Slope Stability Assessment of the Tronox EOFS Residue Storage Facility #6

epoch resources (pty) ltd

mine residue and environmental engineering consultants

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Prepared For

Tronox Mineral Sands (Pty) LRDRD

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SLOPE STABILITY ASSESSMENT OF THE TRONOX EOFS RESIDUE STORAGE FACILITY #6

1. INTRODUCTION

Epoch Resources (Pty) Ltd (Epoch) carried out seepage and slope stability analyses as part of the Bankable Feasibility Study (FS) of the Residue Storage Facility (RSF) for Tronox Mineral Sands (Pty) Ltd (Namakwa) for their Tronox Namakwa Sands East OFS Project (EOFS Project). The Project is located in South Africa's Western Province, 54 km North-west of Lutzville and 385 km north of Cape Town.

The RSF will comprise a Residue Dam (RD) and associated infrastructure (i.e. stormwater diversion, access roads, etc.). The RD is a full containment facility that will store residue over the life of mine behind a two-phase, earth embankment. The embankment will be constructed using a tailings waste product from the plant. The intent of the facility is to store residue produced from the Orange Felspathic Sands mined at the East Mine.

This report documents the undertaking of the seepage assessments for the facility under varying operating conditions, and the consequential slope stability determined in terms of:

- Factor of Safety (*FoS*);
- Reliability Index (*RI*); and
- Probability of Failure (*PoF*).

2. TERMS OF REFERENCE

The terms of reference for the project include seepage and slope stability assessments of the RD. This is to confirm that the required FoS against failure for a short-term, medium-term, and long-term slope are satisfied as per the South African regulatory requirements.

The slope stability assessments investigated the effect of static and pseudo-static conditions on the stability of the proposed RD.

3. SCOPE OF WORKS

The scope of works carried out in addressing the terms of reference as described comprised:

- Review of the geotechnical investigation report (Inroads, 2020);
- Assessment of the RD geometry and seepage control infrastructure;

Physical Address	Block A, 8 Viscount Road, Bedfordview, Germiston, 2008, South Africa
Postal Address	PO Box 751306, Gardenview, 2047, South Africa
Telephone	+27 (11) 656 0380/1, +27 (11) 802 3657
Facsimile	+27 (11) 802 3654
Web Address	www.epochresources.co.za
Company Registration	Epoch Resources (Pty) LRD, No 2005/007908/07
Directors	GJ Wiid, Dr. G Papageorgiou, A Savvas

- A finite element seepage analysis to evaluate the development of the phreatic surface within the RD basin due to recharge associated with rainfall and delivery of slurry water, as well as evaluate the phreatic surface developed in the containment wall;
- Deterministic and probabilistic slope stability analysis of the RD, including the application of the results of the seepage analysis, to determine the FoS, RI, and PoF against failure of the facility; and
- Interpretation and evaluation of the results of the analyses against accepted criteria for the long-term stability of slopes.

4. PERTINENT REGULATIONS AND STANDARDS

South African regulations provide the framework to which the design of an RSF must comply. A multitude of design standard and specifications may be consulted if the South African National Standards (*SANS*) does not provide sufficient spectrum, or where SANS refers to or specifies another standard. In some instances, neither regulation nor design standard may provide enough design framework for compliance in which case industry best practice guidelines may be referred to.

4.1. SOUTH AFRICAN REGULATIONS

The management of clean and dirty/mine contaminated water is regulated by several Acts, namely:

- The National Environment Management: Waste Amendment Act No. 26 of 2014 (*NEMWA*);
- The National Water Amendment Act No. 27 of 2014 (*Water Act*);
- The National Minerals and Petroleum Resource Development Amendment Act No. 49 of 2008 (*Minerals Act*).

It must be noted that NEMWA will be assumed to supersede any similar regulations covered by the older Minerals Act.

5. INFORMATION RECEIVED

During the completion of the slope stability assessment of the RD, several variables needed to be identified and, if need be, quantified. This process required the use of various sources of information. These sources are listed below:

- Geotechnical investigation report in 2020 by Inroads Consulting (Inroads), including rotary core drilling and associated geotechnical laboratory test work; and
- Geotechnical laboratory testing on the residue and tailings products.

The information obtained from the above-named sources is discussed in the section below.

5.1. GEOTECHNICAL INVESTIGATION

A geotechnical investigation of the proposed site was undertaken by Inroads between the 21st of August and the 3rd of September, and the results of the near-surface investigation were published in their report: "*Report on a geotechnical investigation for the proposed residue storage facility for the Tronox Namakwa Sands EOFS project in Brans-se-Baai, Western Cape*".

The focus of the investigation was to determine the geotechnical parameters and depths of the in-situ soil horizons in the vicinity of the RSF for seepage and stability analyses, as well as to identify any problem soils which could affect stability or soil permeability. The location of the test pits (*TPs*) investigated relative to the proposed RSF geometry is illustrated in Figure 5-1.

During 10 to 19 December 2020 and 7 to 13 January 2021 a total of six rotary cored boreholes were drilled to 20 m within the RSF while an additional two holes were drilled in the overburden site. All test pits and boreholes were profiled by Inroads using standard methods and procedures set out in the document "*Guidelines for Soil and Rock Logging in South Africa (2002)".*

5.2. SOIL PROFILE

Inroads undertook to investigate and provide typical soil profiles of 116 Test Pits (TPs) within the area of the RSF. However, due to time constraints, a total of 24 TPs within the RSF were forgone during the investigation. A Tractor Loader Backhoe (TLB) was used to excavate the TPs to depths ranging between 0.2 and 3.5 m. Soil profiling was undertaken during the investigation in an attempt to determine the individual layers, or horizons, of the underlying soils.

The top horizon of the RSF area can be subdivided into two areas, namely the unmined and rehabilitated areas. The unmined area forms the largest portion of the RSF and is comprised of very loose dune sand that extends to an average depth of 2 m. Beneath the dune sand is a layer of silty sand of aeolian origin that was encountered at depths ranging between 0.9 to 3.3 m. The aeolian material was occasionally loose but mostly medium dense to dense silty sand with scattered friable weakly cemented pockets. The aeolian extended to the bottom of most of the test pits with a few test pits contained very dense aeolian material, causing the TLB to partial refuse.

Boreholes NRSF01, NRSF06 to NRSF08 drilled within the unmined area indicate that the aeolian horizon extends to depths greater than 20 m. Standard Penetration Tests (SPT) showcased that the soil horizon becomes very dense with N values increasing from between 20 to 32 at a depth of 2.2 to 3.5 m to mainly above 50 below 3.5 m.

The rehabilitated area comprises very loose fill to a depth of between 1.1 to 3.2 m. The fill material generally extended to the bottom of the pits or was underlain by loose aeolian and in some rare cases by moderately cemented very dense and very soft rock gneiss. No groundwater was encountered in any test pits excavated during the investigation.

Boreholes NRSF02 and NRSF05 drilled along the southern wall of the RSF and within the rehabilitated area, show either very soft rock dorbank or completely weathered granite gneiss to underly the fill and aeolian sand at depths between 4.5 and 12 m.

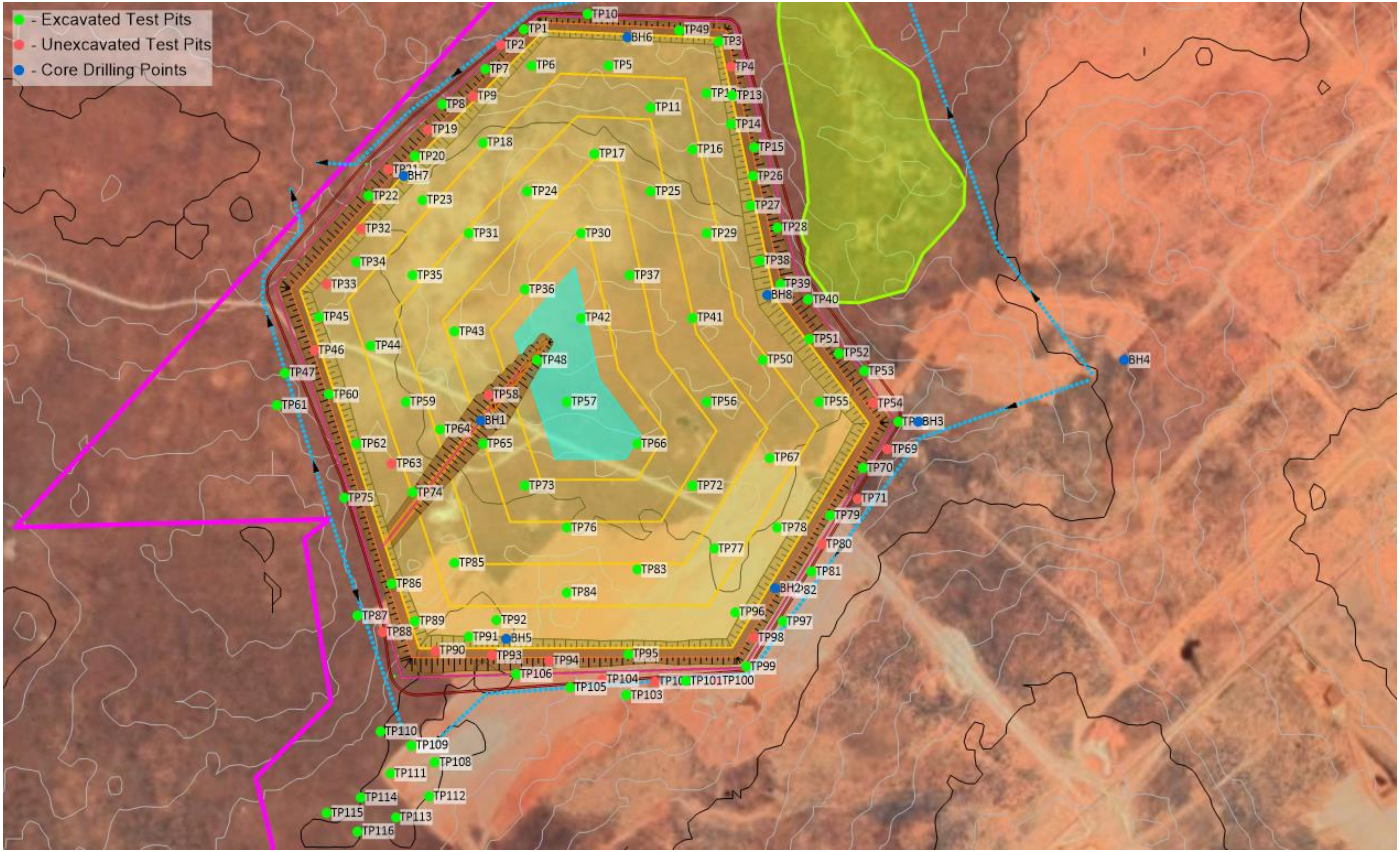


FIGURE 5-1: RSF TEST PIT LOCATIONS

5.3. MATERIAL STRENGTH PARAMETERS AND HYDRAULIC CONDUCTIVITY

Both disturbed and undisturbed representative soil samples were collected during the site investigation that took place from 21st August 2020 to 3rd September 2020. The Particle Size Distributions (PSDs) and Atterberg limits of the dune sand, fill and aeolian sand was determined. The results of the test indicate that the soils present within the RSF basin and beneath the embankment are uniformly non-plastic or slightly plastic. The samples tested consisted mainly of sand fractions, which comprised 87 to 99 % of the samples by mass, with the remainder including fractions of silt and clay. Other tests conducted on the sample include proctor compaction tests, slowly drained shear box tests, oedometer and saturated consolidometer tests. The permeability of the various selected soils samples was determined using the flexible wall triaxial cell test. The hydraulic conductivity values were then utilized in the seepage analyses of the RD. The strength parameters were used in the analysis of the slope stabilities in conjunction with the results of the seepage analyses. Table 5-1 presents the geotechnical parameters recommended by Inroads to be used for the design of the RSF.

TABLE 5-1: GEOTECHNICAL PARAMETERS OF MATERIALS CLASSIFIED IN TEST PITS

Soil Horizon	Layer Thickness (m)	Unified Classification	Φ' (degrees)	C' (kN/m^2)	ρ_d (kN/m^3)	K (m/sec)
Fill & dune sand (very loose in-situ)	3.5	SP	28	0	1400	10^{-4}
Fill & dune sand (compacted to 98 % proctor)		SP	35	0	1600	10^{-5}
Aeolian – silty sand (weakly cemented in places)	3.5	SP / SP - SM	32	0	1600	10^{-6}
Aeolian – silty sand (compacted to 98 % proctor)		SP / SP - SM	37	0	1800	10^{-5}
Weakly cemented aeolian, residual, weak dorbank (Very dense to very soft rock)	15	SP / SP - SM	40	0	1900	10^{-7}

Φ' = effective friction angle; c' = effective cohesion; ρ_d = dry density; k = coefficient of permeability

5.4. GEOTECHNICAL TESTING OF TAILINGS SAMPLES

Geotechnical testing was conducted on a sample of the RAS and EOFS tailing products. The summary of the average result of these tests are listed below:

- Friction Angle – 30°;
- Cohesion – 2 kPa;
- Unit weight – 16.6 kN/m^3; and
- Hydraulic conductivity – 2.3 x 10^{-5} $m.s^{-1}$.

6. RSF LAYOUT

The configurations of the RSF is based on the preferred site determined through a site selection process and has been optimised for efficient use of the available footprint area. The RSF is designed as a full containment facility with a two-phase embankment wall that will be built to the final elevation during the initial construction period with 1VH2.5H slopes for both the upstream and downstream slopes of the facility. Afterwards, the downstream slope will be reshaped to a 1V:5H slope for the closure phase of the project. The containment wall will be built from Product tailings transported from the Primary Concentration Plant (PCP) via conveyors or trucks. Conventional compaction methods will not be undertaken, instead the material will be shaped to the required embankment geometry, during which it is estimated that the traffic load will provide sufficient compaction to yield the required strength parameters, as discussed in the geotechnical investigation report. The method of construction stems from the previous facilities that have been constructed at the project location.

The embankment will also contain a blanket drain to prevent the phreatic surface from rising within the wall and saturating the downstream toe of the facility. Stormwater diversions are included in the design of the facility to prevent high runoff water from pooling at the downstream base of the embankment. The diversions also aim to keep water flowing at high velocity away from the embankment toe to prevent erosion from occurring. An illustration of the RSF, associated infrastructure and mining boundary (EMP boundary) can be seen in Figure 6-1.

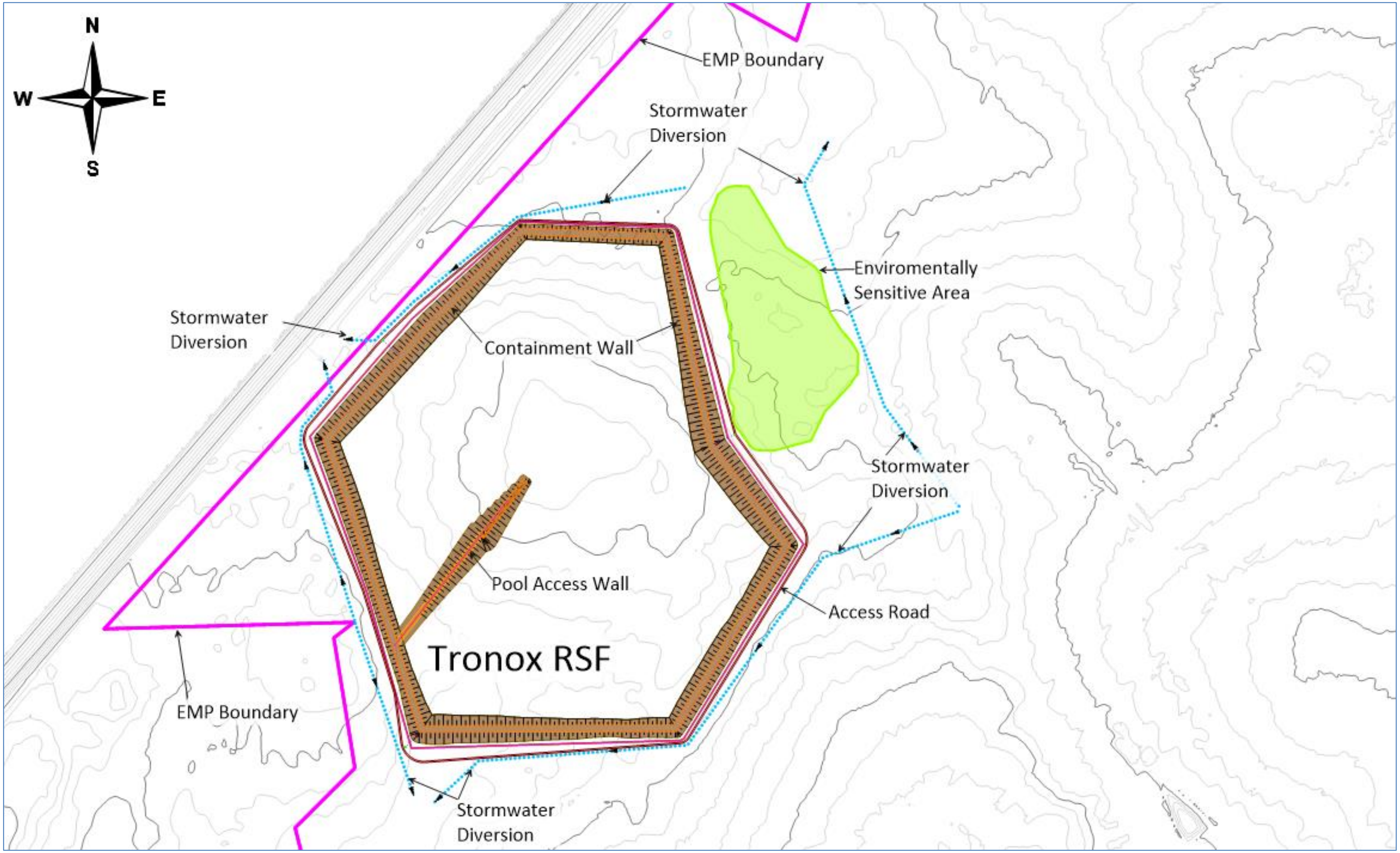


FIGURE 6-1: PLAN VIEW OF THE PROPOSED RSF AT FINAL ELEVATION

7. METHODOLOGY

Seepage and slope stability analyses were carried out based on the configuration of the RD at a critical section where the wall height is the greatest and the pool is the shortest distance from the embankment. The purpose of the analyses was to:

- Determine the phreatic surface in the RD based on various operational conditions; and
- Estimate the factor of safety against failure of the RD based on the shear strength parameters of the residue material, in-situ soils and containment wall construction material, as well as the phreatic surface profile within the RD wall for the different analysed scenarios.

The methodology of determining the seepage regimes within the RD and the associated FoSs against failure comprised:

- Review of the information arising from the geotechnical investigation of the site to incorporate the hydraulic conductivity and shear strength parameters of the in-situ foundation materials;
- Review of the information obtained from tests completed on residue and tailings samples to incorporate the shear strength parameters and hydraulic conductivity of the residue and tailings;
- The development and evaluation of seepage and slope stability models based on the configuration of the RD where necessary, to determine:
  - The likelihood of the phreatic surface rising to unsafe levels;
  - Factors of safety against failure of the facility;
  - The Probability of Failure of the facility; and
  - The Reliability Index of the facility.

Two separate sets of analyses were carried out on two-dimensional models using the GeoStudio 2021 suite. In the first set of analyses, all models conformed to the proposed RD configuration during the operational phase. The second set of analyses investigated a model that conformed to the closure phase. The most critical cross-section of the facility was modelled to obtain the Factor of Safety for the worst-case scenarios. The steady-state seepage regimes within the RD, for the critical cross-section, were determined using GeoStudio’s Seep/W and were imported into Slope/W to analyse their stability.

8. SEEPAGE ANALYSES

Seepage analyses were undertaken to model the development of a phreatic surface within the RD under varying operating conditions. An increase in pore-water pressure, brought on by the onset of seepage, can result in the reduction in the stability of an earth structure’s slope and has other adverse secondary effects, such as:

- Piping (erosive loss of material);
- Loss of effective strength of the material;
- Increase in the liquefaction potential of soils; and

- Increase in the collapse potential of sensitive soils.

It is therefore imperative not only for the designer to take cognisance of the above but also for the construction of the facility to be as per the design and for the operator of the RSF to ensure that best-operating practices are adhered to at all times.

8.1. SEEPAGE METHODOLOGY

Determination of the steady-state phreatic surface generated by the RD pool under varying conditions is conducted using Finite Element Methods (FEM) in the GeoStudio Seep/W suite. The software generates a “mesh” of elements across a typical geometry consisting of:

- RD cross-sectional geometry;
- An assumed residue and/or water level; and
- In-situ soil profile determined by Inroads during the geotechnical investigation.

Seepage analyses of the RD were carried out using the finite element program Seep/W to assess the location of the phreatic surface that would develop under various conditions, such as:

- During the operational phase:
  - Functional drains; and
  - Inactive drains;
- During the closure phase:
  - Functional drains; and
  - Inactive drains;

Each finite mesh element is assigned a set of parameters based on the geotechnical properties of the relevant material’s hydraulic properties and assumed boundary conditions which may include:

- Hydraulic Conductivity;
- Volumetric water content;
- Anisotropy;
- A water source;
- Potential seepage faces; and
- Drainage points.

The phreatic surface may drastically affect the stability of a slope, which is due to the reduction in shear strength along a potential slip surface. The objective, therefore, is to ensure that the phreatic surface is correctly defined before determining the stability of the facility.

