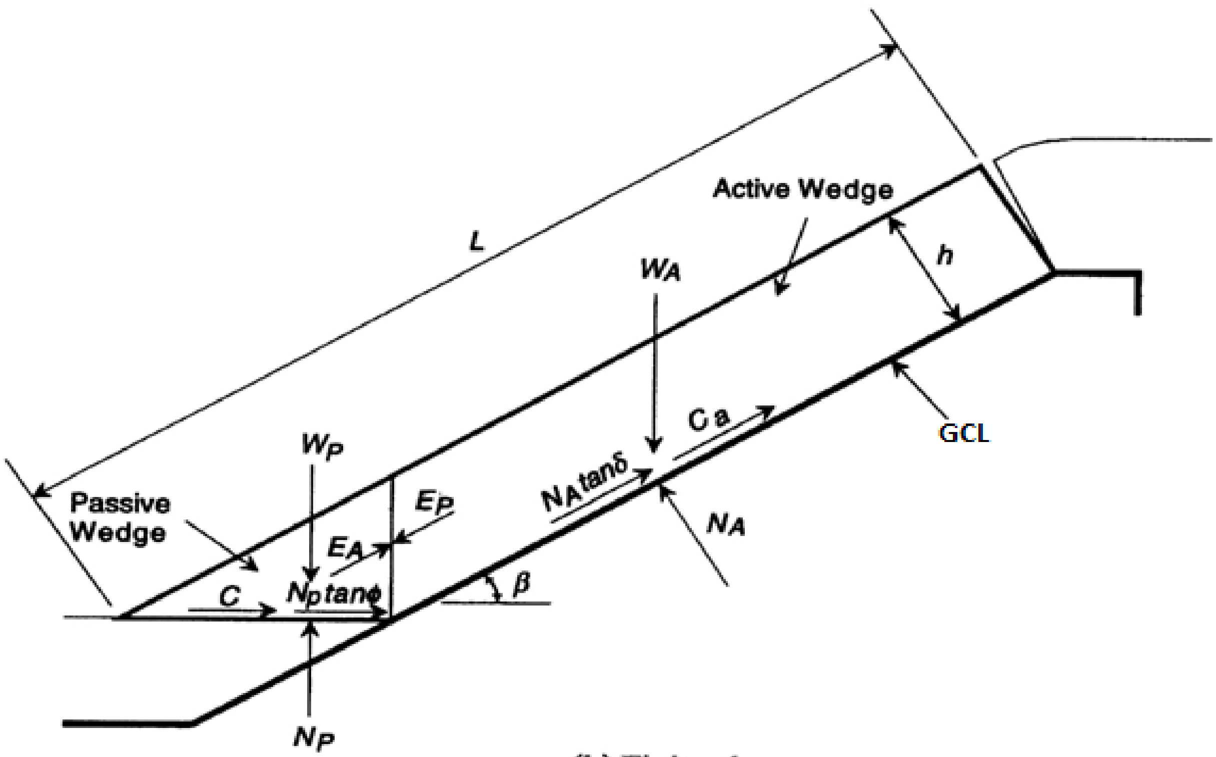
APPENDIX SRA5

CAPPING STABILITY ANALYSIS

Project	Poplars Capping SRA For Permit Variation
Engineer	S Saad
Reviewer	Andrew Kirk
Date	26/10/2021

Capping system stability PSR = 0.0

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	20	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	5	kPa
Thickness of cover soils, h	1.00	m
Height of slope, H	26.8	m
Slope angle, β	18.4	Deg.
Geosynthetic interface shear strengths:		
Cover Soils/GCL friction angle, δ_1	23	Deg.
Cover Soils/GCL cohesion intercept, α_1	2	kPa
GCL/Blinding layer, δ_2	23	Deg.
GCL/Blinding layer, α_2	2	kPa

Parallel submergence ratio, PSR	0.0	
Geosynthetic tensile strengths:		
GCL	12	kN/m

Project	Poplars Capping SRA For Permit Variation
Engineer	S Saad
Reviewer	Andrew Kirk
Date	26/10/2021

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	84.904	m
Thickness of water, h_w	0.000	m
Weight of active wedge, W_A	1498.231	kN
Weight of passive wedge, W_P	30.049	kN
Pore pressure perp. to slope, U_n	0.000	kN
Pore pressure in interwedge surface, U_h	0.000	kN
Force normal to active wedge, N_A	1421.635	kN
Vert pp on passive wedge, U_V	0.000	kN
a	448.738	
b	-833.186	
c	113.815	

Factor of Safety against cover soils sliding **1.71**

2. Integrity of Geosynthetics

(i) GCL

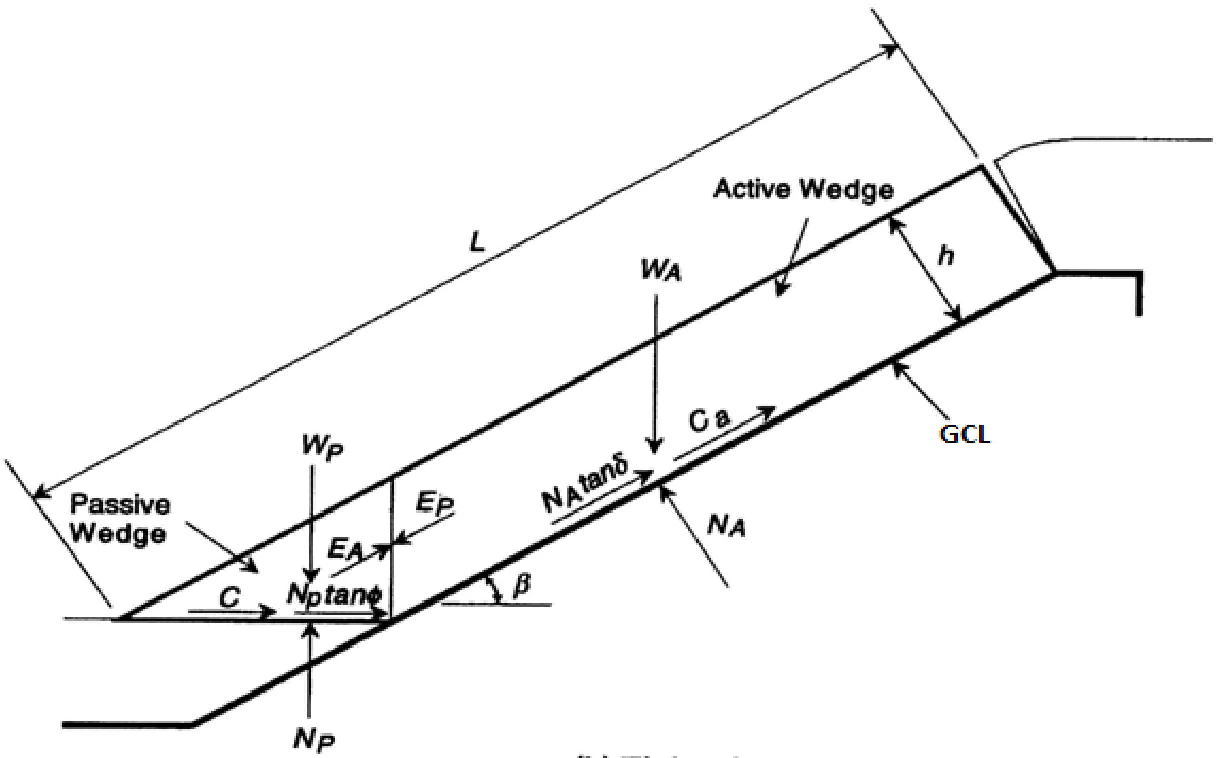
Mobilised shear stress at upper interface	459.744	kN
Shear strength at lower interface	785.360	kN
Tension developed in the GCL	0.000	kN
Tensile strenGCLh of the GCL	12	kN

Factor of Safety against rupture **No Tension**

Project	Poplars Capping SRA For Permit Variation
Engineer	S Saad
Reviewer	Andrew Kirk
Date	26/10/2021

Capping system stability PSR = 0.25

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	20	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	5	kPa
Thickness of cover soils, h	1	m
Height of slope, H	26.8	m
Slope angle, β	18.4	Deg.
Geosynthetic interface shear strengths:		
Cover Soils/GCL friction angle, δ_1	23	Deg.
Cover Soils/GCL cohesion intercept, α_1	2	kPa
GCL/Blinding layer, δ_2	23	Deg.
GCL/Blinding layer, α_2	2	kPa

Parallel submergence ratio, PSR	0.25	
Geosynthetic tensile strengths:		
GCL	12	kN/m

Project	Poplars Capping SRA For Permit Variation
Engineer	S Saad
Reviewer	Andrew Kirk
Date	21/10/2021

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	84.904	m
Thickness of water, h_w	0.250	m
Weight of active wedge, W_A	1540.474	kN
Weight of passive wedge, W_P	30.258	kN
Pore pressure perp. to slope, U_n	200.419	kN
Pore pressure in interwedge surface, U_h	0.313	kN
Force normal to active wedge, N_A	1261.398	kN
Vert pp on passive wedge, U_V	0.939	kN
a	461.421	
b	-770.225	
c	103.804	

