

Appendix 1.0-L

SRK, Tenas Project Management and Control Pond Stability Assessment

FINAL

Tenas Project – Management and Control Ponds Stability Assessment

Tenas Project, British Columbia, Canada
Telkwa Coal Ltd.



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Tenas Project, British Columbia, Canada

Prepared for:

Telkwa Coal Ltd.
1415 Hankin Avenue
Telkwa, BC, V0J 2X0
Canada

+1 778 643 2843
www.allegiancecoal.com.au



Prepared by:

SRK Consulting (Canada) Inc.
350 3rd Avenue North, Suite 600
Saskatoon, SK S7K 6G7
Canada

+1 306 955 4778
www.srk.com



Lead Author: Santiago Pastine

Initials: SP

Reviewer: Calvin Boese **Initials:** CB

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Useful Definitions

This list contains definitions of symbols, units, abbreviations, and terminology that may be unfamiliar to the reader.

CH	High Plasticity Clay
CL	Low Plasticity Clay
EMP	East Management Pond
FoS	Factor of Safety
GC	Clayey gravel
GM	Silty gravel
IS	Infinite strength
LL	Liquid limit
MC	Mohr Coulomb
MP	Management Ponds
NAG	Non-Acid Generating
NMP	North Management Pond
PAG	Potentially-Acid Generating
PGA	Peak ground acceleration
PI	Plasticity Index
PSD	Particle Size Distribution
SC	Clayey sand
SM	Silty sand
USCS	Unified Soil Classification System
TCP	Tenas Control Pond
WMP	West Management Pond

Executive Summary

SRK Consulting (Canada) Inc. was retained by Telkwa Coal Ltd. to provide engineering services to complete a geotechnical assessment of the Tenas Control Pond (TCP) and Management Ponds (MP) for the Tenas Project in central British Columbia. The scope of work included interpretation of historical field and laboratory campaigns, determination of soil stratigraphy, strength of materials, and a slope stability assessment of the TCP and MP embankments.

The purpose of this work was to assess the stability of the TCP and MP, as designed by Telkwa, and to give basic recommendations for constructability at a feasibility design level.

A sequenced steady-state finite element seepage and two-dimensional limit equilibrium slope stability assessment of the TCP and MP were undertaken. Based on the current knowledge of the foundation materials, lower bound material strength parameters were adopted for potential weak layers (CH material), while strength and hydraulic parameters for the overburden, dam, and rock materials were adopted from previous field investigations (SRK, 2018 and SRK, 2019).

SRK included a drainage layer composed by non-acid generating higher permeability material at the toe of the buttress to prevent the development of pore pressures. The results of the analyses showed that the TCP, North Management Pond (NMP), East Management Pond (EMP), and West Management Pond (WMP) configurations are stable, achieve Telkwa's internal requirement and CDA, 2014 design criteria.

The foundation conditions for the NMP and TCP facilities were defined by previous field investigations and designs were assessed accordingly. Characterization of the foundations, under the EMP and WMP, was not as well defined as the dam footprint had shifted during recent design revisions by Telkwa. Based on the limited drilling it was uncertain if the potential weak layers identified in the NMP foundation extended to the EMP and WMP areas.

Considering the main risks and uncertainties related to: 1) foundation conditions, 2) material availability, and 3) performance of the soil-bentonite liner, sensitivity stability assessments were carried out for the EMP and WMP facilities due to lack of drilling data in those areas. The main conclusions of the sensitivity assessments were that:

- Different weak layer thicknesses in the foundation:
 - EMP: the EMP was designed considering the worst-case scenario (10.0 m thick of the CH layer), the facility achieved the design criteria in every scenario; and
 - WMP: the target Factor of Safety (FoS) was achieved even for the worst case scenario.
- Different dam material configurations:
 - Alternatives considering that a portion of the buttress was constructed with a draining material (i.e. NAG material) achieved the design criteria; and
 - If constructing the buttress with the overburden material, some localized failures are obtained in the buttress due to the development of pore pressures which require the addition of a higher permeable material to achieve the design criteria.

- Liner failure:
 - A potential total breakdown of the MP lining has a significant impact on the factor of safety (FoS) for all MPs. Minimum target FoS of 1.5 are not achieved for the overall structure and there is the potential for small localized at the toe of the dam and buttress. However, FoS results were still above 1.3 for the main dam structure and imminent failure would be unlikely; and
 - Constructing a chimney drain connected with the toe drain does improve the stability of the dam to meet minimal design criteria (e.g. CDA), and reducing the buttress slope to 6H :1V would also be required to prevent small localized failures at the toe of the dam and buttress.

Based on the results of this assessment, the following recommendations are being made:

- Undertake further field and laboratory testing to reduce uncertainties related to the foundation conditions, material strength and hydraulic parameters, including:
 - Additional geotechnical boreholes to determine the presence, extension, depth and thickness of weak layers (i.e. CH layers); and
 - Oedometric consolidation tests in selected samples to: (i) determine the pre-consolidation stress and (ii) define if undrained conditions are expected or not.
- Determine the availability and suitability of overburden and NAG materials for construction of the dam;
- Update the stability assessment based on the results of the recommended field and laboratory testing and the material availability and detailed dam design;
- Include a 2-to-3 m thick draining layer at the toe of the buttress in the design to prevent the development of pore pressures which may induce localized instabilities within the buttresses;
- Include a chimney drain in all embankments to provide contingency against a liner failure;
- Design an erosion protection layer for the downstream face of the dam; and
- Include chimney of draining material within the main dam components of the MP.

1 Introduction

1.1 Background

The Tenas Project is a Telkwa Coal (Telkwa) owned coal project located in central British Columbia. The Tenas Project's total mine life is anticipated to be 22 years, including one year of pre-stripping and construction followed by 21 years of production. The mine plan was designed and scheduled in a manner that allows potentially-acid generating (PAG) rock to be deposited into management ponds (MP), including the North Management Pond (NMP), the East Management Pond (EMP) and the West Management Pond (WMP). These MP will be constructed ahead of time, in sequence as per the mine plan developed by Telkwa, and allowed to flood with water to a level that maintains a minimum 1 m water cover above the PAG material placed in the MP. This PAG management plan is intended to reduce oxygen levels so that oxidization of rock will be minimized. The mine plan also includes construction of the Tenas Control Pond (TCP) to aid with onsite water management. The TCP will be constructed at the north of the NMP, and adjacent to the coal processing plant and rail loadout coal stockpile areas.

During 2018 and 2019, SRK conducted field programs to investigate the foundation characteristics for the previous versions of the MP design, and rock geotechnical investigations for the proposed pit slopes. This document presents the stability assessment of the TCP, NMP, EMP and WMP at a feasibility study level, based on the proposed ponds design provided by Telkwa (v.7 20210304) and the previous field investigation results.

1.2 Objectives and Scopes

The purpose of this work is to undertake the stability assessment of the TCP and MP by means of limit equilibrium analyses.

The scope of the study involves the TCP and all MP with focus in the following aspects:

- Field and laboratory data interpretation from 2018 and 2019 campaigns;
- Determination of soil stratigraphy and relevant material parameters;
- Development of representative cross-sections;
- Validation of the TCP and MP facilities design configuration using limit equilibrium analyses for both static and pseudo-static conditions against international standards and best practice guidelines; and
- Recommendations for further foundation characterization and/or design requirements.

The results presented herein are intended for the feasibility study of these facilities and should not be extended beyond this specific scope.

1.3 Reviewed Information

The MP design configuration and properties for this desktop study were based on the results of the field and laboratory campaign and design configurations. The key documents used and referenced in this document are presented below:

- Field investigations for foundation materials (SRK, 2018 and SRK, 2019); and
- Geotechnical stability analysis for MP and TCP (SRK, 2019b); and
- MP and TCP configurations provided by Telkwa (v.7 20210304).

2 Background Information

2.1 Site Conditions

The Tenas Project is located in central British Columbia approximately 25 km south of Smithers and 7 km southwest of Telkwa (Figure 2-1). The project site is located south of the Telkwa River.

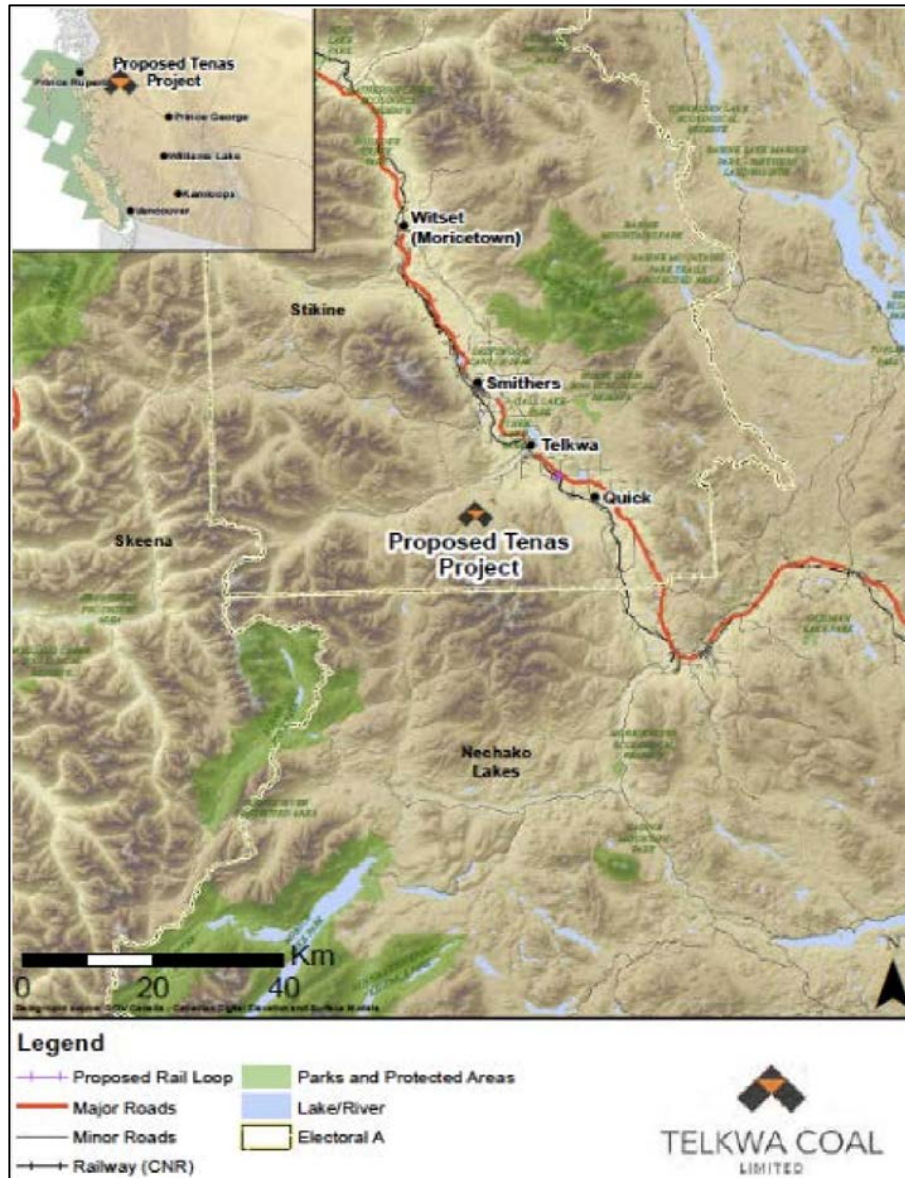


Figure 2-1: Tenas Project Location Plan (SRK, 2019a).

2.2 Geomorphology and Surficial Soils

The site is located within the foothills of British Columbia, between the Bulkley Mountain Range to the north and west, and the low-lying gently sloping relief of the Nechako Plateau.

Bedrock is typically covered by Quaternary soils, except for rocky outcrops along deeply eroded sections in adjacent river and stream valleys. Elevation across the site ranges generally from 600 to 900 m, with maximum elevations along the east side of the property Levson et al, 1997.

The regional surficial soils have been studied by Levson et al.(1997). Two soil units are characterized regionally, and include:

- A widespread, massive unit, interpreted as a till, is located at or near the ground surface elevation. The till is described as diamicton, with particle sizes ranging from clay to boulders within a fine grained or sandy matrix. No thickness information was provided by Levson et al (1997).
- In some areas, the till is underlain by both glaciofluvial stratified sands and gravels, and horizontally bedded sand, silt and clay sequences interpreted to be advanced-phase glacio-lacustrine sediments.

An objective of the geotechnical drilling was to improve the characterization of a glacial paleo-channel located on the northern portion of the project site. The 2019 investigations intercepted finer grained till, more commonly described as a sandy lean clay (CL), as well as coarser-grained soils within the paleo-channel described as a clayey silty sands or gravels. At greater depths within the paleo-channel (i.e. below 30 m) these two broader material types are interfingered. Both of these materials are comparable to the till units described by Levson et al (1997). In 2018, the surficial soils intercepted were mostly sandy clean clays.

Similar to 2018, the 2019 results indicate that the silts and clays are commonly stiff to hard in consistency, and the sands and gravels are dense to very dense.

2.3 Bedrock Geology

The coal that is to be mined from the Tenas Pit is located within the sedimentary sequences of the early Cretaceous-aged Skeena Group. The Skeena Group consists of mudstones, siltstones, sandstones and conglomerates with inter-bedded, bituminous coal seams. The Skeena Group has been historically divided into four units (Palsgrove, 1991), of which two units are economically coal-bearing (Figure 2-2).

Of relevance to this study at Tenas is Unit 1 described by Palsgrove (1991). A geological map of the Tenas Pit area with the Unit 1 outcrops is shown in Figure 2-3. Unit 1 comprises a 50 to 120 m thick sedimentary sequence interbedded with Coal Seam 1 (Seams 1A to 1F). The individual coal seams vary from less than 1 m to 3 m in thickness and can be laterally discontinuous (Palsgrove, 1991).

The Skeena Group unconformably overlies the Hazelton Volcanic Group rocks. Additionally, the project geology map indicates the intrusive unit is in contact with the Skeena Group in all directions outside of the proposed pit footprint (Figure 3-3). However, the Hazelton Volcanic Group was not intercepted in vertical diamond drillholes that extended below the lower-most coal plys at Tenas or the inclined diamond

drillholes that were drilled through the east-limb of the Tenas syncline (i.e. proposed east wall) along the east side of the deposit.

As discussed, the Skeena Group rock have been folded into a syncline at Tenas. On the west side the rocks are shallow, west dipping and gently folded along a west-limb. Toward the east, the bedding dips increase at the axis of the fold, and the rocks are moderately dipping to the east along the east-limb.

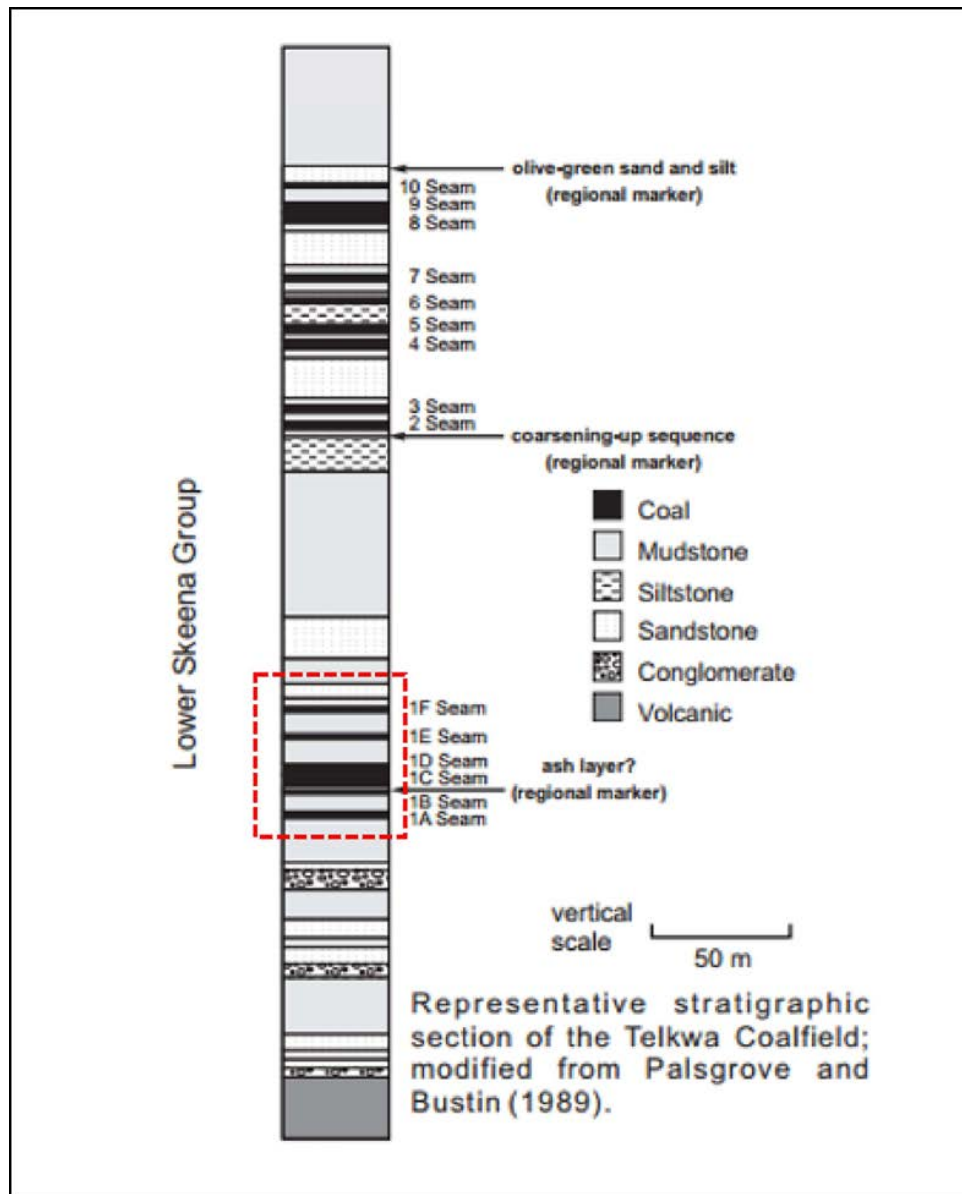


Figure 2-2: Representative Stratigraphy of the Skeena Group (Palsgrove, 1991). Unit 1 shown in Red Dashed Box.

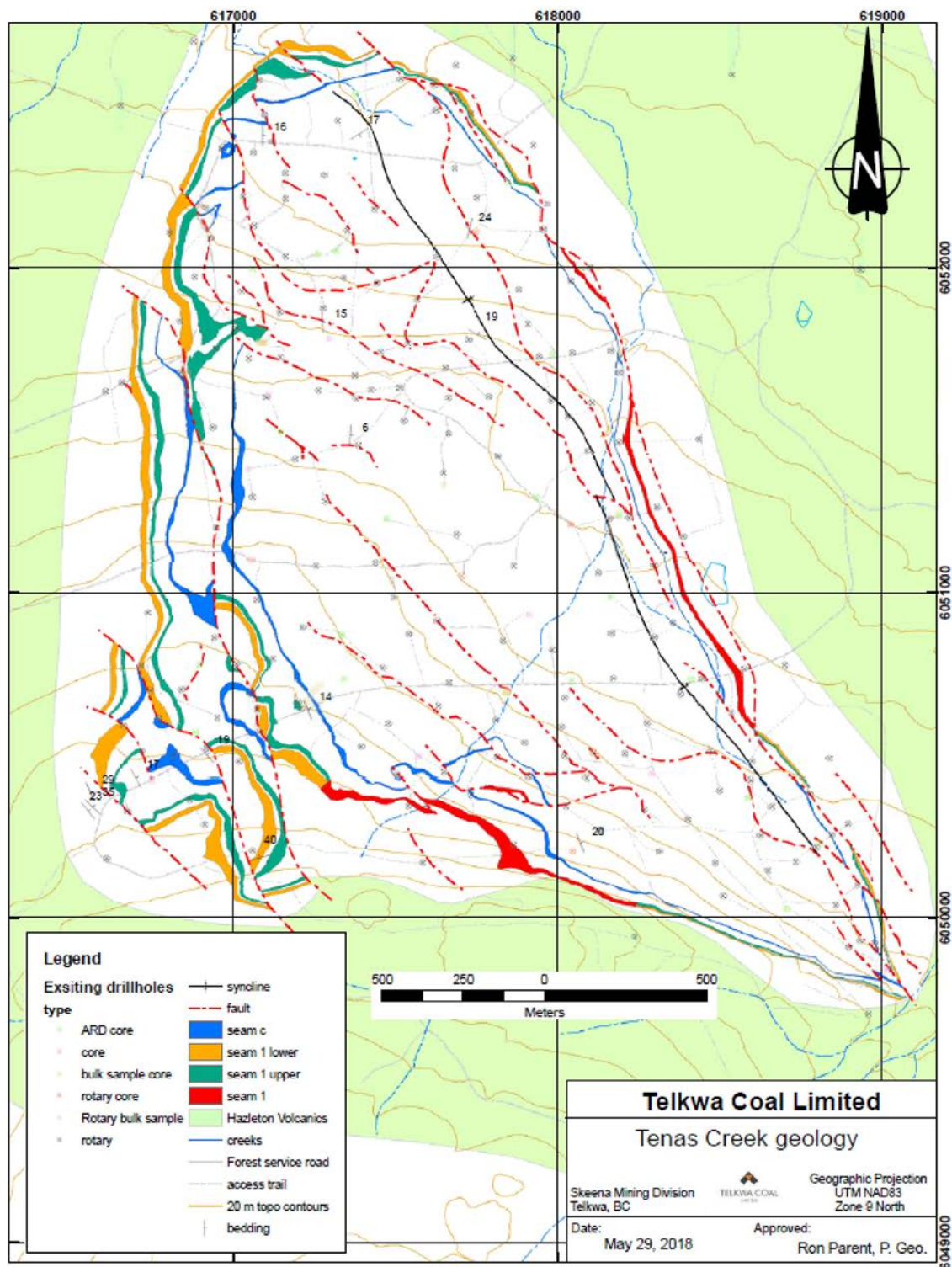


Figure 2-3: Tenas Geology Map (Telkwa, 2018)

2.4 Seismicity

The 2015 National Building Code Seismic Hazard Map (NCR, 2015) indicates that the Telkwa Coal project site is located in a relatively stable seismic area. The peak ground acceleration (PGA) for a two percent probability of exceedance in a 50-year period, i.e. 1 in 2,475 years, is 0.049 times the force of gravity of 9.81 m/s²(g).

2.5 Proposed Mine Plan

The excavation of the Tenas Pit is to be completed in seven phases, each with a subsequent backfill sequence. The non-acid generating (NAG) and PAG waste rock is to be segregated through strategic excavation of the sedimentary sequences. The NAG waste rock is to be used for construction purposes and to backfill the open pit while the PAG is to be placed in the three constructed MP comprising an East, West and North cell (Figure 2-4). The most recent revision of the completed MP (v.7 20210304) was provided by Telkwa for use in this assessment.



Figure 2-4: Mine Layout at the End of Life of Mine.

3 Review of Available Data

3.1 Previous Investigations

Foundation conditions for previous designs of the TCP and MP were studied by SRK in 2018 (SRK, 2018) and 2019 (SRK, 2019a). The MP configuration and footprint has changed for the current embankment design; however, the previous completed investigations and lab testing are still indicative of general site foundation characteristics. The previous investigations included:

- Sonic drilling: 13 geotechnical sonic drill holes in 2018 and 12 geotechnical sonic drill holes in 2019 were carried out to investigate material foundation at a maximum depth of 30.8 m and 67.1 m, respectively, including SPT tests at 3.0 m intervals, sampling and logging according to the Unified Soil Classification System (USCS). A total of 4 and 12 standpipe piezometers were installed in the drill holes for shallow groundwater monitoring in 2018 and 2019 field campaigns, respectively.
- Test Pits: 12 test pits in 2018 and 18 test pits in 2019 were excavated to investigate shallow foundation materials at a maximum depth of 4 m and 4.6 m, respectively, including logging according to the USCS and photographic records of the test pits.
- Diamond drilling: 5 geotechnical diamond drillings in 2018 and 4 geotechnical diamond drilling in 2019 were performed to investigate the rock mass and structural geology conditions, including geotechnical logging, sampling, laboratory testing and downhole geophysics.

A map of the geotechnical investigation area, with all sonic drill holes, test pits and diamond drill holes, is shown in Figure 3-1.

The overburden material is composed by a very stiff to hard sandy lean clay (CL), with low to medium plasticity, with intervals of fat clay (CH), clayey silty sand (SC or SM) or gravel (GC or GM) at various depths. SPT values typically range from 20 to 40 for lean clay (CL) and fat clay (CH) for depths from 0 m to 20 m (Figure 3-2).

Piezometric information and water table estimates were based on a review of monitoring well reading data. Most of the recorded groundwater levels were between 20 m and 40 m, and the shallowest level were approximately 14 m. Groundwater inflow was observed in four test pits during 2018 and in one test pit during 2019.