8.1.1. INPUT PARAMETERS TO SEEPAGE MODEL

The soil USCS classifications and hydraulic conductivities used are listed in Table 8-1.

TABLE 8-1: LIST OF HYDRAULIC PARAMETERS

Material	Anisotropy Ky'/Kx' Ratio	Saturated Hydraulic Conductivity ($m.s^{-1}$)	Saturated/Unsaturated Condition
Residue	0.5	4.03×10^{-8}	Saturated only
Embankment (Tailings)	1	1.00×10^{-5}	Saturated/Unsaturated
Drains	1	1.00×10^{-3}	Saturated only
Aeolian (Silt)	1	1.00×10^{-6}	Saturated/Unsaturated
Aeolian (Slightly Cemented)	1	1.00×10^{-7}	Saturated/Unsaturated

The insitu soils are underlain by fractured bedrock. As the water tightness and preferential flow paths of the bedrock are unknown, for the purpose of the seepage analysis the bedrock was modelled as being an impermeable layer. The natural topography of the site is that of a depression (bowl), with the insitu soil layers and bedrock following the shape of the depression. By modelling the bedrock as impermeable, seepage into the underlying soils is allowed to accumulate above the bedrock and dissipating horizontally. The model was set up in this manner to provide a worse-case scenario for the stability models.

The bedrock is however fissured, with the mounding of seepage expected only in localised pockets under the RSF and as such the model should be reassessed during the detailed design phase of the project once the permeability of the bedrock is confirmed.

A critical 2-Dimensional section was selected for analysis based on the following:

- The height of the RD above ground level;
- The slopes associated with the RD containment walls;
- Relative location of supernatant water from sensitive RSF infrastructure;

8.1.2. CONFIGURATION OF SEEPAGE MODELS

Once all the required input parameters have been allocated as necessary, it is possible to compute the steady-state condition by determining the location of the water table (phreatic surface, or zero pore water pressure) under the given criteria and conditions. The Critical Section of the RD used for the Seepage and Stability analyses are illustrated in Figure 8-1. The typical model setup for the RD along the Critical Section is illustrated in Figure 8-2 to Figure 8-4. The RSF was assessed with a centre banket drain, upstream toe blanket drain and no drains, respectively, with the operating pool located in the centre of the RSF and with the pool located 100 m form the upstream face of the containment wall.

The construction of the facility will be a two-phase process. During the initial phase, the facility will be constructed with 1V:2.5H side slopes for both the upstream and downstream slopes and a 30 m crest to allow adequate space for construction vehicles to spread the tailings material. During the second phase, the slope of the embankment's downstream face will be flattened to a 1V:5H by reshaping the existing material. Subsequently, the crest width will be reduced to 15 meters. All models feature a key with a depth of 0.5 m that extends from the downstream toe of the models to 5 meters past the downstream blanket drain.

It was assumed that the surface layer of dune sand will be removed and sent to the mines processing plant before the construction of the embankment starts. The facility was modelled on top of a 3.5 m layer of silty sand of Aeolian origin, underlain by a 15 m layer of slightly cemented Aeolian silty sand. A layer of bedrock was included beneath the slightly cemented Aeolian silty sand layer to account for the very soft rock dorbank found in some of the boreholes.

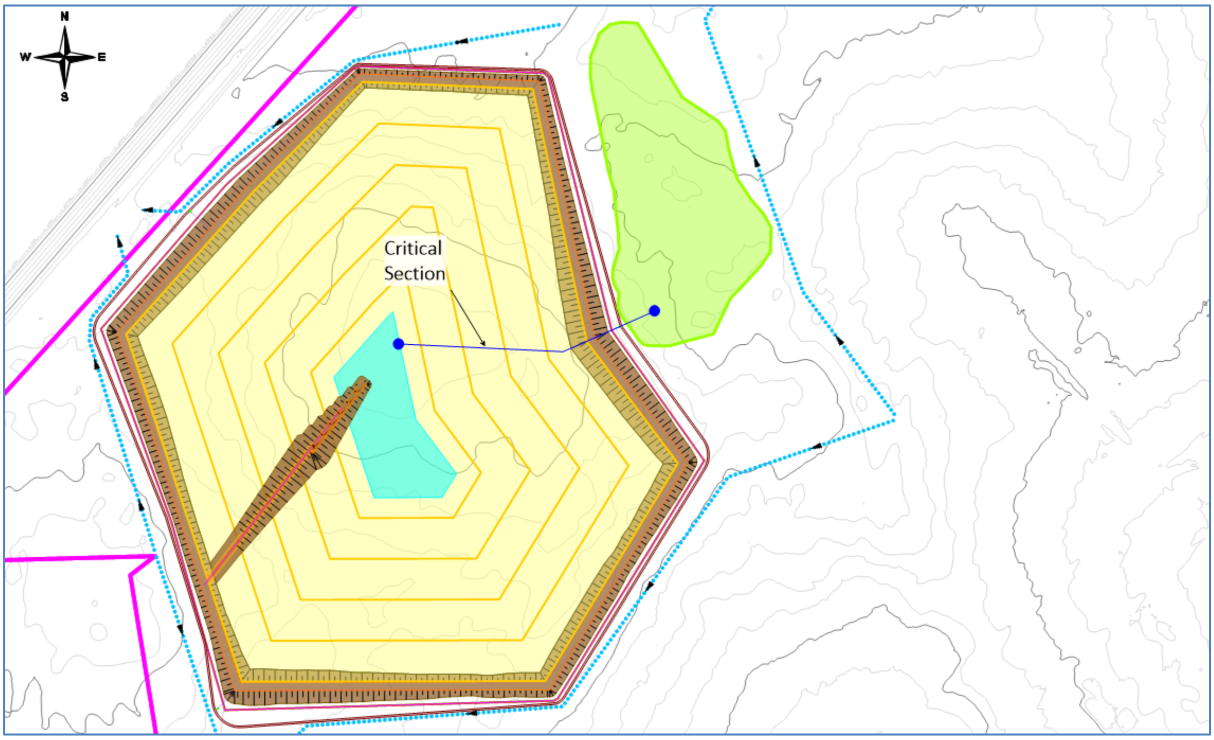


FIGURE 8-1: CRITICAL SECTION ACROSS THE RD

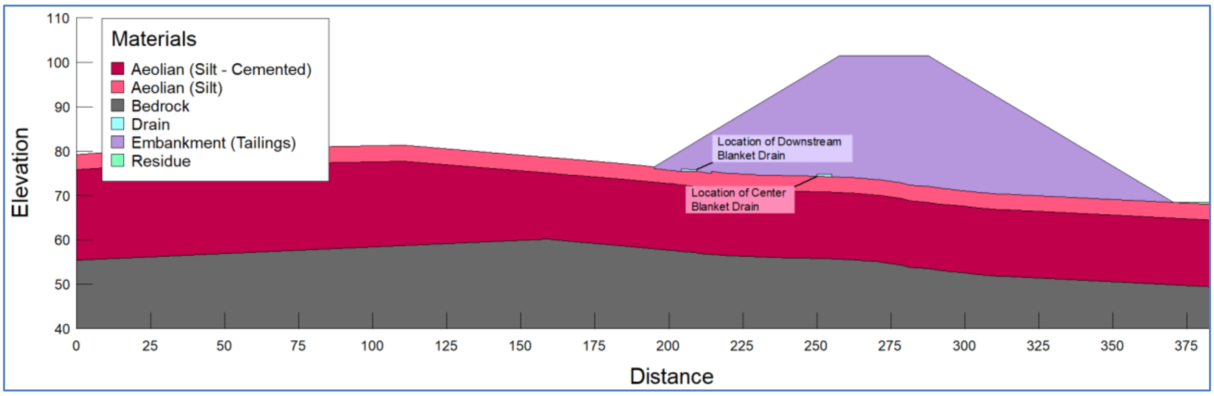


FIGURE 8-2: OPERATIONAL PHASE - INITIAL

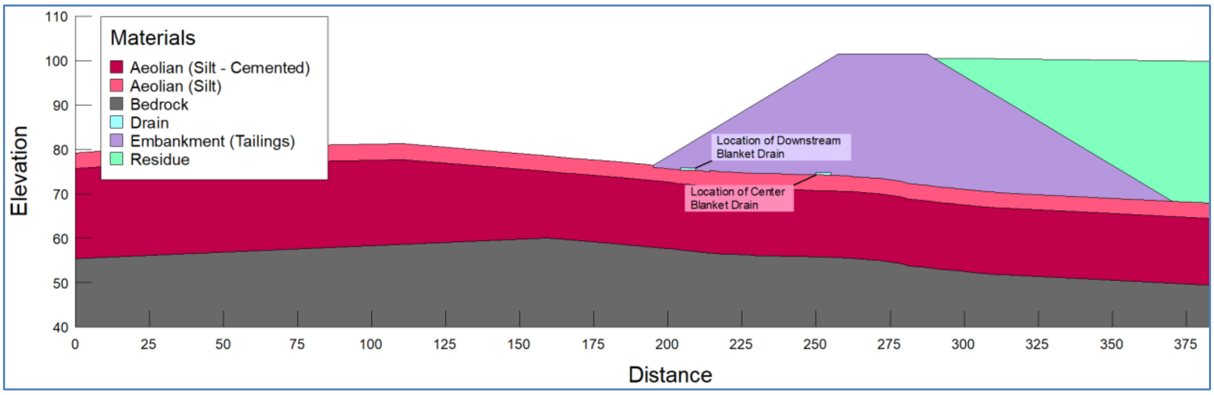


FIGURE 8-3: OPERATIONAL PHASE – RESIDUE AT MAXIMUM CAPACITY

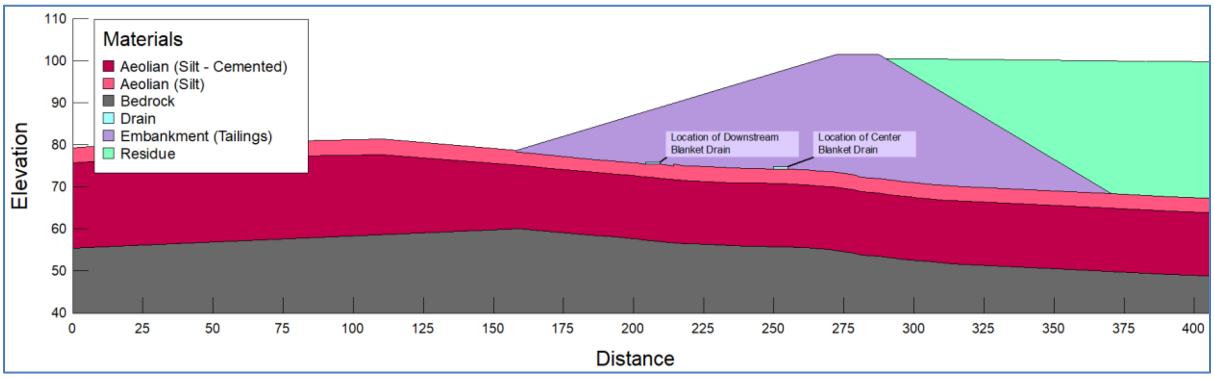


FIGURE 8-4: CLOSURE PHASE

Each scenario was modelled with the pool in the centre of the facility and an upset condition with the pool located 100 m from the upstream face of the containment wall. A water balance conducted by Epoch titled "*Water Balance Study for the Tronox EOFS Residue Storage Facility*", revealed that the pool volume would not exceed 43 328 m^3 at any given point, during the operational life of the facility. The volume of water expected to report to the RSF during the 1 in 200-year return period storm event (including the operational pool) is 300 000 m^3 and discharged off the facility over a period of 30 days. The seepage models assume a pool volume of 300 000 m^3 which is considered a conservative approach as the analysis are run under steady state conditions.

9. SEEPAGE ANALYSIS RESULTS

The critical cross-section was assessed for scenarios the two pool positions. For cosiness, only models analysed with the pool situated 100 m for the embankment wall are discussed in the following section. The results of all the seepage assessments for the RD are provided in Appendix I.

9.1. SEEPAGE ANALYSIS RESULTS OF INITIAL OPERATIONAL PHASE

The model presented in Figure 9-1 illustrates a typical cross-section along the Critical Section during the initial portion of deposition when the residue material starts encroaching on the upstream toe of the facility. This scenario is seen as the worst-case as the deposited material could lead to the saturation of the upstream toe should a significant storm event occur. Further analysis showed that increasing the residue level resulted in an increased FoS. These models were therefore not included in the main body of this report, however, they can be found in appendix I and II.

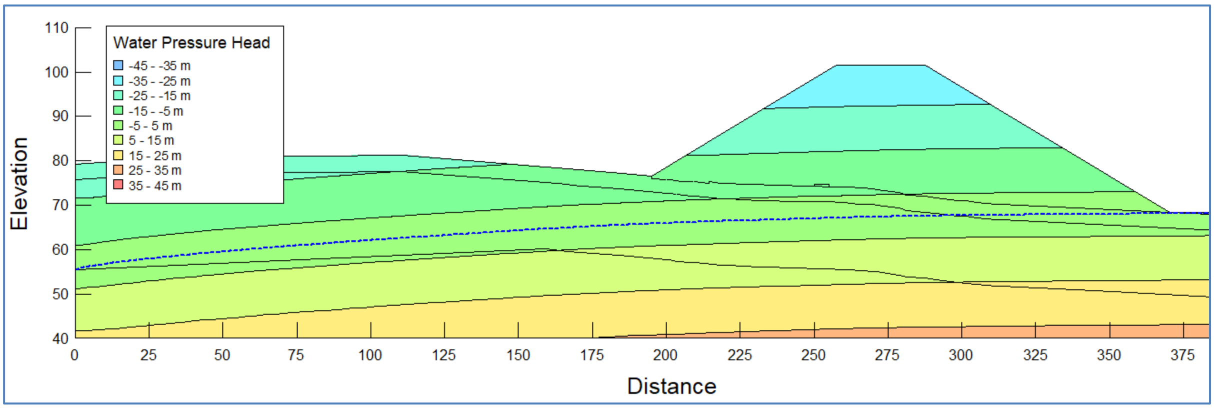


FIGURE 9-1: INITIAL OPERATIONAL PHASE, SEEPAGE ASSESSMENT OF THE RD WITH AN ACTIVE CENTRE BLANKET DRAIN

The embankment illustrated in Figure 9-1 consists of upstream and downstream slopes equal to 1V:2.5H and a 5 m wide centre blanket drain. No further models were included for this scenario as it is shown that the phreatic surface remains below the blanket drain thus indicating that excluding the drains from the analysis would have no significant impact on the phreatic surface within the embankment.

9.2. SEEPAGE ANALYSIS RESULTS OF OPERATIONAL PHASE AT CAPACITY

Figure 9-2 to Figure 9-4 illustrates the effect a blanket drain would have on the phreatic surface within the embankment. It is shown that, due to the topography and underlying soil profile, a central blanket drain is the most effective means by which to decrease the phreatic surface (Figure 9-2). However, similarly due to the topography, deep manholes (exceeding 6 m in depth) will need to be excavated in order to reach the blanket drain outlets. Therefore, it is believed that a downstream toe drain is the most feasible means by which to prevent saturation of the downstream toe.

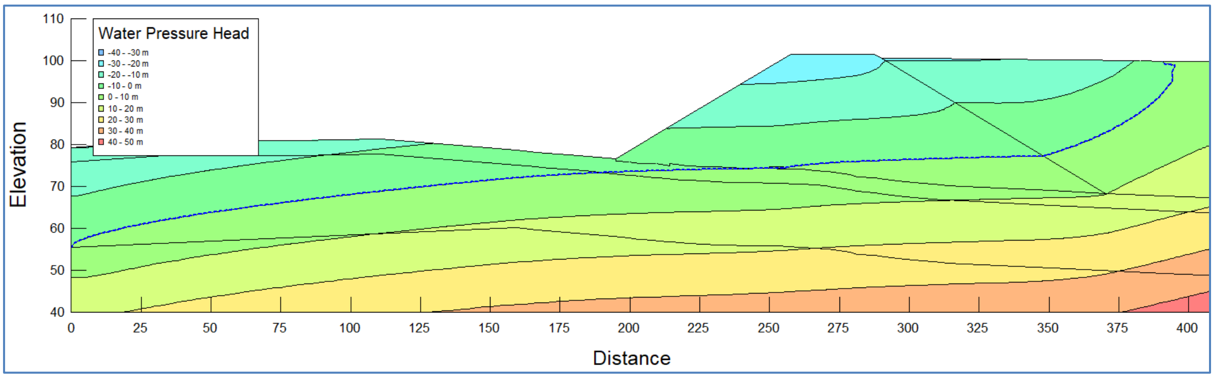


FIGURE 9-2: OPERATIONAL PHASE AT CAPACITY, SEEPAGE ASSESSMENT OF THE RD WITH AN ACTIVE CENTRE BLANKET DRAIN

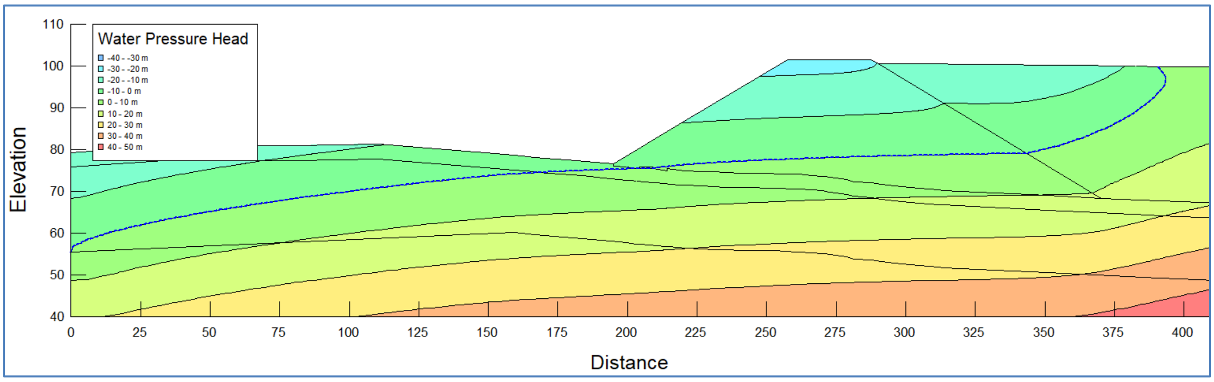


FIGURE 9-3: **OPERATIONAL PHASE AT CAPACITY, SEEPAGE ASSESSMENT OF THE RD WITH AN ACTIVE DOWNSTREAM BLANKET DRAIN**

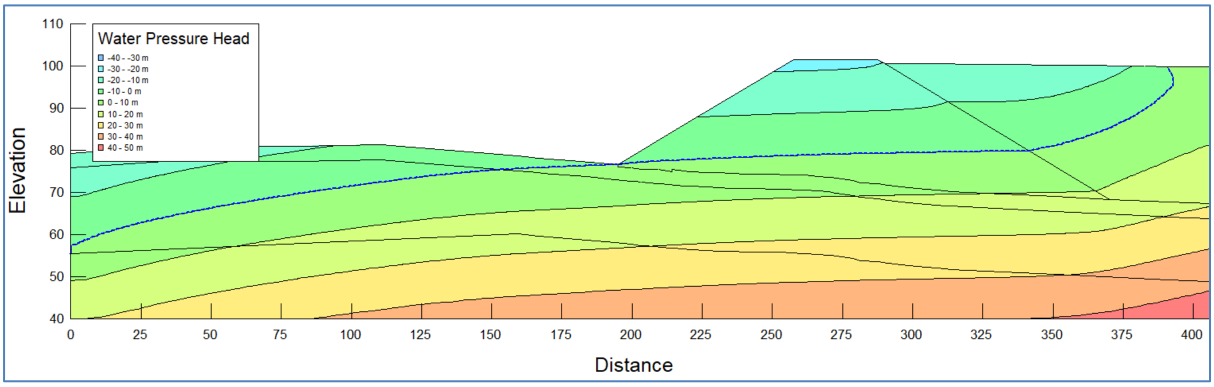


FIGURE 9-4: **OPERATIONAL PHASE AT CAPACITY, SEEPAGE ASSESSMENT OF THE RD WITH NO ACTIVE DRAINS**

9.3. SEEPAGE ANALYSIS RESULTS OF CLOSURE PHASE AT CAPACITY

The closure phase of the facility is depicted in Figure 9-5 to Figure 9-7. It is shown that, as during the operational phase, the downstream blanket drain is an effective means by which the phreatic surface can be decreased within the embankment. The inclined slope of the topography on which the embankment is to be built further improves the separation between the phreatic surface and downstream toe as downstream slopes are reshaped from a 1V:2.5H slope to a 1V:5H slope. This will decrease the likelihood that the downstream toe will become saturated, preventing piping as well as a decrease in the effective strength of the material as it becomes saturated.