Factor of Safety against cover soils sliding **1.52**

2. Integrity of Geosynthetics

(i) GCL

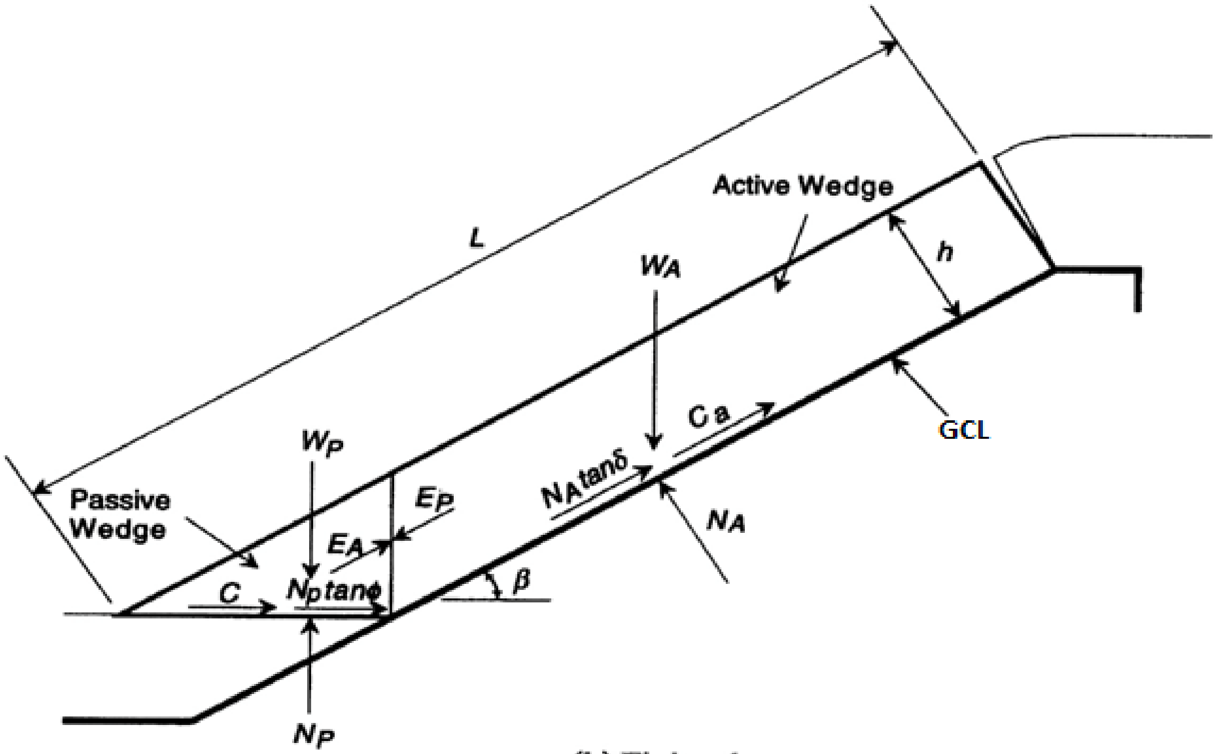
Mobilised shear stress at upper interface	527.457	kN
Shear strength at lower interface	802.459	kN
Tension developed in the GCL	0.000	kN
Tensile strenGCLh of the GCL	12	kN

Factor of Safety against rupture **No Tension**

Project	Poplars Capping SRA For Permit Variation
Engineer	S Saad
Reviewer	Andrew Kirk
Date	26/10/2021

Capping system stability PSR = 0.5

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	20	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	5	kPa
Thickness of cover soils, h	1	m
Height of slope, H	26.8	m
Slope angle, β	18.4	Deg.
Geosynthetic interface shear strengths:		
Cover Soils/GCL friction angle, δ_1	23	Deg.
Cover Soils/GCL cohesion intercept, α_1	2	kPa
GCL/Blinding layer, δ_2	23	Deg.
GCL/Blinding layer, α_2	2	kPa
Parallel submergence ratio, PSR	0.5	
Geosynthetic tensile strengths:		
GCL	12	kN/m

Project	Poplars Capping SRA For Permit Variation
Engineer	S Saad
Reviewer	Andrew Kirk
Date	26/10/2021

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	84.904	m
Thickness of water, h_w	0.500	m
Weight of active wedge, W_A	1582.300	kN
Weight of passive wedge, W_P	30.884	kN
Pore pressure perp. to slope, U_n	398.859	kN
Pore pressure in interwedge surface, U_h	1.250	kN
Force normal to active wedge, N_A	1102.943	kN
Vert pp on passive wedge, U_V	3.758	kN
a	474.042	
b	-707.193	
c	93.904	

Factor of Safety against cover soils sliding **1.34**

2. Integrity of Geosynthetics

(i) GCL

Mobilised shear stress at upper interface	609.563	kN
Shear strength at lower interface	819.557	kN
Tension developed in the GCL	0.000	kN
Tensile strenGCLh of the GCL	12	kN

Factor of Safety against rupture **No Tension**

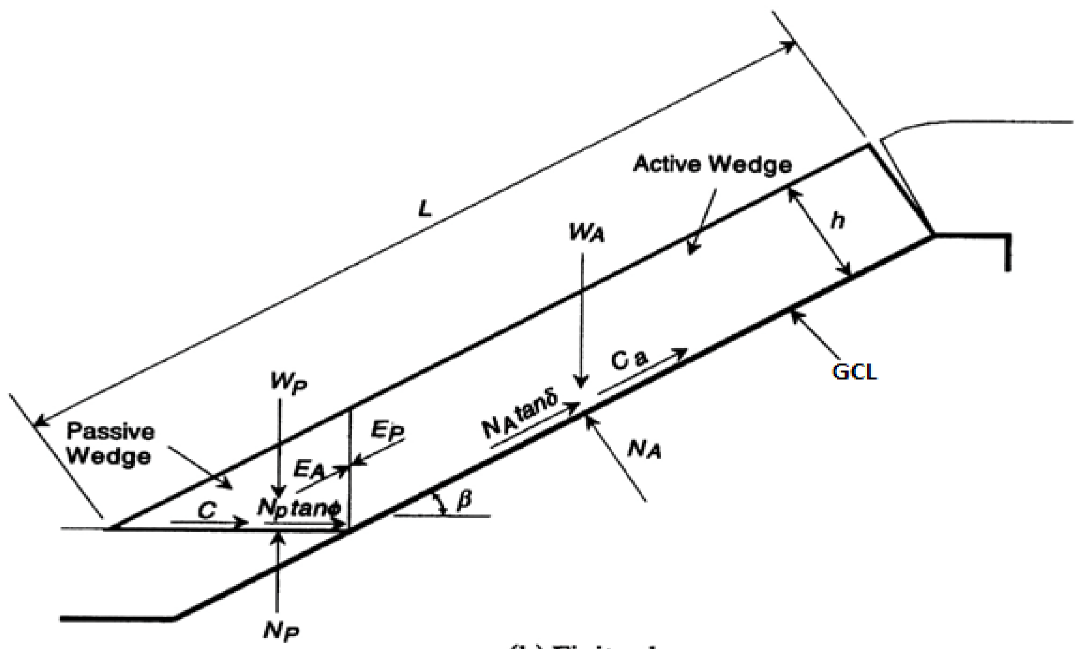
Project	Poplars Capping SRA For Permit Variation
Engineer	S Saad
Reviewer	Andrew Kirk
Date	26/10/2021

Capping system stability: Effect of gas pressure

Aim: To assess the stability of the drainage material and integrity of the geosynthetic lining system

Approach: Use the approach proposed by Koerner & Daniels, 1997.

Geometry:



(b) Finite slope

Input Parameters

Cover soil unit weight, γ	18	kN/m^3
Cover soil internal shear strength, ϕ	25	Deg.
Cover soil cohesion, c	5	kPa
Thickness of cover soil, h	1	m
Height of slope, H	26.8	m
Slope angle, β	18.4	Deg.

N.B. Consider only interface at base of geosynthetics on whi 30

Geosynthetic interface shear strengths:

GCL/Blinding layer, δ_3	23	Deg.
GCL/Blinding layer, α_3	2	kPa

Gas pressure	5.0	kPa

Ignores strength of geosynthetic layers

Project	Poplars Capping SRA For Permit Variation
Engineer	S Saad
Reviewer	Andrew Kirk
Date	26/10/2021

1. Stability of Cover Soil

Calculated Parameters

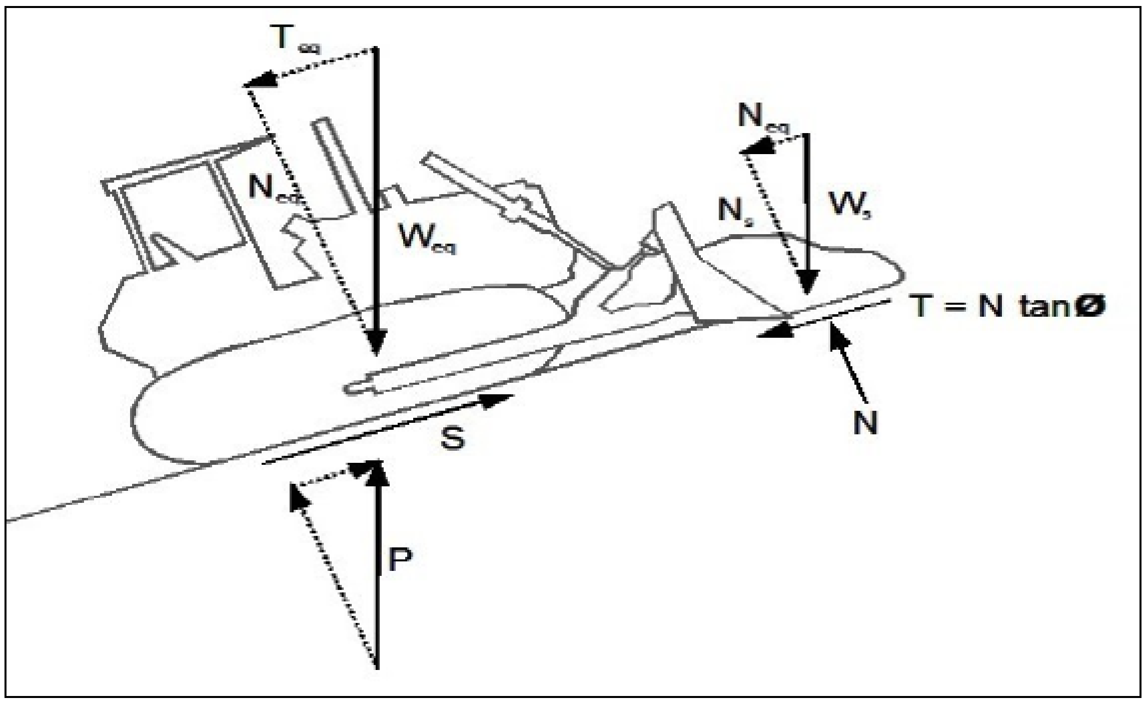
Length of slope, L	84.904	m
Weight of active wedge, W_A	1498.231	kN
Weight of passive wedge, W_P	30.049	kN
Pore pressure perp. to slope, U_n	0.000	
Pore pressure in interwedge surface, U_h	0.000	
Force normal to active wedge, N_A	1421.635	kN
Force normal to active wedge from gas pressure, N_G	424.52	kN
a	448.738	
b	-662.199	
c	62.592	