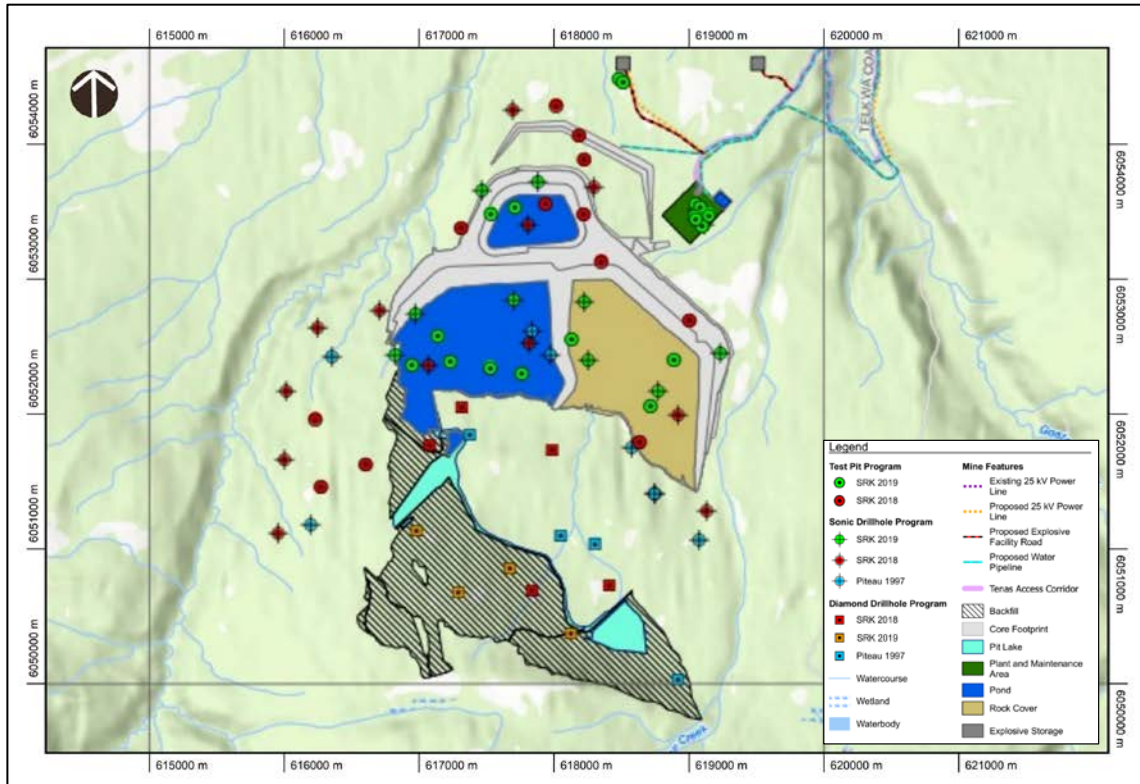


Figure 3-1: 2018 and 2019 SRK Geotechnical Investigation Area with Sonic Drillholes, Test Pits and Diamond Drillholes (SRK, 2020).

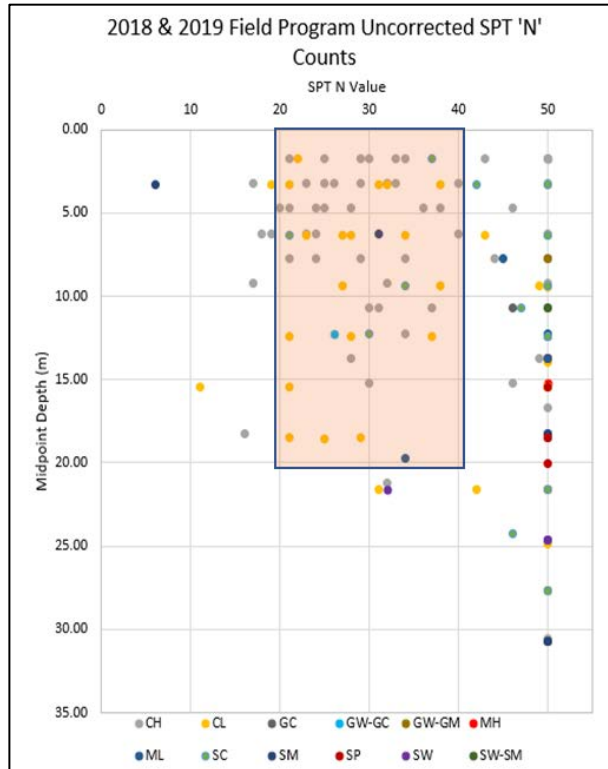


Figure 3-2: 2018 and 2019 SRK Field Campaign. SPT Values vs. Depth.

3.2 Laboratory Testing Data

Disturbed representative samples were recovered from the sonic drill holes and test pits during the 2018 and 2019 SRK field campaigns. Table 3-1 presents the laboratory tests performed in each case.

Table 3-1: Laboratory Tests. 2018 and 2019 SRK Field Campaigns.

Test	No. Tests in 2018	No. Tests in 2019
Particle Size Distribution Tests with Hydrometer	19	73
Atterberg Limits (Plasticity Index)	18	35
Proctor Soil Compaction Tests	4	—
Hydraulic Conductivity	3	—
Direct Shear Tests of Remolded Soil (Consolidated/Drained)	3	10
Soil Resistivity	10	—
Soil pH	10	—
Water Content	18	35

The results of Particle Size Distribution (PSD) tests are presented in Figure 3-3 and Figure 3-4. These charts show that the samples generally have a fines content of 40% - 70% with a typical sand content of 20% - 40% and a gravel content of 0% - 20%. PSDs obtained in 2019 were consistent with the ones obtained during 2018.

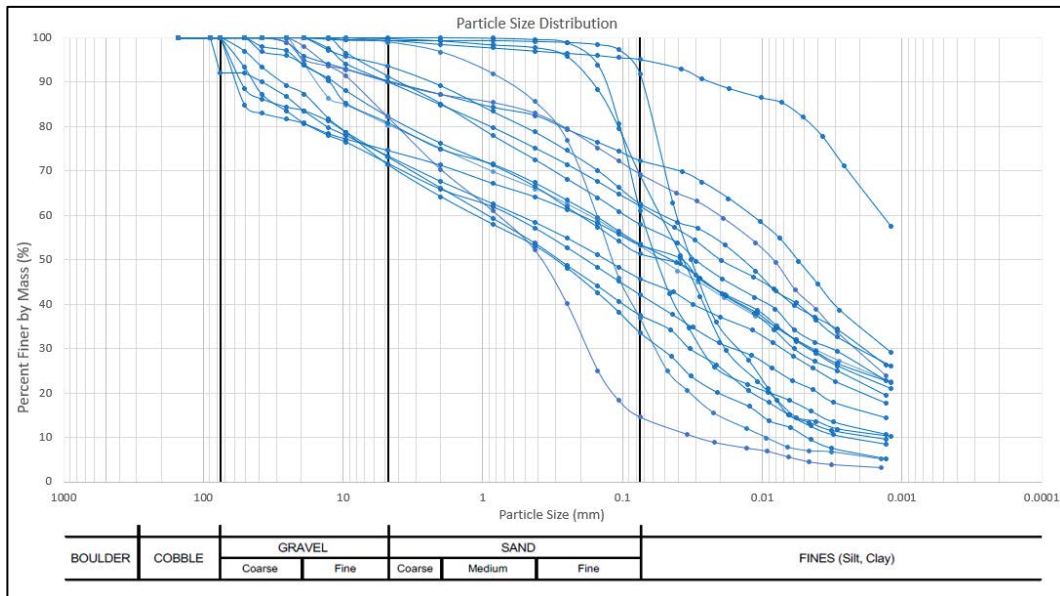


Figure 3-3: 2018 SRK Field Campaign. PSD Curves (SRK, 2019a).

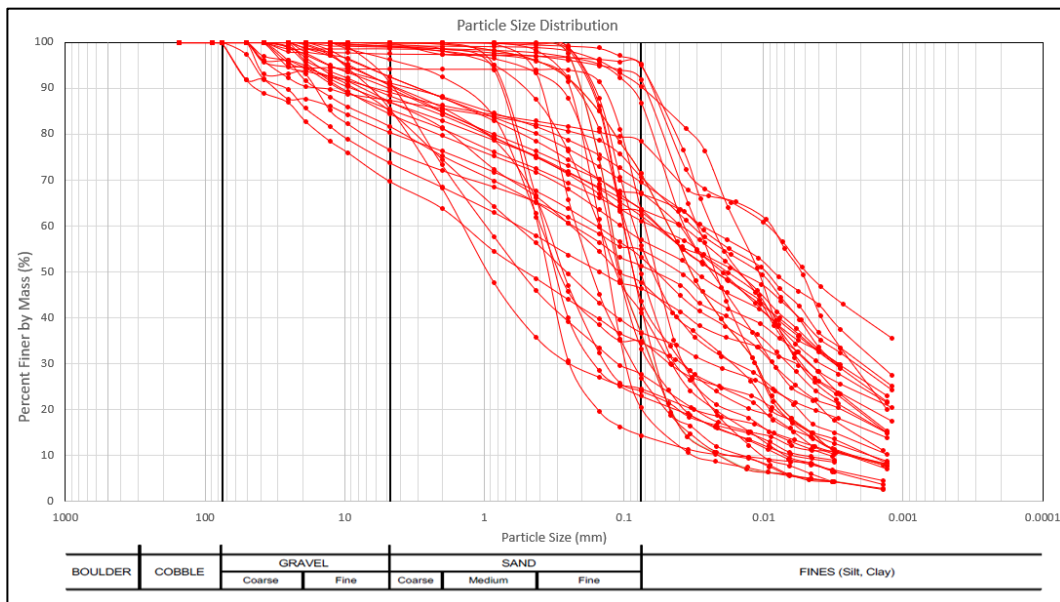


Figure 3-4: 2019 SRK Field Campaign. PSD Curves (SRK, 2019a).

Atterberg Limit tests were completed for soil classification. The results are presented in Figure 3-5 and Table 3-2. Samples classify as low plasticity clays (CL), except two 2018 samples which classify as high plasticity clays (CH). Values obtained during 2018 and 2019 were consistent with historical data.

Table 3-2 also accounts for the natural moisture content, which is in general lower or similar to the plastic limit. This, together with the high SPT values observed during the field investigation, may indicate that the material is over-consolidated.

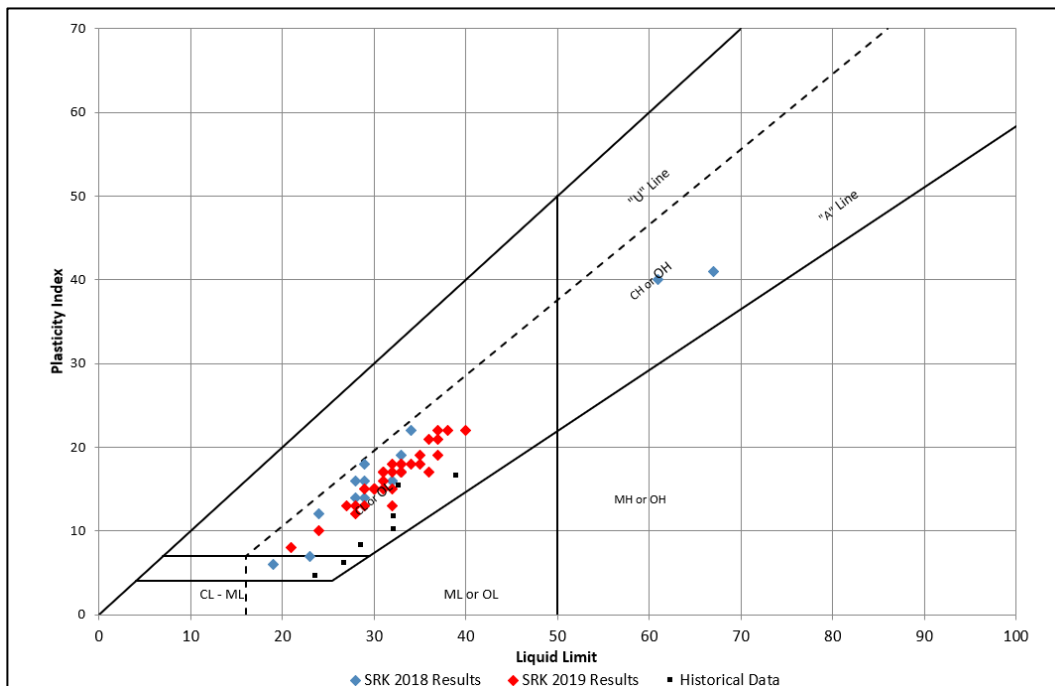


Figure 3-5: Atterberg Limits for 2018 and 2019 Test Results (SRK, 2019a).

Table 3-2: Atterberg Limits and moisture content test results.

Field Campaign	PI [%]	LL [%]	PL [%]	ω [%]
2018	NP 41	NP 67	NP 26	9.3 29.2
2019	8 52	21 40	13 19	6.4 22.1

PI: Plastic Index; LL: Liquid Limit; PL: Plastic Limit; ω : moisture content

Thirteen direct shear tests were performed on remolded test specimens from disturbed soil samples. The testing was conducted on finer-grained portions of the samples collected during the site investigations. A summary of the results is presented in Table 3-3.

Table 3-3: Summary of Remolded Soil Direct Shear Results.

SRK Field Program	Location	Sample ID	Depth (m)	Soil	Average Residual Friction Angle (°)
2018	T18-02S	SPT-05	7.7	CL	23.1
	T18-12S	SPT-07	18.2	CH	21.1
	T18-10S	BLK-01	3.8	CL	11.0
2019	19-19S	35096	34.8	CL	33.9
	19-21S	35035	15.5	CL	24.0
	19-22S	29661	3.4	CL	31.5
	19-22S	29665	16.5	CL	33.3
	19-34S	29622	31.7	CL	22.7
	19-35S	29624	5.8	CL	29.9
	19-35S	35143	22.6	CH	34.5

3.3 Rock Mass Characterization

The geotechnical characterization of the rock mass in MP and TCP zones were obtained from SRK (2018 and 2019). Intact rock strengths were estimated following the ISRM Strength Index (Table 3-4). The rock mass was classified following the Rock Mass Rating (RMR₈₉) developed by Bieniawski in 1989 (Table 3-5).

A brief description of the principal findings is presented herein:

- The lithology intercepted in the drilling program consisted of mudstones, siltstones, and sandstones with inter-bedded, bituminous coal seams.
- The assessed strengths were generally R3 for the sandstones, R2 to R3 for the siltstones, and R2 to R3 for the mudstones. All rock types were observed to be weaker parallel to bedding. Laboratory intact rock testing work supported the field strength estimates from geotechnical logging.
- The Rock Mass Rating ₈₉ (RMR₈₉) was in average, 56 for the mudstones, 58 for the siltstones and 69 for the sandstones. An RMR₈₉ was not assigned for the coal and automatically assigned a 20 as the classification use is limited for very weak rocks.

Table 3-4: Summary of ISRM intact rock strength estimates (Brown, 1982).

Strength Index	Description	Field Identification	Approx. Range of Unconfined Compressive Strength (MPa)
R0	Extremely Weak Rock	Indented by thumbnail	0.25 – 1.0
R1	Very Weak Rock	Crumbles under firm blows with point of geological hammer, can be peeled by a pocketknife	1.0 – 5.0
R2	Weak Rock	Can be peeled by a pocketknife with difficulty, shallow indentations made by firm blow with point of geological hammer	5.0 – 25
R3	Medium Strong Rock	Cannot be scraped or peeled with a pocketknife, specimen can be fractured with single firm blow of geological hammer	25 – 50
R4	Strong Rock	Specimen requires more than one blow of geological hammer to fracture it	50 – 100
R5	Very Strong Rock	Specimen requires many blows of geological hammer to fracture it	100 – 250
R6	Extremely Strong Rock	Specimen can only be chipped with geological hammer	>250

Table 3-5: Summary of the rock mass rating system (Bieniawski, 1989)

Parameter	Range of Values						
UCS (MPa) Rating	>250 (15)	100 – 250 (12)	50 – 100 (7)	25 – 50 (4)	5 – 25 (2)	1 – 5 (1)	<1 (0)
RQD Rating	90 – 100% (20)	75 – 90% (17)	50-75% (13)	25-50% (8)	<25% (3)		
Joint Spacing Rating	>2 m (20)	0.6 – 2 m (15)	200 – 600 mm (10)	60 – 200 mm (8)	<60 mm (5)		
Joint Condition Rating, (JCR)	Very rough No separation Hard wall rock (30)	Slightly rough Separation <1 mm Hard wall rock (25)	Slightly rough Separation <1 mm Soft wall rock (20)	Slickensides, Separation or gouge <5 mm (10)	Soft gouge or Separation >5 mm (0)		
Groundwater (Jw) Rating	Completely Dry (15)	Damp (10)	Moist (7)	Mod. Pressure (4)	Severe (0)		
Total RMR ₈₉ Value (Sum of Ratings for 5 Items) =							
Rating	100 – 81	80 – 61	60 – 41	40 – 21	20 – 0		
Category	5 – Very Good	4 – Good	3 – Fair	2 – Poor	1 – Very Poor		

4 Geotechnical Strength Parameters

Geotechnical strength parameters for the rock mass, foundation soil, PAG, NAG and pond materials were estimated based on the field and laboratory testing programs from 2018 and 2019. Where no information was available, the strength parameters were inferred from SRK (2019b) and by application of engineering judgement.

4.1 Bedrock

Field and laboratory testing results presented in Section 3.2 show that rock mass is of good quality (based on the RMR₈₉). Since the geotechnical strength of the bedrock is much higher than the strength of the foundation soils, the rock mass was considered to be an infinite strength material during the stability assessment.

4.2 Foundation Soils

Foundation soils are composed by alternate layers of clay and sand while some gravel lenses from 0.5 to 3 m thick were found between these layers (SRK, 2018 and SRK, 2019a).

Units weight for CL, CH, and sand layers (mainly clayey sands – SC) were adopted based on the soil classification and SRK's experience with similar materials (Table 4-1).

The critical effective internal friction angle ϕ'_{cv} for CL and CH materials were adopted from direct shear tests; an average value was adopted for CL and lower bound value was conservatively adopted for CH due to the limited testing and uncertainty about this material. For the sand material, a typical value of $\phi'_{cv} = 30^\circ$ was considered. Table 4-1 presents the values adopted during the limit equilibrium analyses.

The effective cohesions c' were adopted equal to 0 kPa since no evidence of cementation was observed during the field and laboratory testing programs.

Table 4-1: Foundation Soils. Geotechnical Strength Parameters.

Material	Unit weight [kN/m ³]	c' [kPa]	ϕ'_{cv} [°]
CH	18	0	20
CL	18	0	27
Sand	20	0	30

4.3 Mine Materials and Construction Materials

Mine materials and construction geotechnical strength parameters were obtained from SRK (2019b) and considered:

- The dam and buttress will be constructed with the foundation soil materials, with the appropriate compaction requirements (e.g. potential for softening under seismic loading, or strength reduction under undrained loads is not expected).
- A soil-bentonite liner will be constructed on bottom of the MPs by diskings bentonite into the surficial soil. For the purpose of this feasibility design study, the geotechnical strength parameters of the liner were conservatively adopted equal to those of the CH layer. More detail of the liner system is presented in Section 5.4.
- PAG and NAG materials are mine rock.

Table 4-2: Mine Material and Construction Materials. Geotechnical Strength Parameters.

Material	γ [kN/m ³]	c' [kPa]	ϕ'_{cv} [°]
Dam	20.5	0	30
Buttress	20.5	0	30
PAG	23	0	35
NAG	23	0	35
Liner	18	0	20

γ : unit weight; c' : effective cohesion; ϕ'_{cv} = effective internal friction angle.

5 TCP and MP Stability Assessment

5.1 Introduction

A stability assessment of the TCP and the MP were undertaken using standard limit equilibrium analyses in Slide 2018 (Rocscience) and followed the suggested acceptance criteria indicated in CDA (2014).

Determinist analyses for non-circular surfaces were performed for static and pseudo-static conditions to identify potential failure surfaces and the associated FoS using GLE/Morgenstern & Price method. The Optimize Surfaces option was used in the analyses.

5.2 Dam Classification

The dam consequence classification was assessed in the dam breach assessment report (SRK, 2021b), according to CDA (2013) guidelines and supported by the Global International Standards on Tailings (GISTM) (ICMM, 2020). Based on the consequence evaluation, a consequence of 'Very High' for all four dams was recommended.

5.3 Design criteria

The stability assessment was performed following CDA (2014) guidelines and suggested acceptance criteria, presented in Table 5-1 and Table 5-2, and Telkwa's internal requirements:

- During or at the end of construction (steady state static loading condition): $FoS > 1.3$ (CDA, 2014); long-term drained (steady state static loading condition): $FoS > 1.6$ (Telkwa's internal requirements which are greater than $FoS > 1.5$ (CDA, 2014)); and
- Pseudo-static condition: $FoS > 1.0$ (CDA, 2014).

Given that the materials used for the embankment construction will be compacted and that the foundation material will be over-consolidated even for the expected loads, potential for softening under seismic loading, or strength reduction under undrained loads is not expected and has not been considered for these materials in the stability assessment. This assumption should be confirmed during the following engineering stages.

For the pseudo-static condition, a horizontal seismic acceleration was adopted from the National Building Code of Canada (NBCC 2015) based on local site classification, Site Class C. The ground acceleration for stability modelling was defined as follows:

$$\alpha_h = \frac{1}{2}PGA \sim 0.025g \quad (PGA: \text{Peak Ground Acceleration}; g: \text{acceleration of gravity})$$

Table 5-1: Target FoS for Slope Stability in Construction, Operation and Transition Phases – Static Assessment (CDA, 2014).

Loading Condition	Minimum FoS	Slope
During or at end of construction	>1.3 depending on risk assessment during construction	Typically downstream
Long term (steady state seepage, normal reservoir level)	1.5	Downstream
Full or partial rapid drawdown	1.2 to 1.3	Upstream slope where applicable

Table 5-2: Target FoS for Stability in Construction, Operation and Transition Phases – Seismic Assessment (CDA, 2014).

Loading Condition	Minimum FoS
Pseudo-static	1.0
Post-earthquake	1.2

5.4 Liner Design

A soil-bentonite liner will be constructed over the pond bottom of the NMP, EMP and over the Ex Pit portion of and the overburden pit slope of the WMP to limit seepage out of the MPs. The recommended liner design is a 0.30 m low permeability soil-bentonite liner that will be constructed in two 0.15 m lifts. This liner system provides a seepage rate $\leq 2,800 \text{ m}^3/\text{day}$, which corresponds to the maximum allowable seepage rate for the 10 Year Dry scenario. To achieve this design criteria the soil-bentonite liner system will require a hydraulic conductivity of $1\text{E-}10 \text{ m/s}$. Therefore the hydraulic conductivity used to define the liner and also used in the seepage analysis is $1\text{E-}10 \text{ m/s}$, as listed in Table 5-3.

Construction of the liner will be completed by pre-disking the native soil after completing preliminary foundation preparation (clearing and grubbing). Then enough bentonite will be spread over the loosened soil and disked in to meet the target of $\leq 1\text{E-}10 \text{ m/s}$ hydraulic conductivity. Placement of a subsequent lift of 0.15 m of native soil, followed by another application of bentonite and diskling will be completed before the final compaction of the 0.30 m soil-bentonite liner system.

A laboratory study will be performed to determine the optimal proportion of bentonite-soil blend to achieve the hydraulic conductivity criteria after which field trials will be performed to refine the construction methodology, equipment selection, and verify the soil-bentonite liner system performs to specification with an in-situ hydraulic conductivity of $\leq 1\text{E-}10 \text{ m/s}$.

More detail of the liner system is presented in the liner design and constructability memorandum in Appendix C. (SRK, 2021a).

5.5 Material Parameters

The material strength and hydraulic parameters for the limit equilibrium analyses are summarized in Table 5-3. The detailed procedure followed to determinate them was presented previously in Section 4.

The NAG and PAG material hydraulic parameters were adopted considering that the material is mine rock. The bedrock and soil permeabilities were obtained from work completed on assessment of the MP (SRK, 2020).

Table 5-3: Material Parameters.

Material	Failure Criterion	Color	γ [kN/m ³]	c' [kPa]	ϕ'_{cv} [°]	k_h [m/s]	k_v/k_h
Dam	Mohr Coulomb		20.5	0	30	1×10^{-07}	1
Buttress	Mohr Coulomb		20.5	0	30	1×10^{-07}	1
NAG Rock	Mohr Coulomb		23	0	35	1×10^{-05}	1
PAG Rock	Mohr Coulomb		23	0	35	1×10^{-05}	1
Liner	Mohr Coulomb		18	0	20	1×10^{-10}	1
CH	Mohr Coulomb		18	0	20	3×10^{-06}	1
CL	Mohr Coulomb		18	0	27	3×10^{-06}	1
Loose Silt	Mohr Coulomb		18	0	25	3×10^{-06}	1
Sand	Mohr Coulomb		20	0	30	3×10^{-05}	1
Bedrock	Infinite Strength		25.5	Infinite Strength		3×10^{-07}	1

γ : unit weight; c' : effective cohesion; ϕ'_{cv} : effective internal friction angle; k_h : horizontal permeability coefficient; k_v : vertical permeability coefficient.

5.6 Geometric Configuration

Eleven cross-sections were defined for limit equilibrium analysis. The cross-section locations were selected to account for the different foundation soil conditions and different ponds' geometric configurations.

Based on the results of the previous field and laboratory testing campaigns, two potentially weaker zones were identified under the EMP and WMP:

- Zone 1 (under EMP): at 15-20 m depth, samples with lower SPT values, higher plasticity and lower direct shear test strength values were founded. This potentially weaker layer is underlined by a more competent material, which may encourage a possible failure along the interface between both materials; and
- Zone 2 (under WMP): SPT values are low, lower direct shear value results and undefined area.

SRK included a drainage layer (composed of NAG material) at the toe of the buttresses of the facilities to prevent the development of pore pressures at the toe of the embankments. Table 5-4 presents the main characteristics of each facility.

SRK also considers that, since the dam material is a low plasticity clay, for closure conditions a downstream erosion protection layer should be placed to protect the dam material in the long-term.

Table 5-4: MP and TCP Main Characteristics.