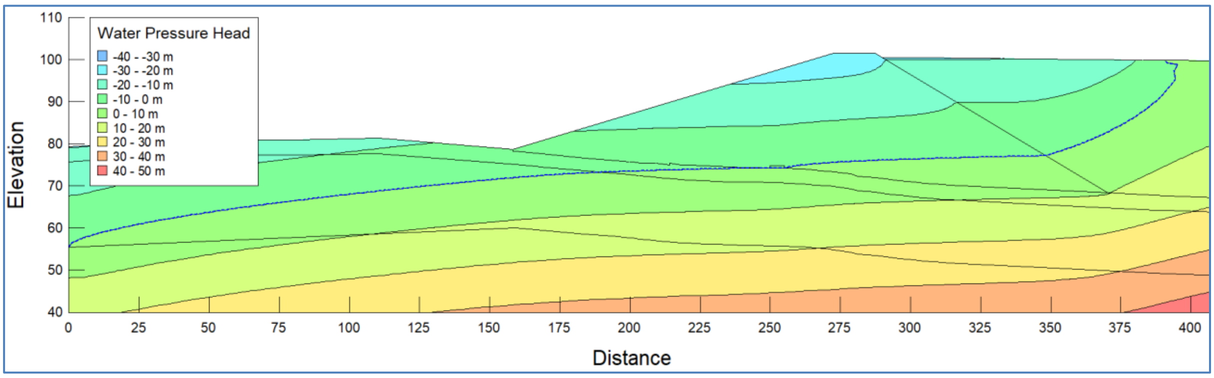


FIGURE 9-5: **CLOSURE PHASE AT CAPACITY, SEEPAGE ASSESSMENT OF THE RD WITH AN ACTIVE CENTRE BLANKET DRAIN**

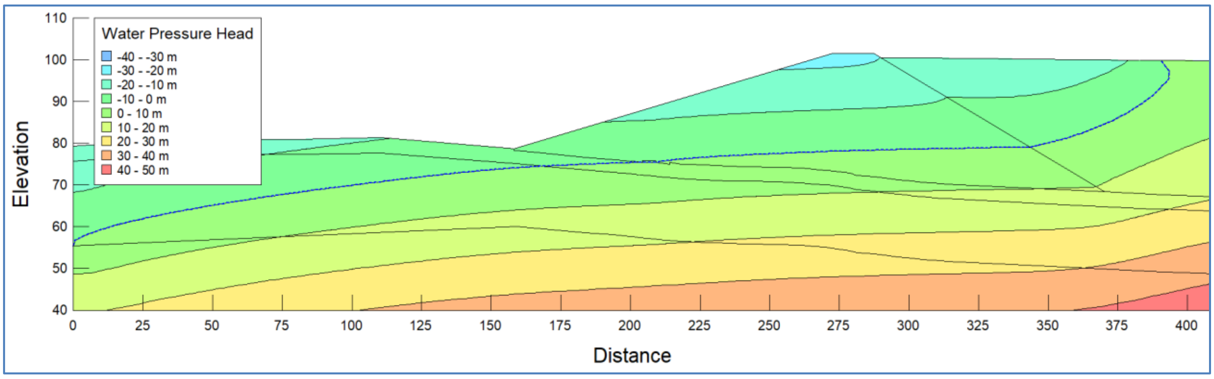


FIGURE 9-6: **CLOSURE PHASE AT CAPACITY, SEEPAGE ASSESSMENT OF THE RD WITH AN ACTIVE DOWNSTREAM BLANKET DRAIN**

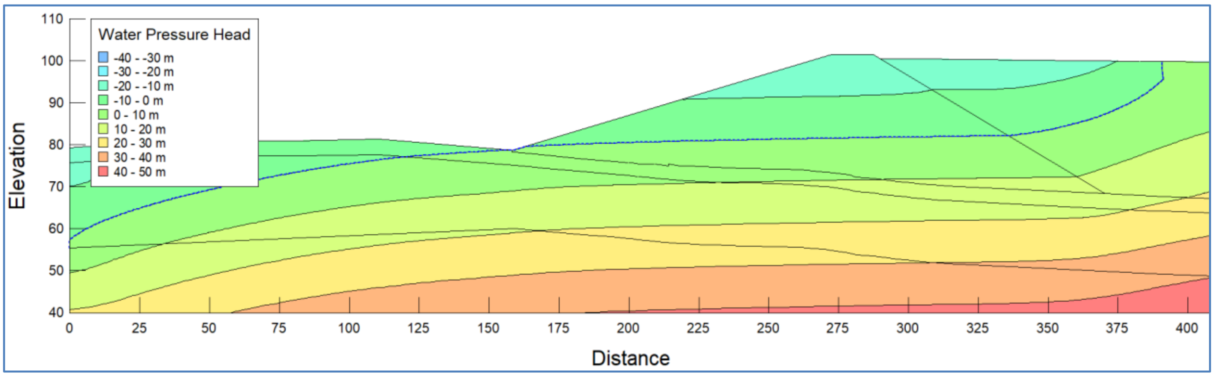


FIGURE 9-7: **CLOSURE PHASE AT CAPACITY, SEEPAGE ASSESSMENT OF THE RD WITH NO ACTIVE DRAINS**

9.4. DISCUSSION OF RESULTS

The high permeability of the embankment material, compared to that of the residue material, results in the phreatic surface decreasing rapidly within the containment wall, it should be noted that the topography and underlying soil profile does not allow water to daylight a distance downstream of the facility. Instead, water seeps from the toe of the facility if no drains are included. Although this does not result in a build-up of pore water pressure through the wall, seepage through the downstream toe of the embankment increases the potential for erosion of the embankment and for piping through the wall to occur. Under these conditions it is recommended that a blanket drain be included in the wall.

It is recommended that the state of the underlying bedrock be further investigated, and the seepage models reassessed in the detailed design phase of the project. Should the permeability of the bedrock be such that the phreatic surface does not build up beneath the footprint of the RSF, the need for the blanket drains may be negated.

Piezometers will be installed in the RSF walls to monitor the phreatic surface within the walls. These are to be installed prior to the commissioning of the facility.

10. SLOPE STABILITY ANALYSIS

A slope stability analysis was completed to assess the safety of the slopes of the RD under varying conditions. The following sections describe the process by which the analysis was undertaken.

10.1. METHODOLOGY

To analyse the stability of a slope requires that the Factor of Safety against the failure of the slope

be determined as well as the associated Probabilities of Failure and the Reliability Index of the analysis. The level of uncertainty associated with the long-term stability of a slope is a function of the level of uncertainty associated with:

- The shear strength parameters of the materials comprising the slope and its foundation as expressed in terms of their friction angle and cohesion; and
- The location of the phreatic surface within the slope.

The risk level, or Probability of Failure that may be tolerated for a given slope, depends on:

- The level of risk to the stakeholders (including downstream property owners, authorities, the mine owner and consultants) are willing to accept;
- The level and extent of quality control and quality assurance undertaken during construction;
- Whether the facility is in the operational phase or post-closure phase; and
- Whether or not the side slopes are monitored.

10.1.1. FACTOR OF SAFETY

The Factor of Safety against the failure of a slope is a ratio between opposing forces: the forces causing failure (gravity forces of the material weight) and the forces preventing failure (shear strength of the soils).

South African legislation as documented in the NEMWA Act No. 59 of 2008 and Regulation 632 (24 July 2015) Chapter 2, 7 (4)(d), says:

"Other design considerations, as appropriate to the particular type of residue stockpile and residue deposit that must be incorporated include:

(d) keeping the pool away at least 50 meters from the walls and a factor of safety not less than 1,5; where there are valid technical reasons for deviating from this, adequate motivation must be provided, and the design must be reviewed by a competent person".

Therefore, the RD has been designed in order to achieve the factor of safety of 1.5 during the operational and closure phase under static loading and pseudo-static loading.

10.1.2. LIMIT EQUILIBRIUM METHODS

The slope stabilities under varying conditions as discussed are determined through Limit Equilibrium Methods (*LEM*) which assesses the equilibrium of forces and moments from a series of pre-defined slices through a potential slip surface of a slope. Many methods of LEM are available which make use of different assumptions of the equilibrium condition that exists between the slices. The following are some advantages of using limit equilibrium methods:

- Provides a single FoS for the whole slope;
- Relatively low calculation effort required; and
- Methods are well-calibrated to field methods.

The RD was assessed using the “Morgenstern-Price” method which takes cognisance of the following:

- Unit weight of each slice (W);
- Normal force to the slip surface (N);
- Shear force acting on the slip surface (S);
- Slice moment (M);
- Slice horizontal force (F);
- Inter-slice normal forces (E);
- Inter-slice shear forces (X); and
- Variable inclination between the results of the ratio of normal and shear forces (δ).

The main reasons for selecting this method are as follows:

- This method makes use of a differential equation that is derived for the equilibrium conditions thus this method ensures that the equilibrium of forces is adhered to;
- Integration along the failure plane ensures more accuracy by considering all materials on the failure plane;
- The solution is obtained once the boundary conditions are met which means that the zero inter-slice forces are present at the last slice which equates to equilibrium being met;
- Provides a single explicit number for Factor of Safety against failure; and
- This method is the most accurate compared to the other LEMs.

Typical slice forces and moment as per the Morgenstern-Price method are illustrated in Figure 10-1.

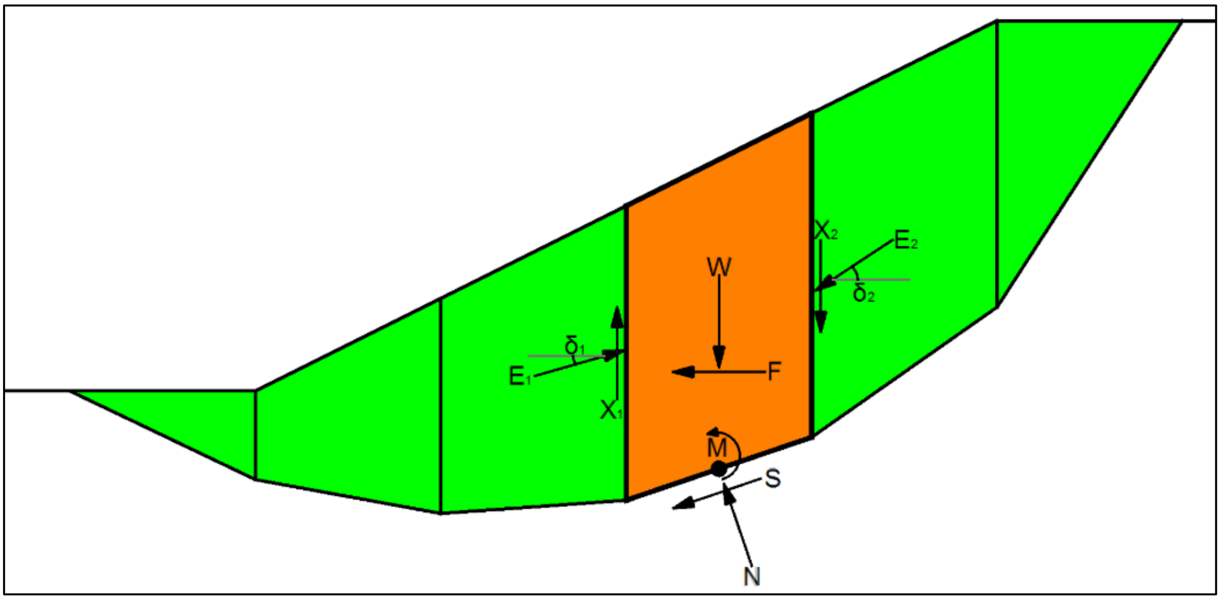


FIGURE 10-1: TYPICAL FORCE AND MOMENT DIAGRAM FOR THE MORGENSTERN-PRICE LEM

The forces and moments are solved assuming a state of equilibrium for each slice within an assumed slip surface iteratively until the solution converges to a constitutive FoS for the entire slip surface. A slip surface which presents the lowest FoS solution is considered the critical slip surface to which the RD design caters for.

As there is an infinite number of slip surfaces that may be analysed, with any of which yielding or not yielding the most critical slip surface, specialised software has been developed to efficiently determine the location and FoS of a critical slip surface. For the Kakula RSF, GeoStudio's 2018 version of Slope/W was used which utilises the method as explained to determine the critical slip surface within a user-defined region. The required inputs for the LEM to operate are:

- Initial pore water pressures (determined with seepage modelling);
- A material failure criterion (Mohr-Coulomb);
- Soil strength parameters including;
    - ➢ Cohesion (c');
    - ➢ Friction Angle (φ'); and
    - ➢ Bulk Density.

10.1.3. PROBABILISTIC ANALYSIS

To allow for variability in the input parameters, a probabilistic analysis is conducted. The software is provided with the probabilistic distribution of the design parameters which includes:

- Type of distribution i.e. Normal distribution, Log-normal distribution etc.;
- The mean; and
- The standard deviation.

A finite number of Monte Carlo trials are conducted which selects, at random, combinations of new parameters within the defined probabilistic distribution. These randomly selected parameters are applied to the critical slip surface which is determined by the deterministic analysis. The FoS from each of the Monte Carlo simulations is recorded as it converges to an overall solution from which a *Reliability Index (RI)* and *Probability of Failure (PoF)* is determined. A sufficient number of Monte Carlo trials are required to ensure that all materials strength parameter distributions have been accounted for in the stability analyses.

The PoF is defined as the number of Monte Carlo trials that resulted in a FoS less than one represented as a percentage of the total number of trials conducted. For long term slopes, a PoF less than 0.0007% (<1:143 000) is widely accepted. Recommended PoFs for short- and medium-term slopes should not exceed 0.07% (1:1 430) and 0,007% (1:14 300) respectively (Cole, 1993).

The RI is defined as the number of standard deviations separating the defined failure FoS of 1.0 from mean FoS that the Monte Carlo simulation converged towards. A Reliability Index of 4.83 correlates to the minimum acceptable PoF, thus values greater than (>) 4.83 is considered acceptable for a long term, or permanent slope.

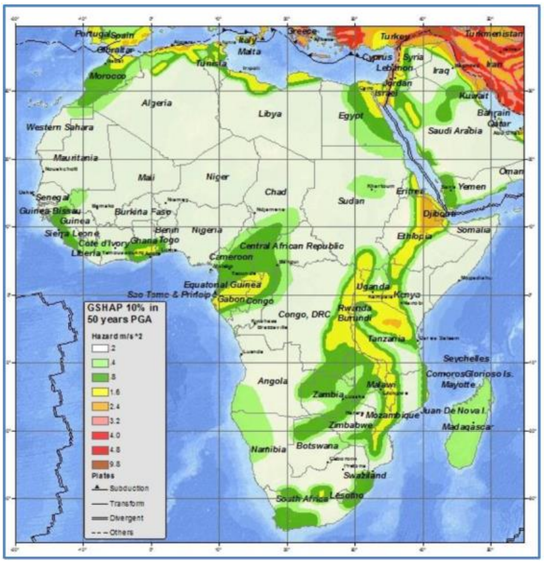
10.1.4. SEISMICITY ASSESSMENT

The horizontal force imposed on the structure when undertaking a pseudo-static analysis is derived from the Peak Ground Acceleration (PGA) parameter. PGA values are based on prior earthquakes and fault studies and are measured as factors of the earth's gravitational acceleration (i.e. 1g is equivalent to 9.81 $m.s^{-2}$).

The minimum allowable Factor of Safety for side slopes, according to NEMWA, is 1.5. Deviations from the prescribed minimum FoS must be substantiated by the designer.

The Peak Ground Acceleration (PGA) for Namakwa will be about 0.04g, based on a 10% probability of exceedance in 50 years from the Global Seismic Hazard Assessment Program (GSHAP) study (Figure 3-1) and between 0.02g and 0.03g (10% probability of exceedance in 50 years) based on the PGA map produced by the Council of Geoscience for South Africa.

A value of 0.03g was used in the stability assessments for the RSF.



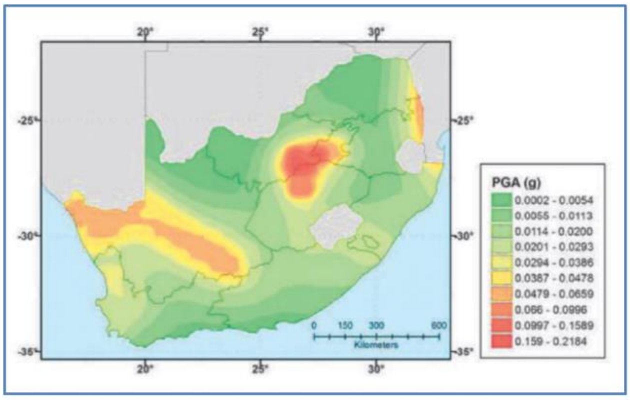


FIGURE 10-2: PEAK GROUND ACCELERATION (GHSAP (LEFT) AND COUNCIL OF GEOSCIENCE (RIGHT).

10.2. INPUT PARAMETERS TO THE SLOPE STABILITY MODELS

The slope stability model was defined in terms of the physical configuration of the structure and its foundations as well as the geotechnical properties of the residue and tailings material, and the foundation material. Two types of slope stability analyses are conducted:

- Static analyses which determine the FoS without the addition of PGA (i.e. an earthquake event); and
- Pseudo-static analysis which incorporates the PGA into the assessment to determine FoS during a seismic event.

10.2.1. CONFIGURATION OF THE STABILITY MODELS

The configuration of the slope stability model and its foundations is comprised of the following:

- The same geometry that was used in the associated seepage analysis;
- The phreatic surface determined by the associated seepage analysis;
- In-situ soils modelled with engineering properties obtained from laboratory testing;
- Pseudo-static analysis performed with a PGA of 0.03 g;

It is envisaged that the RD will be constructed in 2 phases as is illustrated in Figure 10-3 and Figure 10-4.

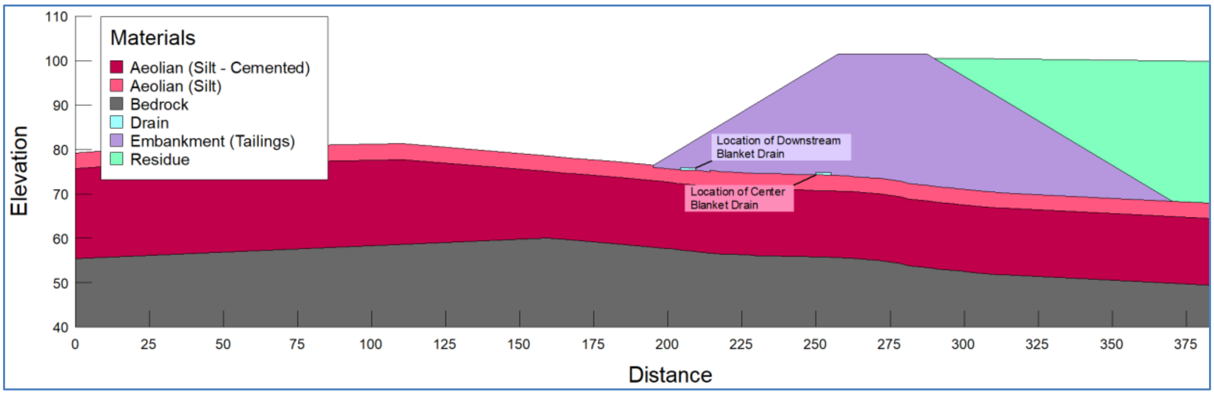


FIGURE 10-3: OPERATIONAL PHASE AT CAPACITY

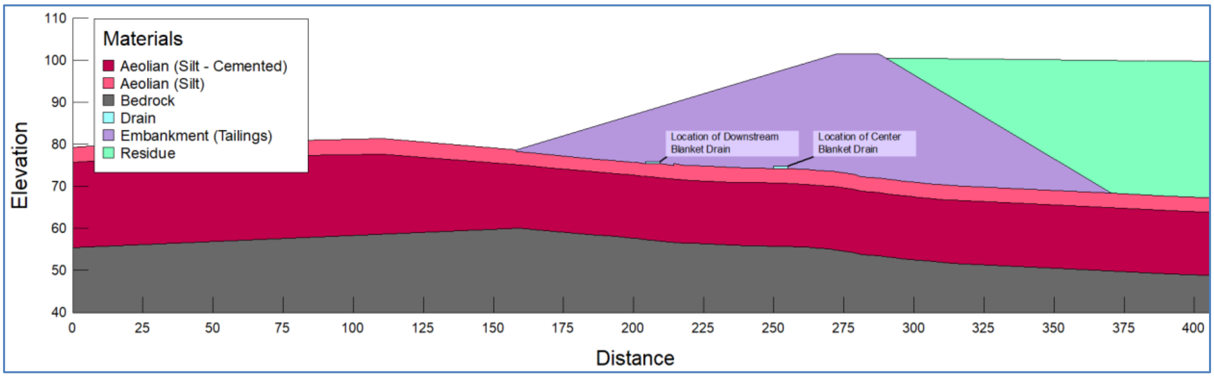


FIGURE 10-4: CLOSURE PHASE AT CAPACITY

The geometry used to analyse the operational and closure phase of the RD cross-section along the Critical section is listed in Table 10-1.

TABLE 10-1: SUMMARY OF RD GEOMETRY FOR STABILITY ASSESSMENT

Feature	Operational Phase	Closure Phase
Crest Elevation (m.a.m.s.l.)	101.5	101.5
Minimum Toe Elevation (m.a.m.s.l.)	74.26	74.41
Maximum Wall Height (m)	27.24	27.09
Crest Width (m)	30	15
Upstream Slope	1V:2.5H	1V:2.5H
Downstream Slope	1V:2.5H	1V:5H

10.2.2. MATERIAL PROPERTIES

The input geotechnical parameters used in the slope stability analysis of the RD is summarised in Table 10-2. It was assumed that RAS or EOFS tailings would be used to construct the containment wall of the facility. It was also assumed that the layer of dune sand that covers the area will be removed and sent to the mines processing plant. The remaining predominant soil profile consists of silty Aeolian sand that becomes weakly cemented with depth. It was assumed that a 3.5 m deep layer of Aeolian material overlays a 15 m deep layer of weakly cemented material before encountering bedrock in the form of very soft rock dorbank.