Factor of Safety against GCL sliding **1.37**

Project	Poplars Capping SRA For Permit Variation
Engineer	S Saad
Reviewer	Andrew Kirk
Date	26/10/2021

Capping system stability: Effect of plant loading

Geometry:



Input Parameters

Cover soils unit weight (dry), gdry	18	kN/m^3
Cover soil internal shear strength, ϕ	25	Deg.
Cover soil cohesion, c	5	kPa
Thickness of cover soil, h	1	m
Height of slope, H	26.8	m
Slope angle, β	18.4	Deg.

Geosynthetic interface shear strengths:

Cover Soils/GCL friction angle, δ_1	23	Deg.
Cover Soils/GCL cohesion intercept, α_1	2	kPa
GCL/Blinding layer, δ_2	23	Deg.
GCL/Blinding layer, α_2	2	kPa

Project	Poplars Capping SRA For Permit Variation
Engineer	S Saad
Reviewer	Andrew Kirk
Date	26/10/2021

I (Influence Factor)	1	
Wb (Buldozer Weight) (CAT D6H LGP)	201	kN
w (Track Length)	3.2	m
b (Track Width)	0.91	m
Force per unit area	34.51	kPa
Equivalent Force/ unit width	110.44	kN/m
acceleration of plant	2	m/s^2
acceleration due to gravity	9.81	m/s^2
Dynamic Force per unit width	22.52	
Effective Equipment Force normal to failure Plane	104.79	
Cohesive Force Along Failure plane of Passive Wedge	15.84	

Geosynthetic tensile strengths:

GCL	10.8	kN/m

1. Stability of Cover Soil

Calculated Parameters

Length of slope, L	84.904	m
Weight of active wedge, W_A	1498.231	kN
Weight of passive wedge, W_P	30.049	kN
Pore pressure perp. to slope, U_n	0.000	
Pore pressure in interwedge surface, U_h	0.000	
Force normal to active wedge, N_A	1421.635	kN
a	503.180	
b	-883.839	
c	95.663	

Factor of Safety against cover soil sliding **1.64**

2. Integrity of Geosynthetics

(i) GCL

Mobilised shear stress at upper interface	478.696	kN
Shear strength at lower interface	785.360	kN
Tension developed in the GCL	0.000	kN
Tensile strength of the GCL	10.8	kN

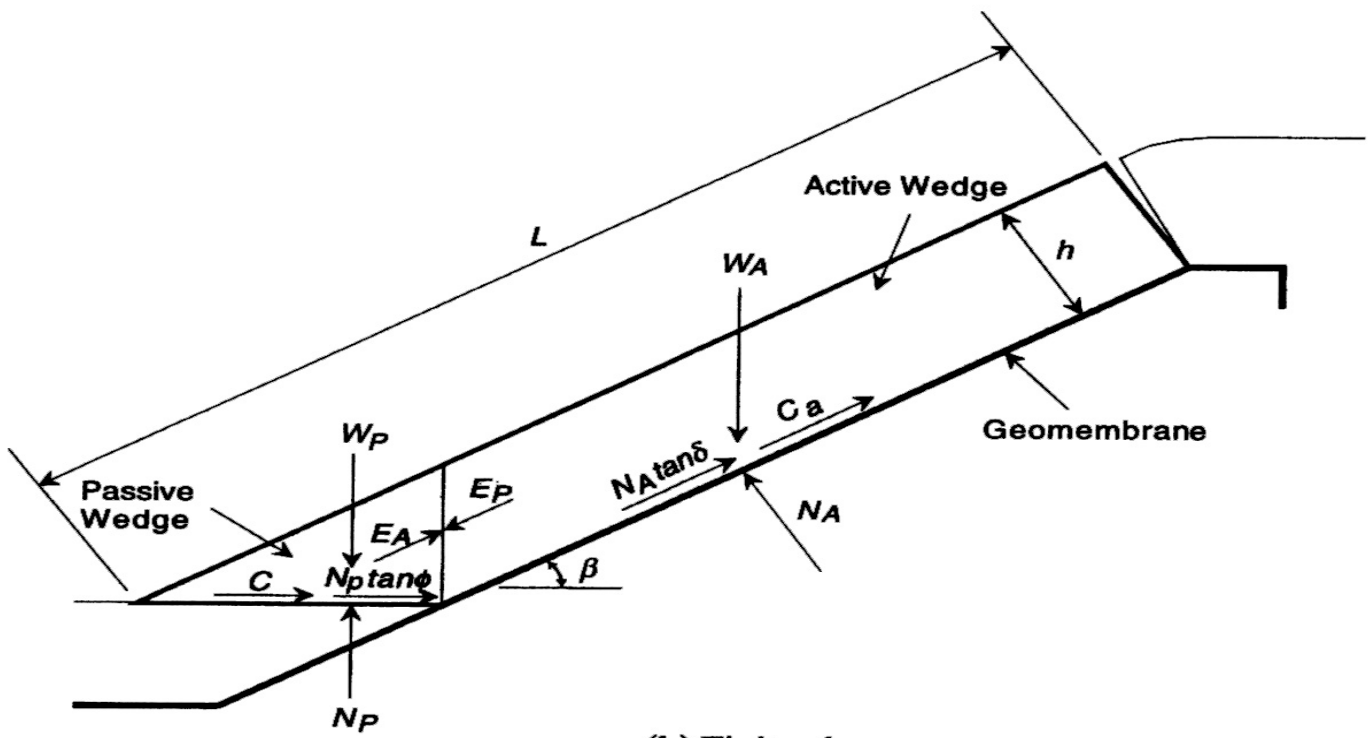
Factor of Safety against rupture **No Tension**

PROJECT **Poplars Capping SRA For Permit Variation**

Job No. BF5048 | Made By: S Saad | Date: 26/10/2021
Checked: J Davies
Reviewed: A Kirk

Capping system stability PSR = 0.0

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	20	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	5	kPa
Thickness of cover soils, h	1.00	m
Height of slope, H	26.8	m
Slope angle, β	18.4	Deg.
Geosynthetic interface shear strengths:		
Cover soil/Geocomposite friction angle, d1	23	Deg.
Cover soil/Geocomposite cohesion intercept, a1	2	kPa
Geocomposite/Geomembrane friction angle, d2	23	Deg.
Geocomposite/Geomembrane cohesion intercept, a2	2	kPa
Geomembrane/Blinding layer, δ_3	23	Deg.
Geomembrane/Blinding layer, α_3	2	kPa
Parallel submergence ratio, PSR	0.0	
Geosynthetic tensile strengths:		
Geocomposite	15	kN/m
Geomembrane	11	kN/m

PROJECT	**Poplars Capping SRA For Permit Variation**				
Job No.	BF5048	Made By:	S Saad	Date:	26/10/2021
		Checked:	J Davies		
		Reviewed:	A Kirk		

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	84.904	m
Thickness of water, h_w	0.000	m
Weight of active wedge, W_A	1498.231	kN
Weight of passive wedge, W_P	30.049	kN
Pore pressure perp. to slope, U_n	0.000	kN
Pore pressure in interwedge surface, U_h	0.000	kN
Force normal to active wedge, N_A	1421.635	kN
Vert pp on passive wedge, U_V	0.000	kN
a	448.738	
b	-833.186	
c	113.815	

Factor of Safety against cover soils sliding **1.71**

2. Integrity of Geosynthetics

(i) Geocomposite

Mobilised shear stress at upper interface	459.744	kN
Shear strength at lower interface	785.360	kN
Tension developed in the GT	0.000	kN
Tensile strength of the GT	15	kN

Factor of Safety against rupture **No Tension**

(ii) Geomembrane

Shear strength at upper surface	785.360	kN
Mobilised shear stress at upper interface	459.744	kN
Shear strength at lower interface	785.360	kN
Tension developed in the GM	0.000	kN
Tensile strength of the GM	11	kN