Parameter	Units	NMP	EMP	WMP	TCP	Notes
PAG Volume (saturated)	m ³	2,549,926	11,380,614	26,880,946	—	Total volume with water in voids
CR Volume (saturated)	m ³	147,796	773,496	2,113,551	—	Total volume with water voids
Top of PAG Elevation	masl	826.0	847.1	844.8	—	Based on stage-storage curves
NAG Cover thickness	m	1.0	1.0	1.0	—	—
NAG Volume	m ³	267,726	1,031,559	941,924	—	Based on stage-storage curves
Top of NAG Elevation	masl	827.0	848.1	845.8	—	—
Min. OVB Cover Thickness	m	1.0	1.0	1.0	—	From spillway invert to top of NAG. Additional OVB may be placed above the spillway, as-needed
Min. OVB Volume	m ³	272,757	1,059,380	956,852	—	Additional OVB volume may be placed above spillway invert as-needed
Spillway Elevation	masl	828.0	850.0	850.0	798.0	Also top of OVB (minimum)
Volume at Spillway	m ³	3,238,206	14,245,049	30,893,272	839,058	—
Water Level above top of PAG	masl	2.0	2.0	2.0	—	Spillway invert, 2m above PAG elevation
Water Level	masl	828.0	850.0	850.0	798.0	Spillway Elevation at closure
Dam Crest	masl	830.0	852.0	852.0	800.0	As provided

Figure 5-1 presents the locations of the cross-sections and the potential weaker zones. Figure 5-2 to Figure 5-14 show the cross-sections with the respective dimensions and location relative to the TCP and MP area:

- TCP-01: highest section of the TCP's dam;
- TCP & NMP-01: near several drill holes and test pits;
- NMP-01: highest section of the northwest area of NMP's dam, and near to several drill holes and test pits;
- NMP-02: middle of the west side of the NMP;

- NMP &WMP-01: transversal section; near several drill holes and test pits;
- WMP-01: highest WMP's dam section;
- WMP-02: nearest to the Tenas Open Pit; near several drill holes and test pits;
- EMP & WMP-01: highest section of the EMP&WMP embankment;
- EMP-01: highest EMP's dam section;
- EMP-02: near several drill holes and test pits; and
- EMP-03: middle of the east side of the EMP.

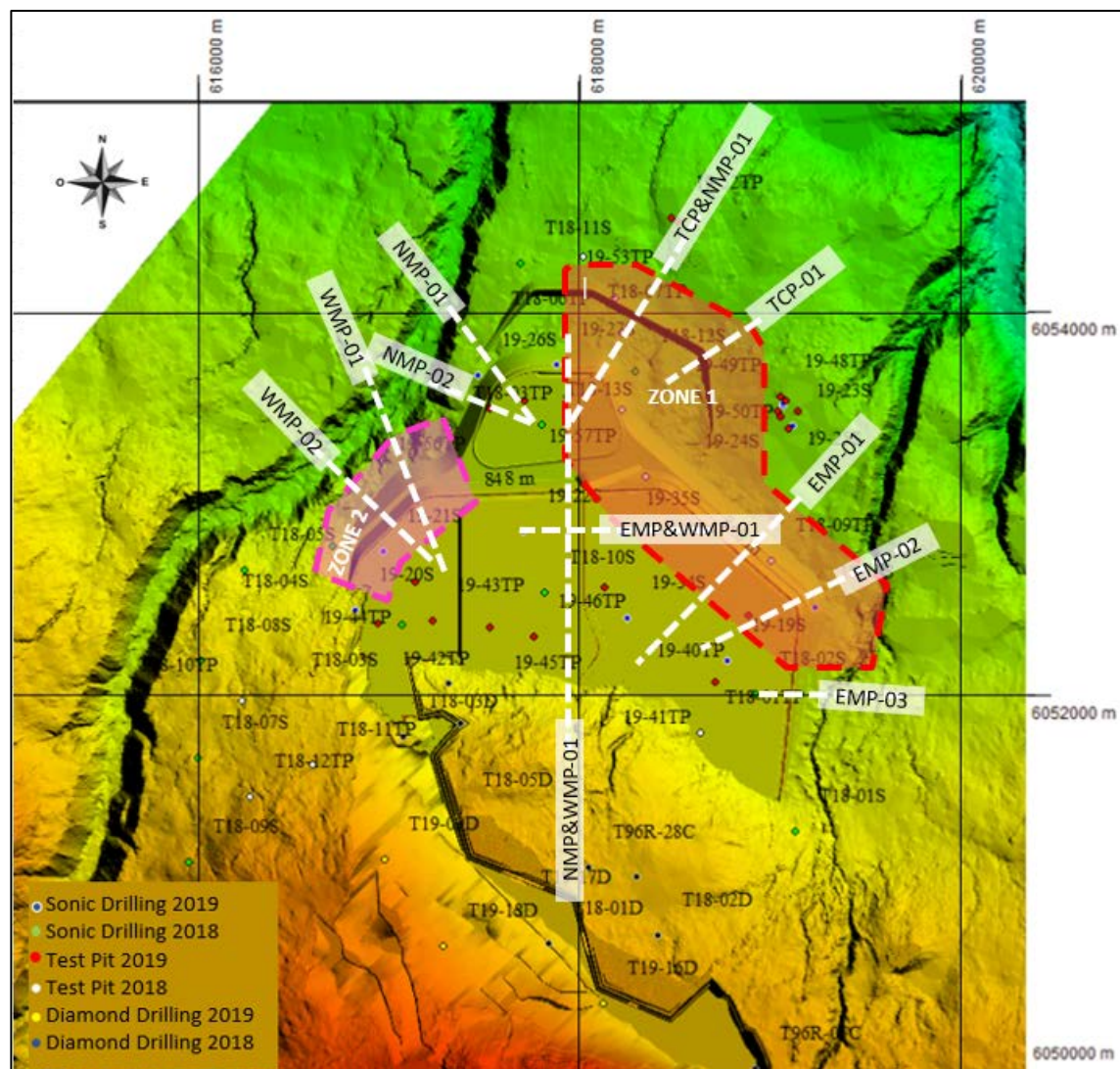


Figure 5-1: Cross-section Locations and Potential Weaker Zones.

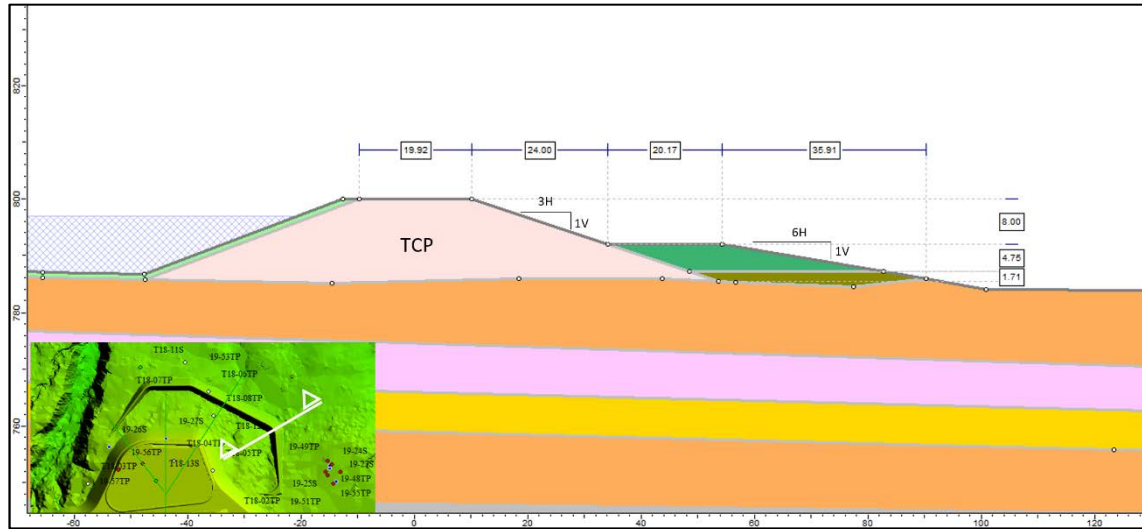


Figure 5-2: TCP-01 Section. Geometric Configuration.

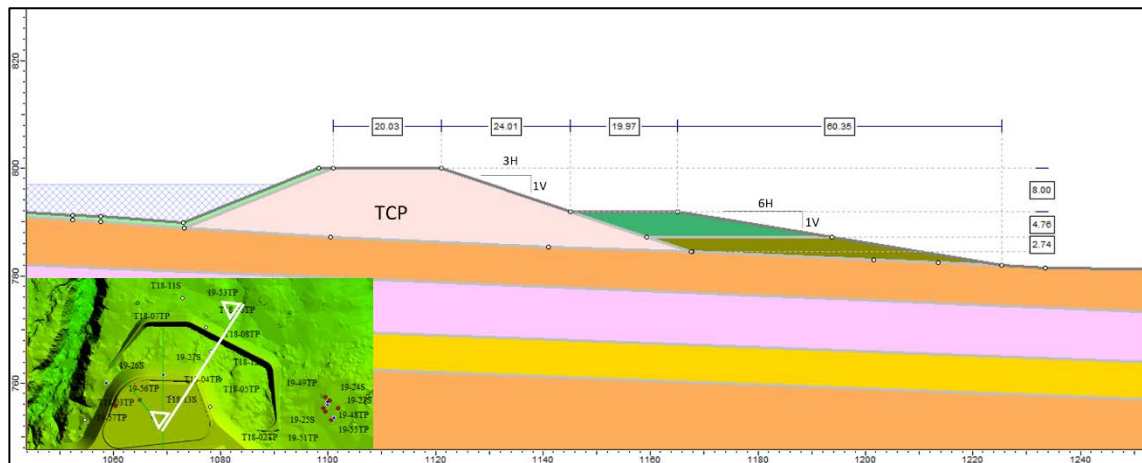


Figure 5-3: TCP&NMP-01 (TCP) Section. Geometric Configuration.

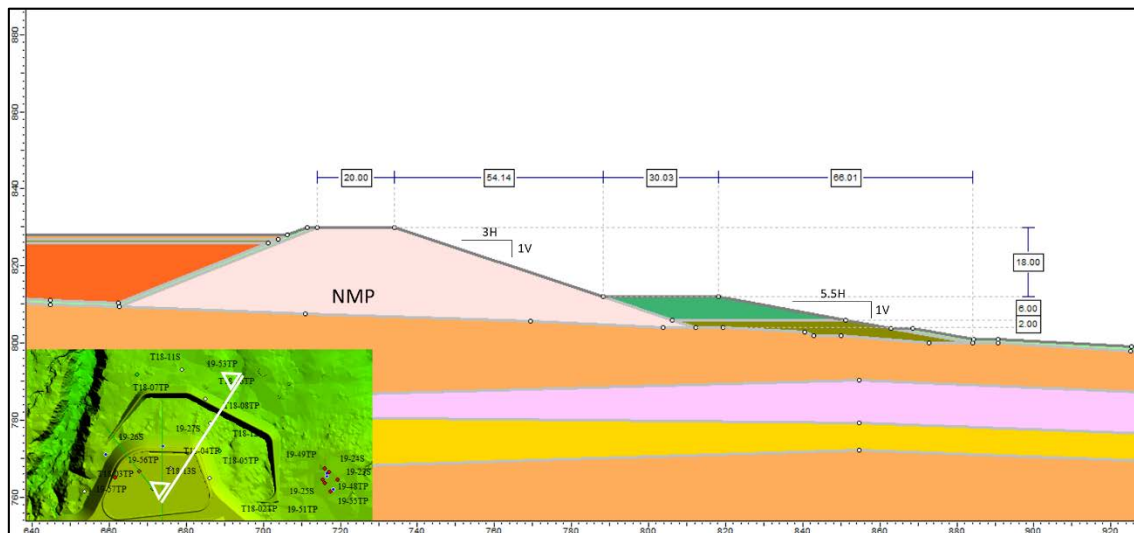


Figure 5-4: TCP&NMP-01 (NMP) Section. Geometric Configuration.

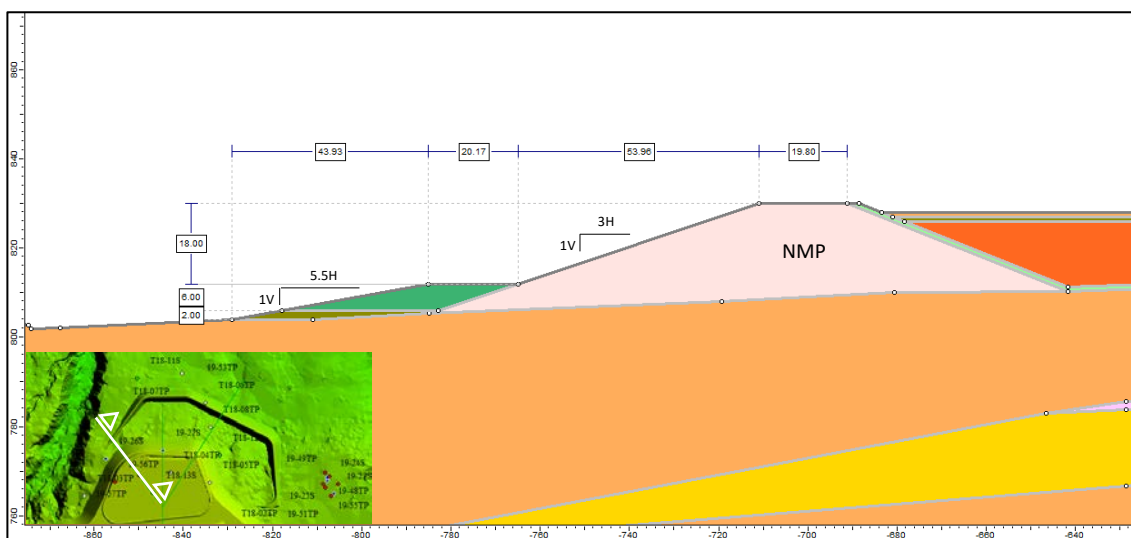


Figure 5-5: NMP-01 Section. Geometric Configuration.

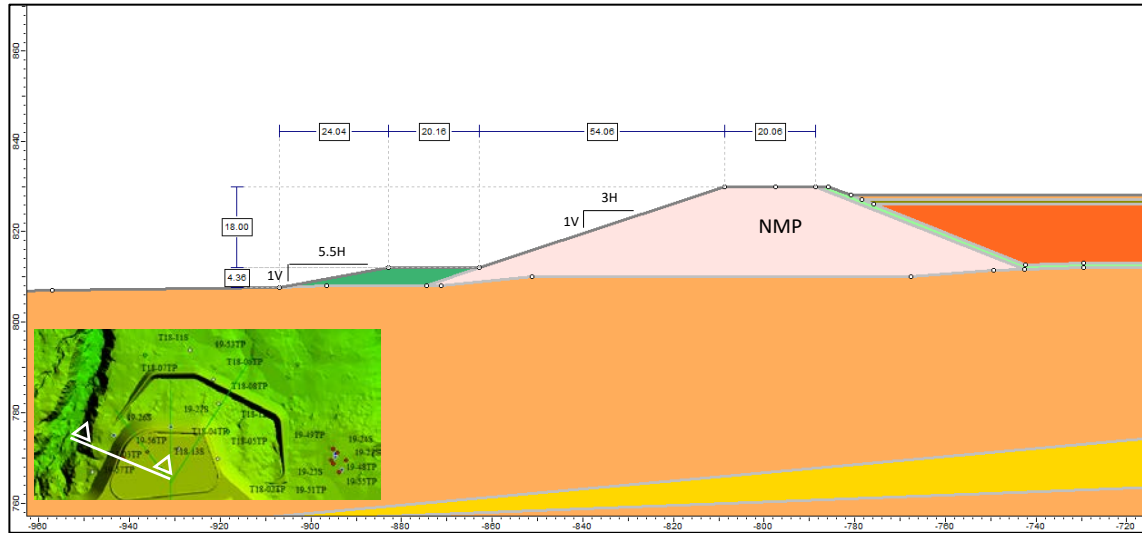


Figure 5-6: NMP-02 Section. Geometric Configuration.

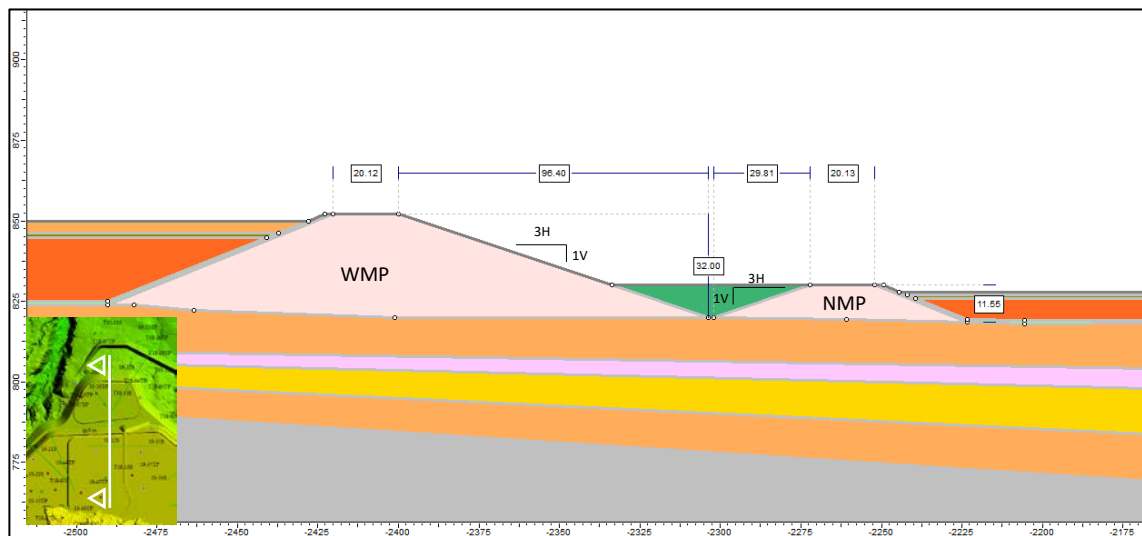


Figure 5-7: NMP&WMP-01 (WMP) Section. Geometric Configuration.

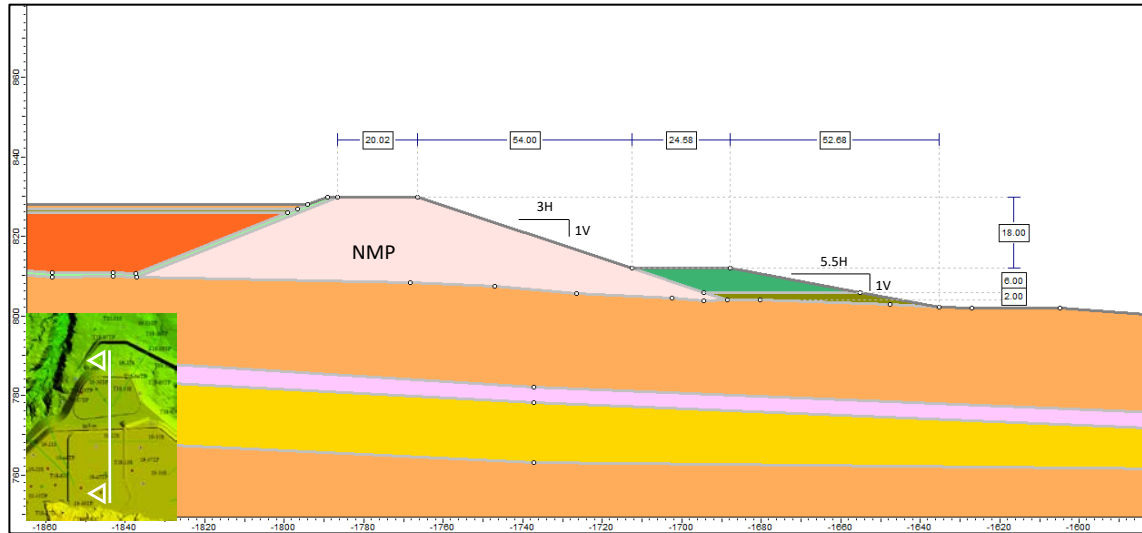


Figure 5-8: NMP&WMP (NMP) Section. Geometric Configuration.

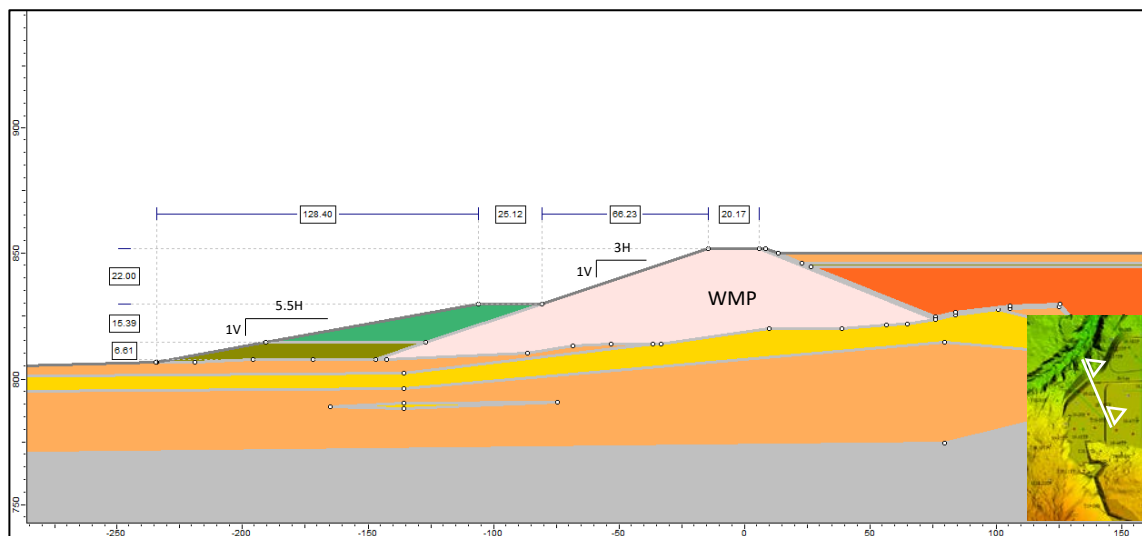


Figure 5-9: WMP-01 Section. Geometric Configuration.

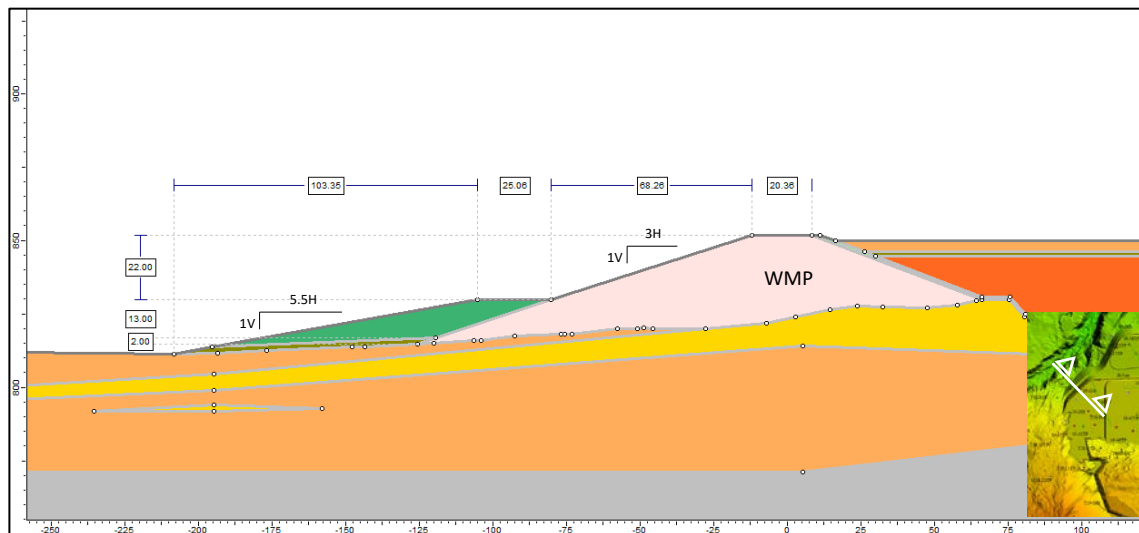


Figure 5-10: WMP-02 Section. Geometric Configuration.

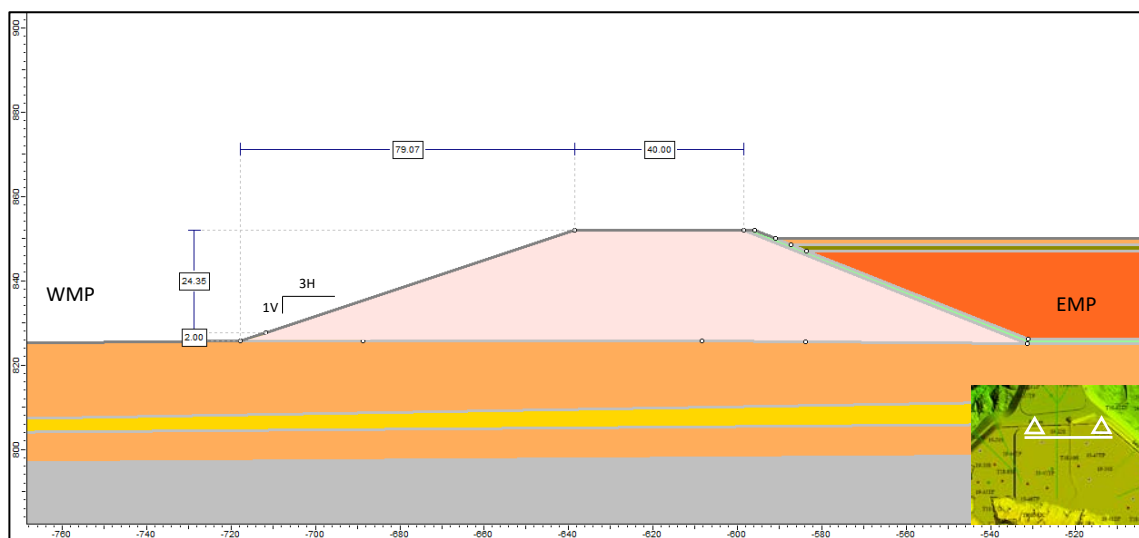


Figure 5-11: EMP&WMP-01 Section. Geometric Configuration.