TABLE 10-2: GEOTECHNICAL PARAMETERS ASSOCIATED WITH THE RELEVANT MATERIALS FOR SLOPE STABILITY ANALYSIS

Region	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Residue	15.0	33	0
Embankment (Tailings)	16.0	30	2
Aeolian (Silt)	16.0	32	0
Aeolian (weakly cemented)	19.0	40	0

11. RD STABILITY RESULTS

The results of the slope stability assessment have been separated into three sections (Section 11.1, 0 and 11.3). The first section considers results from the analysis of the upstream face of the embankment with the residue encroaching on the toe of the upstream wall. The second section investigates the stability of the downstream face of the operation phase of the facility once the maximum deposition capacity of the RD has been reached. Finally, section 11.3 discusses the FoS against a failure of the downstream face of the closure phase. All critical slip surfaces generated for static conditions are provided in Appendix I and for pseudo-static conditions in Appendix II.

11.1. SLOPE STABILITY ANALYSIS RESULTS (OPERATIONAL PHASE – INITIAL DEPOSITION)

The model discussed in this section features an upstream face with a 1V:2.5H slopes, with variation in pool size and the location of the blanket drain, if included. The results obtained from the slope stability assessment of the upstream face of the facility during initial residue deposition are summarised in Table 11-1 with S indicating static loading conditions and PS indicating pseudo-static loading conditions.

From Table 11-1 it can be seen that a minimum FoS of 1.561 was obtained for static load conditions while FoSs for pseudo-static conditions were equal to or greater than 1.427 with the lowest FoS noted for the analyses containing a pool 100 m from the upstream face of the embankment.

TABLE 11-1: OPERATIONAL PHASE AT INITIAL RESIDUE (SLOPE STABILITY ASSESSMENT RESULTS)

Load condition	Active Drains			Pool Position		Deterministic	Probabilistic		
	Centre Blanket Drain	Downstream Blanket Drain	No Drains	Centre of RSF	100 m from embankment	FoS	FoS	PoF	RI
S	X			X		1.588	1.588	0	7.7004
S	X				X	1.566	1.561	0	8.1578
S		X		X		1.588	1.588	0	7.7004
S		X			X	1.566	1.561	0	8.1578
S			X	X		1.588	1.588	0	7.7004
S			X		X	1.566	1.561	0	8.1578
PS	X			X		1.462	1.462	0	5.2435
PS	X				X	1.427	1.427	0	5.7552
PS		X		X		1.462	1.462	0	5.2435
PS		X			X	1.427	1.427	0	5.7551
PS			X	X		1.462	1.462	0	5.2435
PS			X		X	1.427	1.427	0	5.7551

Figure 11-1 illustrates a typical critical slipe surface resulting from a seismic analysis on the upstream face of the embankment. Although a substantial slip surface has resulted from the analysis, it is noted that the greater majority of the embankment has remained untouched, implying that the wall will remain stable enough for the repair of the upstream face to take place. It should also be noted that the upstream face is a short to medium term slope as it will be covered with residue as residue deposition progresses. Therefore, it is argued that a minimum FoS of 1.427 is adequate for the upstream slope of the facility. Results of the stability analysis showcasing the stability of the upstream slope at the point where the elevation of residue and supernatant pond is such that the phreatic surface within the embankment is just below the centre blanket drain can be found in Appendix I and Appendix II. It was found that FoS improve as deposition takes place, thus the results of the analysis were excluded from the main report.

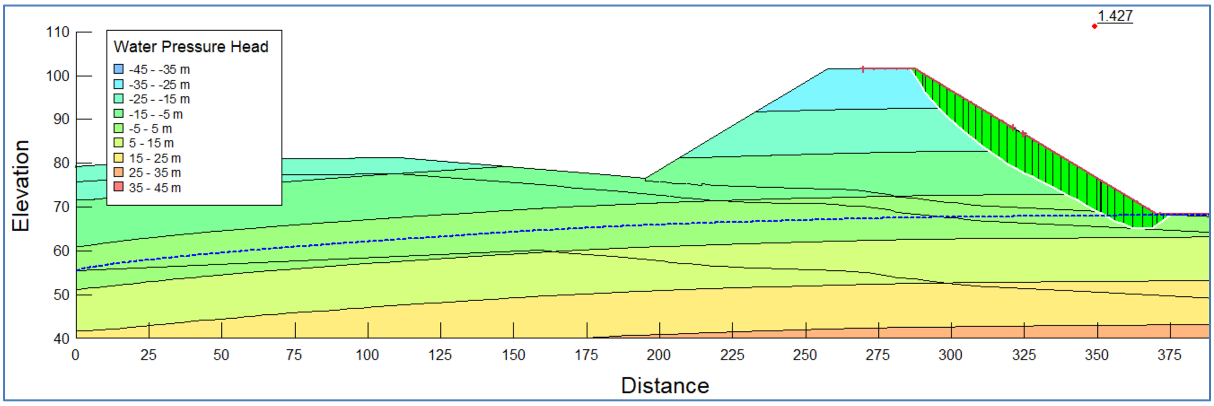


FIGURE 11-1: UPSTREAM FACE OF THE OPERATIONAL PHASE WITH INITIAL RESIDUE DEPOSITION (SEISMIC LOADING)

11.2. Slope Stability Analysis Results (Operational Phase – Maximum Residue Capacity)

It is shown that the FoSs are above the minimum required by SA regulations, for static loading with a minimum of 1.517. The Reliability Index for all models remain above the minimum required 4.83 and the Probability of failure does not exceed 0.07%.

The seismic analysis revealed the downstream face of the embankment for the operational phase to have FoSs exceeding or achieving the minimum required value of 1.5 within an acceptable margin for models analysed with a blanket drain. The analysis of models where drains were excluded indicated that the FoSs decreases to 1.386 (Figure 11-2) if the phreatic surface is allowed to build up and saturate the downstream toe of the facility.

Table 11-2: Operational Phase at maximum residue capacity (Slope Stability Assessment Results)

Load condition	Active Drains			Pool Position		Deterministic	Probabilistic		
	Centre Blanket Drain	Downstream Blanket Drain	No Drains	Centre of RSF	100 m from embankment	FoS	FoS	PoF	RI
S	X			X		1.648	1.648	0	8.1097
S	X				X	1.648	1.648	0	8.1097
S		X		X		1.648	1.648	0	8.1097
S		X			X	1.648	1.648	0	8.1097
S			X	X		1.567	1.567	0	9.5781
S			X		X	1.517	1.517	0	9.0344
PS	X			X		1.518	1.518	0	5.8095
PS	X				X	1.518	1.518	0	5.7951
PS		X		X		1.518	1.518	0	5.7951
PS		X			X	1.518	1.518	0	5.7951
PS			X	X		1.440	1.440	0	6.4015
PS			X		X	1.386	1.386	0	5.5390

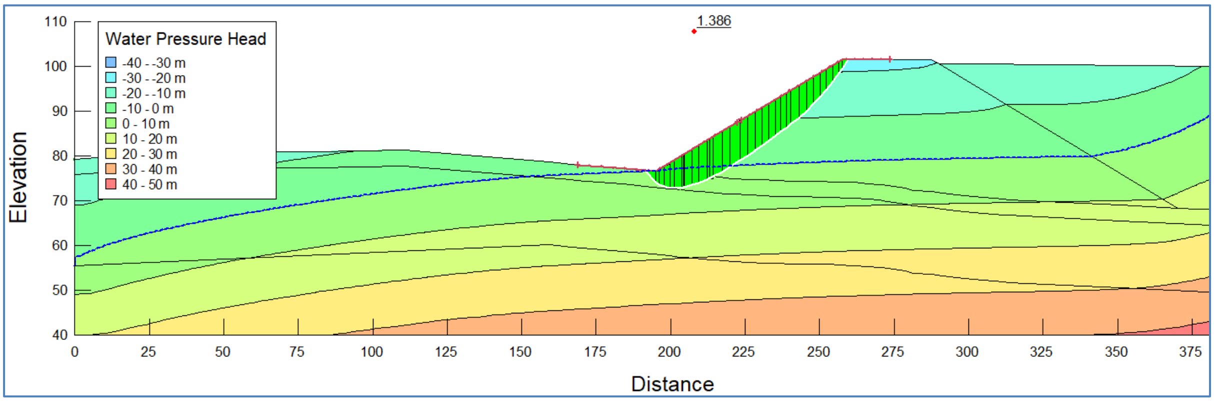


Figure 11-2: Downstream Face of the Operational Phase with residue at Maximum Capacity (seismic loading)

11.3. SLOPE STABILITY ANALYSIS RESULTS (CLOSURE PHASE – MAXIMUM RESIDUE CAPACITY)

The results of the analysis on the downstream face of the closure phase of the RD is shown to far exceed the minimum requirements both in terms of the FoS and RI. A minimum FoS of 2.435 was noted for static loading and 2.097 for pseudo-static loading (Table 11-3 & Figure 11-3). The minimum value for the RI is shown to be 10.792, significantly higher than the required value of 4.83.

TABLE 11-3: CLOSURE PHASE AT MAXIMUM RESIDUE CAPACITY (SLOPE STABILITY ASSESSMENT RESULTS)

Load condition	Active Drains			Operating Pool		Deterministic	Pool Position		
	Centre Blanket Drain	Downstream Blanket Drain	No Drains	Centre of RSF	100 m from embankment	FoS	FoS	PoF	RI
S	X			X		3.094	3.094	0	13.617
S	X				X	3.094	3.094	0	13.719
S		X		X		3.094	3.094	0	13.617
S		X			X	3.094	3.094	0	13.719
S			X	X		2.658	2.658	0	17.136
S			X		X	2.435	2.435	0	16.240
PS	X			X		2.677	2.677	0	10.816
PS	X				X	2.677	2.677	0	10.792
PS		X		X		2.677	2.677	0	10.816
PS		X			X	2.677	2.677	0	10.792
PS			X	X		2.307	2.307	0	12.175
PS			X		X	2.097	2.097	0	10.987

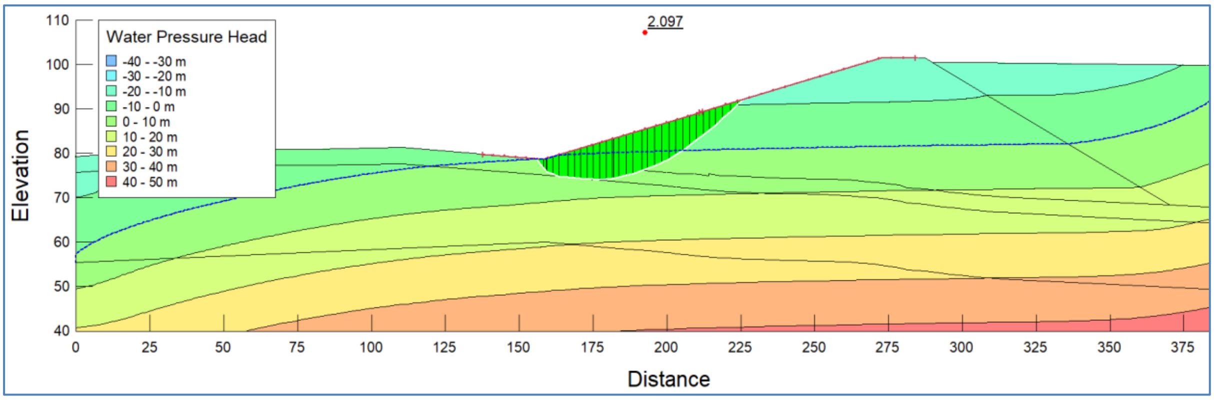


FIGURE 11-3: DOWNSTREAM FACE OF THE CLOSURE PHASE WITH RESIDUE AT MAXIMUM CAPACITY (SEISMIC LOADING)

11.4. DISCUSSION OF RESULTS

The results of the slope stability assessment indicate that the facility is stable under static loads for the short, medium and long-term slopes under all scenarios considered. A blanket drain is required to achieve FoS above the minimum required value of 1.5 for the downstream slope of the operational phase in the event of pseudo-static conditions. Additionally, it is advised to include the drain as a means to prevent water seeping through the downstream toe of the embankment.

Similarly, to the downstream face, the upstream face of the embankment yielded FoS greater than 1.5 for static conditions. However, all pseudo-static loading conditions resulted in FoS below 1.5 with a minimum of 1.427. It is argued that the upstream slope will be buttressed with residue as residue deposition takes place, and the resultant slip surface does not compromise the majority of the wall. As such FoS greater than 1.4 are considered acceptable for the upstream short term slope under pseudo-static conditions.

It is recommended that the stability models be reassessed in the detailed design phase of the project should the seepage models change once the bedrock permeability has been confirmed.

12. CONCLUSIONS

The following conclusions were drawn from the seepage and slope stability analysis of the facility:

- The geometry of the RD adheres to the minimum acceptable FoS for both interim slopes and long-term slopes;
- Functional drains are effective in reducing the phreatic surface through the RD and preventing saturation of the downstream toe which could lead to piping and subsequent instability of the downstream slope;
- The drains functions as an effective means by which to intercept the movement of groundwater generated by the supernatant pool for the given topography and soil profiles assumed in this analysis.
- The FoS of the downstream slope against slope failure are above the 1.5 required for static and pseudo-static conditions provided active drains are included in the design;
- The FoSs of the analyses conducted on the upstream slope are satisfactory (i.e. greater than 1.5) for static loading conditions. Values lower than 1.5 were noted (with a minimum of 1.427) during the pseudo-static analysis of the upstream face of the facility, although, it should be taken into consideration that upstream face is a temporary slope that will be buttressed with residue as deposition progresses;
- The probabilities of failure for all models are below 0.007; and
- Should a slope failure occur, it is believed that the robust design will prevent a wall breach from occurring while allowing adequate time for repairs to be undertaken.

13. RECOMMENDATIONS

In consideration of the analyses and contents of this report, it is recommended that:

- The designed side slopes of the RD should be adhered to ensure the modelled factors of safeties are achieved;
- A competent and reputable construction team must undertake the construction of the RSF; and

- The drains provided for the RD were shown to be critical in preventing saturation of the downstream toe, therefore it will be necessary to ensure that these are constructed according to design specifications.
- The permeability and degree of fissuring in the bedrock be confirmed and the seepage and stability models be re-assessed to the confirm the need for the blanket drain;

14. REFERENCES

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Tuluka, G. M. (2010). *Seismic Hazard Assessment and Volcanogenic Seismicity for the Democratic Republic of Congo and Surrounding Areas, Western Rift Valley of Africa.* Johannesburg: University of Witwatersrand.

Report Author	Reviewer	Project Manager
Stephan Barkhuizen	**Georgia Vagis**	**Andrew Savvas**

epoch resources (pty) ltd

APPENDIX I STATIC RD SEEPAGE AND SLOPE STABILITY RESULTS

INFORMATION

In an attempt to avoid confusion, tables have been created by which to identify the information presented below, for static loading conditions. Results are separated based on the amount of residue deposited within the basin and RD phase they are associated with (i.e. Scenario A refers to the operational phase of the RD when residue deposition is in the initial stage).

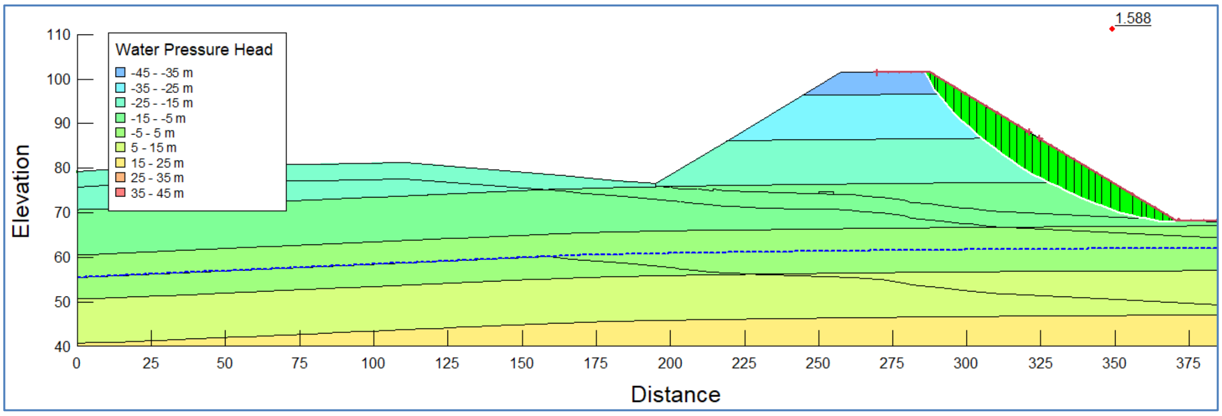
SUMMARY OF SCENARIOS ANALYSED

Scenario	Phase	Deposition Capacity reached	Slope Considered
A	Operational	Initial capacity	Upstream
B	Operational	Partial capacity	Upstream
C	Operational	Maximum capacity	Downstream
D	Closure	Maximum capacity	Downstream
E	Operational (Fissured Bedrock)	Maximum capacity	Downstream

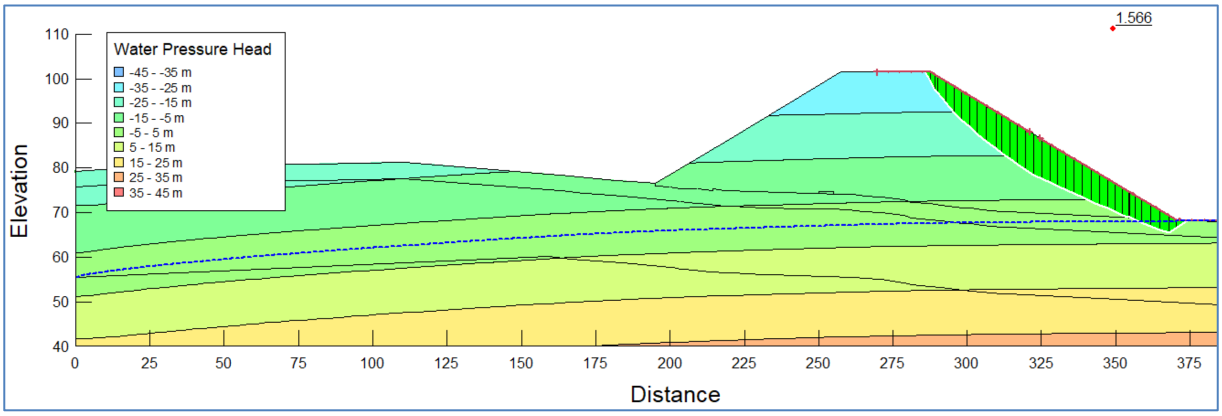
Each scenario is further subdivided into subsections based on the active drainage condition and operating pool level. A table has been included at the start of results for each scenario, as shown below.