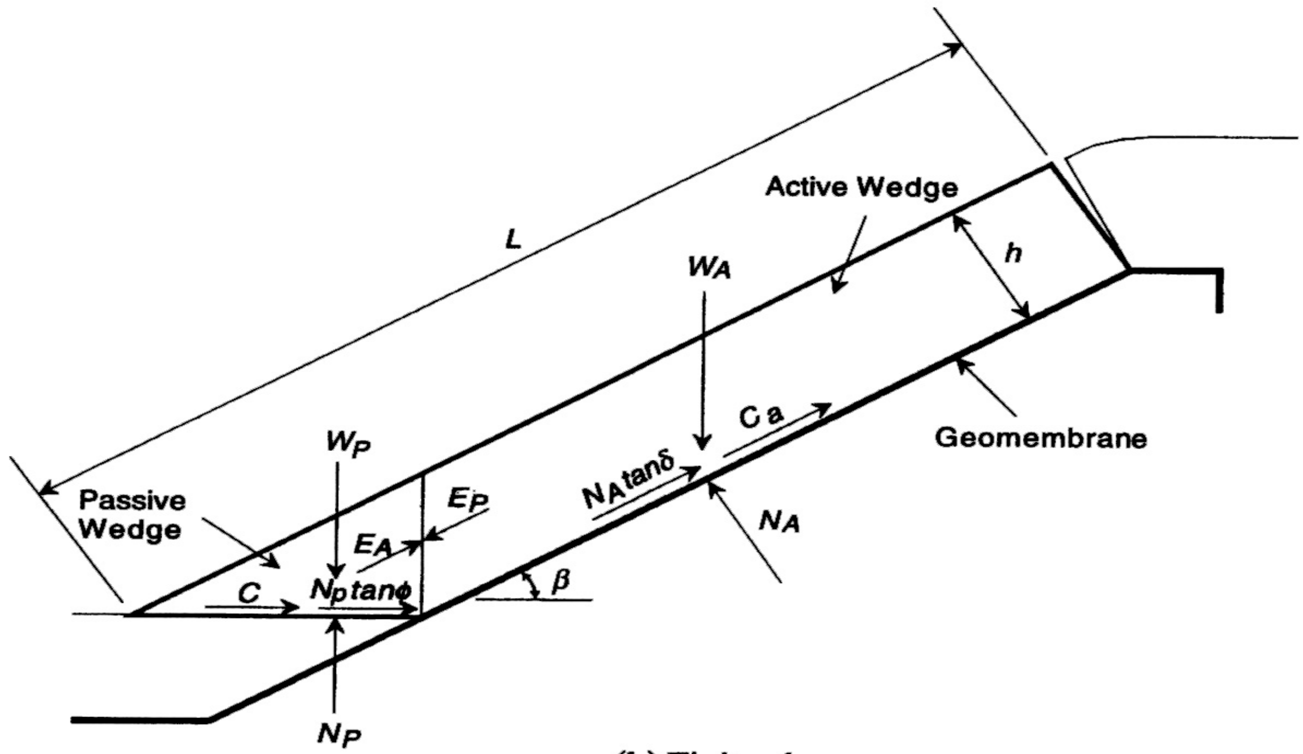
Factor of Safety against rupture **No Tension**

PROJECT **Poplars Capping SRA For Permit Variation**

Job No.	BF5048	Made By:	S Saad	Date:	26/10/2021
		Checked:	J Davies		
		Reviewed:	A Kirk		

Capping system stability PSR = 0.25

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	20	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	5	kPa
Thickness of cover soils, h	1	m
Height of slope, H	26.8	m
Slope angle, β	18.4	Deg.
Geosynthetic interface shear strengths:		
Cover soil/Geocomposite friction angle, d1	23	Deg.
Cover soil/Geocomposite cohesion intercept, a1	2	kPa
Geocomposite/Geomembrane friction angle, d2	23	Deg.
Geocomposite/Geomembrane cohesion intercept, a2	2	kPa
Geomembrane/Blinding layer, d3	23	Deg.
Geomembrane/Blinding layer, a3	2	kPa
Parallel submergence ratio, PSR	0.25	
Geosynthetic tensile strengths:		
Geocomposite	15	kN/m
Geomembrane	11	kN/m

PROJECT	**Poplars Capping SRA For Permit Variation**				
Job No.	BF5048	Made By:	S Saad	Date:	26/10/2021
		Checked:	J Davies		
		Reviewed:	A Kirk		

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	84.904	m
Thickness of water, h_w	0.250	m
Weight of active wedge, W_A	1540.474	kN
Weight of passive wedge, W_P	30.258	kN
Pore pressure perp. to slope, U_n	200.419	kN
Pore pressure in interwedge surface, U_h	0.313	kN
Force normal to active wedge, N_A	1261.398	kN
Vert pp on passive wedge, U_V	0.939	kN
a	461.421	
b	-770.225	
c	103.804	

Factor of Safety against cover soils sliding **1.52**

2. Integrity of Geosynthetics

(i) Geocomposite

Mobilised shear stress at upper interface	527.457	kN
Shear strength at lower interface	802.459	kN
Tension developed in the GT	0.000	kN
Tensile strength of the GT	15	kN

Factor of Safety against rupture **No Tension**

(ii) Geomembrane

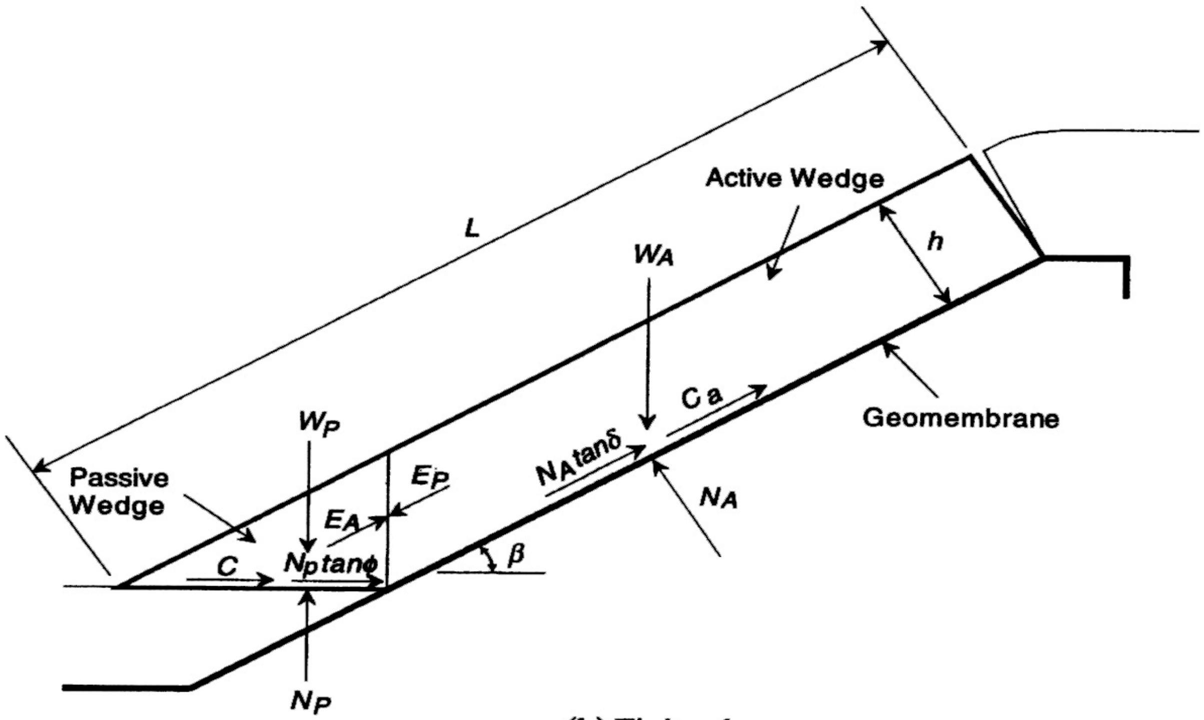
Shear strength at upper surface	802.459	kN
Mobilised shear stress at upper interface	527.457	kN
Shear strength at lower interface	802.459	kN
Tension developed in the GM	0.000	kN
Tensile strength of the GM	11	kN

Factor of Safety against rupture **No Tension**

PROJECT	**Poplars Capping SRA For Permit Variation**				
Job No.	BF5048	Made By:	S Saad	Date:	26/10/2021
		Checked:	J Davies		
		Reviewed:	A Kirk		

Capping system stability PSR = 0.5

Geometry:



(b) Finite slope

Input Parameters

Cover soils unit weight (dry), γ_{dry}	18	kN/m^3
Cover soils unit weight (saturated), γ_{sat}	20	kN/m^3
Cover soils internal shear strength, ϕ	25	Deg.
Cover soils cohesion, c	5	kPa
Thickness of cover soils, h	1	m
Height of slope, H	26.8	m
Slope angle, β	18.4	Deg.
Geosynthetic interface shear strengths:		
Cover soil/Geocomposite friction angle, d1	23	Deg.
Cover soil/Geocomposite cohesion intercept, a1	2	kPa
Geocomposite/Geomembrane friction angle, d2	23	Deg.
Geocomposite/Geomembrane cohesion intercept, a2	2	kPa
Geomembrane/Blinding layer, d3	23	Deg.
Geomembrane/Blinding layer, a3	2	kPa
Parallel submergence ratio, PSR	0.5	
Geosynthetic tensile strengths:		
Geocomposite	15	kN/m
Geomembrane	11	kN/m

PROJECT	**Poplars Capping SRA For Permit Variation**				
Job No.	BF5048	Made By:	S Saad	Date:	26/10/2021
		Checked:	J Davies		
		Reviewed:	A Kirk		

1. Stability of Cover Soils

Calculated Parameters

Length of slope, L	84.904	m
Thickness of water, h_w	0.500	m
Weight of active wedge, W_A	1582.300	kN
Weight of passive wedge, W_P	30.884	kN
Pore pressure perp. to slope, U_n	398.859	kN
Pore pressure in interwedge surface, U_h	1.250	kN
Force normal to active wedge, N_A	1102.943	kN
Vert pp on passive wedge, U_V	3.758	kN
a	474.042	
b	-707.193	
c	93.904	

Factor of Safety against cover soils sliding **1.34**

2. Integrity of Geosynthetics

(i) Geocomposite

Mobilised shear stress at upper interface	609.563	kN
Shear strength at lower interface	819.557	kN
Tension developed in the GT	0.000	kN
Tensile strength of the GT	15	kN