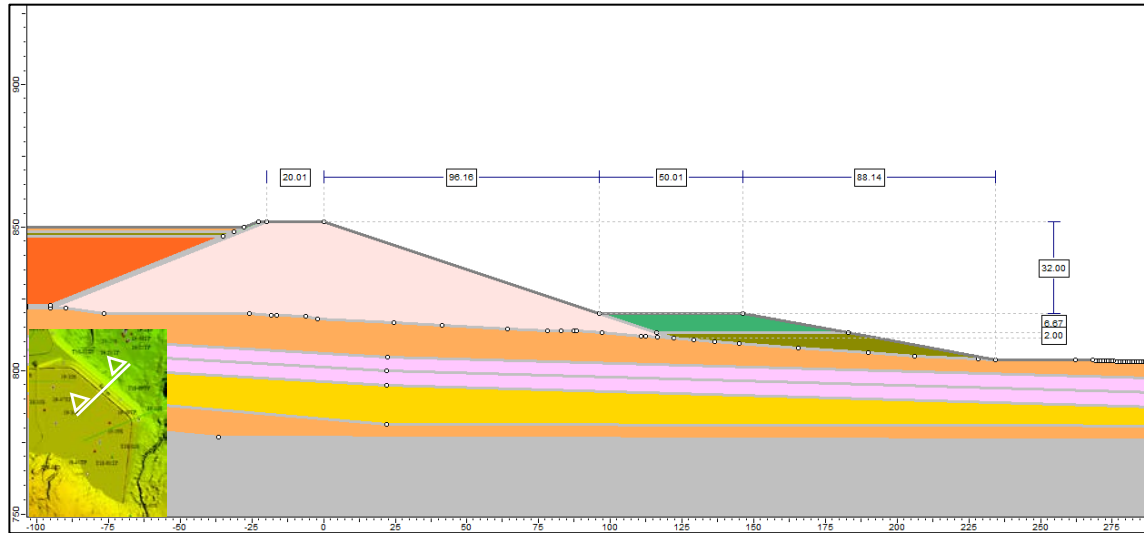


Figure 5-12: EMP-01 Section. Geometric Configuration.

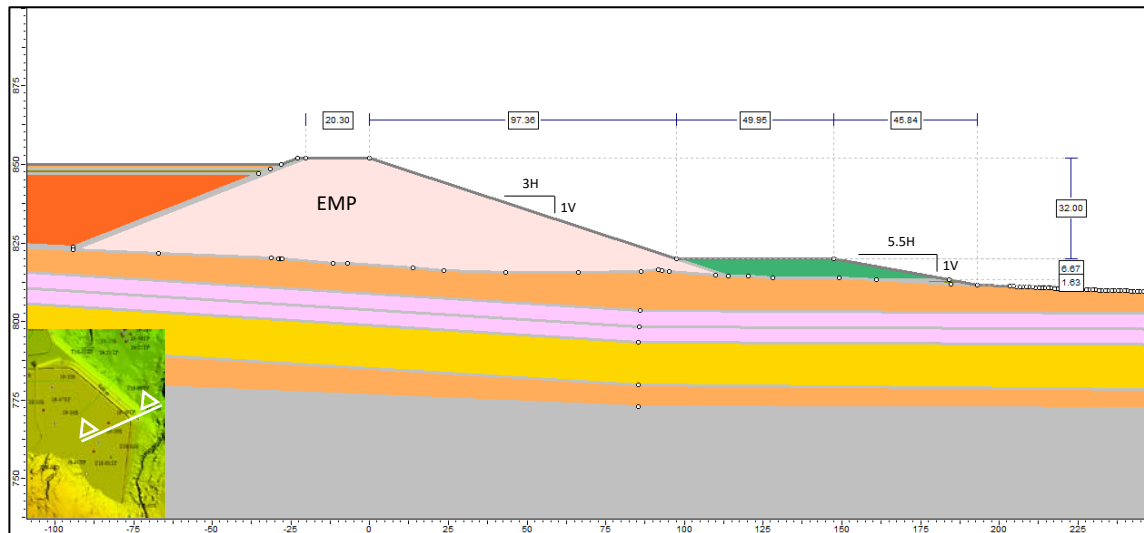


Figure 5-13: EMP-02 Section. Geometric Configuration.

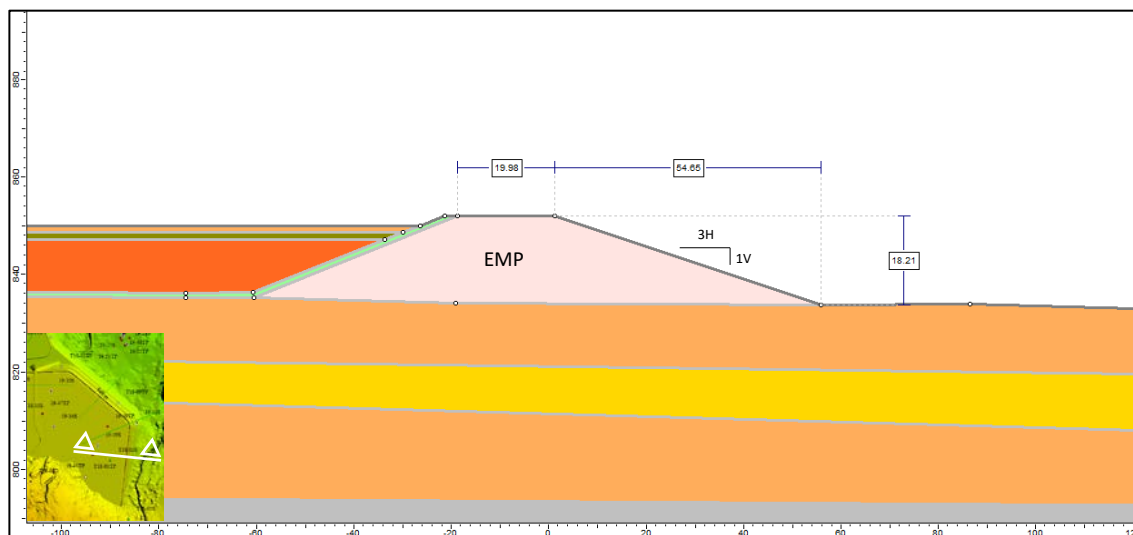


Figure 5-14: EMP-03 Section. Geometric Configuration.

5.7 Piezometric Conditions

A steady state groundwater flow was computed to determine the pore pressure distribution along the Management and Tenas Control Ponds and foundation terrain. Table 5-5 presents the water levels at closure for each facility.

Table 5-5: Pond Water Elevations.

MP	Pond Water Elevation [masl]	Comments
EMP	850	2 m below the dam crest
WMP	850	2 m below the dam crest
NMP	828	2 m below the dam crest
TCP	798	3 m below the dam crest

The groundwater elevation considered is the maximum target operating level. A 1 m contingency storage was considered to the spillway invert.

For the dividing embankment between the EMP and WMP, different piezometric levels were considered for the containment materials (e.g. PAG and NAG) in EMP and the foundation since it is expected that PAG and NAG will be deposited in the WMP before a steady state condition is achieved. Figure 5-15 presents the groundwater conditions assumed for the dividing embankment.

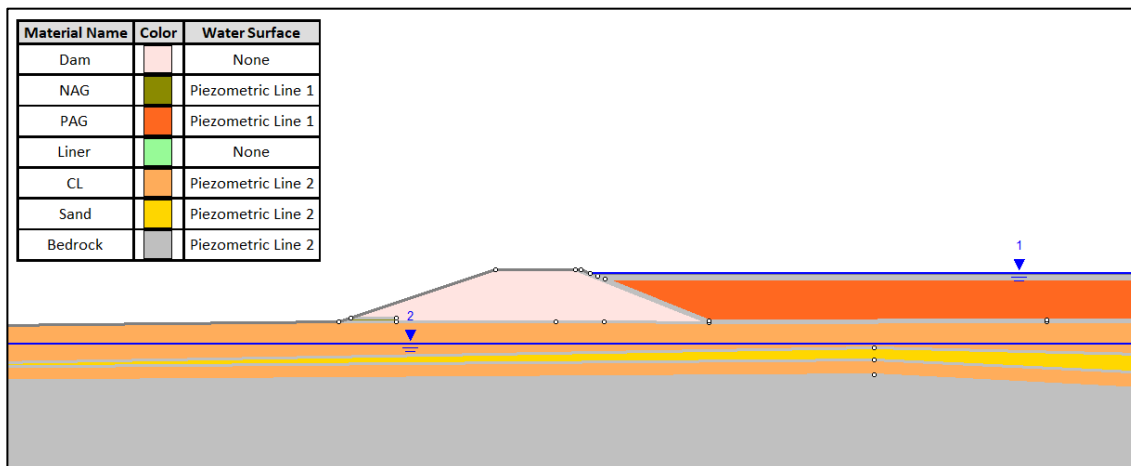


Figure 5-15: Dividing Embankment. Groundwater Condition for full EMP.

5.8 Modelling Scenarios

Every facility was verified for the end of operation condition (i.e. including the NAG and CL layers and maximum water elevation) except the EMP&WMP-01 section, which was modelled considering that only the EMP is full at the end of operation condition for this structure (based on the mining schedule provided by Telkwa and to represent worse state conditions for this structure). In every case, a minimum FoS of 1.6 was required (see Section 5.3), considering a long-term steady state condition.

5.9 Stability Results

The results of the limit equilibrium analyses are presented in Table 5-6. The reported FoS are for the buttress and overall scale failures, based on the foundation conditions assumed in Section 5.5, as per the previous field investigations. The minimum target FoS were achieved in every section. Failure surfaces and the associated FoS for every scenario are included in Appendix B.

Table 5-6: 2D Limit Equilibrium Results.

Section	Static Main Embankment (Buttress)	Pseudo-static Main Embankment (Buttress)
TCP-01	2.23 (1.85)	1.89 (1.71)
TCP&NMP-01 (TCP)	1.88 (1.80)	1.59 (1.55)
TCP&NMP-01 (NMP)	1.77 (1.86)	1.43 (1.62)
NMP-01	1.99 (1.79)	1.77 (1.65)
NMP-02	1.76 (1.69)	1.58 (1.55)
NMP&WMP-01 (WMP)	1.78 (N/A)	1.65 (N/A)
NMP&WMP-01 (NMP)	2.07 (1.80)	1.80 (1.66)
WMP-01	2.44 (1.81)	2.14 (1.71)
WMP-02	2.40 (1.86)	2.10 (1.75)
EMP&WMP-01	1.85 (N/A)	1.70 (N/A)
EMP-01	1.98 (1.69)	1.70 (1.47)
EMP-02	1.81 (1.69)	1.37 (1.57)
EMP-03	1.63 (N/A)	1.50 (N/A)

Values in brackets are for buttress FoS.

6 Sensitivity Assessment

Several stability sensitivity assessments were undertaken for EMP and WMP in sections EMP-01, EMP-02 and WMP-01 to determine the impact of varying foundation and construction conditions in potential failure surfaces and the associated FoS. The foundation conditions for the NMP and TCP facilities were defined by previous field investigations and designs were assessed accordingly.

Sensitivity assessment was made only for EMP and WMP due to the lack of drilling data in those areas. Figure 3-1 shows that the NMP and TCP areas are well covered by the drilling program while some gaps were detected for the EMP and WMP areas. The following scenarios were analyzed:

- Different CH layer thickness;
- Different dam material configurations;
- Different stratigraphy configurations;
- A total breakdown of the liner; and
- Internal seepage considering a chimney drain.

6.1 Different CH Layer Thickness

A sensitivity assessment on the FoS considering the different thickness of the CH layer were carried out for EMP-01 and WMP-01 sections as presented in Figure 6-1 and Figure 6-2.

The results of the limit equilibrium analyses are presented in Table 6-1, comparing the FoS obtained against the base case.

Based on the results, it can be seen that the thicker the CH layer, the lower FoS. The stability results are presented in Appendix B.

Since section EMP-01 was designed for the worst-case (10.0 m thick CH layer) the minimum target FoS were achieved in every scenario. For section WMP-01, the sensitivity analyses presented a minimum FoS higher than the target FoS. This may indicate that the section could be optimized if during further field testing it is confirmed that the CH layer is not as thick as assumed.

Table 6-1: Limit Equilibrium Results. CH Layer Sensitivity Assessment.

Section	Static Main Embankment (Buttress)	Pseudo-static Main Embankment (Buttress)
EMP-01 – 0.0 m CH thickness	2.20 (1.78)	1.94 (1.66)
EMP-01 – 5.0 m CH thickness	2.09 (1.78)	1.81 (1.61)
EMP-01 – 10.0 m CH thickness (Base case)	1.98 (1.69)	1.70 (1.47)
WMP-01 (Base case – no CH)	2.44 (1.81)	2.14 (1.71)
WMP-01 – 10.0 m CH thickness	2.14 (1.84)	1.87 (1.69)

Values in brackets are for buttress scale failures.

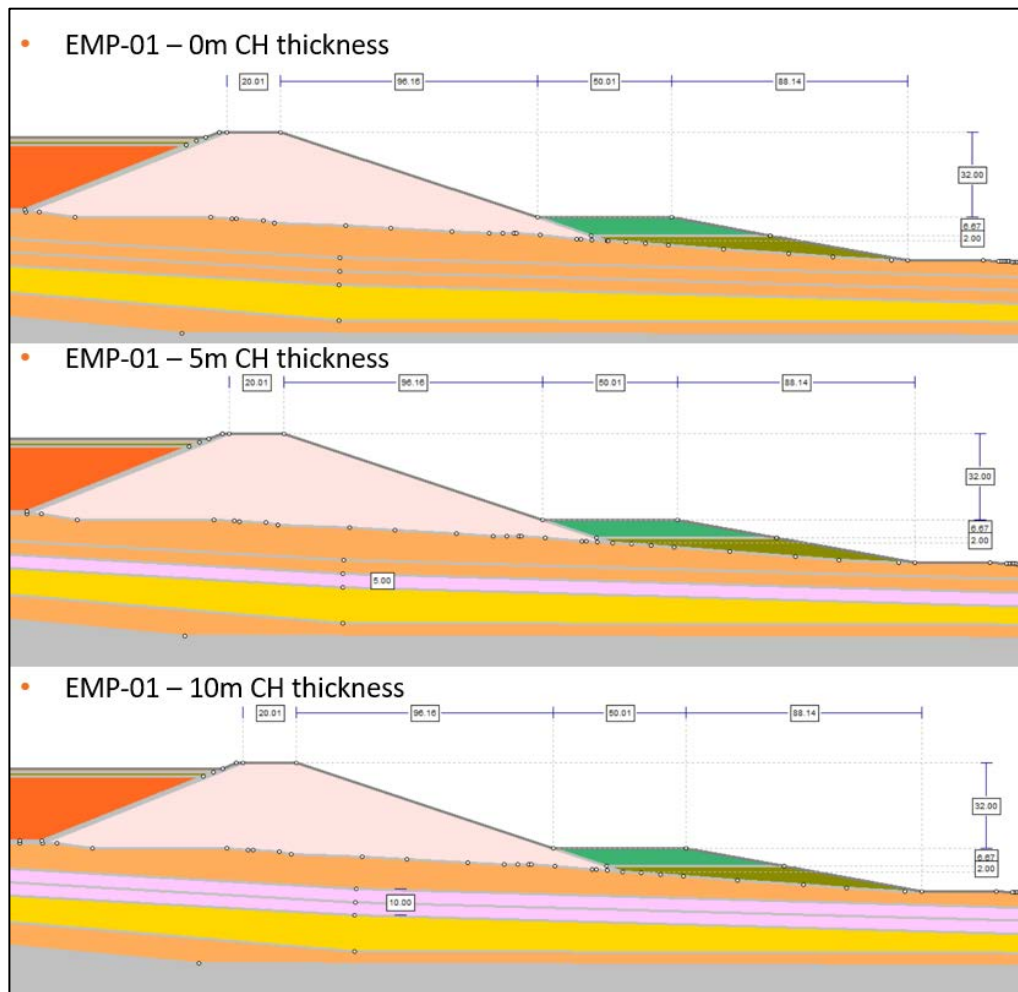


Figure 6-1: EMP-01 Section. Stratigraphy Configurations.

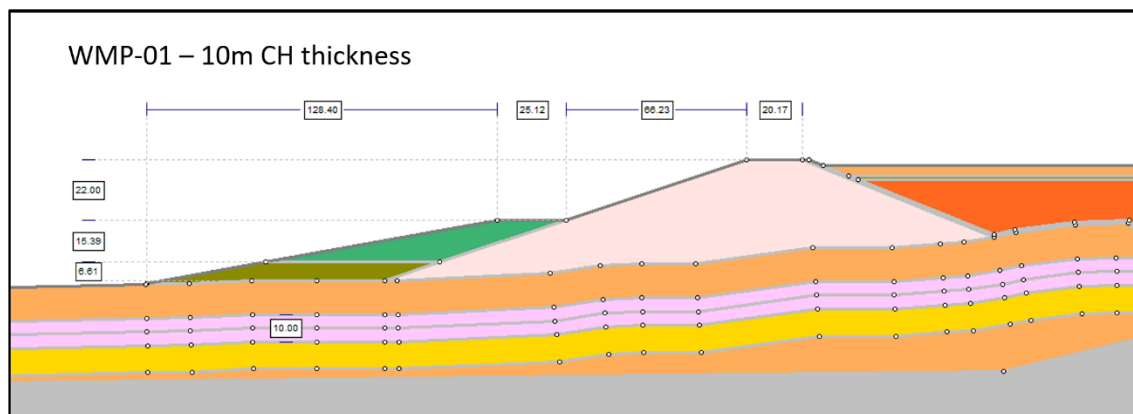


Figure 6-2: WMP-01 Section. Stratigraphy Configuration.

6.2 Different Dam Materials Configurations

A sensitivity assessment considering different materials' configurations for dam construction was carried out for EMP-01 taking into account the uncertainties related to the availability NAG and overburden material volumes. The following alternatives were assessed:

- Alternative 1: buttress constructed entirely with NAG material (Figure 6-3);
- Alternative 2: dam upstream face constructed with overburden, dam body constructed with NAG material and buttress constructed with overburden (Figure 6-4); and
- Alternative 3: dam upstream face constructed with overburden; dam body and buttress constructed with NAG material (Figure 6-5).

The results of the limit equilibrium analyses are presented in Table 6-2, and Appendix B, comparing the FoS obtained against the base case.

Based on the results, it can be seen that the performance of Alternatives 1 and 3 are practically the same as the base case since the piezometric levels are quite similar. Minimum differences are due to the better shear strength of the NAG material.

For Alternative 2, the FoS decreases slightly and local failures are present at the toe of the buttress as a result of the development of pore pressures due to the lower permeability characteristics of the overburden material.

Table 6-2: Limit Equilibrium Results. Different Embankment Design Configurations.

Section	Static Main Embankment (Buttress)	Pseudo-static Main Embankment (Buttress)
EMP-01 – (Base case ²)	1.98 (1.69)	1.70 (1.47)
EMP-01 – Alternative 1	2.00 (1.67)	1.71 (1.46)
EMP-01 – Alternative 2	1.93 (1.57)	1.70 (1.39)
EMP-01 – Alternative 3	2.05 (1.66)	1.78 (1.46)

¹ Values in brackets are for buttress scale failures.

² Shown in Figure 5-12

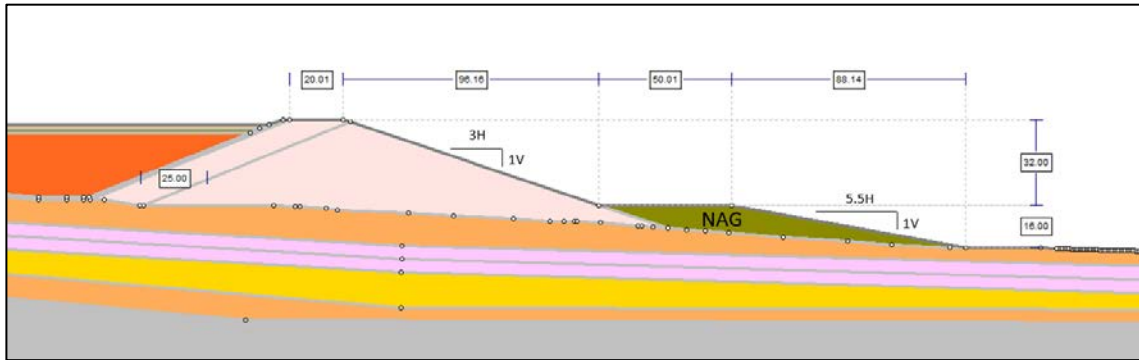


Figure 6-3: EMP-01 – Alternative 1.

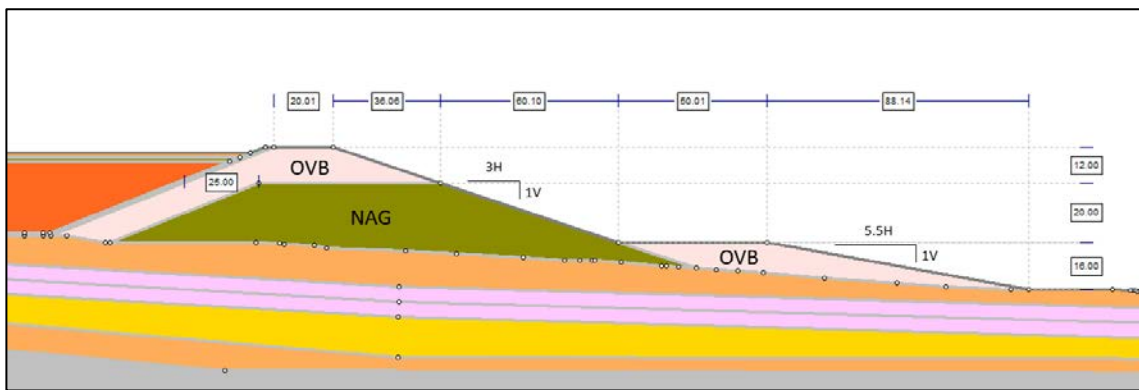


Figure 6-4: EMP-01 – Alternative 2.

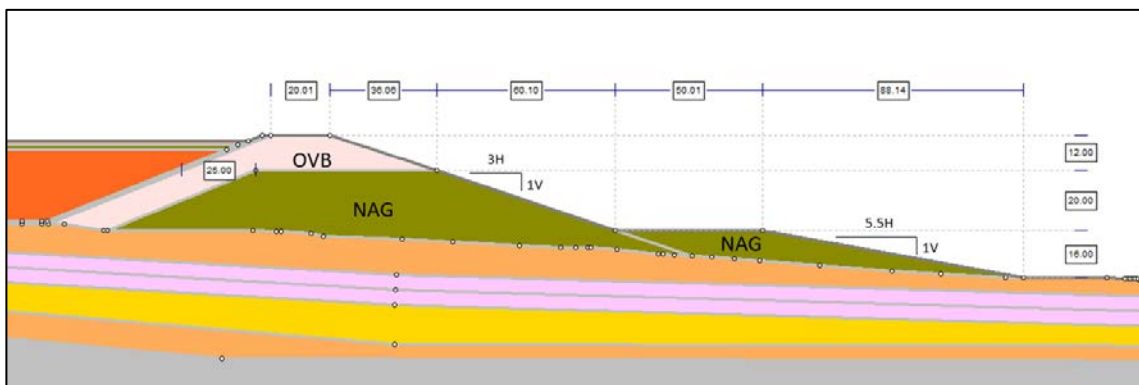


Figure 6-5: EMP-01 – Alternative 3.

6.3 Different Stratigraphy Configurations

A sensitivity assessment of the FoS considering different stratigraphy configurations was carried out for WMP-01 considering the results of the current ongoing field testing program being completed in Q1, 2021. A foundation composed by a 9 m thick CH layer, followed by a layer of 6 m loose silt and a CL layer until bedrock was assessed (Figure 6-6).

The target FoS was achieved as presented in Table 6-3 and Appendix B.

Table 6-3: Limit Equilibrium Results. WMP Stratigraphy Configurations.

Section	Static Main Embankment (Buttress)	Pseudo-static Main Embankment (Buttress)
WMP-01 – (Base case)	2.44 (1.81)	2.14 (1.71)
WMP-01 – (Sensitivity case)	2.21 (1.85)	1.94 (1.65)

Values in brackets are for buttress scale failures.

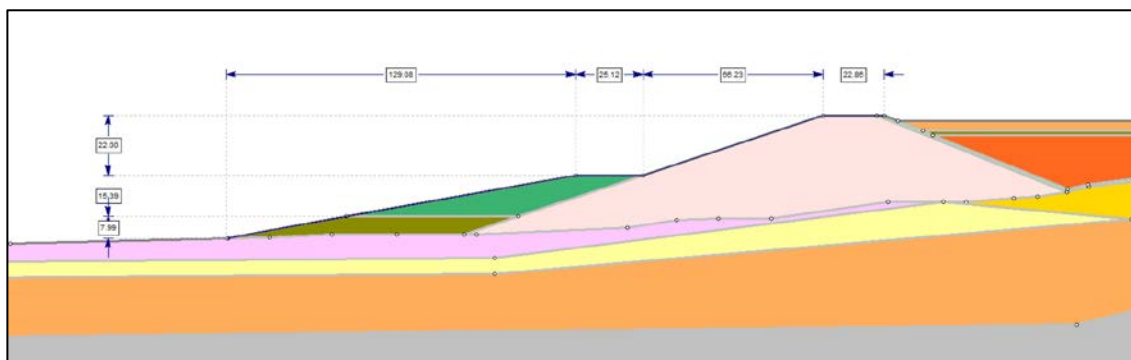


Figure 6-6: WMP-01 – Sensitivity Stratigraphy Configuration.