OPERATIONAL PHASE – INITIAL

Scenario	Active Drains			Operating Pool	
	centre blanket drain	downstream blanket drain	No Drains	Min	Max
A 1	X			X	
A 2	X				X
A 3		X		X	
A 4		X			X
A 5			X	X	
A 6			X		X



SCENARIO A1



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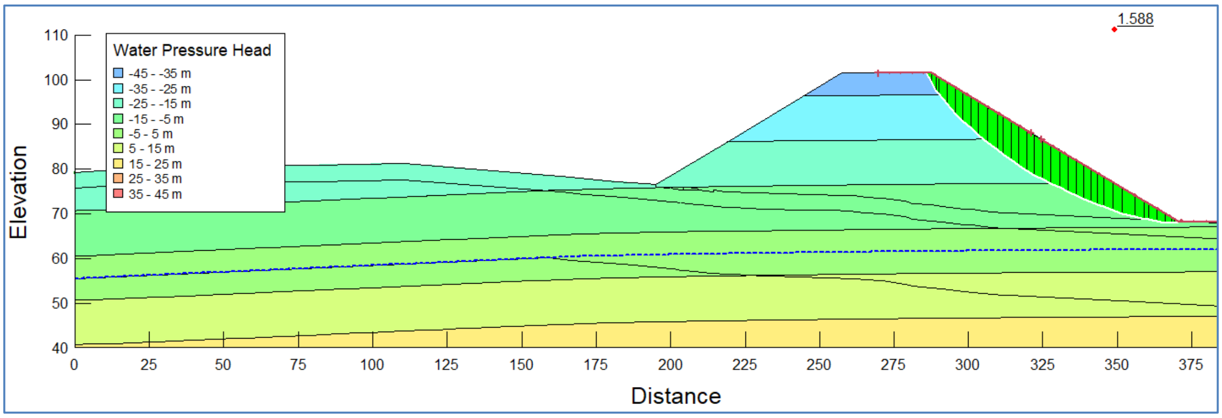

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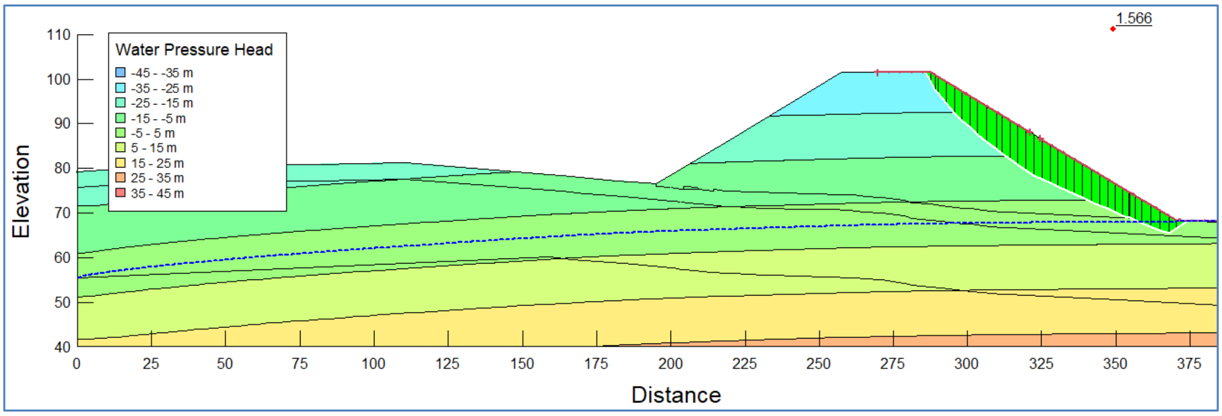

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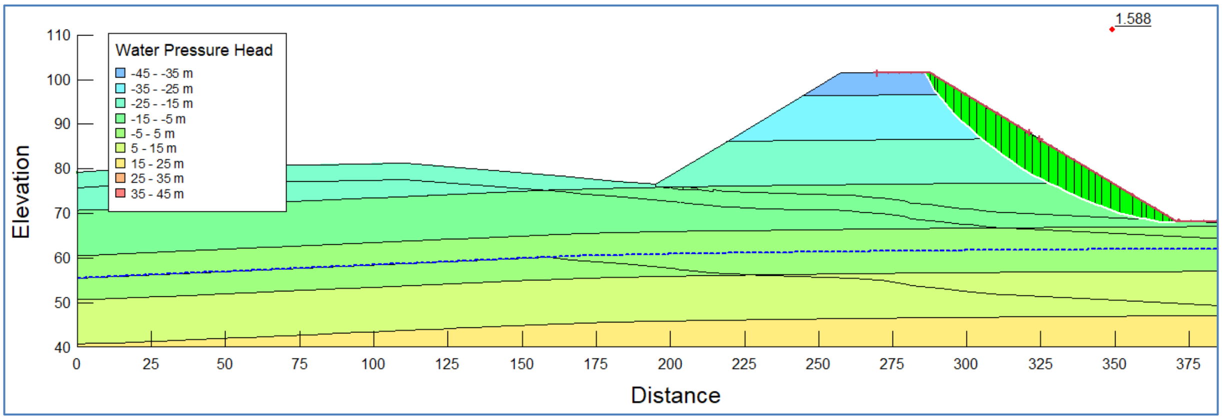

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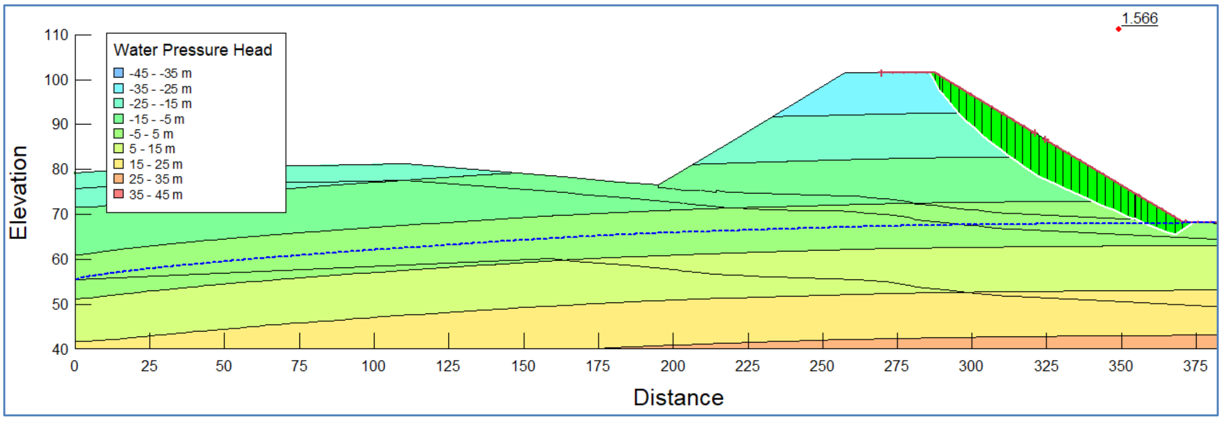
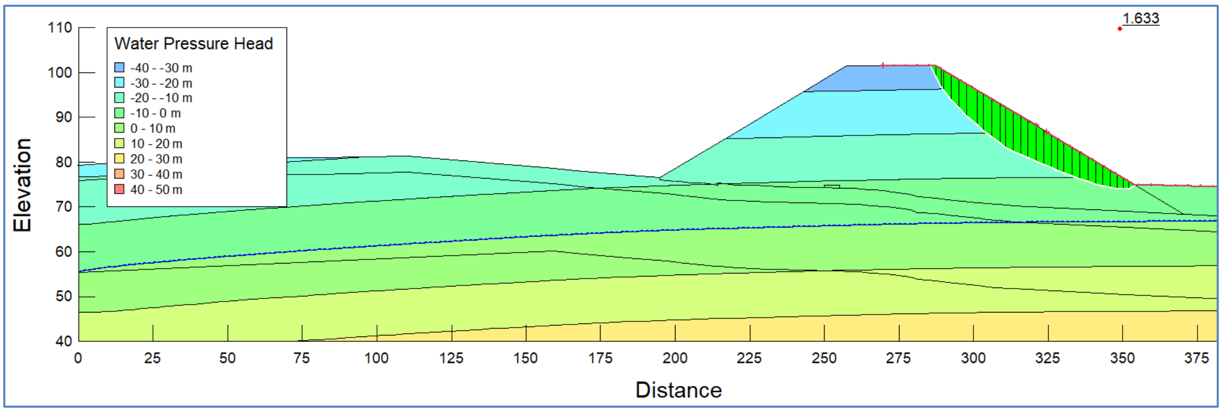


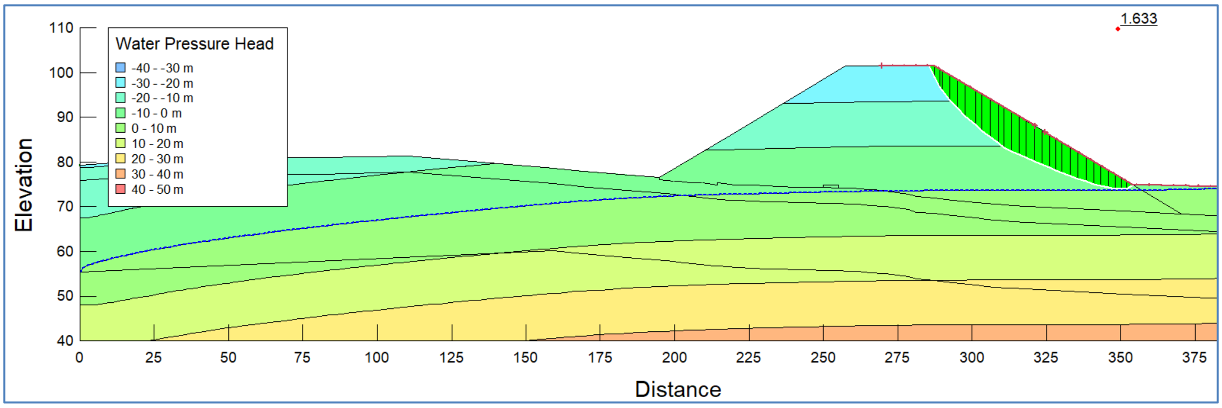
SCENARIO A6

OPERATIONAL PHASE – PARTIAL CAPACITY REACHED

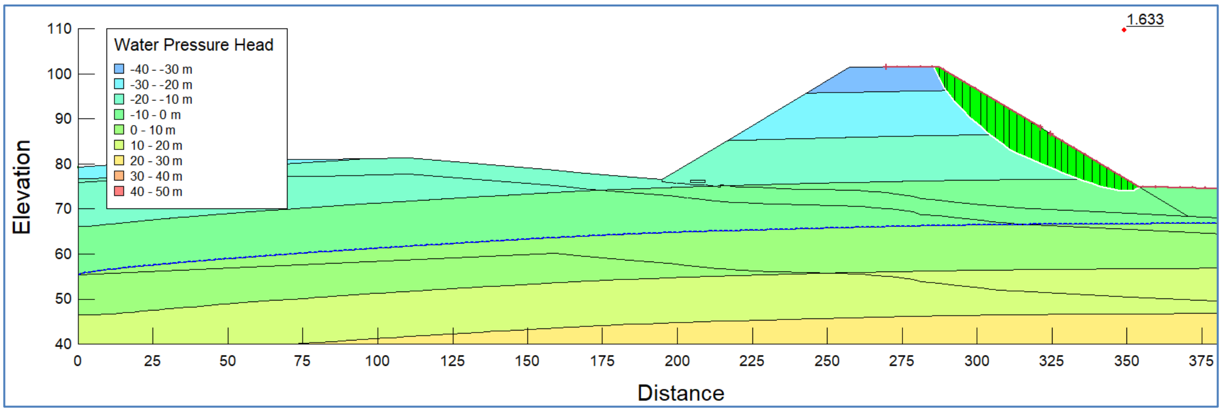
Scenario	Active Drains			Operating Pool	
	centre blanket drain	downstream blanket drain	No Drains	Min	Max
B 1	X			X	
B 2	X				X
B 3		X		X	
B 4		X			X
B 5			X	X	
B 6			X		X



SCENARIO B1



SCENARIO B2



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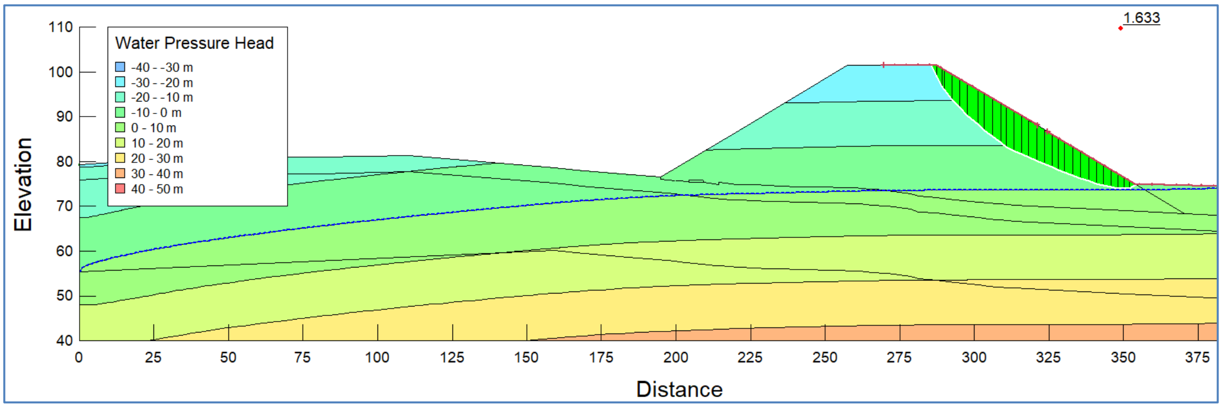

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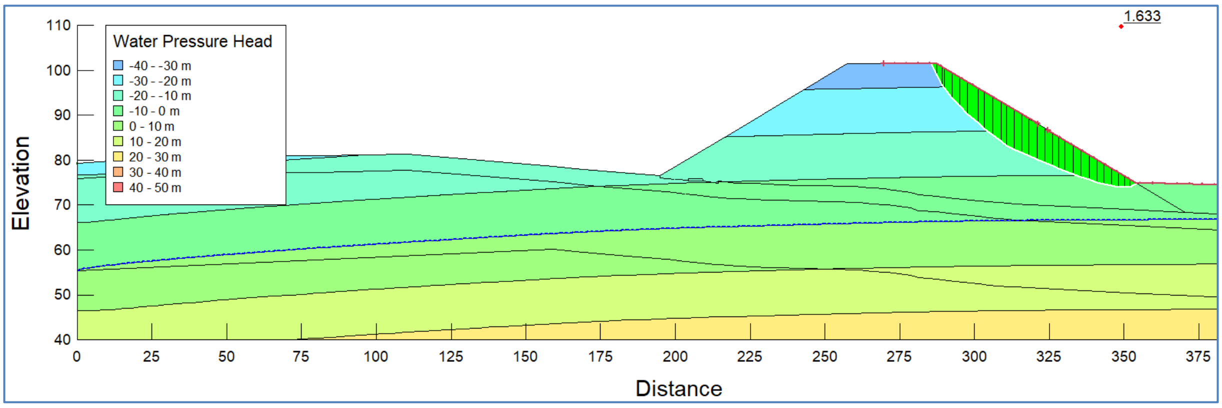

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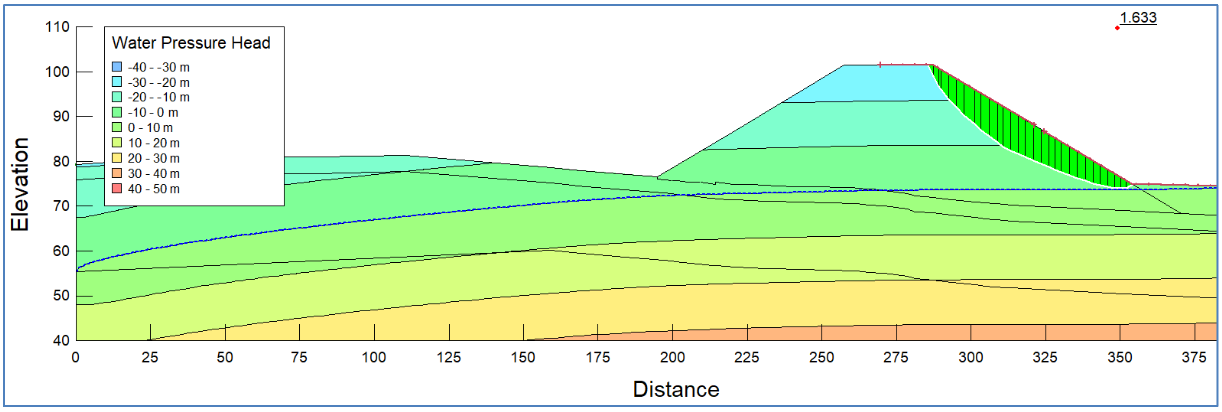
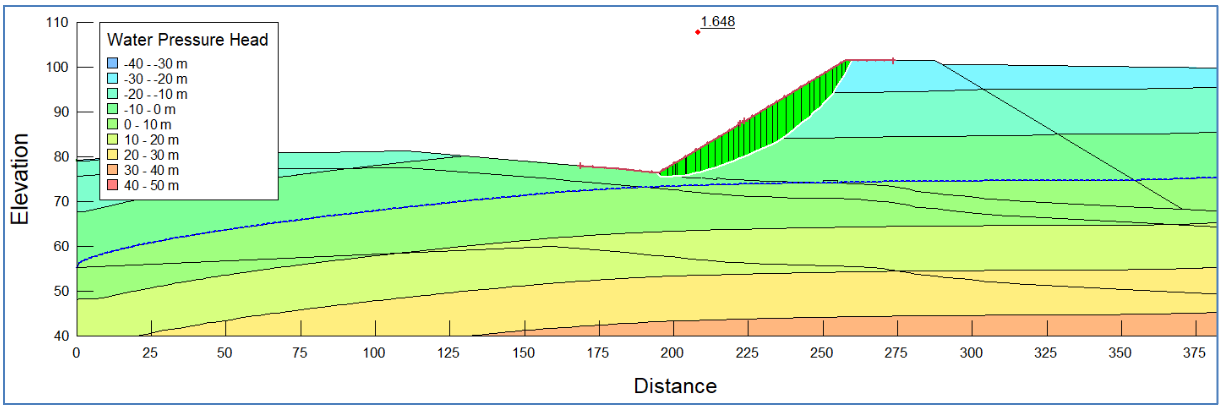


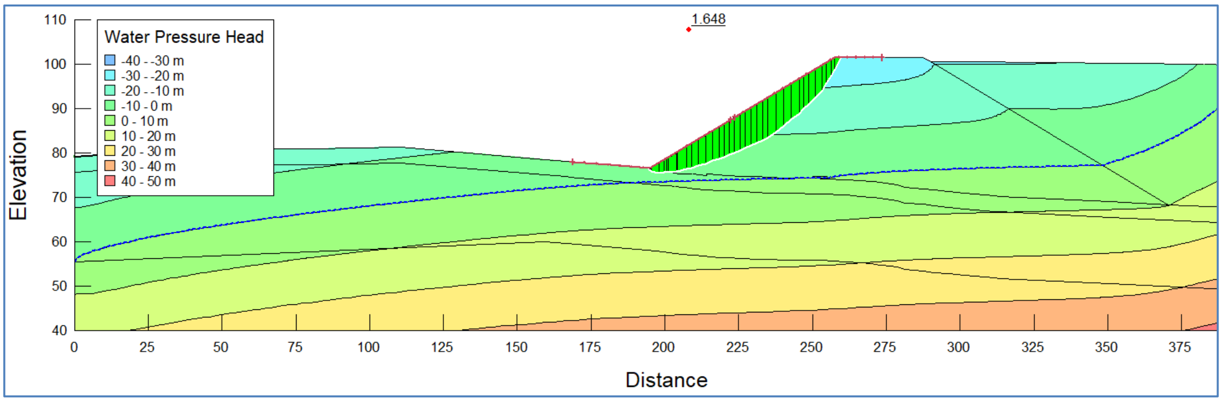
Scenario B6

Operational Phase – Maximum Capacity Reached

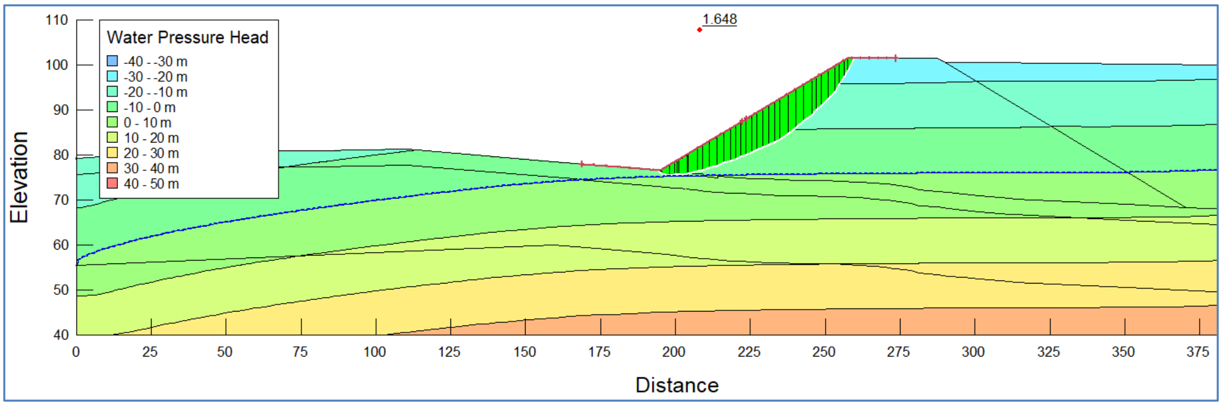
Scenario	Active Drains			Operating Pool	
	centre blanket drain	downstream blanket drain	No Drains	centre blanket drain	downstream blanket drain
C 1	X			X	
C 2	X				X
C 3		X		X	
C 4		X			X
C 5			X	X	
C 6			X		X