Factor of Safety against rupture **No Tension**

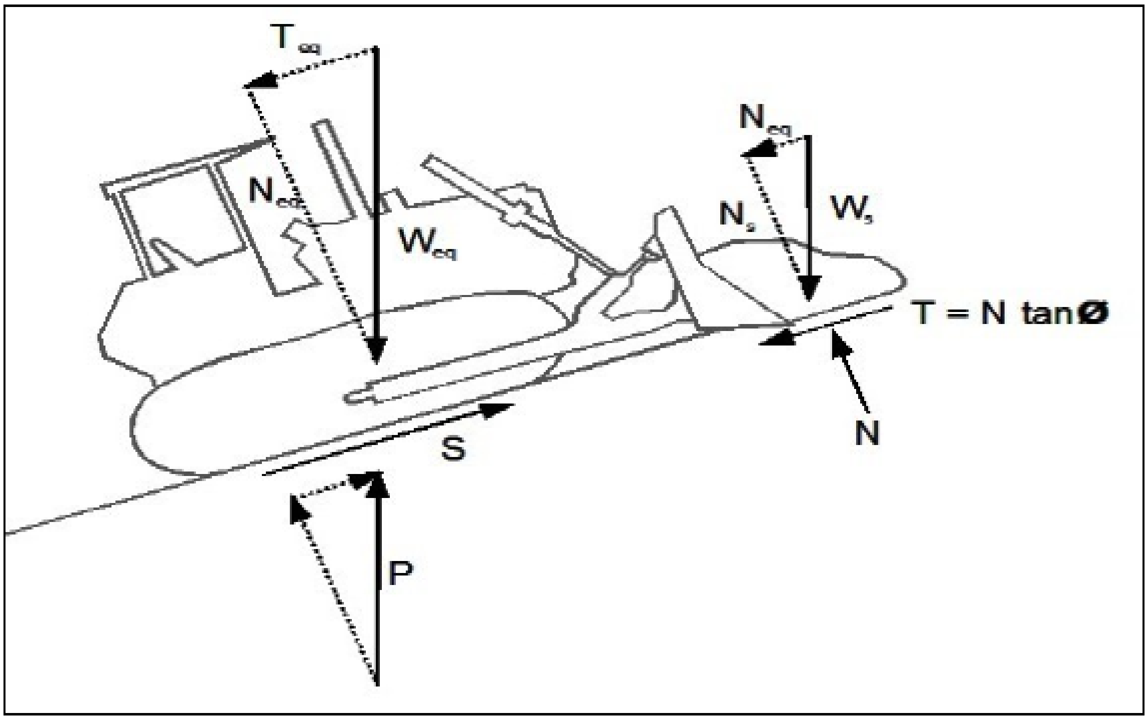
(ii) Geomembrane

Shear strength at upper surface	819.557	kN
Mobilised shear stress at upper interface	609.563	kN
Shear strength at lower interface	819.557	kN
Tension developed in the GM	0.000	kN
Tensile strength of the GM	11	kN

Factor of Safety against rupture **No Tension**

Capping system stability: Effect of plant loading

Geometry:



Input Parameters

Cover soils unit weight (dry), gdry	18	kN/m^3
Cover soil internal shear strength, ϕ	25	Deg.
Cover soil cohesion, c	5	kPa
Thickness of cover soil, h	1	m
Height of slope, H	26.8	m
Slope angle, β	18.4	Deg.

Geosynthetic interface shear strengths:

Cover soil/Geocomposite friction angle, d1	23	Deg.
Cover soil/Geocomposite cohesion intercept, a1	2	kPa
Geocomposite/Geomembrane friction angle, d2	23	Deg.
Geocomposite/Geomembrane cohesion intercept, a2	2	kPa
Geomembrane/Blinding layer, d3	23	Deg.
Geomembrane/Blinding layer, a3	2	kPa

PROJECT **Poplars Capping SRA For Permit Variation**

Job No. BF5048 Made By: S Saad Date: 26/10/2021

Checked: J Davies

Reviewed: A Kirk

I (Influence Factor)	1	
Wb (Buldozer Weight)	230	kN
w (Track Length)	3.2	m
b (Track Width)	0.91	m
Force per unit area	45	kPa
Equivalent Force/ unit width	144	kN/m
acceleration of plant	2	m/s^2
acceleration due to gravity	9.81	m/s^2
Dynamic Force per unit width	29.3578	
Effective Equipment Force normal to failure Plane	136.6381	
Cohesive Force Along Failure plane of Passive Wedge	15.84038	
Geosynthetic tensile strengths:		
Geocomposite	15	kN/m
Geomembrane	11	kN/m

1. Stability of Cover Soil

Calculated Parameters

Length of slope, L	84.904	m
Weight of active wedge, W_A	1498.231	kN
Weight of passive wedge, W_P	30.049	kN
Pore pressure perp. to slope, U_n	0.000	
Pore pressure in interwedge surface, U_h	0.000	
Force normal to active wedge, N_A	1421.635	kN
a	519.724	
b	-899.232	
c	97.653	

Factor of Safety against cover soil sliding **1.61**

2. Integrity of Geosynthetics

(i) Geocomposite

Mobilised shear stress at upper interface	486.659	kN
Shear strength at lower interface	785.360	kN
Tension developed in the Geotextile	0.000	kN
Tensile strength of the Geotextile	20	kN

Factor of Safety against rupture **No Tension**

Shear strength at upper surface	785.360	kN
Mobilised shear stress at upper interface	486.659	kN
Shear strength at lower interface	785.360	kN
Tension developed in the Geomembrane	0.000	kN
Tensile strength of the Geomembrane	12	kN

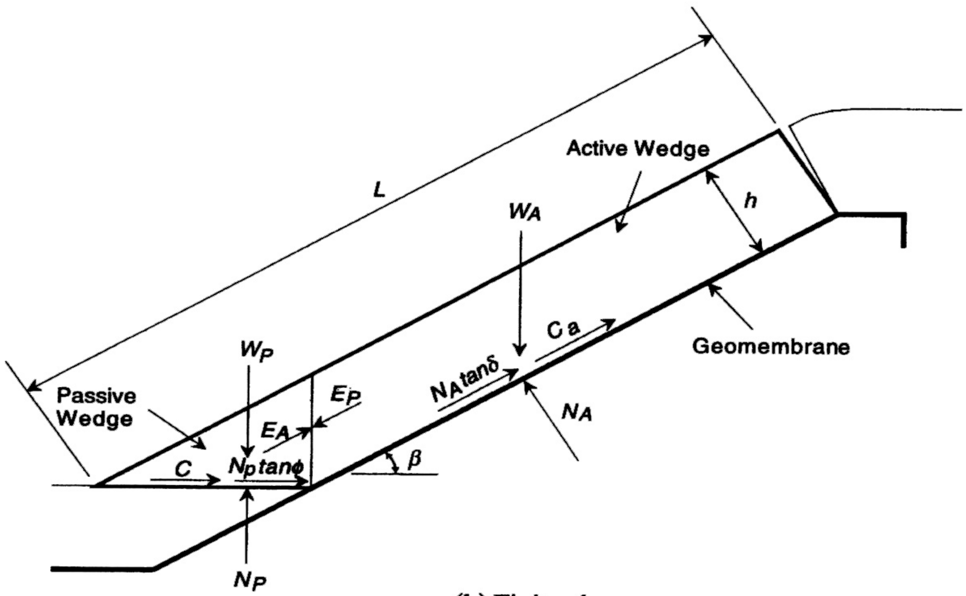
Factor of Safety against rupture **No Tension**

Capping system stability: Effect of gas pressure

Aim: To assess the stability of the drainage material and integrity of the geosynthetic lining system

Approach: Use the approach proposed by Koerner & Daniels, 1997.

Geometry:



(b) Finite slope

Input Parameters

Cover soil unit weight, γ	18	kN/m^3
Cover soil internal shear strength, ϕ	25	Deg.
Cover soil cohesion, c	5	kPa
Thickness of cover soil, h	1	m
Height of slope, H	26.8	m
Slope angle, β	18.4	Deg.

N.B. Consider only interface at base of geosynthetics on which gas pressure acts.

Geosynthetic interface shear strengths:

Geomembrane/Blinding layer, δ_3	23	Deg.
Geomembrane/Blinding layer, α_3	2	kPa

Gas pressure	5.0	kPa

Ignores strength of geosynthetic layers

1. Stability of Cover Soil

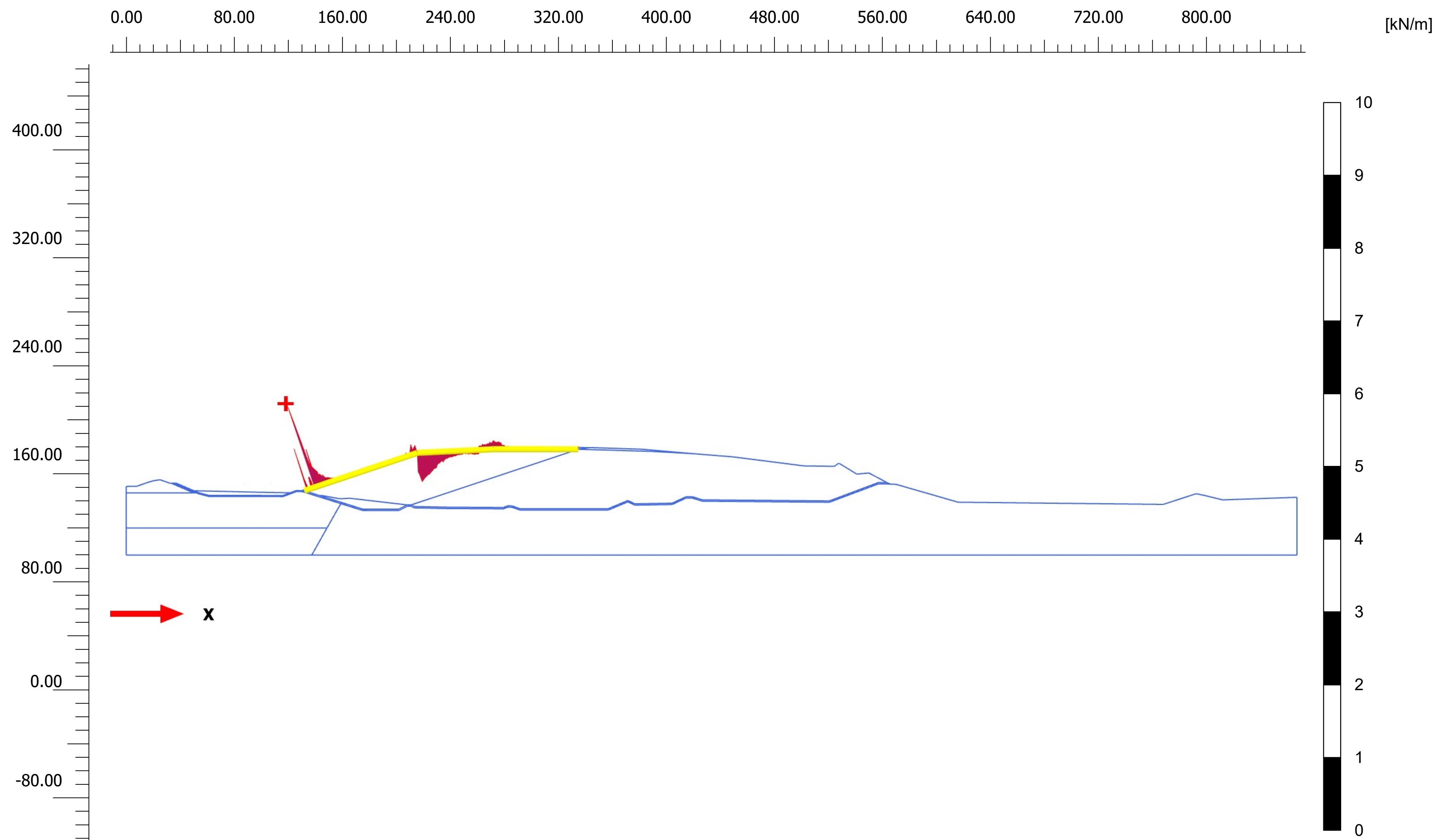
Calculated Parameters

Length of slope, L	84.904	m
Weight of active wedge, W_A	1498.231	kN
Weight of passive wedge, W_P	30.049	kN
Pore pressure perp. to slope, U_n	0.000	
Pore pressure in interwedge surface, U_h	0.000	
Force normal to active wedge, N_A	1421.635	kN
Force normal to active wedge from gas pressure, N_G	424.52	kN
a	448.738	
b	-646.359	
c	62.592	

Factor of Safety against geomembrane sliding **1.34**

APPENDIX SRA6

CAPPING INTEGRITY ANALYSIS



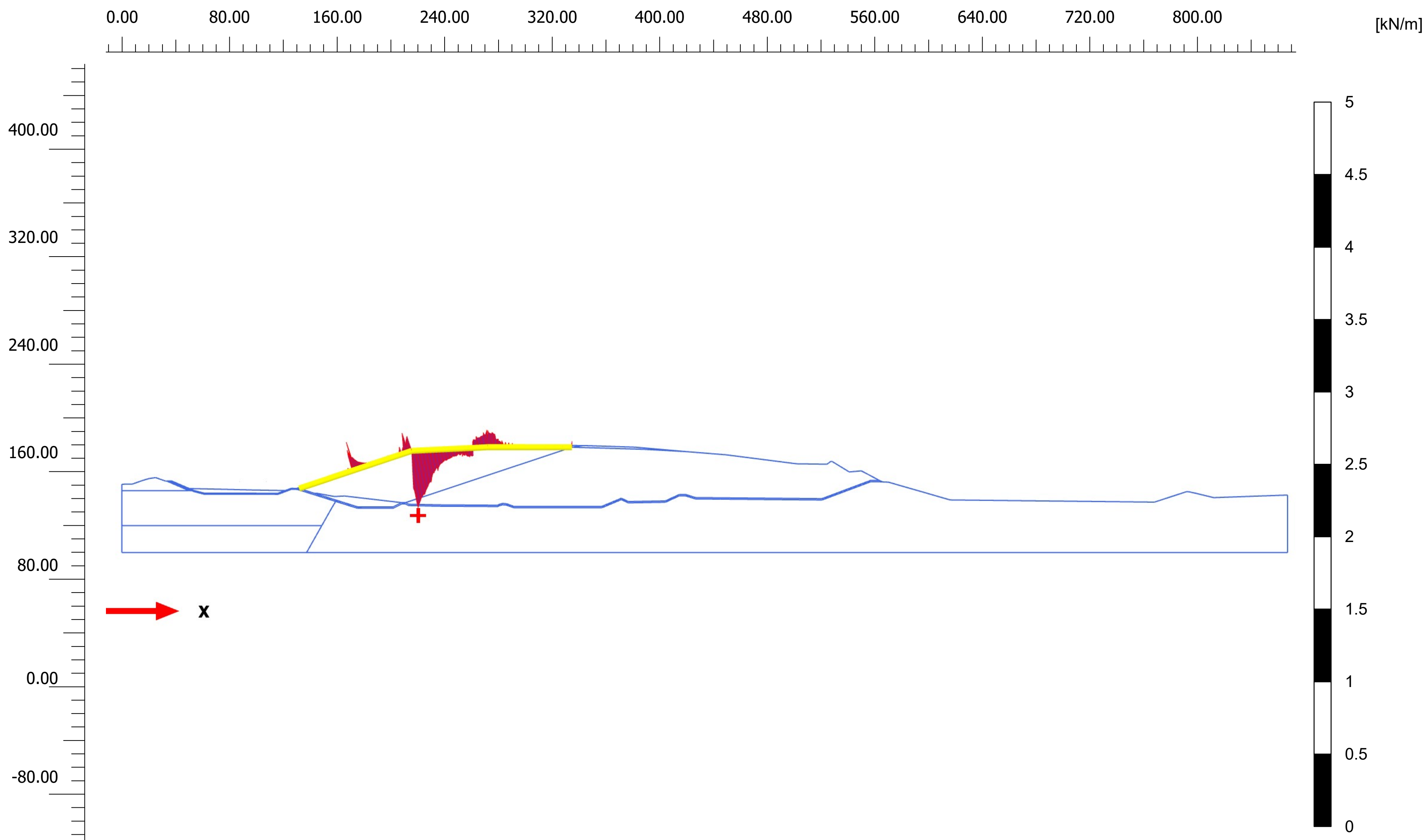
Axial forces N (scaled up 50.0 times) (Time 2524 day)

Maximum value = 1.249 kN/m (Element 251 at Node 42056)

Minimum value = 0.000 kN/m (Element 199 at Node 42114)

PLAXIS® 2D CONNECT Edition	*Project description* Poplars EP Consolidation		*Date* 03/11/2021
	Project filename Poplars EPVA Revised S ...	*Step* 547	*Company* Sirius Environmental Ltd

Output Version 21.1.0.479



Axial forces N (scaled up 100 times) (Time 2524 day)

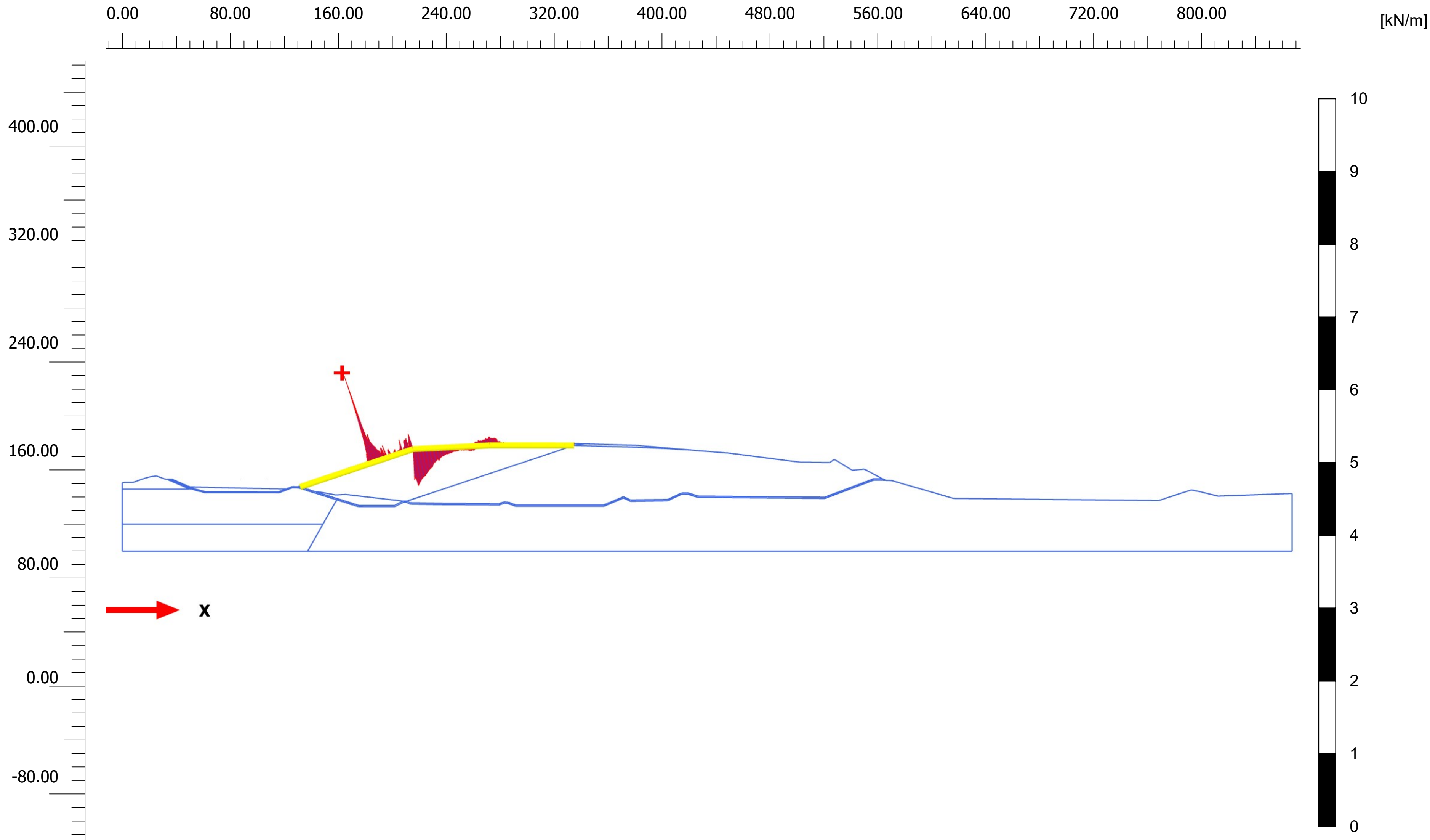
Maximum value = 0.4251 kN/m (Element 64 at Node 32902)