6.4 Liner Failure

A sensitivity assessment was completed considering a total breakdown of the liner for EMP-01, EMP-02 and WMP-01. Under this scenario, the minimum CDA requirements were considered (see Table 5-1 and Table 5-2).

The results of the analyses are presented in Table 6-4, showing a significant decrease in the FoS. In every case, the minimum target FoS were not achieved for the static condition; however, FoS results were still above 1.3 and imminent failure would be unlikely.

Table 6-4: Limit Equilibrium Results for Liner Failure.

Section	Base Condition		Liner Failure	
	Static Main Embankment (Buttress)	Pseudo-static Main Embankment (Buttress)	Static Main Embankment (Buttress)	Pseudo-static Main Embankment (Buttress)
EMP-01	1.98 (1.69)	1.70 (1.47)	1.65 (1.46)	1.44 (1.28)
EMP-02	1.81 (1.69)	1.37 (1.57)	1.43 (1.29)	1.26 (1.19)
WMP-01	2.44 (1.81)	2.14 (1.71)	1.77 (1.47)	1.58 (1.42)

Values in brackets are for buttress scale failures.

Based on these results, the performance of a chimney drain under a potential total breakdown of the liner was also assessed. The chimney drain was collocated at the center of the embankment considering a 5.0 m width and a maximum elevation equal to the maximum water elevation at the ponds (Figure 6-7).

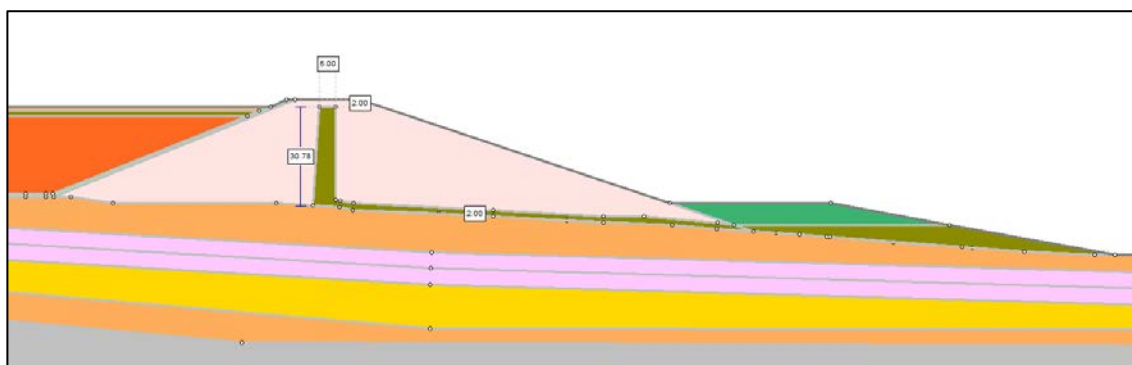


Figure 6-7: EMP-01. Chimney Drain Configuration.

The results of the limit equilibrium analyses showed that, under a total failure of the liner, the chimney drain does improve the stability of the main dam sufficiently to meet the CDA minimum requirements. Table 6-5 presents the FoS obtained, comparing the results obtained with and without the chimney drain. Figure 6-8 and Figure 6-9 show the piezometric level and failures surfaces for EMP-01 with and without the chimney drain. Failure surfaces and the associated FoS for the other sections are included in Appendix B.

It is noted that the buttress does not quite meet stability requirement due to pore pressures developed at the toe of the buttress. In the scenario that a chimney drain existed, and liner failure occurred, the buttress could be enhanced to meet long term stability requirements (i.e. FoS > 1.5) by modifying the existing buttress slope from 5.5H:1V to a slope of 6H:1V.

Table 6-5: Limit Equilibrium Results. Liner Failure and Chimney Drain.

Section	Liner Failure		Chimney Drain	
	Static Main Embankment (Buttress)	Pseudo-static Main Embankment (Buttress)	Static Main Embankment (Buttress)	Pseudo-static Main Embankment (Buttress)
EMP-01	1.65 (1.46)	1.44 (1.28)	1.71 (1.48)	1.48 (1.29)
EMP-02	1.43 (1.29)	1.26 (1.19)	1.52 (1.42)	1.32 (1.30)
WMP-01	1.77 (1.47)	1.58 (1.42)	1.93 (1.49)	1.72 (1.41)

Values in brackets are for buttress scale failures.

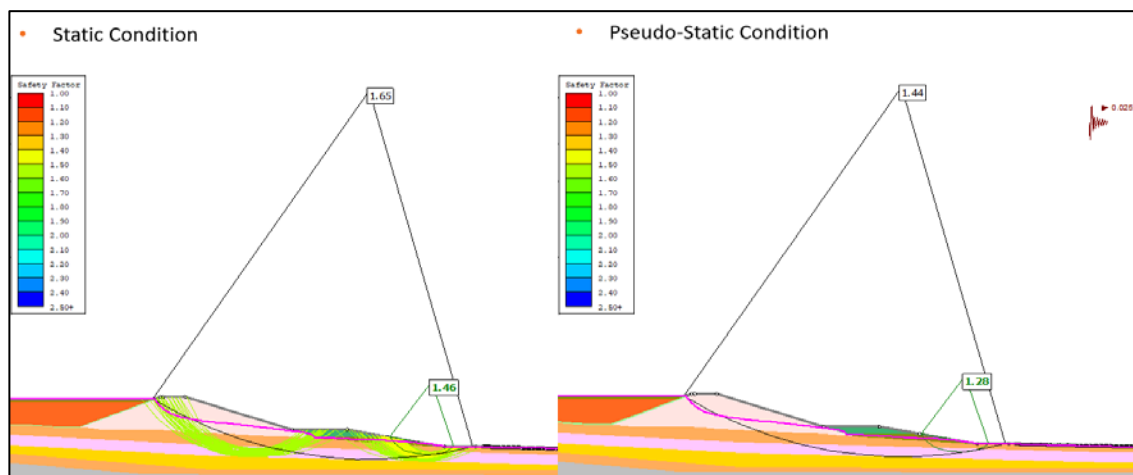


Figure 6-8: EMP-01. Liner Failure.

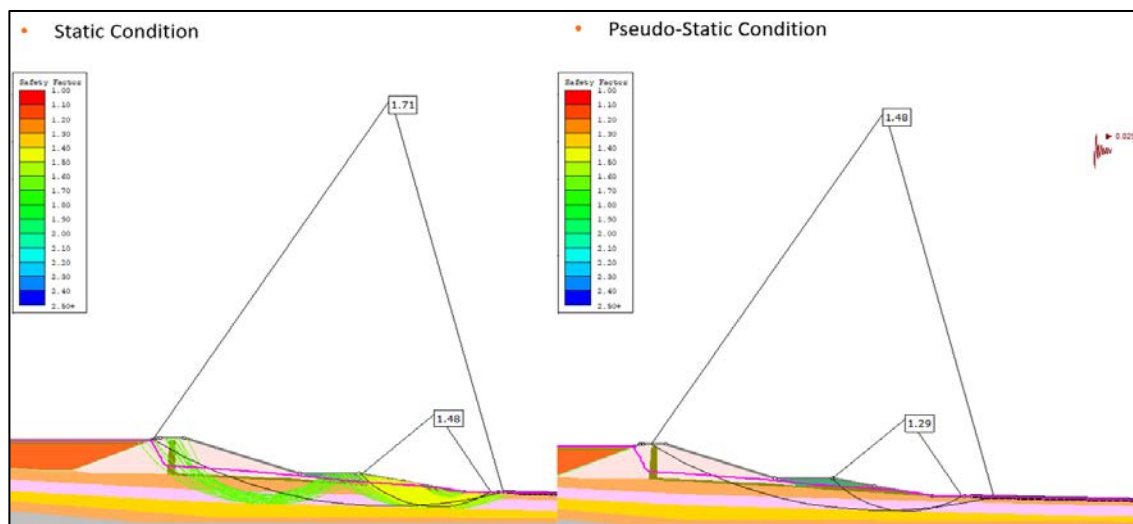


Figure 6-9: EMP-01. Liner Failure with Chimney Drain.

7 Technical Specifications

In this chapter, technical specifications for embankment, buttress, drainage material, construction and quality controls are provided at an early design stage level. Technical specifications should be further updated during the detailed design engineering.

7.1 Material Characteristics

The type of material used for the embankments depends on the parameters adopted in the design. Key aspects to consider are:

- Geomechanical parameters; and
- Permeability.

The material characteristics and its adequacy to quality standards must be proven by field and laboratory testing.

Field obtained geotechnical parameters must be equal to or better than those adopted in the design. A correct characterization of the strength parameters (effective cohesion and effective internal friction angle) and parameters associated with the condition of peak strength (degree of compaction, optimum moisture content, 95% Proctor modified density) shall be required. Additional characteristic parameters shall be evaluated during the construction works to check compliance with minimum material requirements during construction.

7.1.1 Adopted Parameters

The embankments are composed by a clayey to silty overburden material present on site and NAG rock. The lining system is composed by a 0.3 m thick low permeability soil-bentonite (SRK, 2021a). SRK carried out a geotechnical campaign on site where samples were extracted, and laboratory tests were performed. Test results, complemented with empirical correlations and SRK's experience in projects of similar characteristics, were used to determine the geotechnical and hydraulic parameters used for the embankment design. Geotechnical and hydraulic parameters are summarized in Table 7-1.

Table 7-1: Geotechnical and Hydraulic Parameters of the Embankment Materials.

Material	γ [kN/m ³]	c' [kPa]	ϕ'_{cv} [°]	k_v [m/s]	k_h [m/s]
Embankment dam	20.5	0	30	1×10^{-07}	1×10^{-07}
Embankment buttress	20.5	0	30	1×10^{-07}	1×10^{-07}
NAG rock	23.0	0	35	1×10^{-05}	1×10^{-05}
Soil-bentonite liner	18.0	0	20	1×10^{-10}	1×10^{-10}

Where:

- γ : unit weight;
- c' : effective cohesion;
- ϕ_{cv}' : effective internal friction angle;
- k_v : vertical permeability coefficient; and
- k_h : horizontal permeability coefficient.

7.1.2 On-site Soil

The selected area for the MP is located over and at the north and north-east of the projected Tenas open pit. The overburden material is composed principally of a very stiff to hard lean clay (CL), with low to medium plasticity, with intervals of high plasticity clay (CH), clayey silty sand (SC or SM) or gravel (GC or GM) at various depths (with an overburden total thickness equal or higher than 30 m in most of the extension).

This layer is mainly a silty/clayey soil of low plasticity that can be used as filling material. The soil must be free from organic material and with plasticity index lower than 20 ($PI < 20$) and a liquid limit lower than 40 ($LL < 40$).

7.1.3 Selected Soil

Any fine or granular material can be used for the embankment construction as long as it is insoluble and inorganic.

If the soil natural moisture content is greater than the optimum moisture, it must be reduced to the optimum level required for adequate compaction of the material. To this end, material may have to be disposed in drying benches before construction.

7.1.4 Tests to be Made on the Material used in the Embankment

Material parameters and target density shall be determined by the Telkwa Coal appointed group at the beginning of extraction and tested randomly from haul trucks and on lifts. During embankment construction, in situ density tests shall be required to approve the works. An appointed identity (Inspector) shall be responsible for certifying laboratory test results and may indicate additional tests as required. In case the material is a mixture of overburden and NAG rock, tests for different mixes shall be necessary to determine the optimum content of fine soil.

Routine Tests

These shall be performed to establish soil type and in-situ condition: natural moisture content, Atterberg limits, unit weight and visual-manual inspection according to standards ASTM D2216, ASTM D2488 and ASTM D4318.

Particle size distribution curves shall be obtained according to standards ASTM D0422 (granular material) and ASTM D2487 (fine material). These shall be obtained for samples of overburden, NAG waste rock and mixtures of both.

Proctor Compaction Tests

Modified Proctor testing according to standard ASTM D1557 shall be performed on overburden and overburden-NAG waste rock mixture samples.

Permeability Tests

Permeability tests shall be performed according to standards ASTM D2434 (granular material) and ASTM D5084 (fine material).

7.2 Quality Controls

7.2.1 Compaction

The in-situ soil density can be measured by two methods:

- Sand-cone method (ASTM D1556): an excavation is made, and the weight of the extracted material is determined. The excavation volume is computed by filling the hole with sand using a calibrated recipient; and
- Nuclear methods (ASTM D2922, ASTM D3017): based on the interaction between gamma radiation and existing electrons in the soil particles. The difference between energy emitted and received is measured, which is proportional to the soil density.

A systematic on-site control program of the degree of compaction must be elaborated and implemented to determine the density and moisture content of the compacted material and compare the results with the specified limits.

The degree of compaction grade shall be verified in each lift. Control of the fill material shall be carried out continuously by visual inspection of the operations (unloading, spreading, humidification, mixing and compaction).

The number of control tests is presented in Table 7-2. If needed, the Inspector may determine additional control points. No new lift shall be spread until the degree of compaction (moisture and density) has been tested and accepted. Each lift shall be accepted if all density measurements are greater than 90% MP (Modified Proctor). The top lift shall be accepted if all density measurements are greater than 94% MP.

The water content during compaction must be in the range $w_{opt} \pm 3\%$, where w_{opt} is the optimum moisture content obtained by Modified Proctor testing.

If the lift is rejected, it shall be scarified, its moisture adjusted, and it shall be re-compacted until acceptable results are obtained.

7.2.2 Permeability Tests

The soil permeability governs the hydraulic performance of the embankment. Permeability shall be verified by the use of test embankments either prior to or during construction, by Guelph infiltrometer or in situ tests (ASTM D5126).

Permeability results for the soil bentonite liner must be less than or equal to $k = 1 \cdot 10^{-10} \text{ m/s}$ and permeability results for the NAG rock must be higher or equal to $k = 1 \cdot 10^{-4} \text{ m/s}$, in accordance with hypotheses made during the geotechnical design of the embankment.

7.2.3 Topographic Survey

A topographic survey along the site will be performed in the following milestones:

- Before the start of work;
- After removal of the top and/ or organic soil layer;
- After surface compaction of each lift;
- After the filling of each dam embankment; and
- Annually after embankment construction.

The survey shall be considered concluded in each sector when two consecutive readings have differences less than 50 mm.

The levelling shall be checked in each lift. Variations in the final lift greater than 50 mm shall not be allowed (30 mm for roads).

7.2.4 Technical Specifications for Quality Control

Table 7-2 summarizes the works and tests to be made for quality control of the embankment.

Table 7-2: Technical Specifications for Quality Control of the Embankments.

Control		Test	Quantity
Materials	At origin	1 Modified Proctor	The greater of one test per 1,000 m ³ of material or 1 per day
		1 PSD 1 Atterberg limits	The greater of one test per 5,000 m ³ of material or 1 every three days
	At destination	Examine batches unloaded from the trucks, discarding those that at first sight contain remains of topsoil and/ or organic matter. Mark batches that present any anomaly in appearance. Take samples from the marked batches to repeat the tests that were performed at the place or origin.	
Extension		Check the thickness and width of the lifts	
Compaction	Embankment materials	5 moisture content 5 in-situ density	The greater of one test per 5,000 m ³ of material or 1 per day
	Soil-bentonite liner	1 moisture content 1 in-situ density	Every 100 linear meters
Geometry		Surveying bases, every 50 m plus singular points. Width and transversal slopes at the same points.	

7.3 Execution of the Management Ponds

7.3.1 General

During the construction procedure (cleaning, excavation, embankment construction and operations), additional information related to foundation characteristics and slope materials can be obtained. This information can contradict the hypotheses previously assumed.

Weather and groundwater conditions during construction can significantly affect the water content of the embankment material or create hydraulic conditions different from those assumed, which could in turn cause changes in the embankment design. For this reason, the project must be continuously evaluated to make sure that the conditions found during construction are compatible with the final design.

The development of an adequate geotechnical campaign, technical specifications for construction and quality controls allow to mitigate the risk of requiring a re-design during construction. The definition of ranges for acceptable parameters ensures flexibility in design tolerances and enables foreseeing actions on possible variabilities in material quality. The design of adequate constructive methodologies and its correct control can help prevent contingent quality defects due to negligence and the additional associated costs.

7.3.2 Equipment

Haul trucks, water trucks, excavators, graders and vibratory and padfoot rollers shall be used. The most suitable equipment will be selected by Telkwa Coal based on the soil characteristics and construction requirements.

7.3.3 Previous Works

Surface soils, trunks, roots and topsoil materials shall be removed until soils with no appreciable content of organic matter is found.

The surface must be clean, clear and shall be compacted by successive passes of vibratory and/ or padfoot equipment. The type of equipment, number of passes and circulation speed will be determined by the constructor to guarantee the recommended compaction requirements are achieved.

Where subsidence is detected, it shall be necessary to fill the holes with suitable fill material, level the surface and continue with the compaction.

The terrain slope must be uniform to ensure adequate rainwater runoff.

7.3.4 Placement and Compaction

In this section, preliminary placement and compaction specifications are provided. This will be furtherly studied and updated during the detailed design engineering.

Previous Studies

Compaction equipment and use shall be determined from studies in test fields. From these studies the optimum combination of type of equipment, lift thickness, speed and number of passes shall be specified.

Lifts

The embankments are divided in three zones with different placement and compaction requirements. Each zone is executed by lifts of uniform thickness, parallel to the design grade, covering the total width of the cross section and in lengths compatible with the spreading, mixing and compaction methods. A typical place and compaction procedure during construction operations is shown in Figure 7-1.

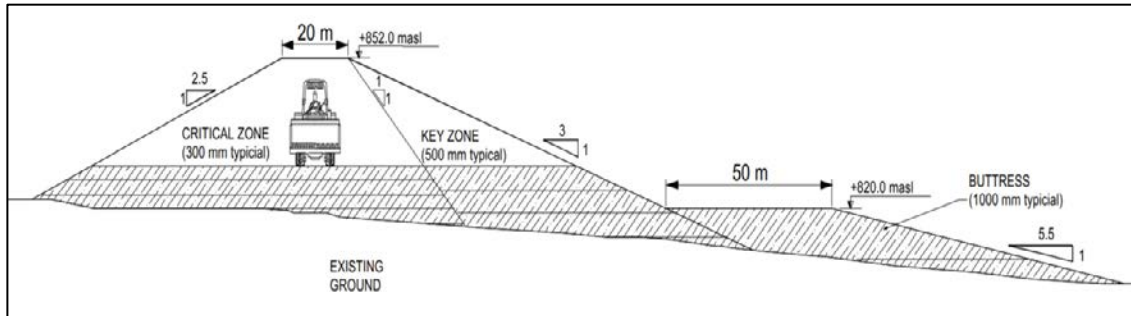


Figure 7-1: Typical Compaction Procedure. EMP-01 Section (not at scale).

Recommendations assuming overburden materials:

- Critical zone: lifts of 300 ± 100 mm thick and density $> 94\%$ MP;
- Key zone: lifts of 500 ± 125 mm thick and density $> 90\%$ MP; and
- Buttress zone: lifts of 1000 ± 200 mm thick .

In the detailed design report, different materials and lift thicknesses will be considered and a range of acceptable values provided.

Water content during compaction must be in the range $w_{opt} \pm 3\%$, where w_{opt} is the optimum moisture content obtained by Modified Proctor test.

The lift thickness shall be adequate to ensure required degree of compaction is achieved.

Each lift shall be mechanically scarified to ensure breaking of soil clods. Clods that cannot be broken shall be removed before compaction.

The whole surface must be treated with enough passes to achieve a uniform degree of compaction. The compaction procedure shall start from the boundaries of the embankment to the center with parallel passes that overlap at least half the width of the compaction equipment.

Placement of the next fill shall be carried out upon authorization by the Inspector, i.e. after the compaction and state of the previous layer and the material quality were verified by laboratory tests.

Wetting

If addition of water is needed to achieve the compaction degree, wetting shall be done at the extraction point, or in the lift, ensuring correct soil mixture, with uniform moisture content throughout the thickness.

Drying

If the water content is greater than the optimum, the soil shall be removed and left to dry or it shall be mixed with drier material or with appropriate substances.

Levelling

Levelling shall be checked for each layer. For the topmost layers within the critical zone, variations greater than 100 mm shall not be admitted.

Minimum transversal slopes must be maintained to ensure rainwater runoff during construction.

The construction shall start from the lowest points of the transversal profile and shall continue by overlapping lifts until reaching the full width of the embankment.

7.3.5 Measures to Minimize the Environmental Impact

In general terms, Telkwa Coal must ensure the maintenance of air, water and soil quality in the zone of operation and its environment during and after the execution of the works.

During construction, preventive measures to minimize the environmental impact are:

- Distribute the soil masses to haul in a reasonable way, ensuring maximum possible compensation, disposing the remnant material conveniently in cut zones or excavations in order to minimize soil haulage;
- Minimize the opening of temporal access roads to the working area and/or the borrow site;
- Use blasting techniques only if strictly necessary;
- Ensure sound technical maintenance of the power park to reduce gas leaks, fuel spills and annoying noises;
- Avoid or minimize the creation of dust clouds during embankment construction by means of irrigation; and
- Need to have a water management plan for construction of the embankments and/or buttresses.

7.3.6 Security Specifications

Risks

Selected risks related with safety during embankment construction should be defined by the constructor and responsible of health and safety before the start of construction works. As a guidance, selected risk examples are presented in Table 7-3.

Table 7-3: Selected Risks Related with Working Conditions.

Risk	Working Condition Generating Risk
Burial and flattening	Earth sliding and falling on people located at the toe of the embankment
Machinery overturning	Soil overturning: vehicles near edge of slopes or embankments
Falls at different levels	Personnel close to the edge of the slopes or embankments: supervising, executing works near the edge of the slopes and/ or access roads

Preventive Actions During Construction

Personnel access to the working area must be limited during dumping and spreading of fill material. Authorized personnel must be out of reach of the vehicles and machinery used during the execution of these works. Absence of any other personnel in the zone must be ensured.

During sampling and testing, the working area must be correctly marked and delimited with containment fences, stoppers or similar systems to avoid the access of machinery and vehicles.

As the embankment execution progresses, the slope boundaries must be correctly identified and marked considering the criteria established in Table 7-4.

Table 7-4: Indicative Criteria.

Embankment Height [m]	Delimitation/ Protection	
	Concerning Works	Concerning Other Personnel
H < 2.0 m	Delimitation of slope boundaries with reflective vertical rods (PVC, reflective pegs or similar) spaced 2.0 to 3.0 m approximately; stopper or similar mesh; freestanding fences	Work fencing
H > 2.0 m	Delimitation of slope boundaries with stopper mesh or similar 1.0 m from the edge; freestanding fences	Work fencing

8 Monitoring and Maintenance Requirements

Maintenance, surveillance, and monitoring include all activities carried out to provide continued proper operation of all the facilities and ancillary structures, or to adjust infrastructure to ensure operation in conformance with performance objectives.

This section presents tentative maintenance, surveillance, and monitoring schedules at a feasibility design level. These requirements should be updated and included in an Operation, Maintenance and Surveillance (OMS) Manual as part of the detailed engineering design.

A trigger action response plan (TARP) should be included in the OMS Manual to be developed during the detailed engineering design stage to manage the critical monitoring parameters. The TARP will provide trigger levels, potential consequences of a failure, and will define the actions that need to be taken in case a trigger is reached.

8.1 Maintenance

8.1.1 Preventative Maintenance

Preventative maintenance is the planned, recurring maintenance activities conducted at fixed or approximate frequency and not typically arising from results of surveillance activities. The following preventative maintenance activities are required for the MP:

- Service and repair of pumps.
- Repair and replacement of pipelines.
- Calibration and repair of instrument monitoring.
- Service of electrical system and supply.

The frequency and procedures for the above maintenance must be according to manufacture instructions. The relevant procedural documents from the manufacturer should be referenced in the updated version of this document.

8.1.2 Corrective Maintenance

Corrective maintenance is the repair of MP components conducted in response to results of surveillance activities or incidents. Corrective maintenance is conducted on an as required basis. The corrective maintenance activities required for the MP are presented in Table 8-1.

Table 8-1: Corrective Maintenance Activities.

MP Components	Activity
Embankments crests and outer slopes	Grade over minor cracks
	Mark, investigate and repair large cracks on slopes or crest. Repair should only commence once investigation into cause has been completed.
	Repair erosion gullies on slope and along crest.
	Maintain the design downstream angle of the perimeter wall.
Spillway	Remove obstructions and vegetation (trees, shrubs, and weeds) in the inlet and chute.
	Maintain design dimensions of spillway and erosion protection measures
Instrumentation	Repair or replace damaged instruments and associated components, including labels, damage, functionality.

8.2 Surveillance

8.2.1 Inspection and Site Observation

Inspections and site observations are used to identify and track visible changes in the condition of the ponds. Their objective is to identify any conditions that might indicate changes in the facility performance and therefore require maintenance, investigation, documentation and reporting, or emergency responses.

Inspections are to be conducted by personnel with appropriate training and are to target specific activities. They are required on a routine basis and after unusual or extreme events.