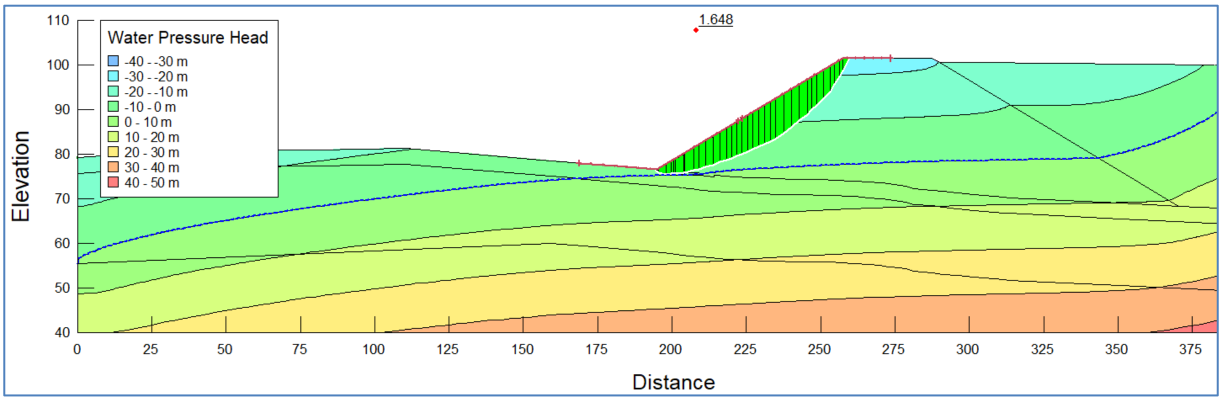
Scenario C1



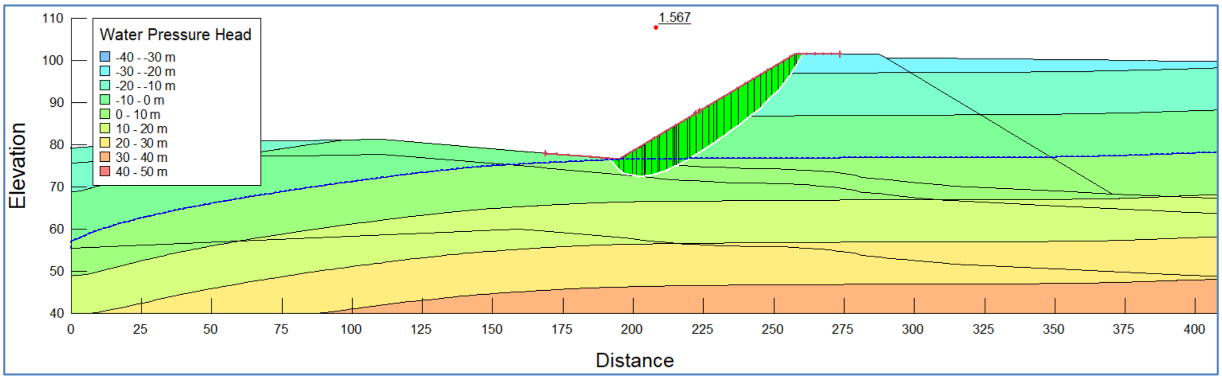
SCENARIO C2



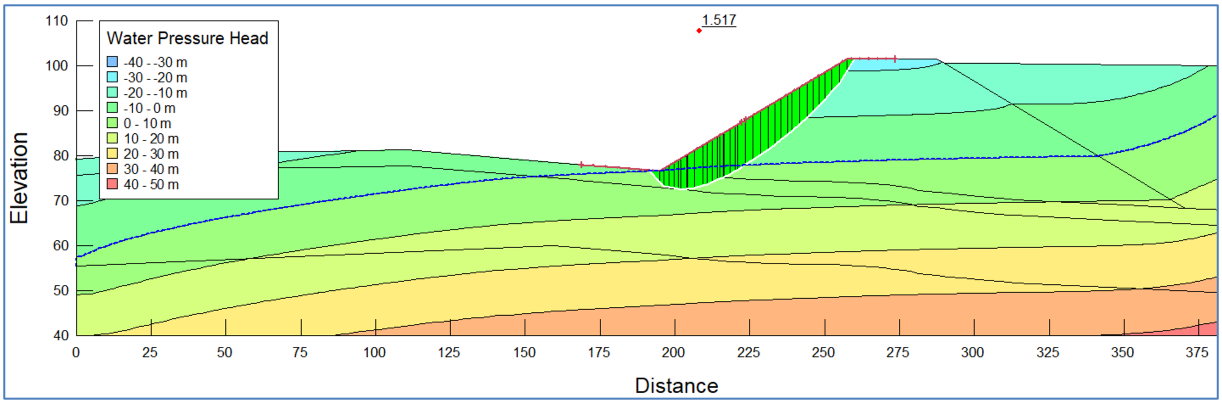
SCENARIO C3



SCENARIO C4



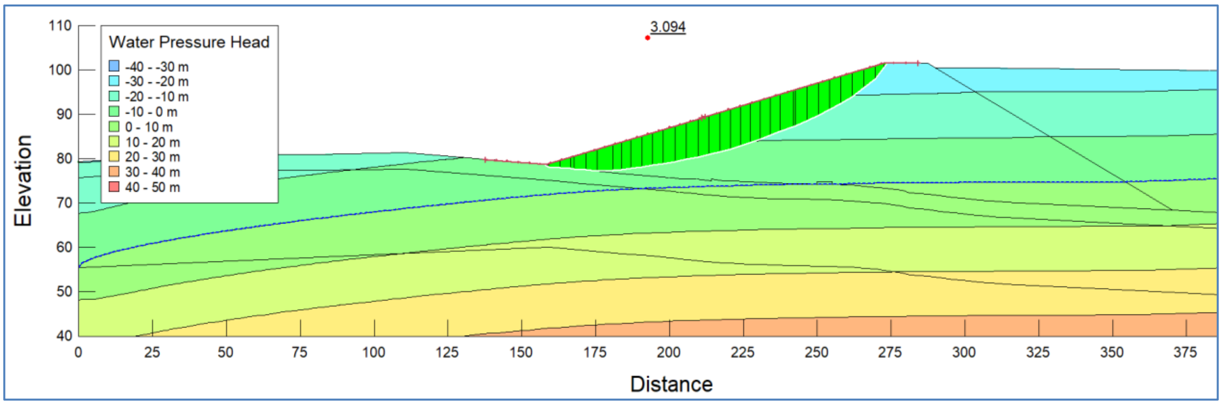
SCENARIO C5



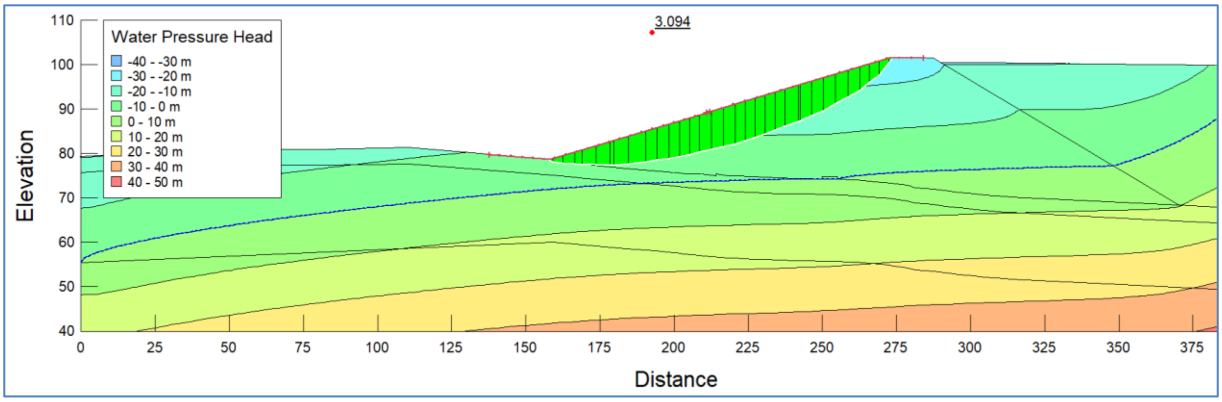
SCENARIO C6

CLOSURE PHASE – MAXIMUM CAPACITY REACHED

Scenario	Active Drains			Operating Pool	
	centre blanket drain	downstream blanket drain	No Drains	centre blanket drain	downstream blanket drain
D 1	X			X	
D 2	X				X
D 3		X		X	
D 4		X			X
D 5			X	X	
D 6			X		X



SCENARIO D1



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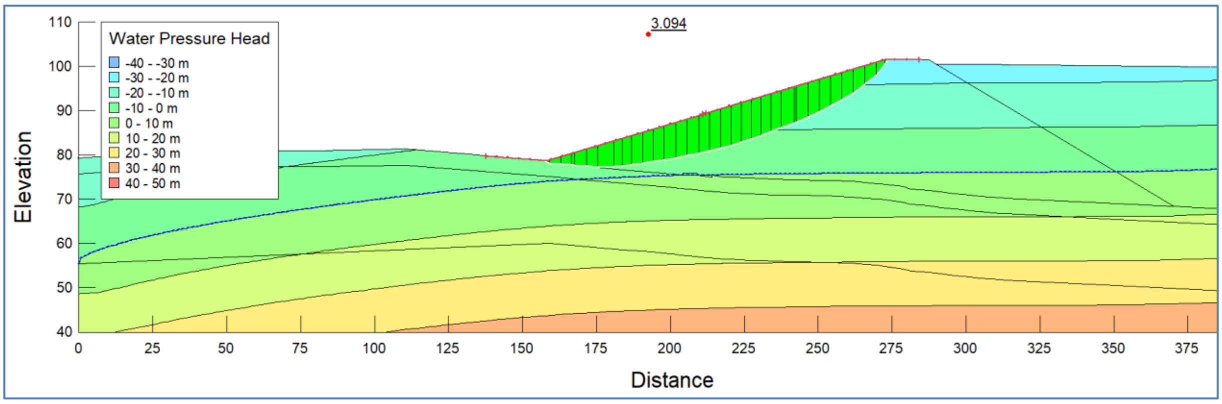
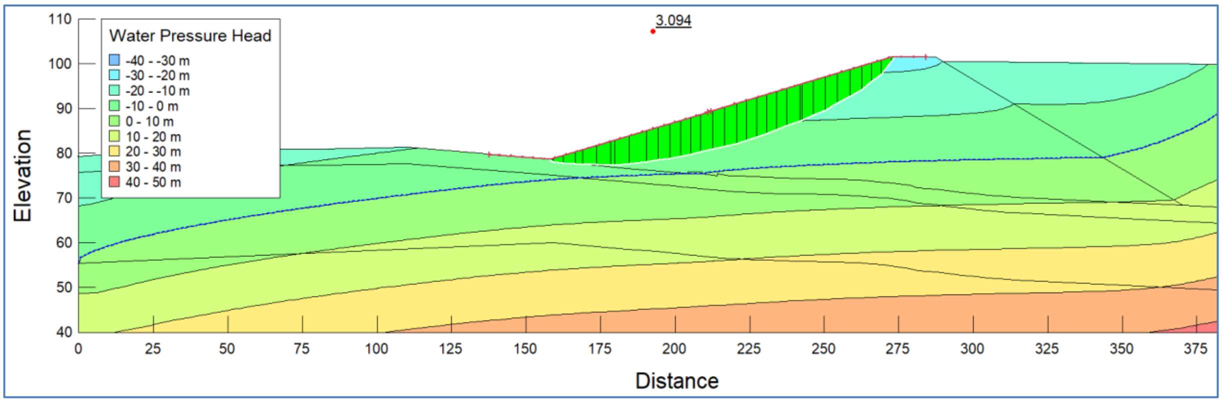


SCENARIO D3



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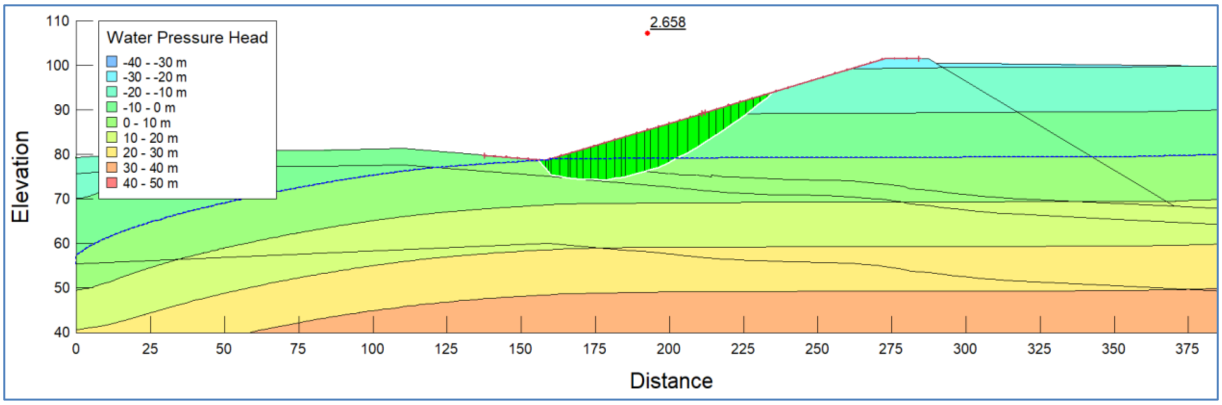
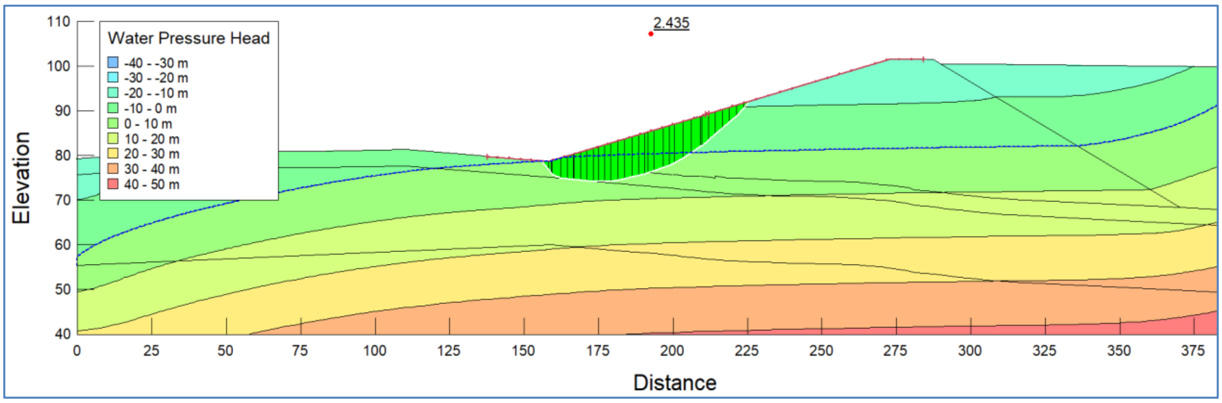


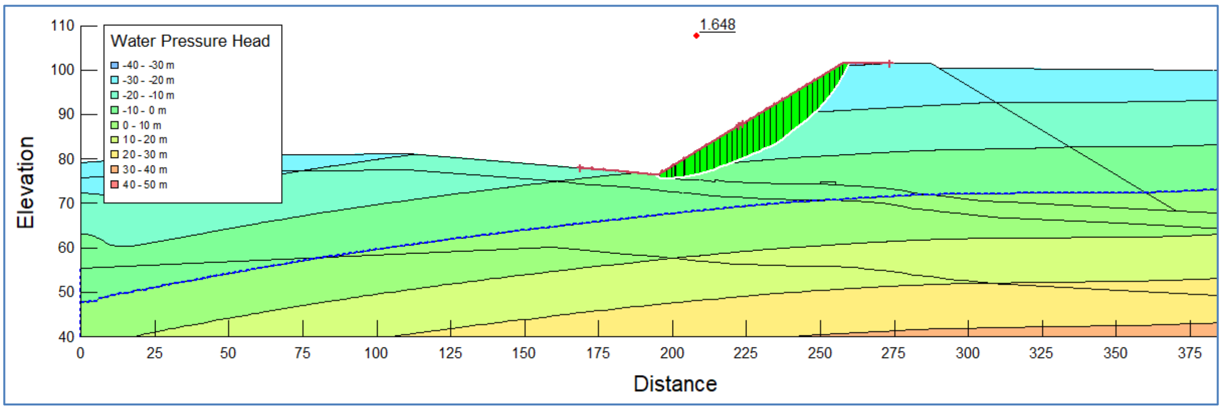
SCENARIO D5



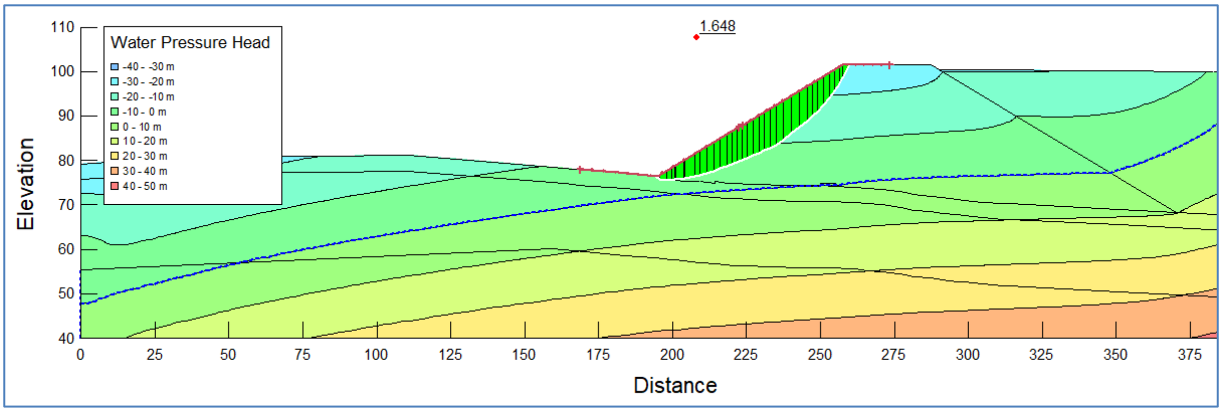
SCENARIO D6

OPERATIONAL PHASE – MAXIMUM CAPACITY REACHED (FISSURED BEDROCK)

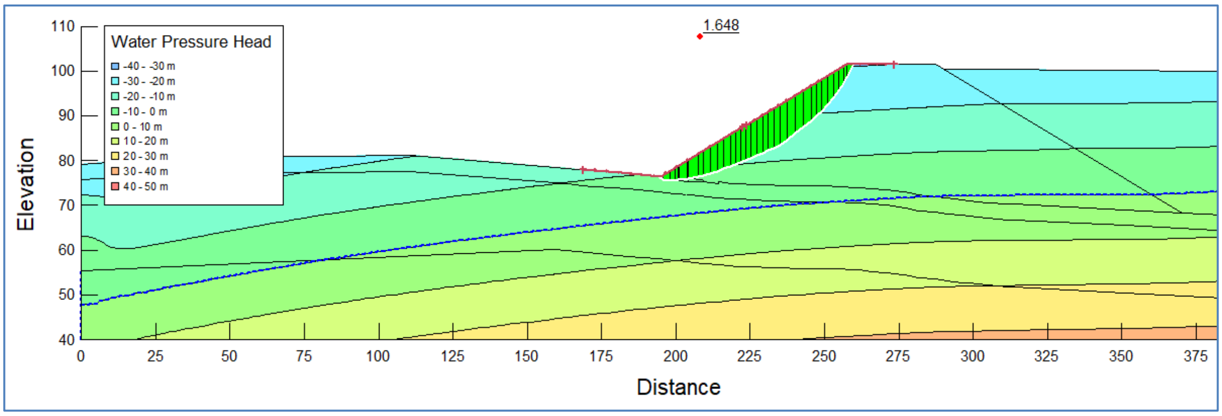
Scenario	Active Drains			Operating Pool	
	centre blanket drain	downstream blanket drain	No Drains	centre blanket drain	downstream blanket drain
E 1	X			X	
E 2	X				X
E 3		X		X	
E 4		X			X
E 5			X	X	
E 6			X		X



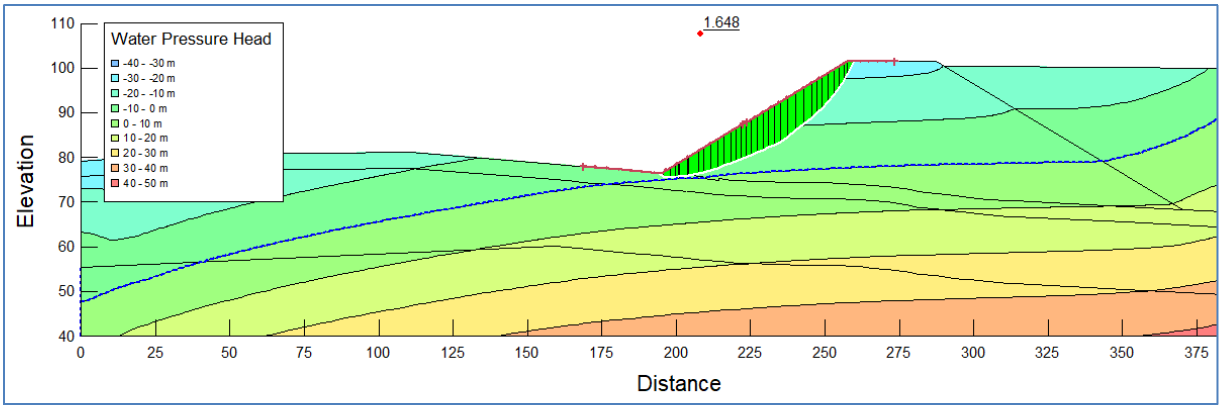
SCENARIO E1



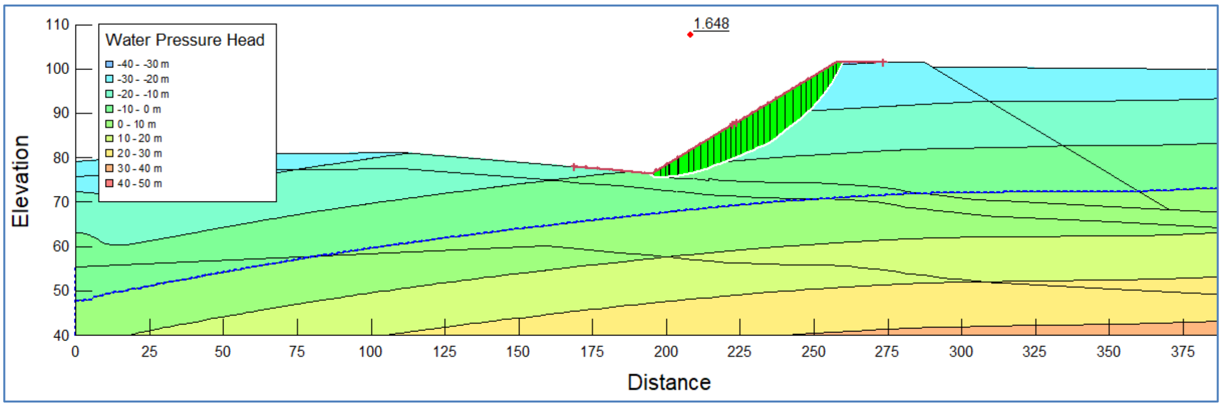
SCENARIO E2



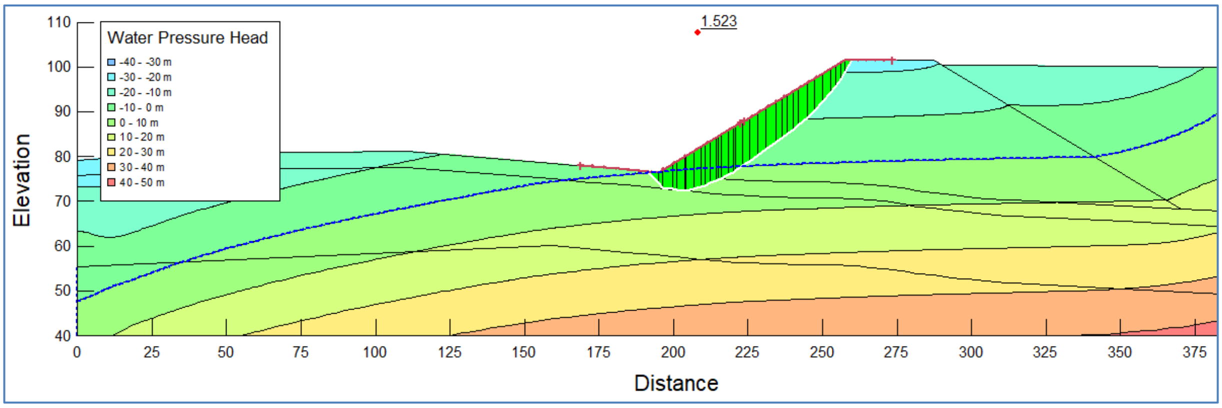
SCENARIO E3



SCENARIO E4



SCENARIO E5


1.523
Water Pressure Head
-40 - -30 m
-30 - -20 m
-20 - -10 m
-10 - 0 m
0 - 10 m
10 - 20 m
20 - 30 m
30 - 40 m
40 - 50 m
Elevation
110
100
90
80
70
60
50
40
0
25
50
75
100
125
150
175
200
225
250
275
300
325
350
375
Distance

Scenario E6

APPENDIX II PSEUDO-STATIC RD SEEPAGE AND SLOPE STABILITY RESULTS

INFORMATION

As in APPENDIX I, tables have been created by which to identify the information presented below for pseudo-static loading conditions. Results are separated based on the residue capacity and RD phase they are associated with (i.e. the operational phase with residue at maximum capacity is identified as Scenario C).