Minimum value = 0.000 kN/m (Element 121 at Node 40748)

PLAXIS® 2D CONNECT Edition

Project description		*Date*
Poplars EP Consolidation		03/11/2021
Project filename	*Step*	*Company*
Poplars EPVA Revised S ...	551	Sirius Environmental Ltd

Output Version 21.1.0.479



Axial forces N (scaled up 50.0 times) (Time 2524 day)

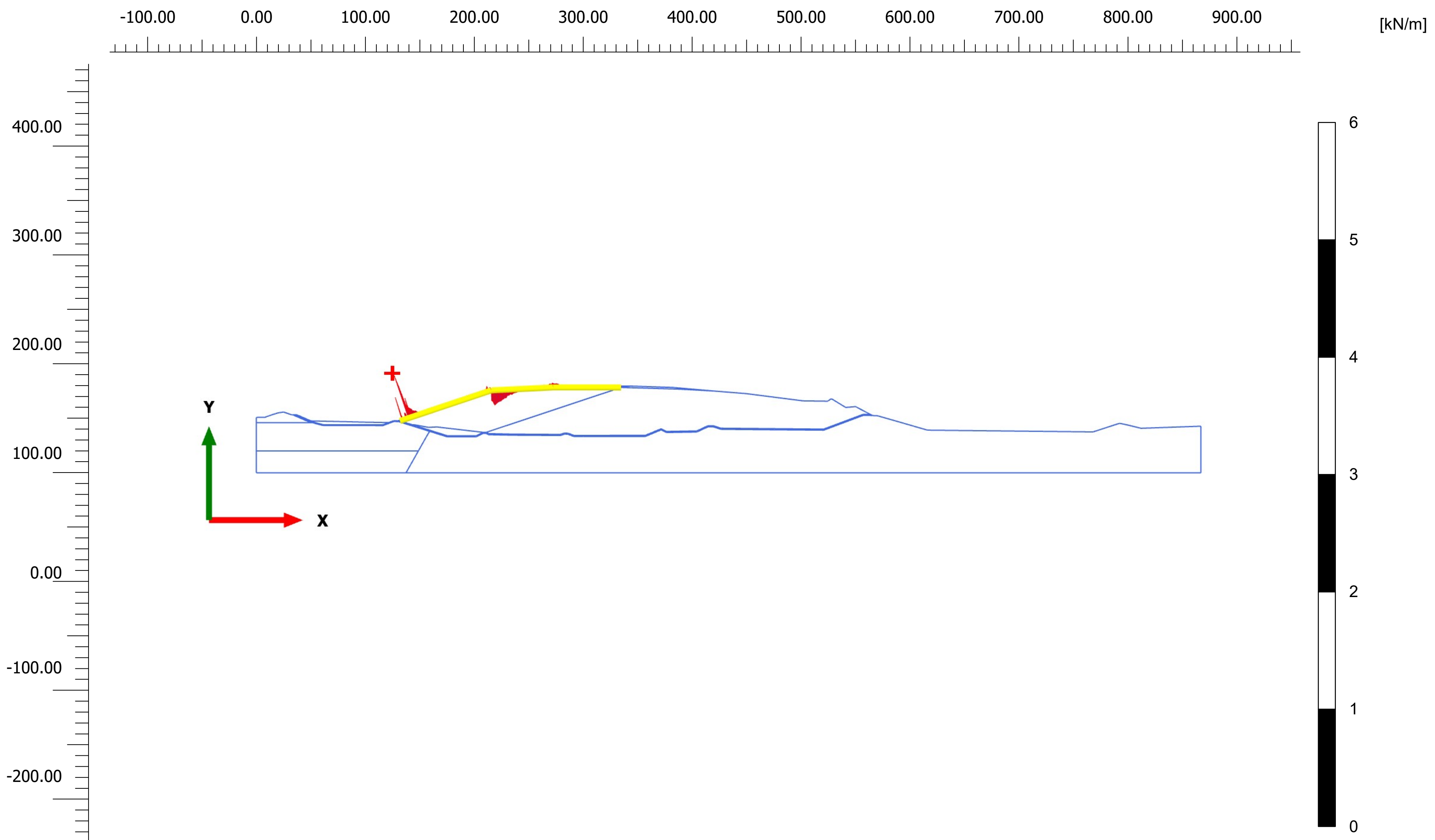
Maximum value = 1.347 kN/m (Element 199 at Node 42114)

Minimum value = 0.000 kN/m (Element 122 at Node 40234)

PLAXIS® 2D CONNECT Edition

Project description			*Date*
Poplars EP Consolidation			03/11/2021
Project filename	*Step*	*Company*	
Poplars EPVA Revised S ...	549	Sirius Environmental Ltd	

Output Version 21.1.0.479



Axial forces N (scaled up 100 times) (Time 2524 day)

Maximum value = 0.4099 kN/m (Element 251 at Node 42056)

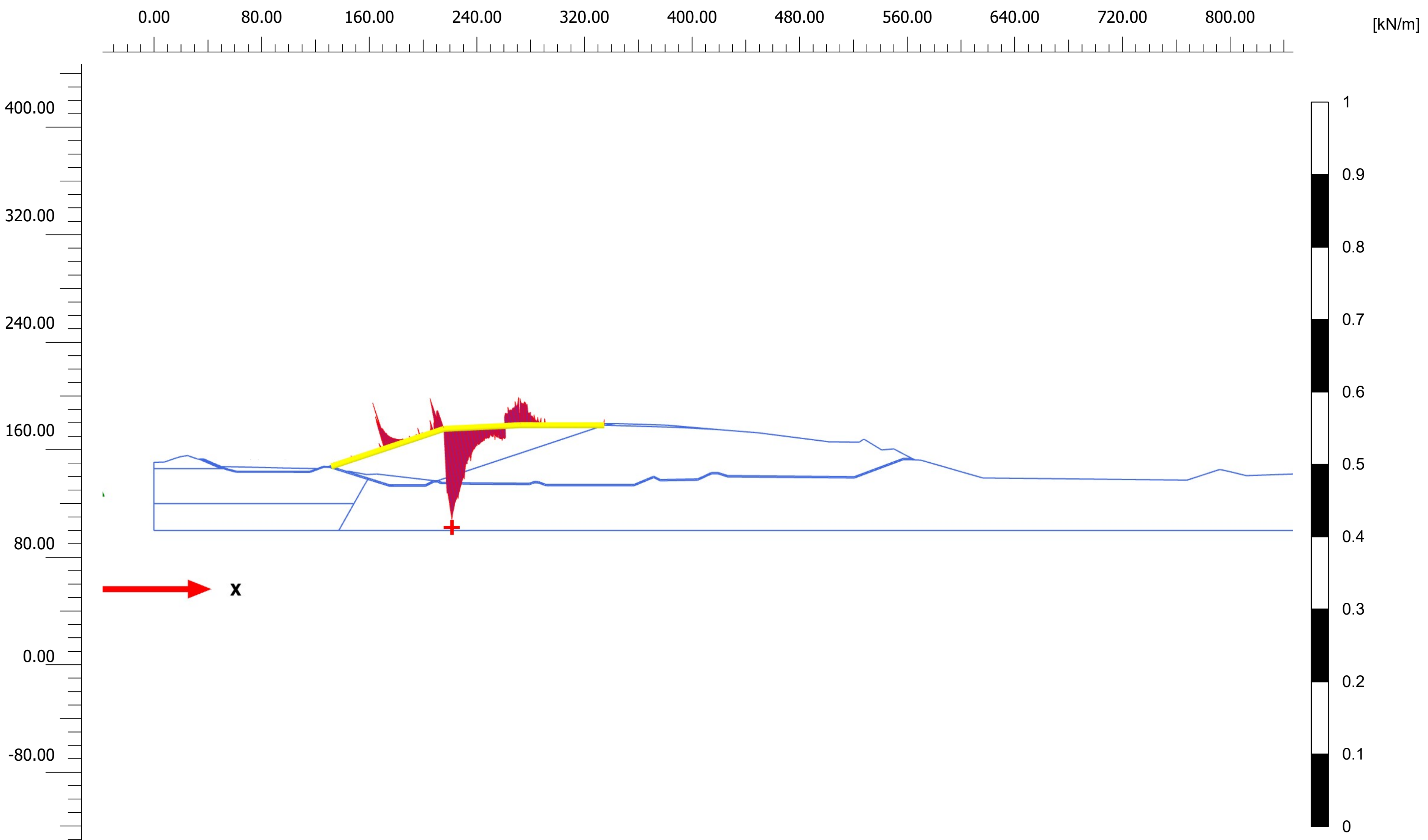
Minimum value = 0.000 kN/m (Element 199 at Node 42114)