Site observation is conducted by all personnel working on or adjacent to a pond as part of their daily activities, including those who do not have specialized training and competencies related to tailings management (e.g., equipment operators, security personnel). All personnel working on or adjacent to the MP need to be provided with training to understand the changes that, if observed, need to be documented and reported.

If an issue is observed the following actions should be taken:

- Document all issues in accordance with Section 9.2.
- Provide maintenance as appropriate in accordance with Table 8-1.
- Engage the Engineer of Record (EoR) for any unfamiliar or significant issues under the discretion of the MP manager.
- Increase inspection frequency according to the magnitude of the issue under the discretion of the MP manager or geotechnical engineer.
- Record safety hazards (e.g. cracks in roads and embankments, spillages of any nature, etc.), mark and/or block access and implement the appropriate HSE procedures.

8.2.2 Routine Inspection

The inspection items and their frequencies are presented in Table 8-2. The frequency of inspections should be increased for items where issues are detected. The frequency would depend on the magnitude of the issue under the discretion of the MP manager or geotechnical engineer.

Table 8-2: Inspection Activities and Frequencies.

MP Component	Inspection Item / Potential Issues	Frequency
Embankment crests and outer slopes	General MP inspection from vehicle	daily
	Available operational and pond freeboard along the perimeter of the MP	daily
	Appearance of new movement, cracks, erosion, seepage, bulging, deformations.	weekly
	Status of existing cracks, erosion, seepage, or deformations.	daily
Spillway	Debris and significant vegetation in spillway inlet and chute	weekly
	Erosion protection condition	monthly
	Inlet and chute cracks, bulging, deformations	monthly
Instrumentation	Condition of monitoring instrumentation and associated components, including labels, damage, functionality	monthly

8.2.3 Event Driven Inspection

Event driven inspections are conducted during (if safe and practical to do so) and after unusual or extreme events that may impact the facilities (e.g., heavy rainfall, windstorms, rapid snow melt, seismic events, exceedance of minimum freeboard). Inspections should also be conducted in response to significant changes to normal operations, nearby construction activity, or other unusual events.

If a major event occurs, all activities listed in Table 8-2 should be conducted. A qualified geotechnical engineer should be engaged to inspect the facilities immediately.

8.3 Monitoring

8.3.1 Instrument Monitoring

Instrument monitoring provides information on parameters or characteristics that cannot be detected through site observation or inspections, cannot be observed with sufficient precision and accuracy, or needs to be monitored at high frequency or continuously.

A tentative location for piezometers was selected based on the critical section of each MP. Proposed piezometers will be located at the dam body and the buttress, as presented in Figure 8-1 and Figure 8-2.

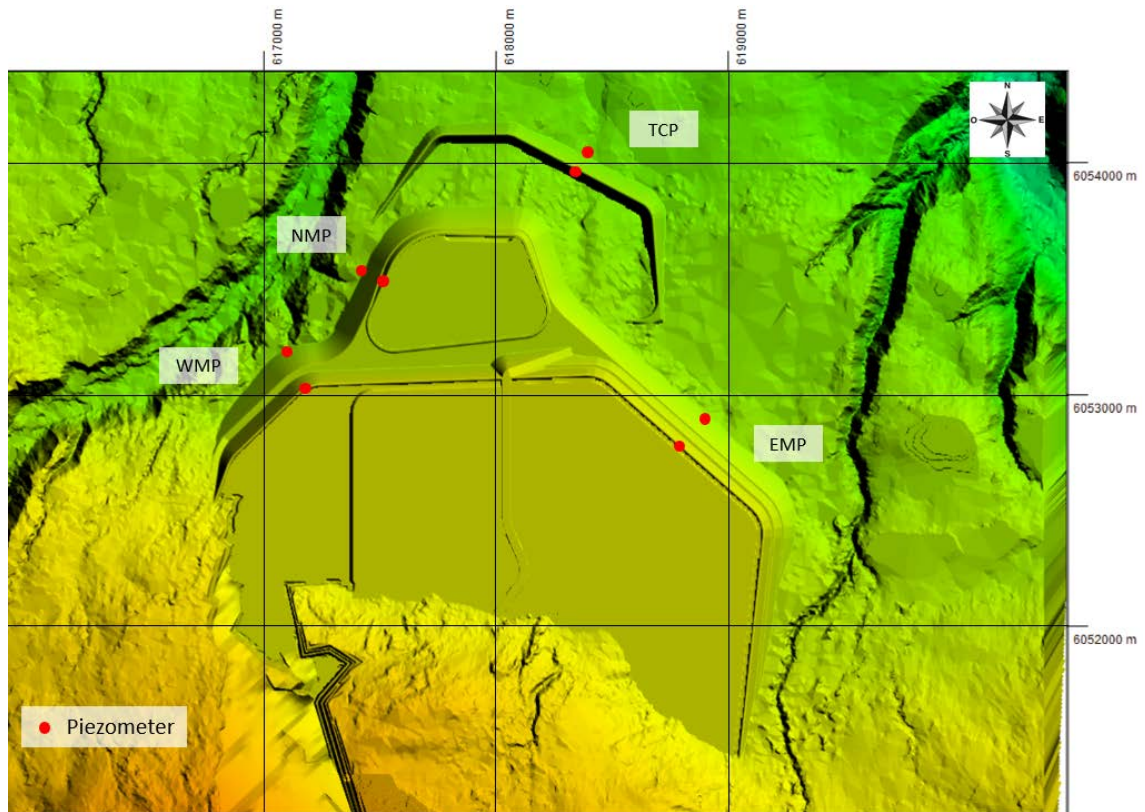


Figure 8-1: Piezometer proposed locations.

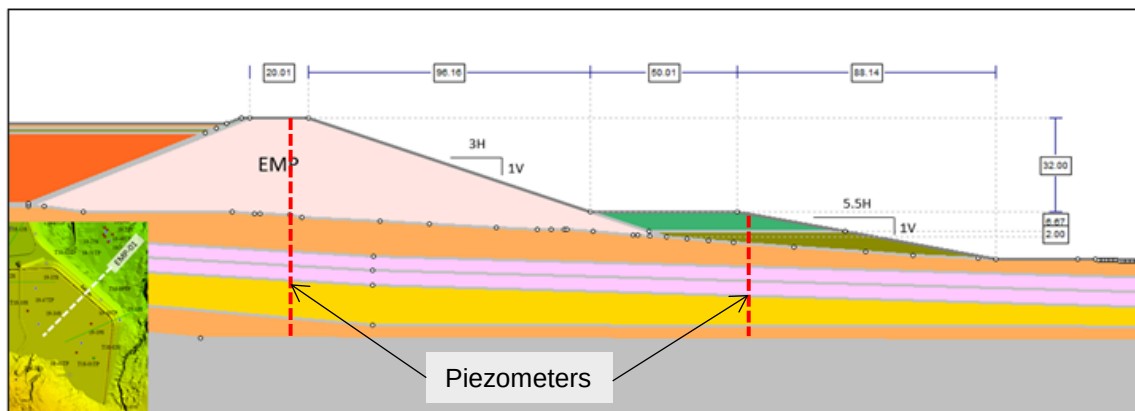


Figure 8-2: Piezometer Proposed Locations. Section View (Illustrative).

8.3.2 Monitoring Parameters and Schedule

The monitoring parameters applicable to the MP, their method of data acquisition, and frequency of data acquisition are presented in Table 8-3. Monitoring data is to be documented and reported as discussed in Section 9.2.

Table 8-3: Monitoring Parameters, Methods and Frequency of Data Acquisition.

Parameter	Method	Frequency
Climate		
Wind speed and direction	Measured on site weather station	hourly
Precipitation	Measured using rain gauge on site weather station	daily
Evaporation	Measured using evaporation pan on site weather station	daily
Elevations and freeboards		
Pond level	Measured using a vertical stake in the ponds	weekly
Pond freeboard	Calculated from pond elevations and dam crest	weekly
Embankment		
Pore pressures (piezometers)	Measured using piezometers located in the embankment	weekly
Crest settlement	Measured using manual survey equipment with survey beacons located at regular intervals on the crest	weekly
Survey		
MP survey	Surveyed using manual survey equipment or LiDAR drone, includes impoundment surface, embankments, and auxiliary features.	Annual

9 Reporting Requirements

9.1 Audits

9.1.1 Compliance Audits

The purpose of the compliance audit is to ensure ongoing compliance with the design intent. These audits are to focus on the identification of deficiencies by visual examination of the MP embankments and all appurtenant structures, as well as a review of all surveillance and monitoring data.

Compliance audits shall be conducted by a suitably qualified geotechnical consultant annually and in line with the facilities Licence Conditions, at minimum.

9.1.2 Dam Safety Review

A dam safety review (DSR) is a systematic review and evaluation of all aspects of design, construction, operation, maintenance, and surveillance, and governance affecting the dams, to evaluate the design criteria with current standards, operational compliance with design intent, stability and functionality of the dams, and to identify appropriate remedial measures.

A DSR is to be conducted by a suitably qualified geotechnical consultant (different from the EoR), and is to comply with the Canadian Dam Association guidelines (CDA, 2016).

9.2 Documentation and Reporting

All surveillance, monitoring and maintenance activities are to be documented as they assist in the identification of trends as well as changing conditions at the facility. Site staff should develop checklists or templates to record pertinent information in a consistent manner. Routine maintenance records should include:

- Relevant inspection reports;
- Monitoring data record sheets (piezometers, freeboards, etc.);
- Maintenance activity records, memos, reports;
- Quality control records;
- Photographs of all issues identified through inspection and site observations;
- Equipment logs, inventories of spares, materials, tool and equipment; and
- Maintenance schedules.

These records will document the frequency of problems that may be important when assessing the site conditions and reliability of various components.

10 Conclusion and Recommendations

10.1 Conclusion

A sequenced steady-state finite element seepage and two-dimensional limit equilibrium slope stability assessment of the TCP and MP were undertaken. Based on the current knowledge of the foundation materials, lower bound material strength parameters were adopted for potential weak layers (CH material), while strength and hydraulic parameters for the overburden, dam and waste rock materials were adopted from previous field investigations (SRK, 2018 and SRK, 2019a).

SRK included a drainage layer composed by NAG material at toe of the buttress to prevent the development of pore pressures. The results of the analyses showed that the TCP, NMP, EMP and WMP configurations are stable and achieve Telkwa's internal requirement and CDA (2014) design criteria.

The foundation conditions for the NMP and TCP facilities were defined by previous field investigation and designs were assessed accordingly. Characterization of the foundations, under the EMP and WMP, was not well defined as the dam footprint had shifted during recent design revisions by Telkwa. Based on the limited drilling it was uncertain if the potential weak layers identified in the NMP foundation extended to the EMP and WMP areas.

Considering the main risks and uncertainties related to: 1) foundation conditions, 2) material availability, and 3) performance of the soli-bentonite liner, sensitivity assessments were carried out for the EMP and WMP facilities. The main conclusions of the sensitivity assessments were that:

- Different weak layer thicknesses in the foundation:
 - EMP: the EMP was designed considering the worst-case scenario (10.0 m thick of the CH layer), the facility achieved the design criteria in every scenario; and
 - WMP: the target FoS was achieved even for the worst case scenario.
- Different dam material configurations:
 - Alternatives considering the buttress constructed with a draining material (i.e. NAG material) achieved the design criteria; and
 - If constructing the buttress with the overburden material, some localized failures are obtained in the buttress due to the development of pore pressures.
- Liner failure:
 - A potential total breakdown of the MP lining has a high impact in the FoS. Minimum target FoS' are not achieved for the overall structure and there is also the potential for small localized failures at the toe of the dam and buttress. However, the FoS results were still above 1.3 for the main dam structure and imminent failure would be unlikely. Constructing a chimney drain connected with the toe drain does improve the stability of the dam to meet minimal design criteria, but additional buttress enhancements may also be required to prevent small localized failures at the toe of the dam and buttress.

10.2 Recommendations

Based on the results of this assessment, the following recommendations are being made:

- Undertake further field and laboratory testing to reduce uncertainties related to the foundation conditions and material strength and hydraulic parameters, including:
 - Additional geotechnical boreholes to determine the presence, extension, depth and thickness of weak layers (i.e. CH layers); and
 - Oedometric consolidation tests in selected samples to: (i) determine the pre-consolidation stress and (ii) define if undrained conditions are expected or not.
- Determine the availability of overburden and NAG material for construction of the dam;
- Update the stability assessment based on the results of the recommended field and laboratory testing and the material availability;
- Include a 2-to-3 m thick draining layer at the toe of the buttress in the design to prevent the development of pore pressures which may induce localized instabilities within the buttresses;
- Design an erosion protection layer on the downstream face of the dam; and
- Complete a detailed design report for the Management and Tenas Control Ponds.

Closure

This final report, Tenas Project – Management and Control Ponds Stability Assessment, was prepared by

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Santiago Pastine
Consultant

and reviewed by

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Calvin Boese, MSc., PEng
Principal Consultant

All data used as source material plus the text, tables, figures, and attachments of this document have been reviewed and prepared in accordance with generally accepted professional engineering and environmental practices.

References

- Bieniawski, Z. (1989). *Engineering Rock Mass Classifications*. New York: Wiley.
- Brown, E. (1982). *Rock Characterization, Testing and Monitoring. International Society of Rock Mechanics Suggested Methods*. Oxford: Pergamon Press.
- CDA. (2013). *Dam Safety Guidelines 2007 (2013 Edition)*.
- CDA. (2014). *Application of Dam Safety Guidelines to Mining Dams*.
- CDA. (2016). *Dam Safety Reviews*.
- ICMM. (2020). *Global Industry Standard on Tailings Management*.
- Levson et al. (1997). *Quaternary Geology and Ice-Flow Studies in the Smithers and Hazelton Map Areas (93L and M): Implications for Exploration: British Columbia Geological Survey Branch. Geological Fieldwork 1997. Paper 1998-1*.
- NCR. (2015). *2015 National Building Code Seismic Hazard Calculation for site: 54.6163 N, 127.152 W: National Resources Canada. Accessed May 15, 2018*.
- Palsgrove, R. (1991). *Stratigraphy, Sedimentology and Coal Quality of the Lower Skeena Group, Telkwa Coalfield, Central British Columbia*.
- Read, J., & Stacey, P. (2009). *Guidelines for Open Pit Slope Design*. CSIRO.
- SRK. (2018). *Tenas Project - Factual Data Report for 2018 Geotechnical Investigation*.
- SRK. (2019a). *Tenas Project - Factual Data Report for 2019 Field Investigation*.
- SRK. (2019b). *Geotechnical Stability Analysis for Management and Control Ponds*.
- SRK. (2020). *Tenas Project Test Pit and Drillhole Locations*.
- SRK. (2021a). *Liner Design and Construction Methodology for Tenas Management Ponds 2CT025.008 04192021*.
- SRK. (2021b). *Tenas Project Dam Breach Assessment Report 1CT027-003 04072021*.
- Telkwa. (2018). *Project Geological Map: Telkwa Coal Ltd*.

Appendix A TCP and MP Stability Assessment Outputs

A2. Geotechnical strength and hydraulic parameters

Table A - 1: Geotechnical strength and hydraulic parameters

Material	Failure Criterion	Color	γ [kN/m ³]	c' [kPa]	ϕ'_{cv} [°]	k_h [m/s]	k_v/k_h
Dam	Mohr Coulomb		20.5	0	30	1×10^{-07}	1
Buttress	Mohr Coulomb		20.5	0	30	1×10^{-07}	1
NAG Rock	Mohr Coulomb		23	0	35	1×10^{-05}	1
PAG Rock	Mohr Coulomb		23	0	35	1×10^{-05}	1
Liner	Mohr Coulomb		18	0	20	1×10^{-10}	1
CH	Mohr Coulomb		18	0	20	3×10^{-06}	1
CL	Mohr Coulomb		18	0	27	3×10^{-06}	1
Loose Silt	Mohr Coulomb		18	0	25	3×10^{-06}	1
Sand	Mohr Coulomb		20	0	30	3×10^{-05}	1
Bedrock	Infinite Strength		25.5	Infinite Strength		3×10^{-07}	1

A3. TCP

A3-1. TCP-01

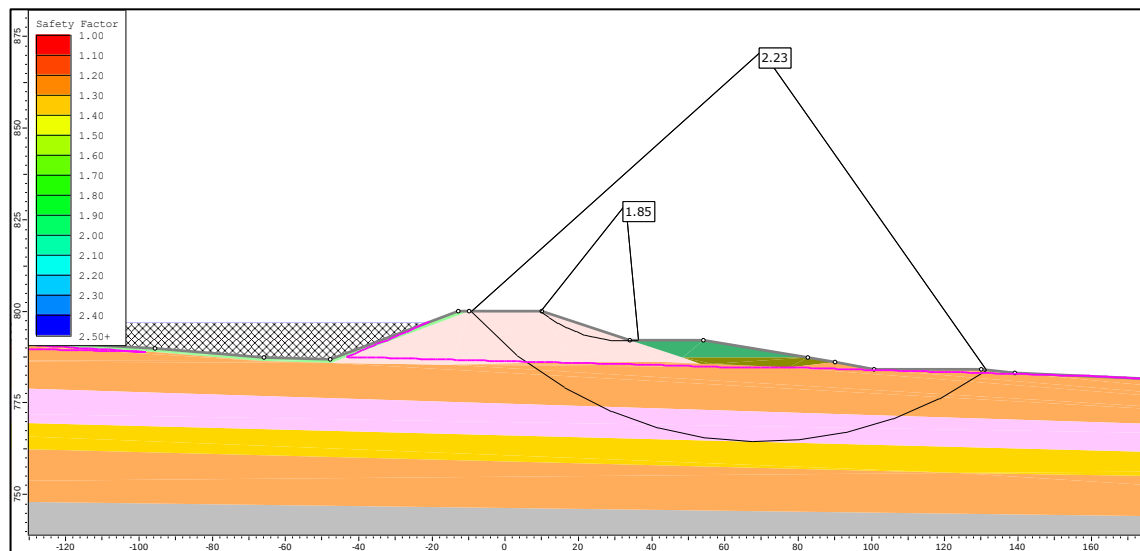


Figure A - 2: TCP-01. Static condition.

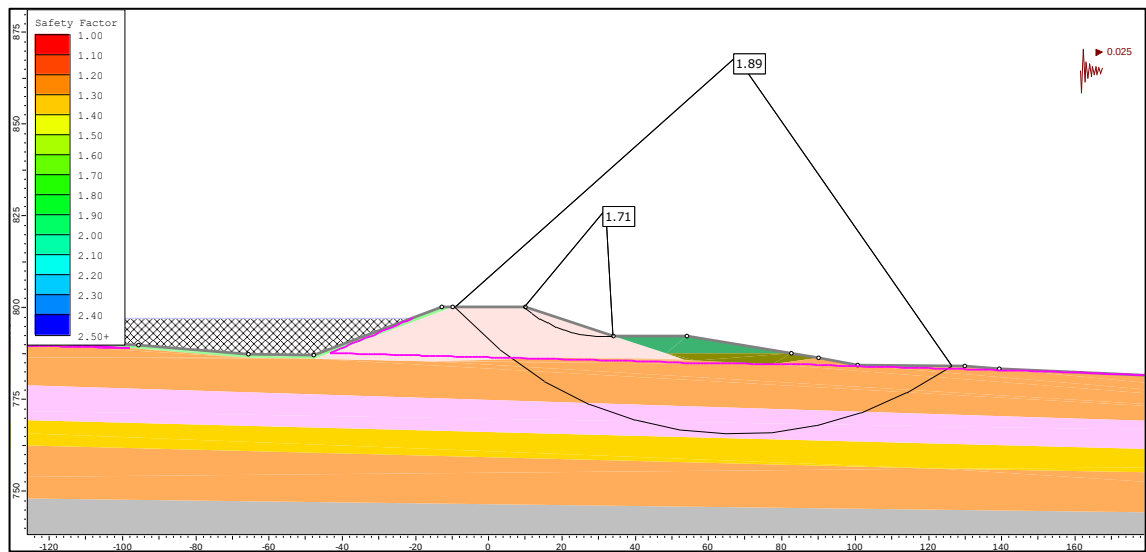


Figure A - 3: TCP-01. Pseudo-static condition.

A3-2. TCP&NMP-01

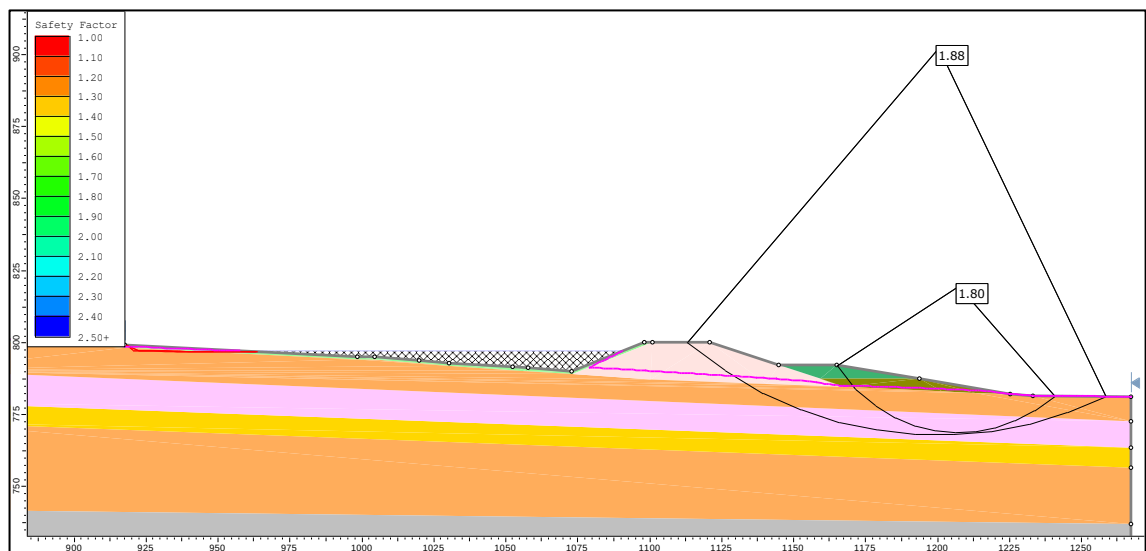


Figure A - 4: TCP&NMP-01 (TCP). Static condition.

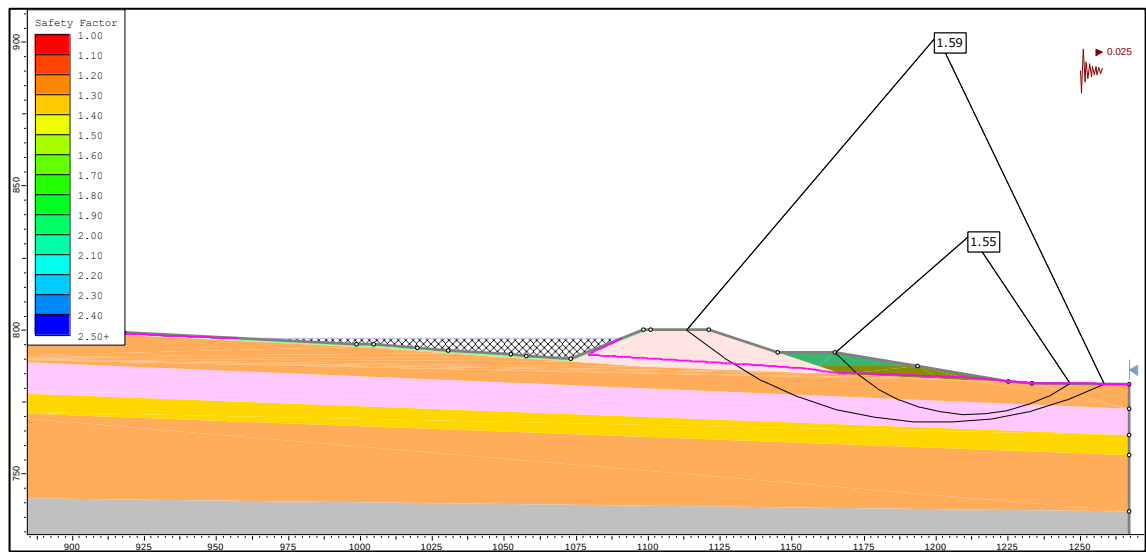


Figure A - 5: TCP&NMP-01 (TCP). Pseudo-static condition.

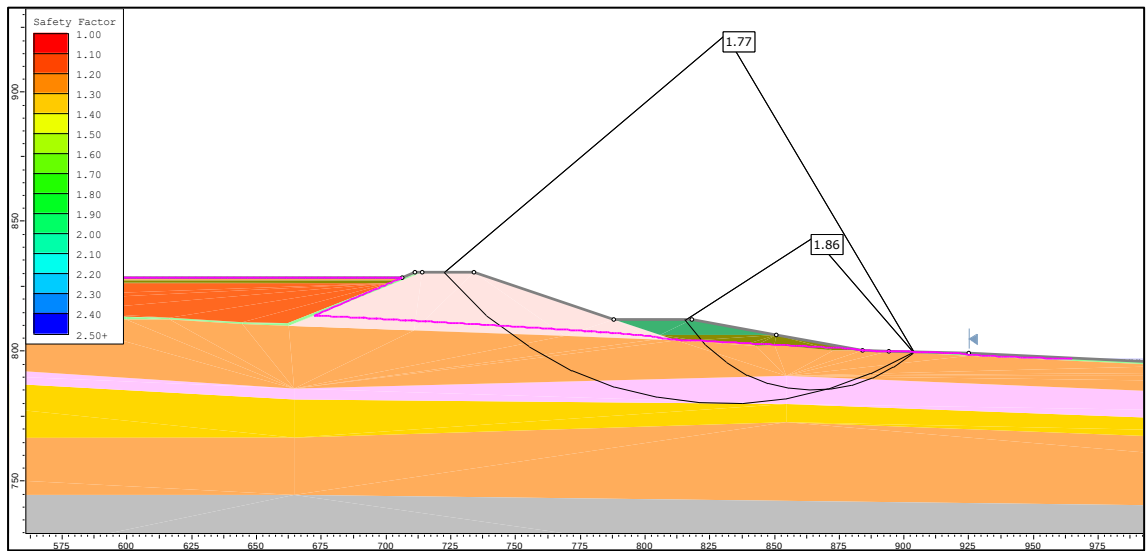


Figure A - 6: TCP&NMP-01 (NMP). Static condition.