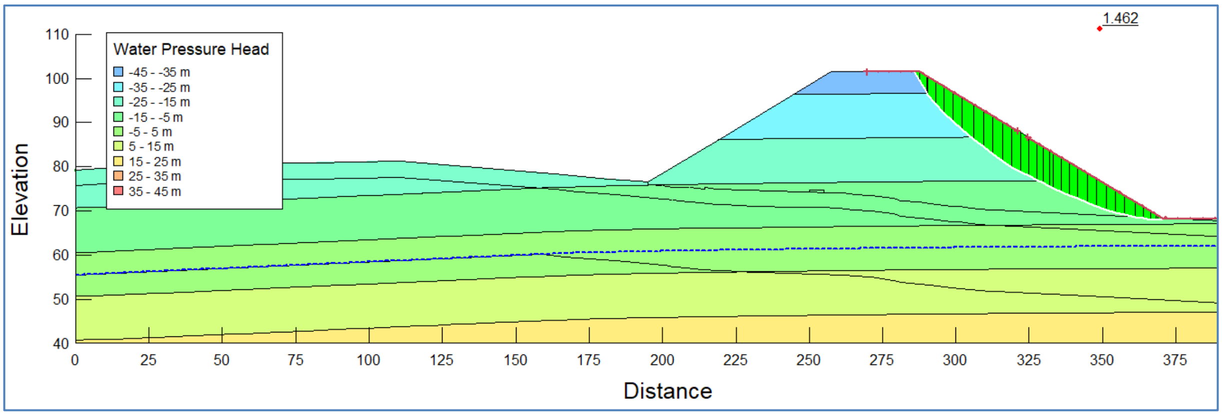
SUMMARY OF SCENARIOS ANALYSED

Scenario	Phase	Deposition Capacity reached	Slope Considered
A	Operational	Initial capacity	Upstream
B	Operational	Partial capacity	Upstream
C	Operational	Maximum capacity	Downstream
D	Closure	Maximum capacity	Downstream
E	Operational (Fissured Bedrock)	Maximum capacity	Downstream

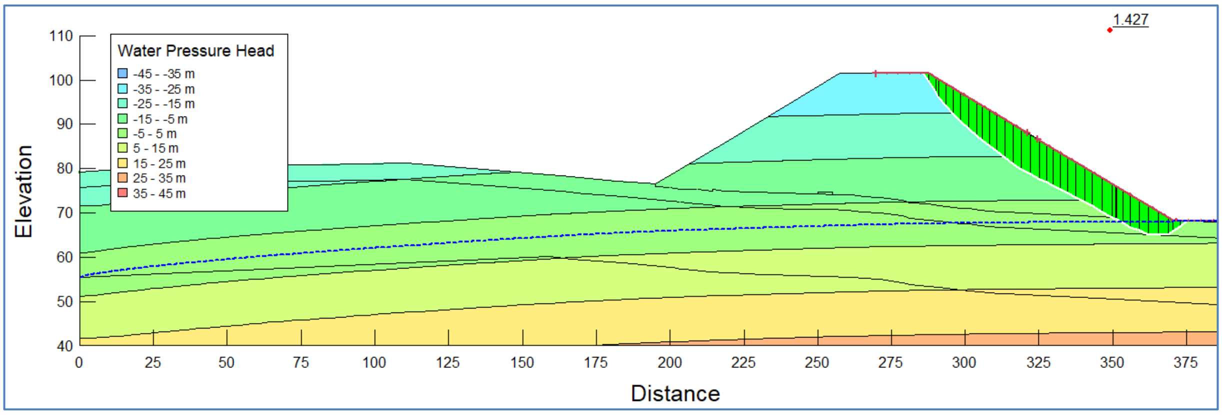
Each scenario is further subdivided into subsections based on the active drainage condition and operating pool level. A table has been included at the start of results for each scenario, as shown below.

OPERATIONAL PHASE – INITIAL

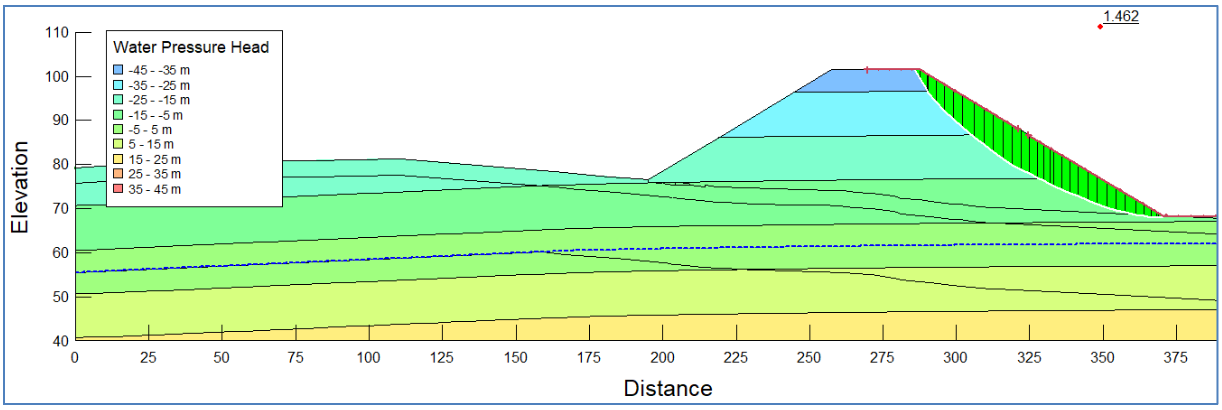
Scenario	Active Drains			Pool position	
	centre blanket drain	downstream blanket drain	No Drains	Centre of RSF	100 m from embankment
A 1	X			X	
A 2	X				X
A 3		X		X	
A 4		X			X
A 5			X	X	
A 6			X		X



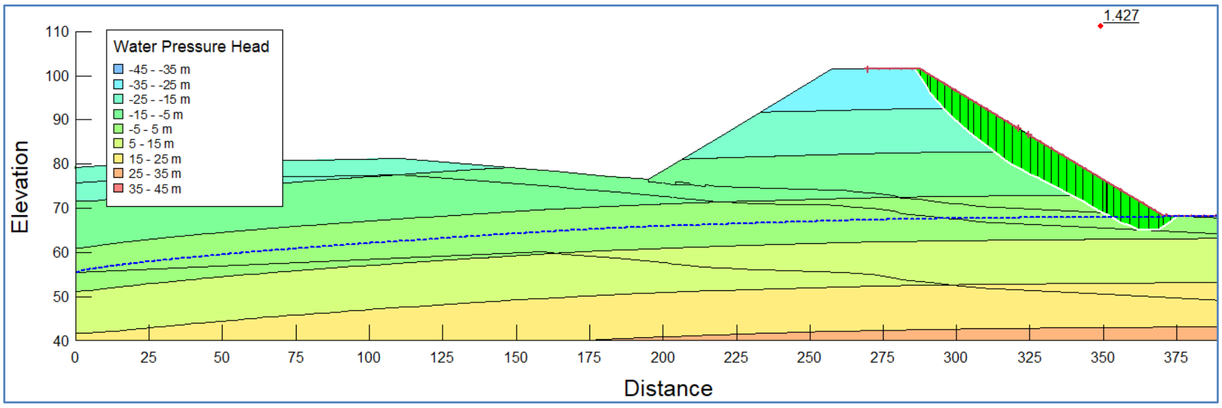
Scenario A1



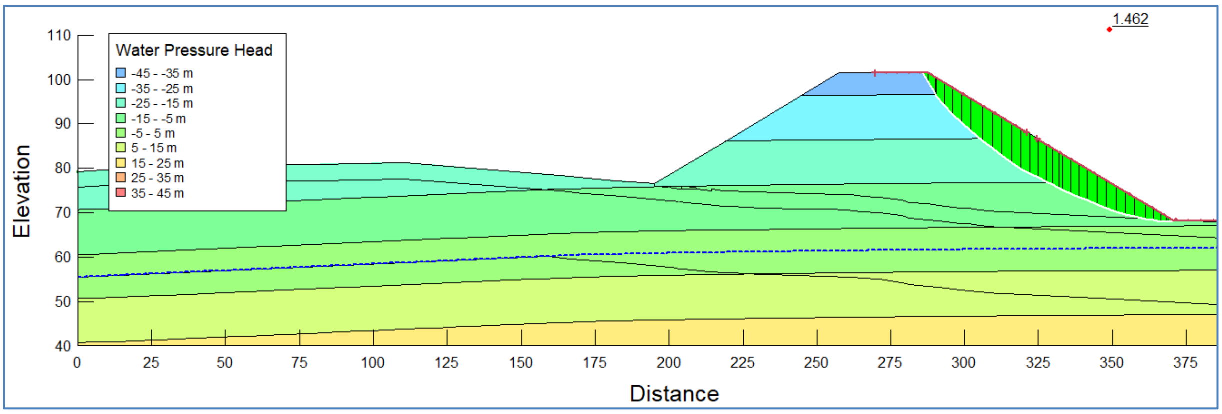
Scenario A2



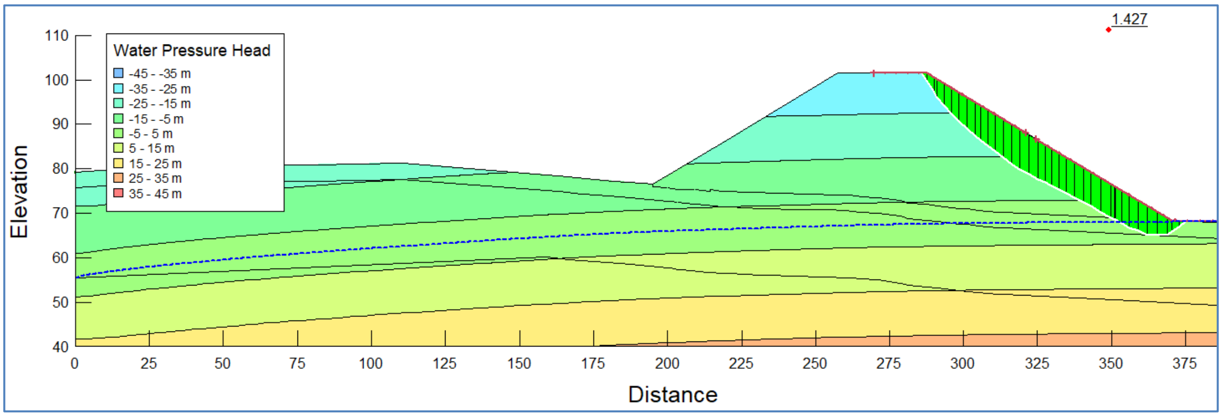
Scenario A3



SCENARIO A4



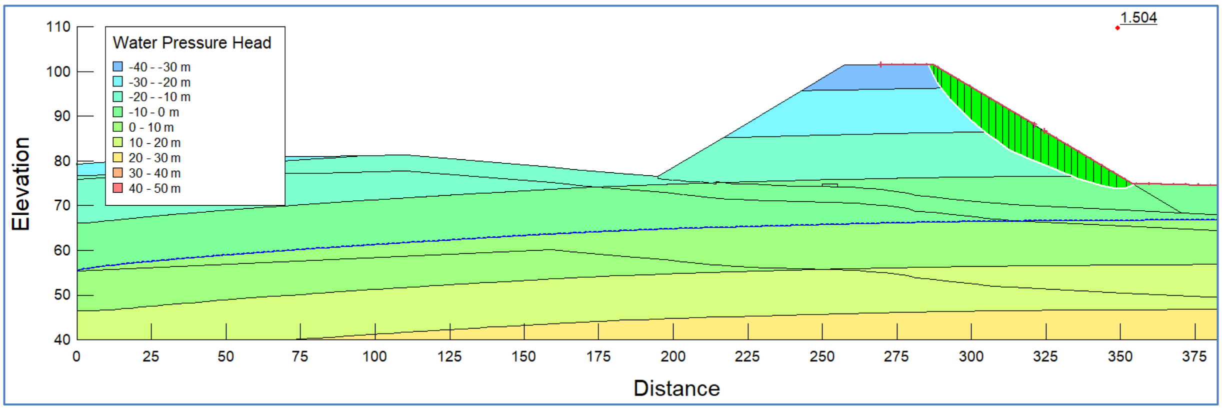
SCENARIO A5



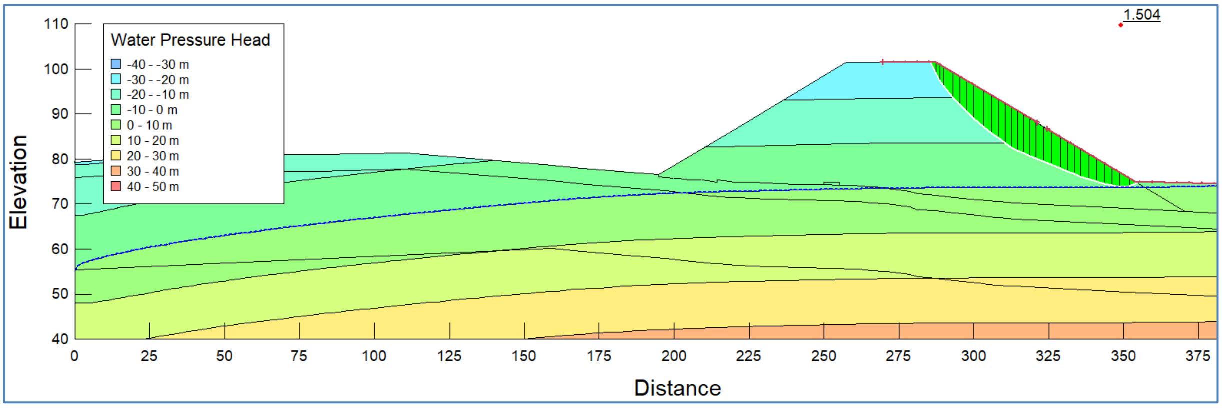
SCENARIO A6

OPERATIONAL PHASE – PARTIAL CAPACITY REACHED

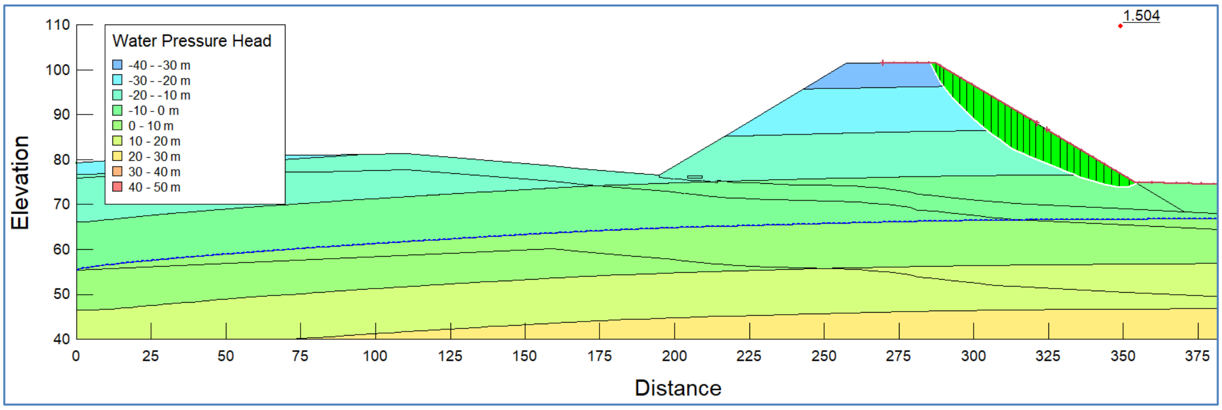
Scenario	Active Drains			Pool position	
	centre blanket drain	Centre of RSF	Centre of RSF	Centre of RSF	100m from embankment
B 1	X			X	
B 2	X				X
B 3		X		X	
B 4		X			X
B 5			X	X	
B 6			X		X