PLAXIS® 2D CONNECT Edition

Project description		Date
Poplars EP Consolidation		02/11/2021

Project filename	Step	Company
Poplars EPVA Revised S ...	547	Sirius Environmental Ltd

Output Version 21.1.0.479



Axial forces N (scaled up 500 times) (Time 2524 day)

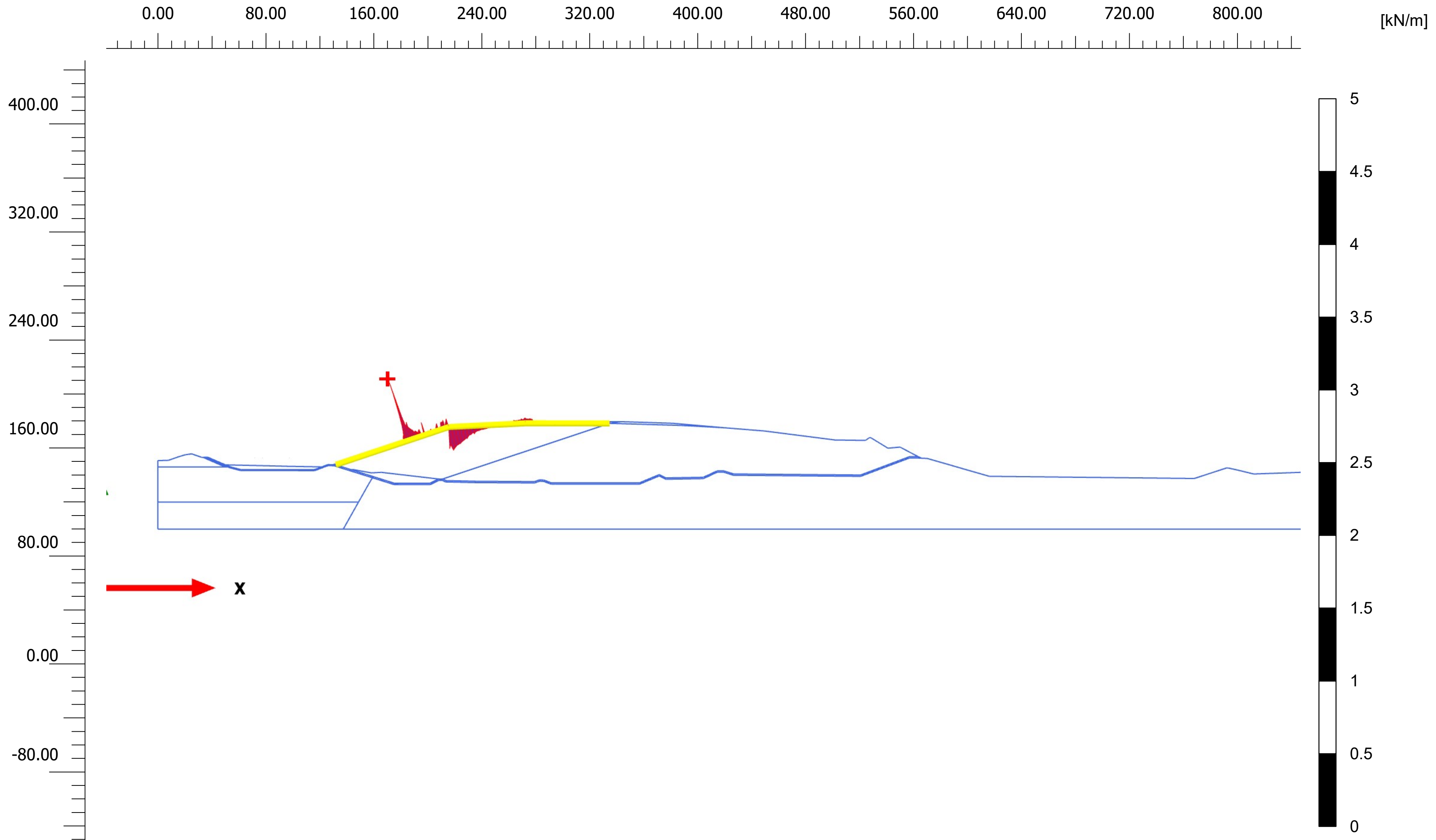
Maximum value = 0.1354 kN/m (Element 64 at Node 32902)

Minimum value = 0.000 kN/m (Element 121 at Node 40748)

PLAXIS® 2D CONNECT Edition

Project description			Date
Poplars EP Consolidation			02/11/2021
Project filename	Step	Company	
Poplars EPVA Revised S ...	551	Sirius Environmental Ltd	

Output Version 21.1.0.479



Axial forces N (scaled up 100 times) (Time 2524 day)

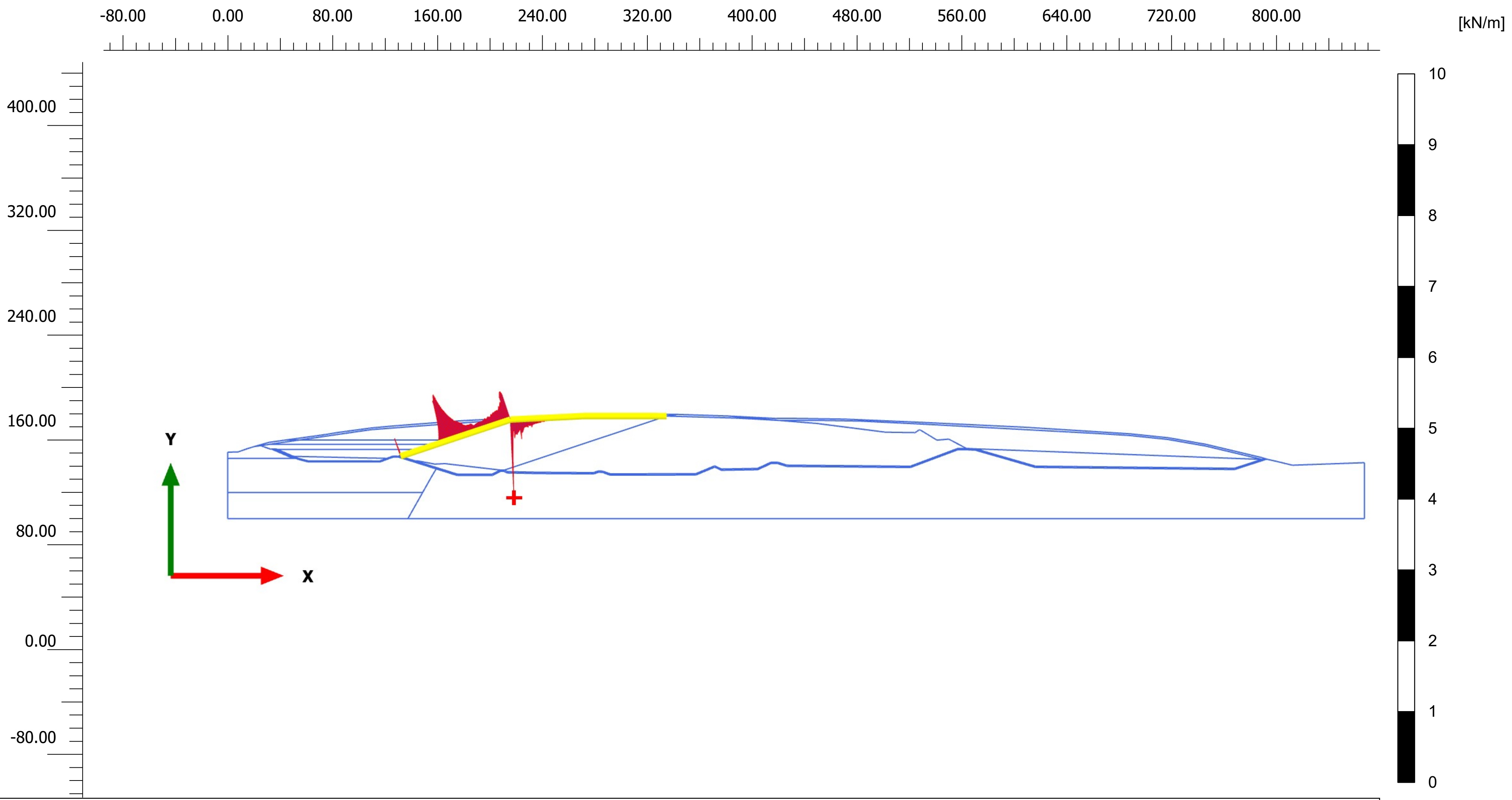
Maximum value = 0.4521 kN/m (Element 199 at Node 42114)

Minimum value = 0.000 kN/m (Element 121 at Node 40742)

PLAXIS® 2D CONNECT Edition

Project description			*Date*
Poplars EP Consolidation			02/11/2021
Project filename	*Step*	*Company*	
Poplars EPVA Revised S ...	549	Sirius Environmental Ltd	

Output Version 21.1.0.479



Axial forces N (scaled up 50.0 times) (Time 3523 day)

Maximum value = 1.087 kN/m (Element 65 at Node 33010)

Minimum value = 0.000 kN/m (Element 230 at Node 43284)

Project description Poplars EP Consolidation			*Date* 09/11/2021
Project filename Poplars EPVA Revised Secti ...	*Step* 974	*Company* Sirius Environmental Ltd	