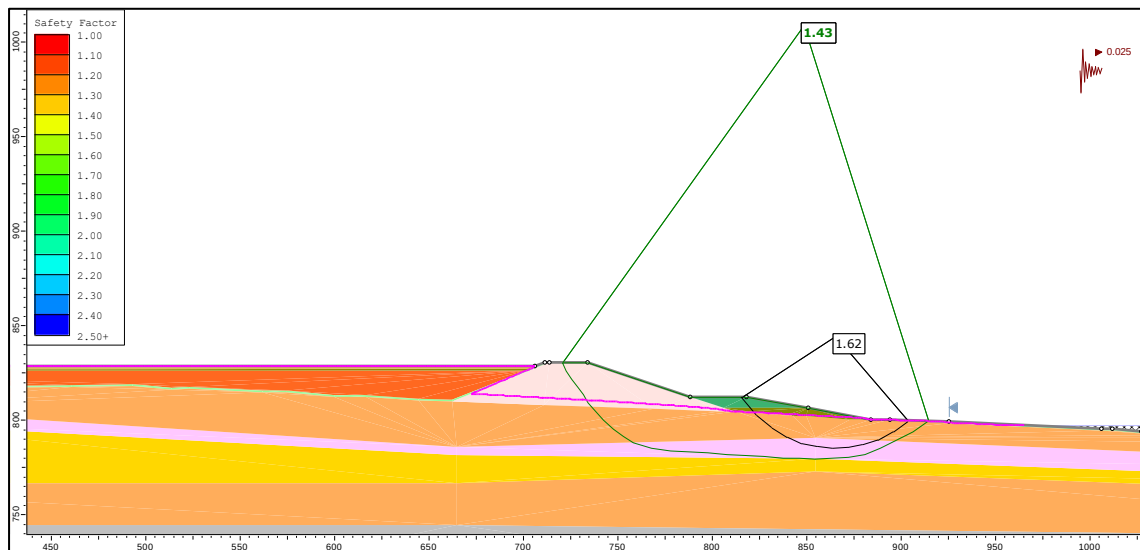


Figure A - 7: TCP&NMP-01 (NMP). Pseudo-static condition.

A4. NMP

A4-1. NMP-01

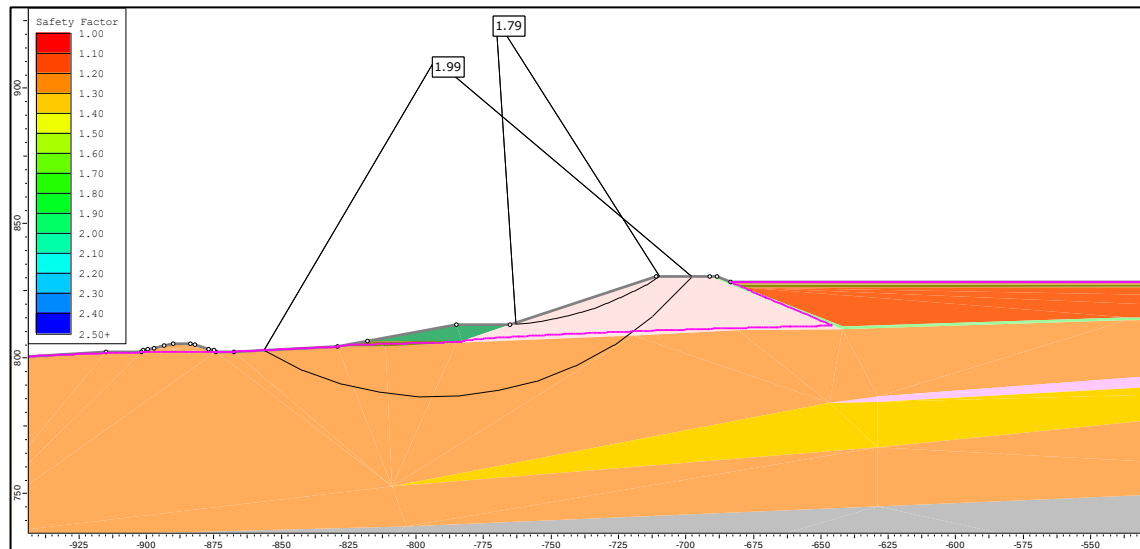


Figure A - 8: NMP-01. Static condition.

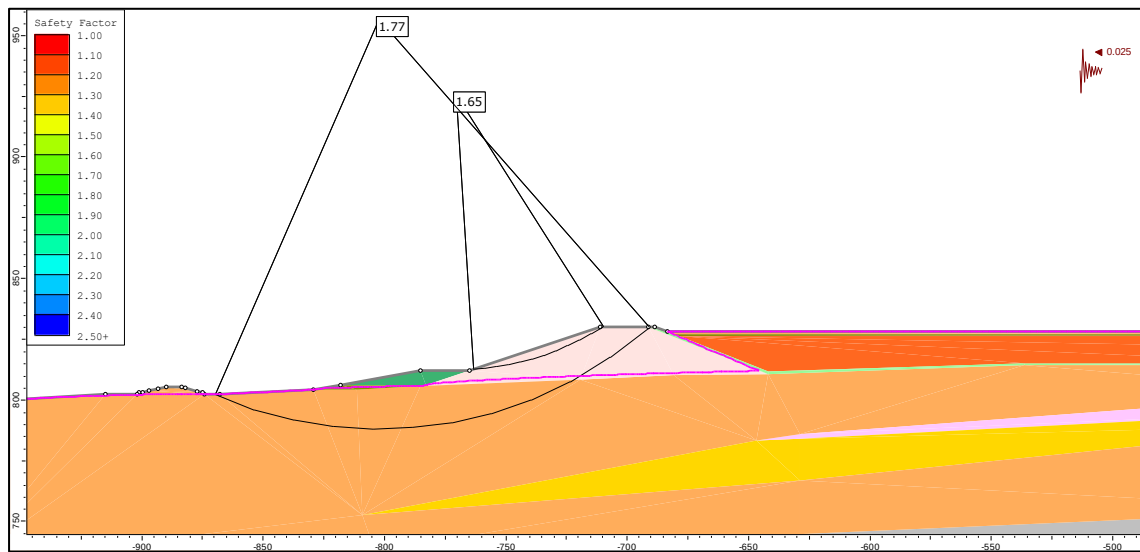


Figure A - 9: NMP-01. Pseudo-static condition.

A4-2. NMP-02

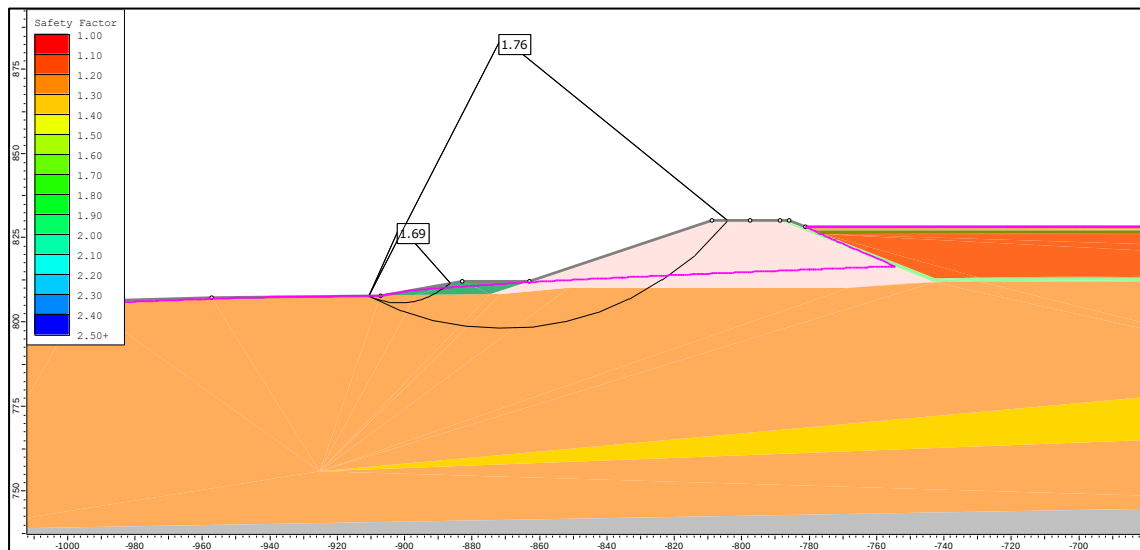


Figure A - 10: NMP-02. Static condition.

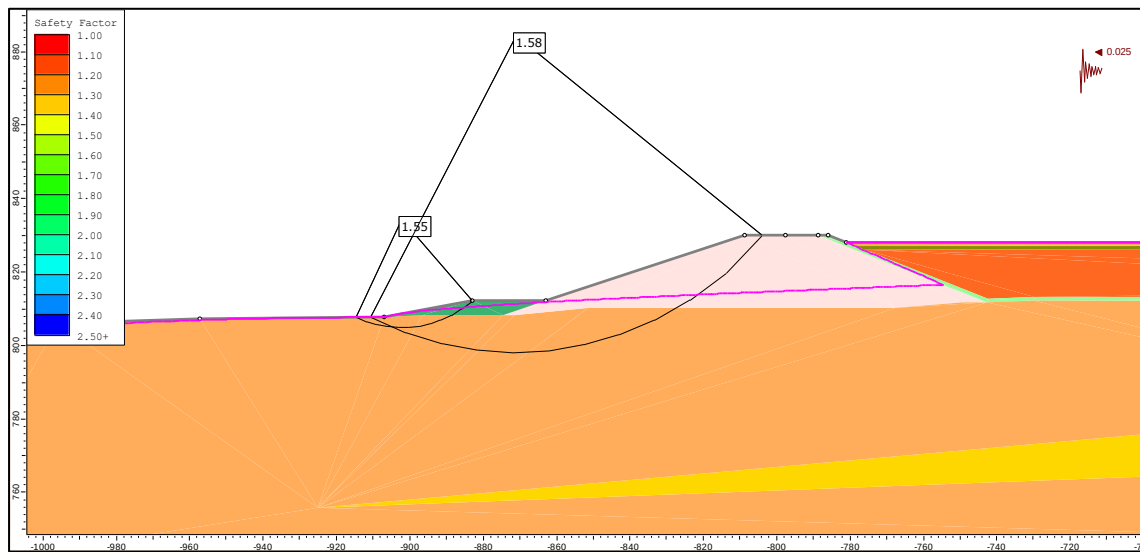


Figure A - 11: NMP-02. Pseudo-static condition.

A4-3. NMP&WMP-01

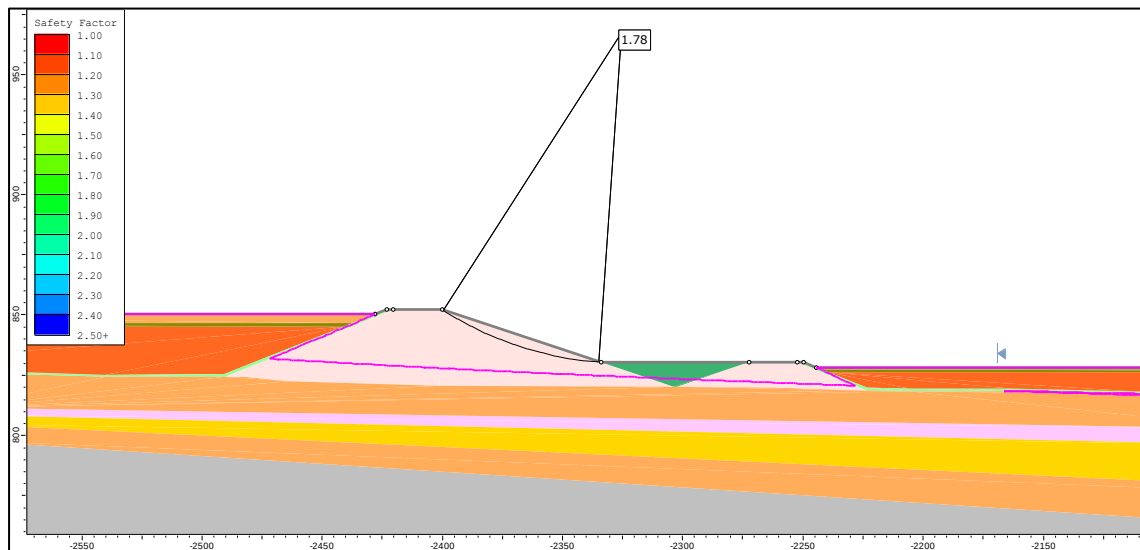


Figure A - 12: NMP&WMP-01 (WMP). Static condition.

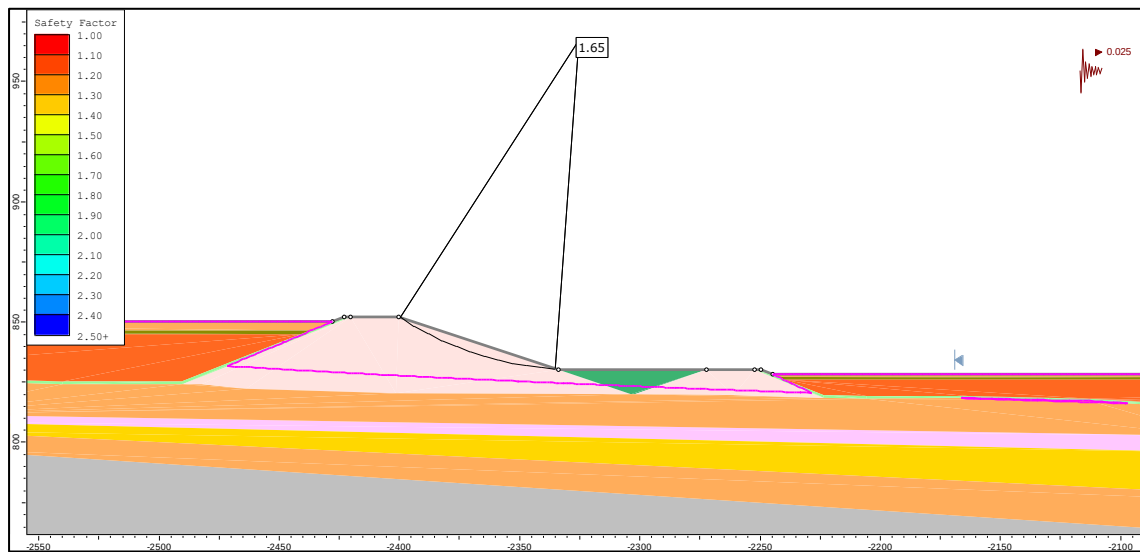


Figure A - 13: NMP&WMP-01 (WMP). Pseudo-static condition.

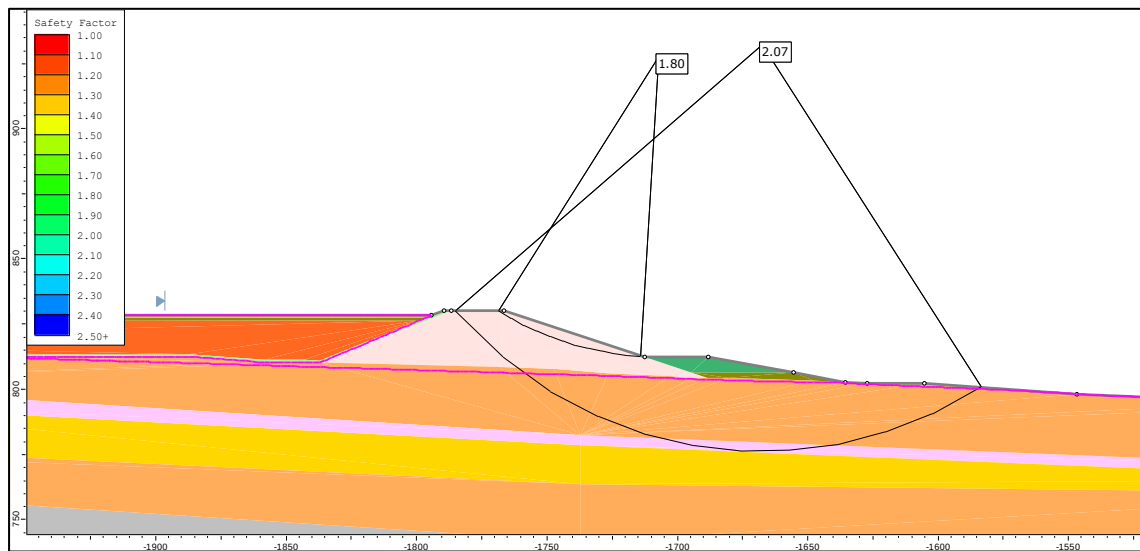


Figure A - 14: NMP&WMP-01 (NMP). Static condition.

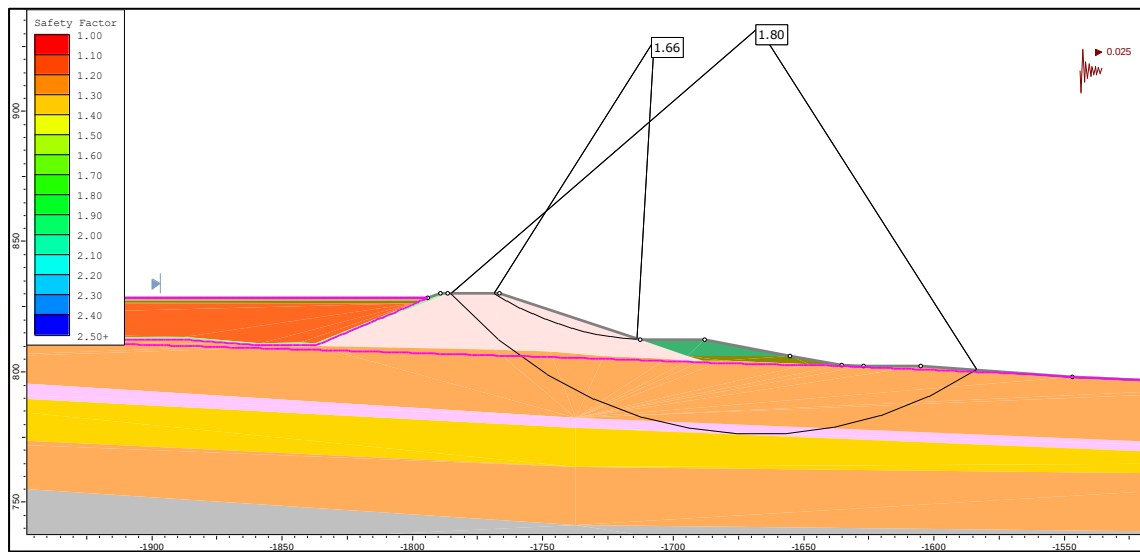


Figure A - 15: NMP&WMP-01 (NMP). Pseudo-static condition.

A5. WMP

A5-1. WMP-01

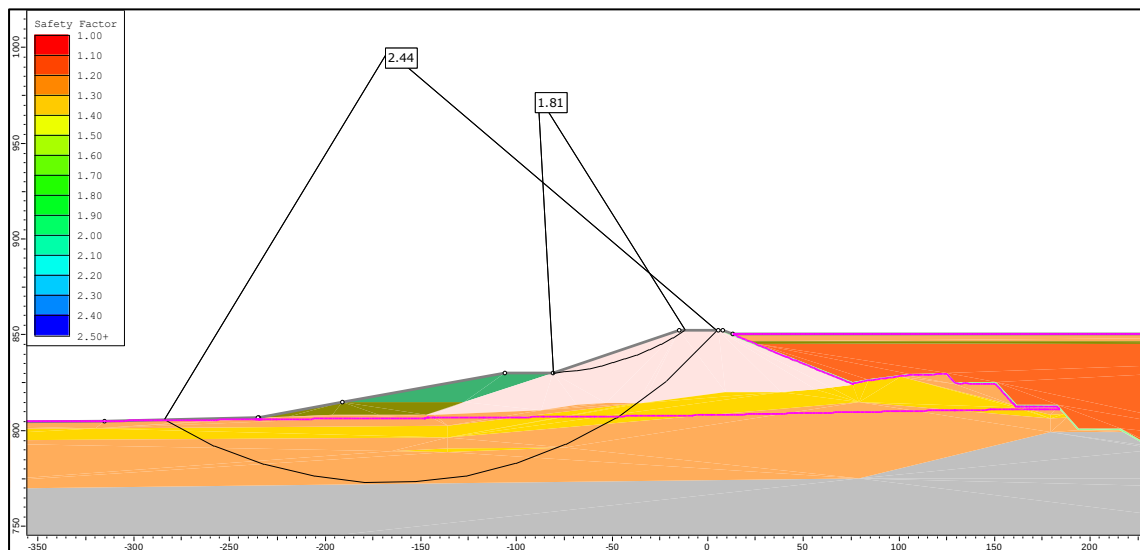


Figure A - 16: WMP-01. Static condition.

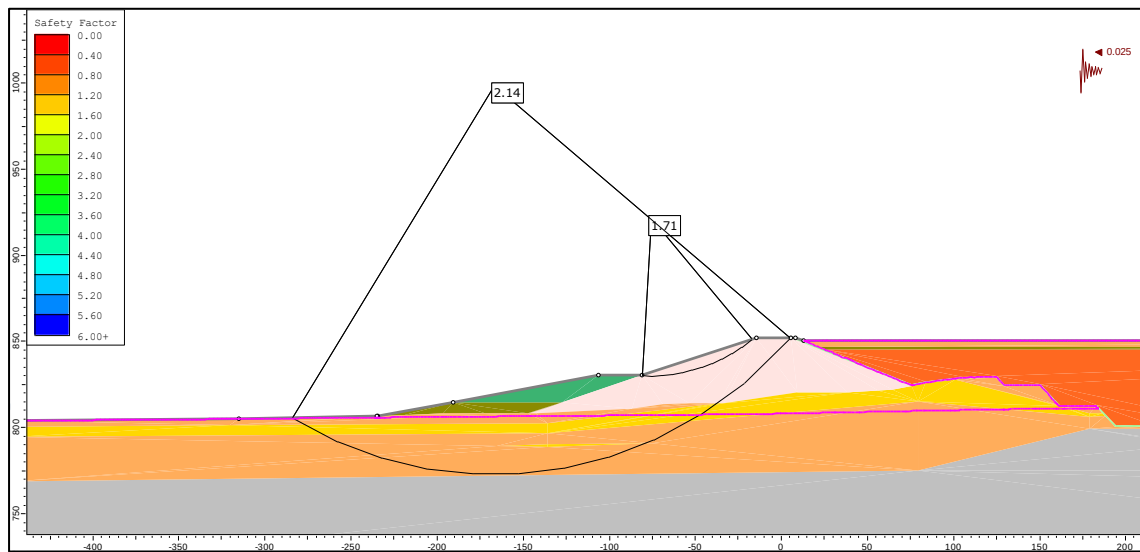


Figure A - 17: WMP-01. Pseudo-static condition.

A5-2. WMP-02

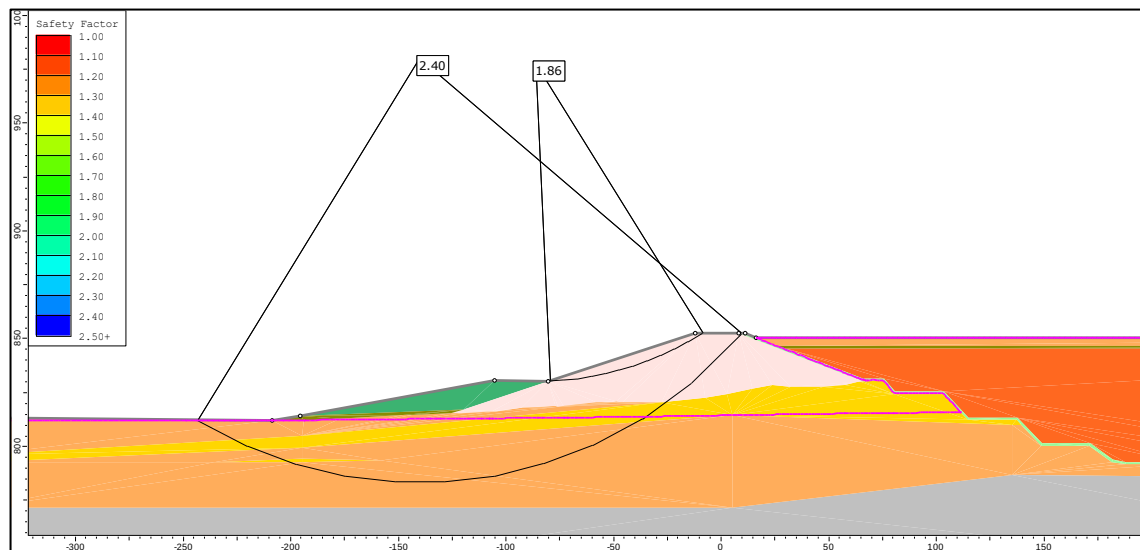


Figure A - 18: WMP-02. Static condition.