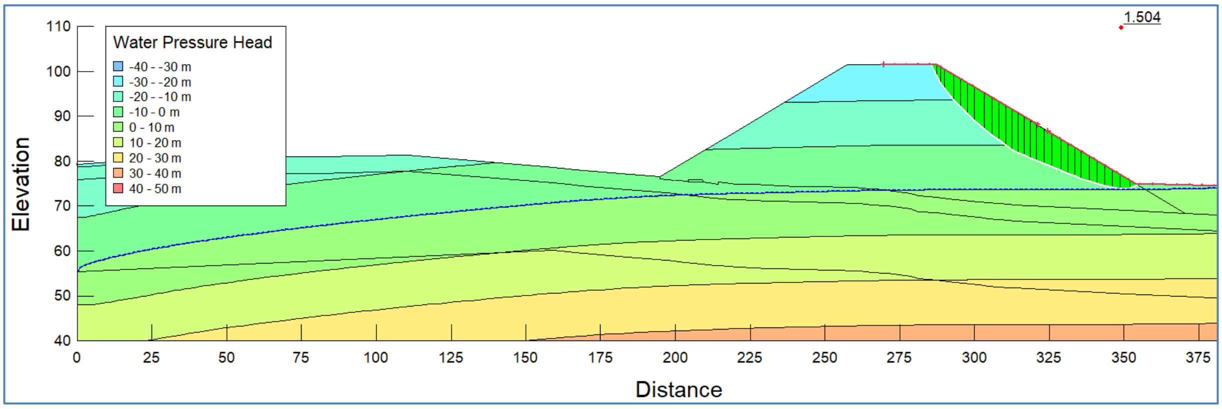
SCENARIO B1



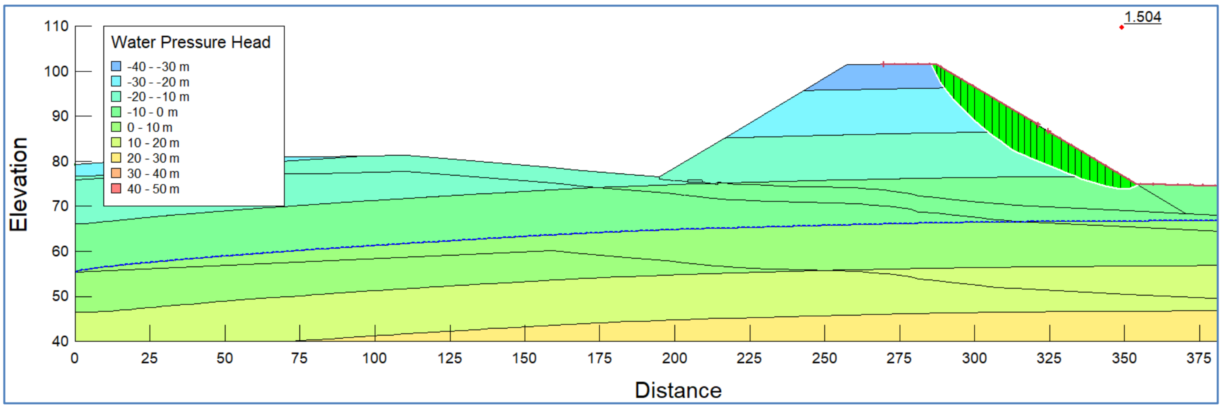
SCENARIO B2



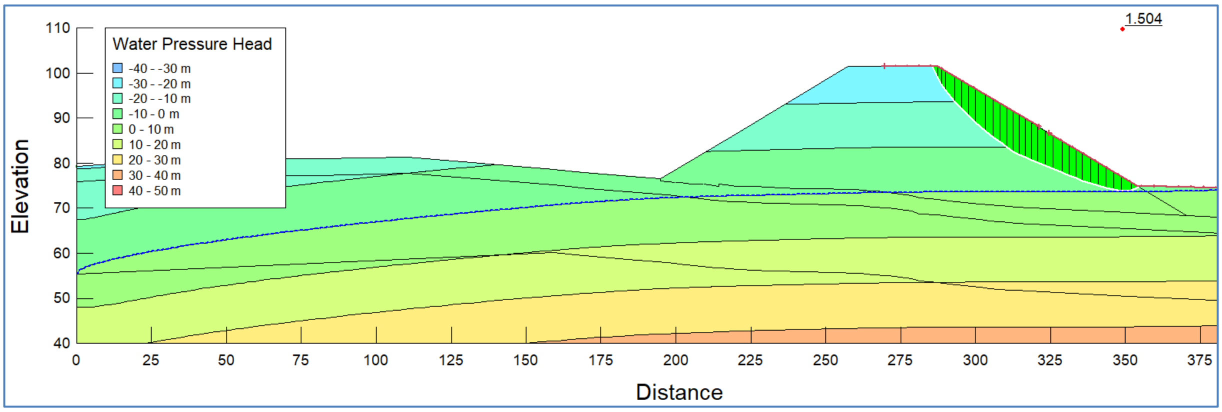
SCENARIO B3



SCENARIO B4



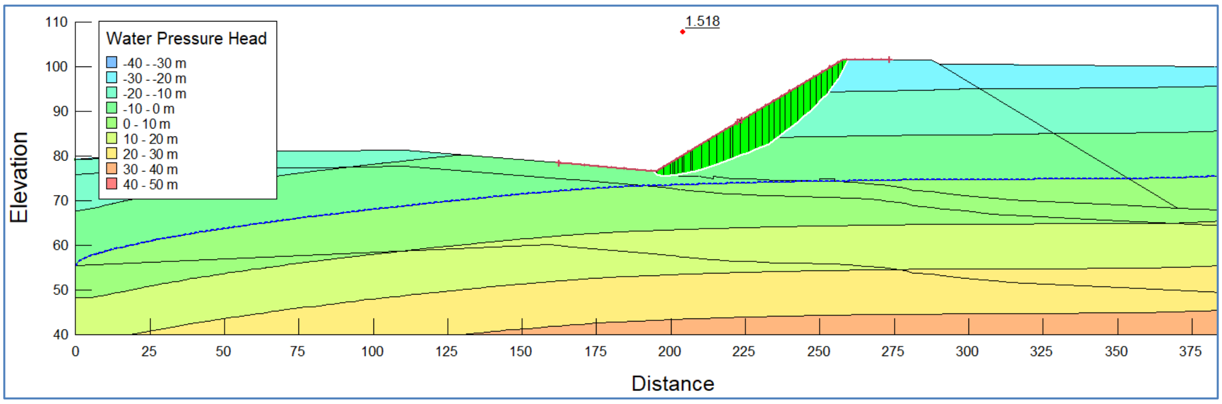
SCENARIO B5



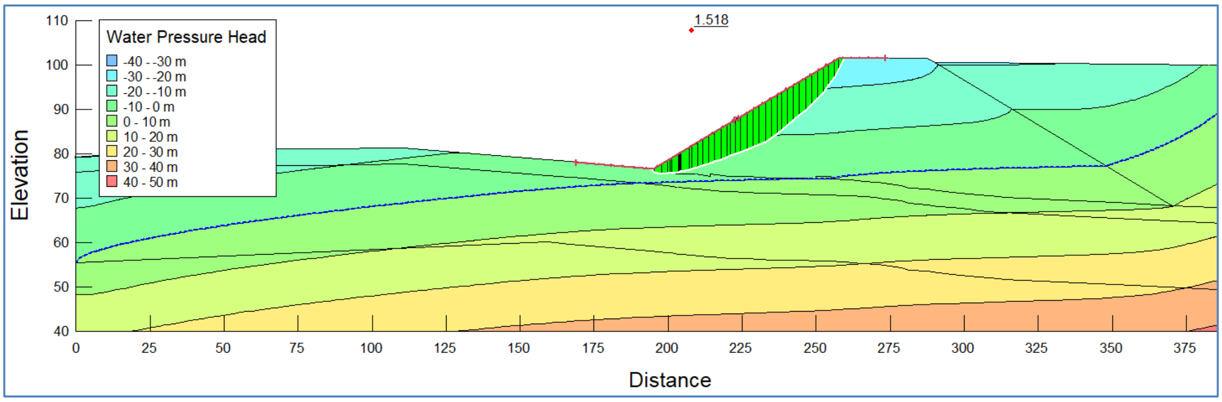
Scenario B6

Operational Phase – Maximum Capacity Reached

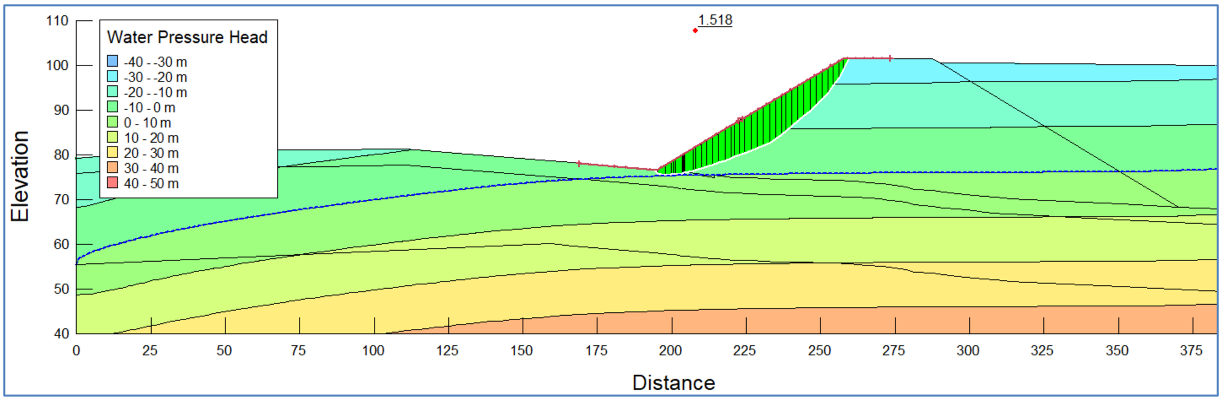
Scenario	Active Drains			Pool Position	
	centre blanket drain	downstream blanket drain	No Drains	Centre of RSF	100 m from embankment
C 1	X			X	
C 2	X				X
C 3		X		X	
C 4		X			X
C 5			X	X	
C 6			X		X



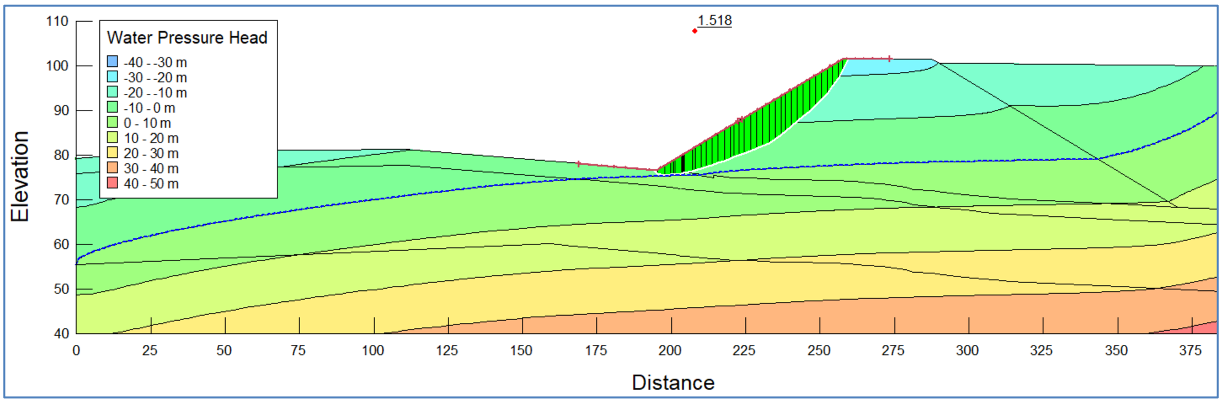
Scenario C1



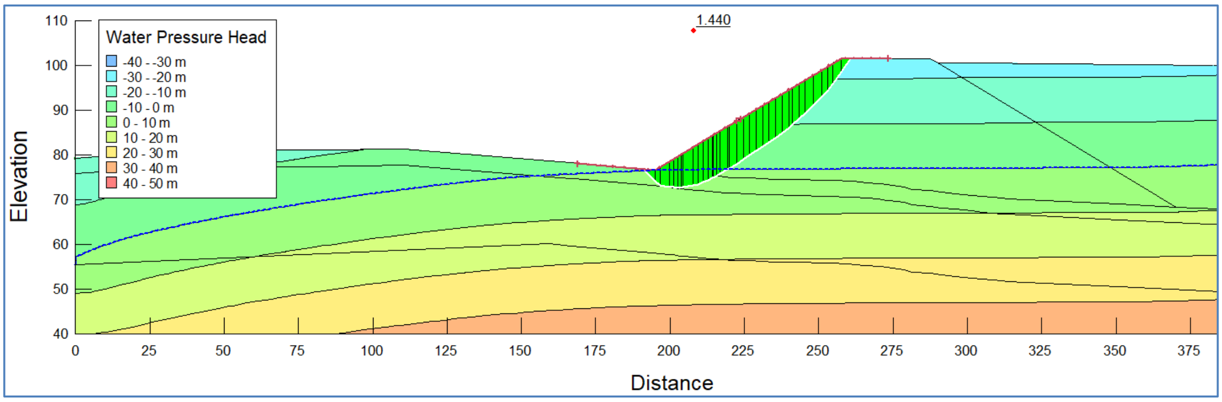
SCENARIO C2



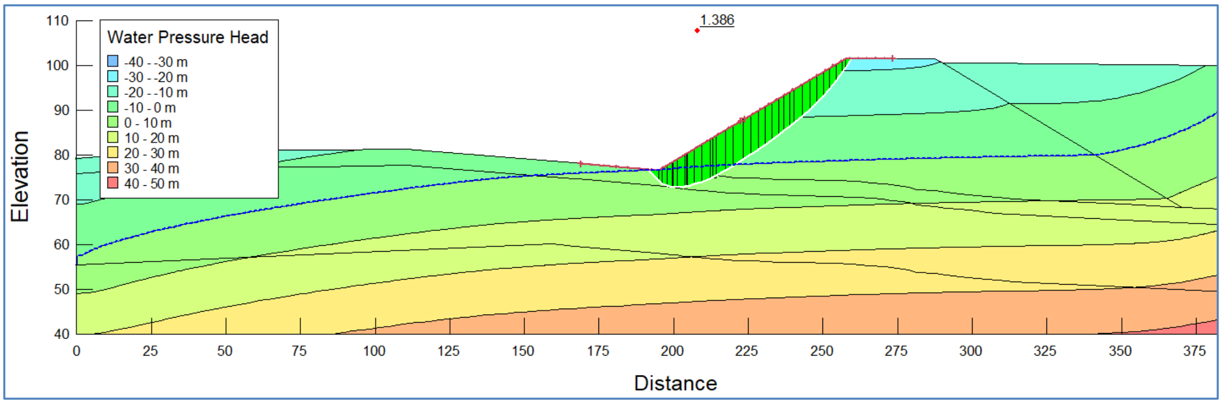
SCENARIO C3



SCENARIO C4



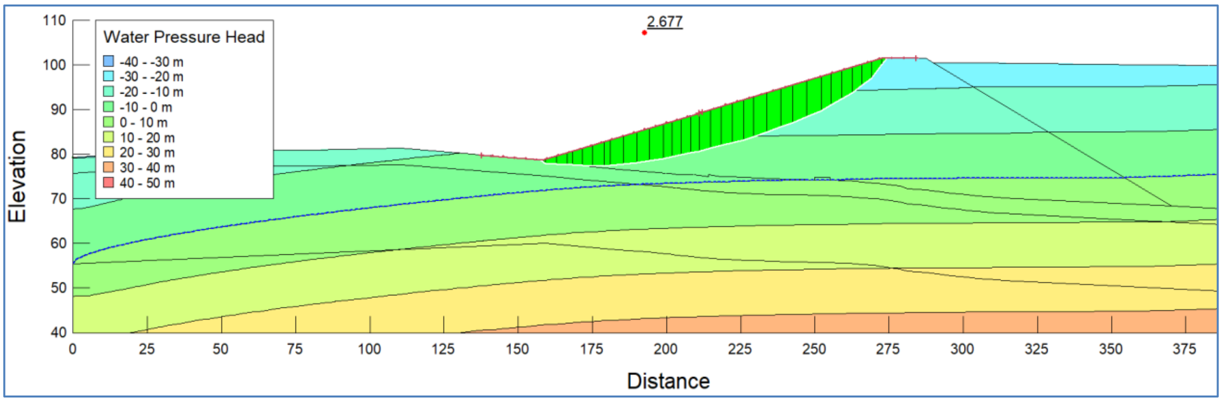
SCENARIO C5



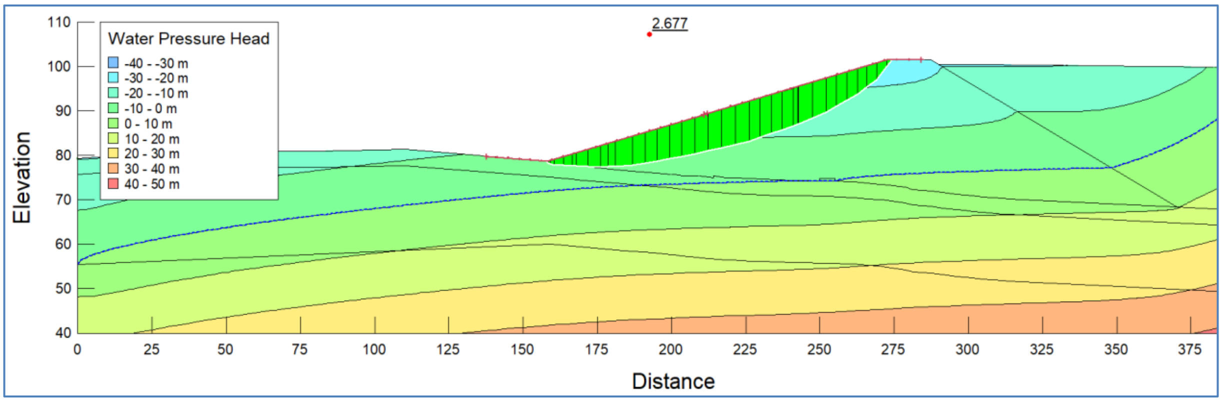
SCENARIO C6

CLOSURE PHASE – MAXIMUM CAPACITY REACHED

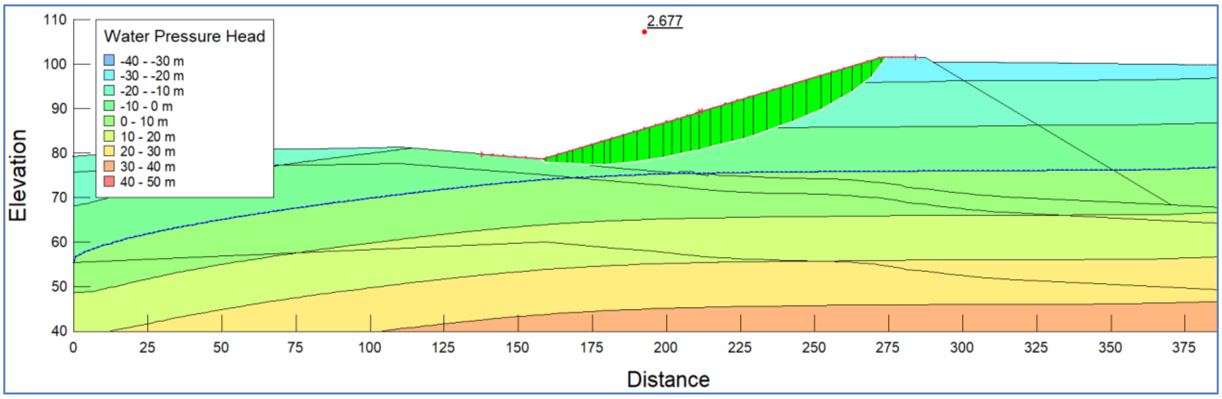
Scenario	Active Drains			Pool position	
	centre blanket drain	downstream blanket drain	No Drains	Centre of RSF	100 m from embankment
B 1	X			X	
B 2	X				X
B 3		X		X	
B 4		X			X
B 5			X	X	
B 6			X		X



Scenario D1



Scenario D2



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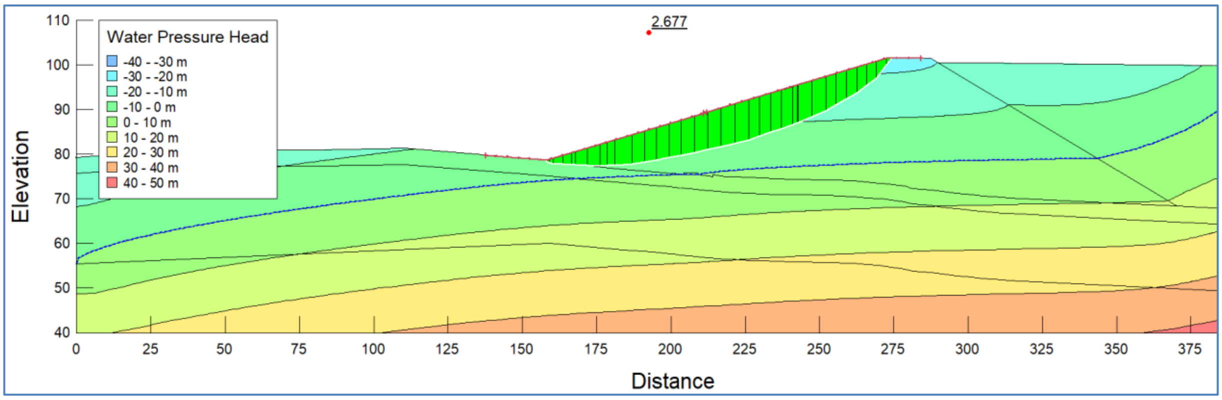

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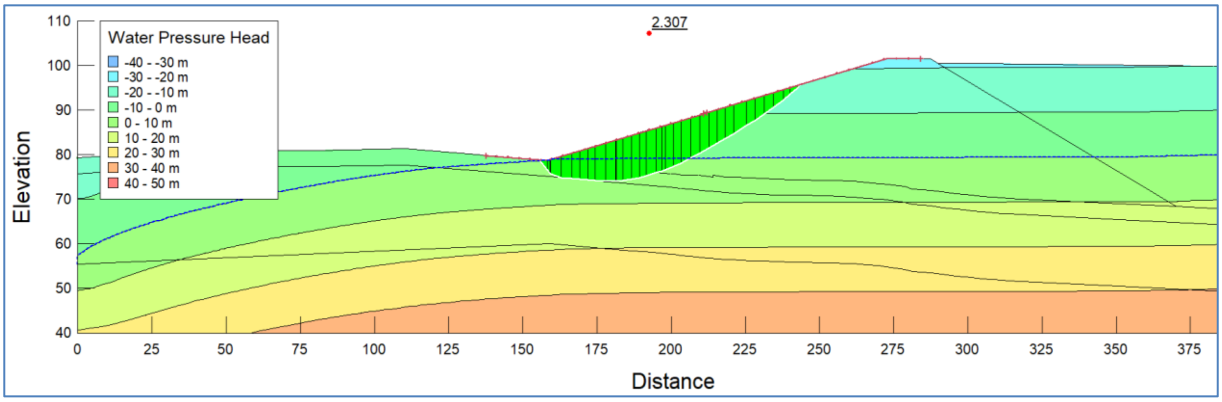

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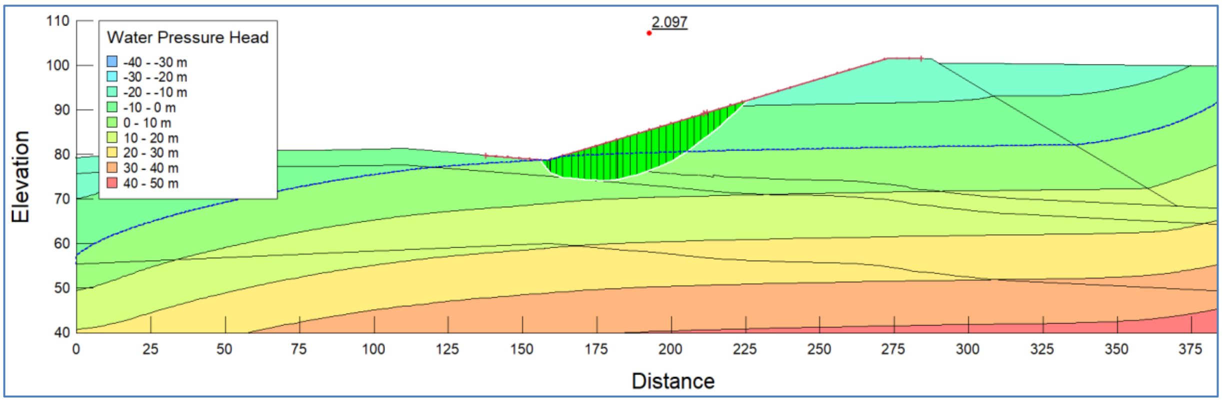

Scenario D6

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