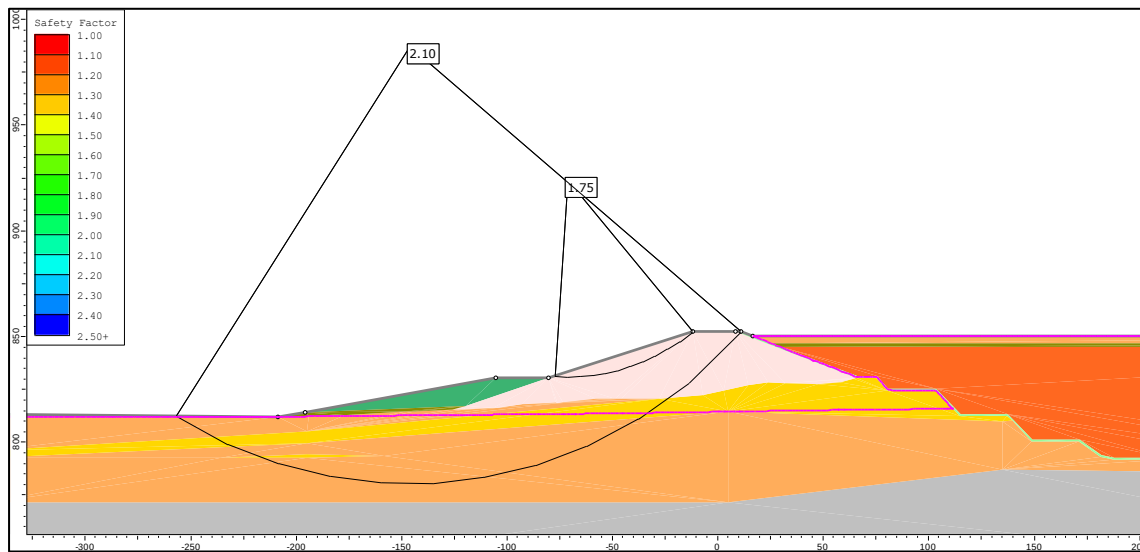


Figure A - 19: WMP-02. Pseudo-static condition.

A6. EMP

A6-1. EMP&WMP-01

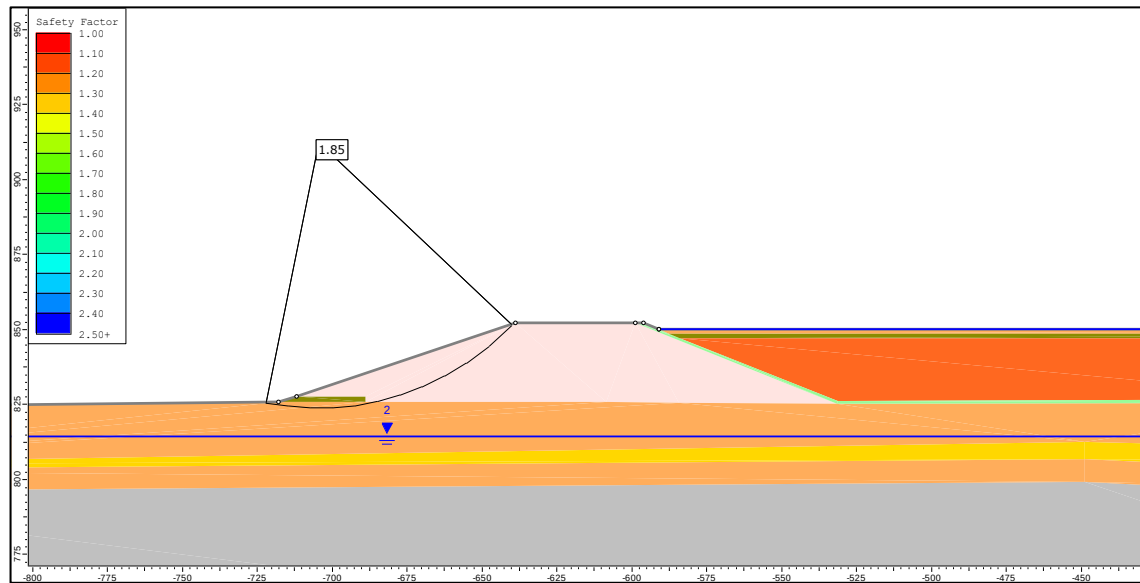


Figure A - 20: EMP&WMP-01. Static condition.

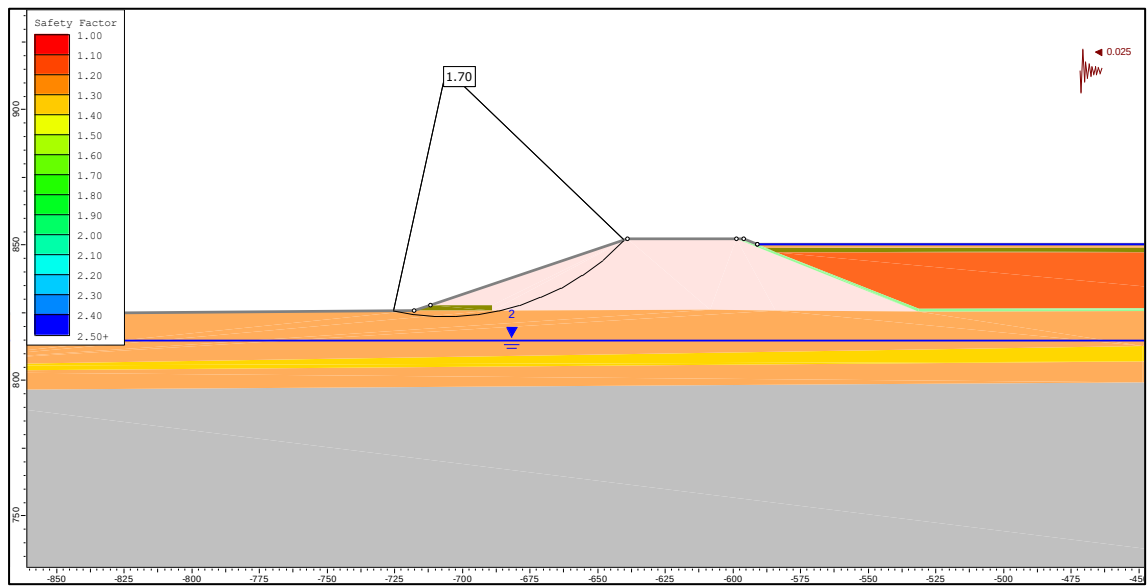


Figure A - 21: EMP&WMP-01. Pseudo-static condition.

A6-2. EMP-01

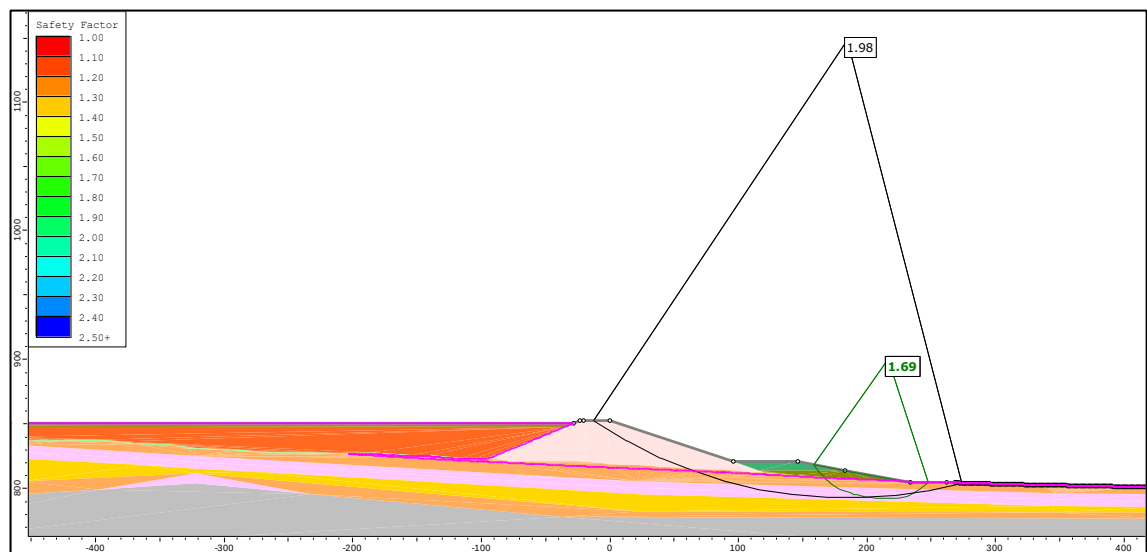


Figure A - 22: EMP-01. Static condition.

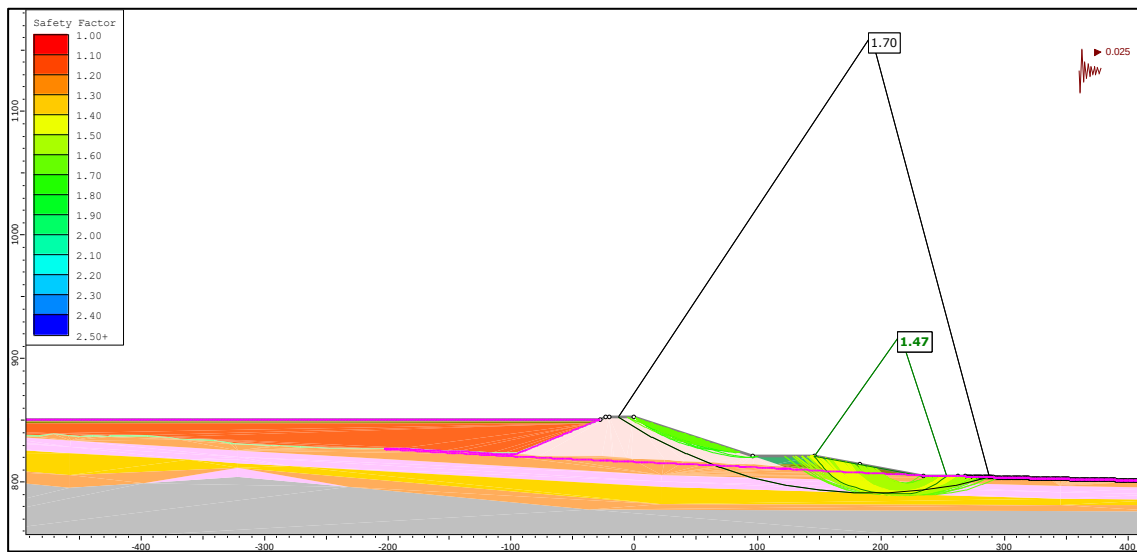


Figure A - 23: EMP-01. Pseudo-static condition.

A6-3. EMP-02

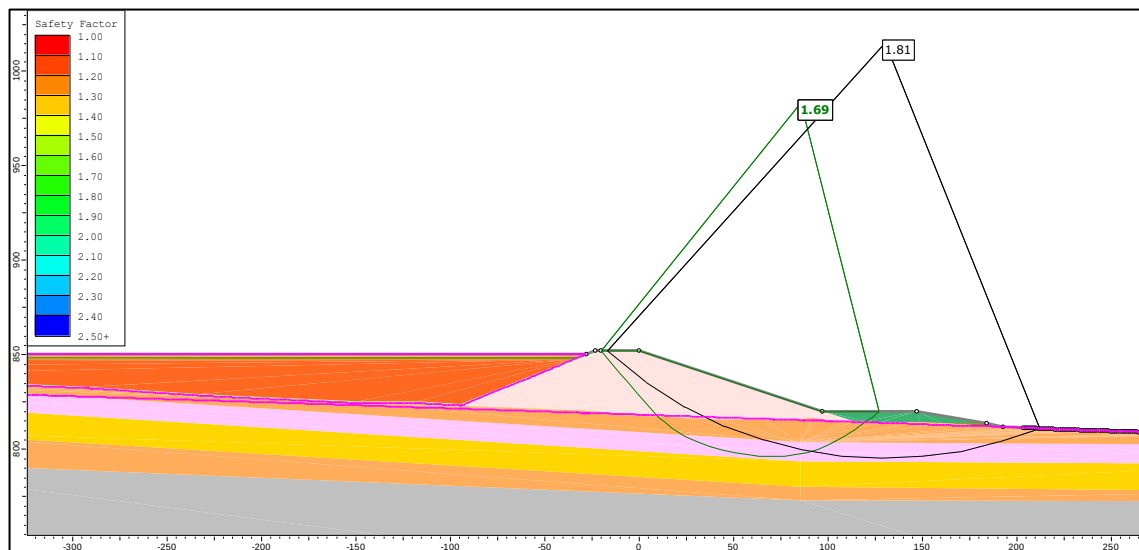


Figure A - 24: EMP-02. Static condition.

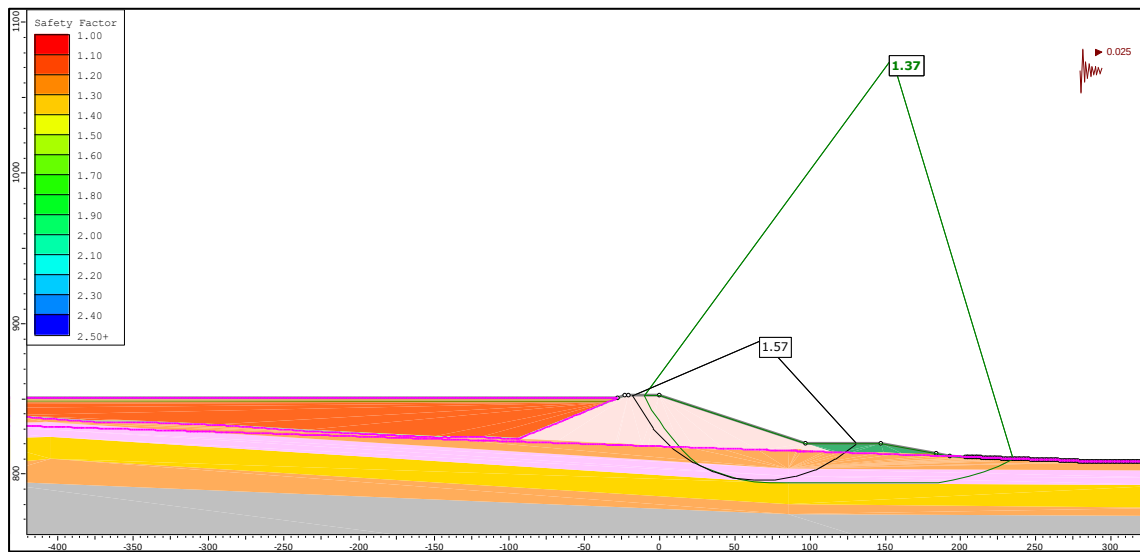


Figure A - 25: EMP-02. Pseudo-static condition.

A6-4. EMP-03

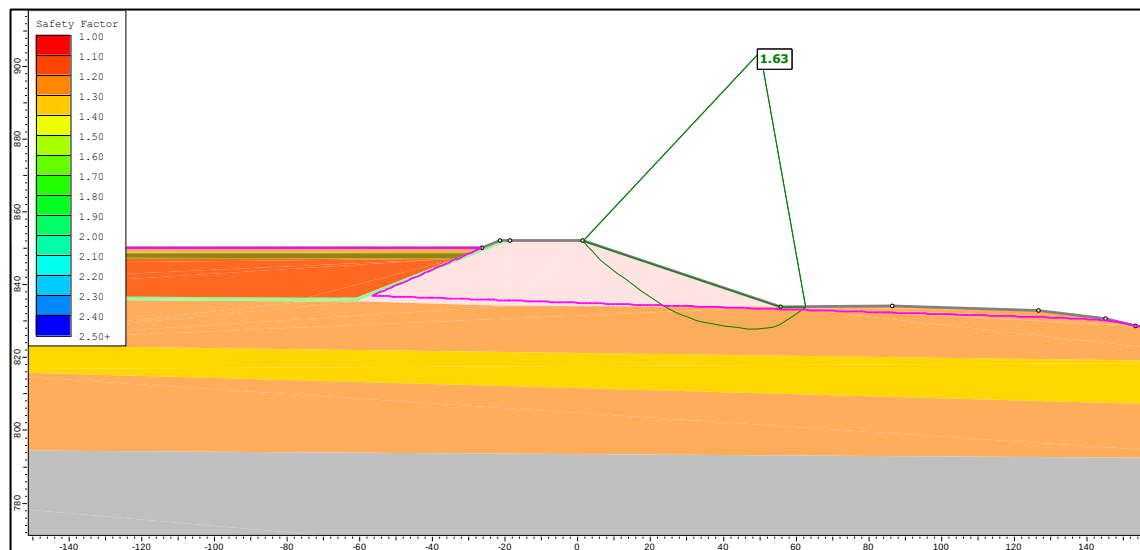


Figure A - 26: EMP-03. Static condition.

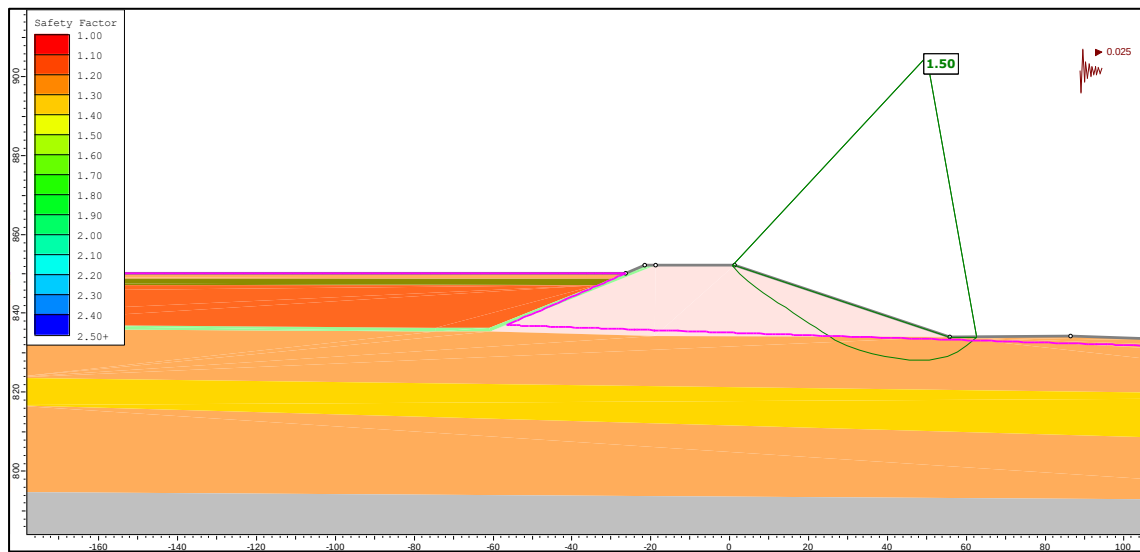


Figure A - 27: EMP-03. Pseudo-static condition.

Appendix B Sensitivity Assessment

Appendix B. Sensitivity Assessment

B1. Different CH layer thickness

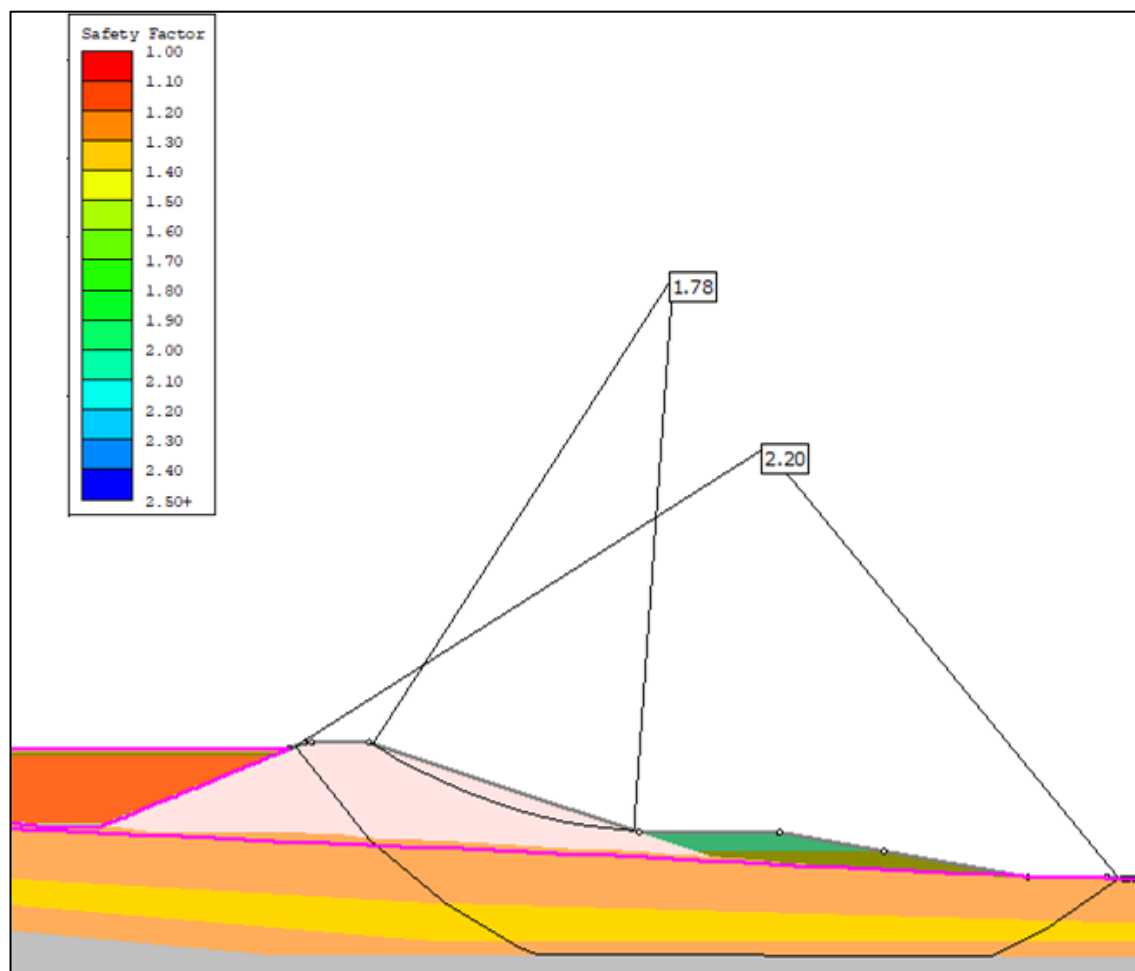


Figure B - 1: EMP-01. Static condition. 0m CH layer thickness.

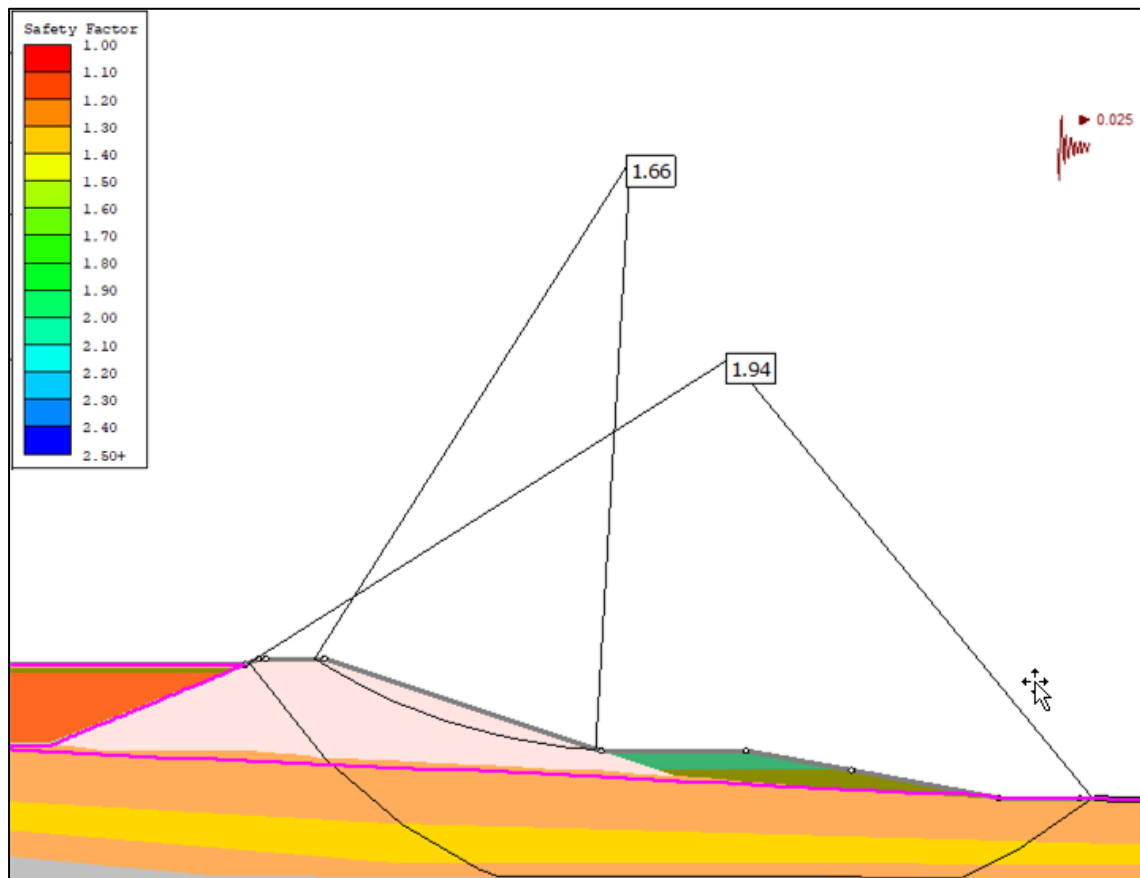


Figure B - 2: EMP-01. Pseudo-static condition. 0m CH layer thickness.

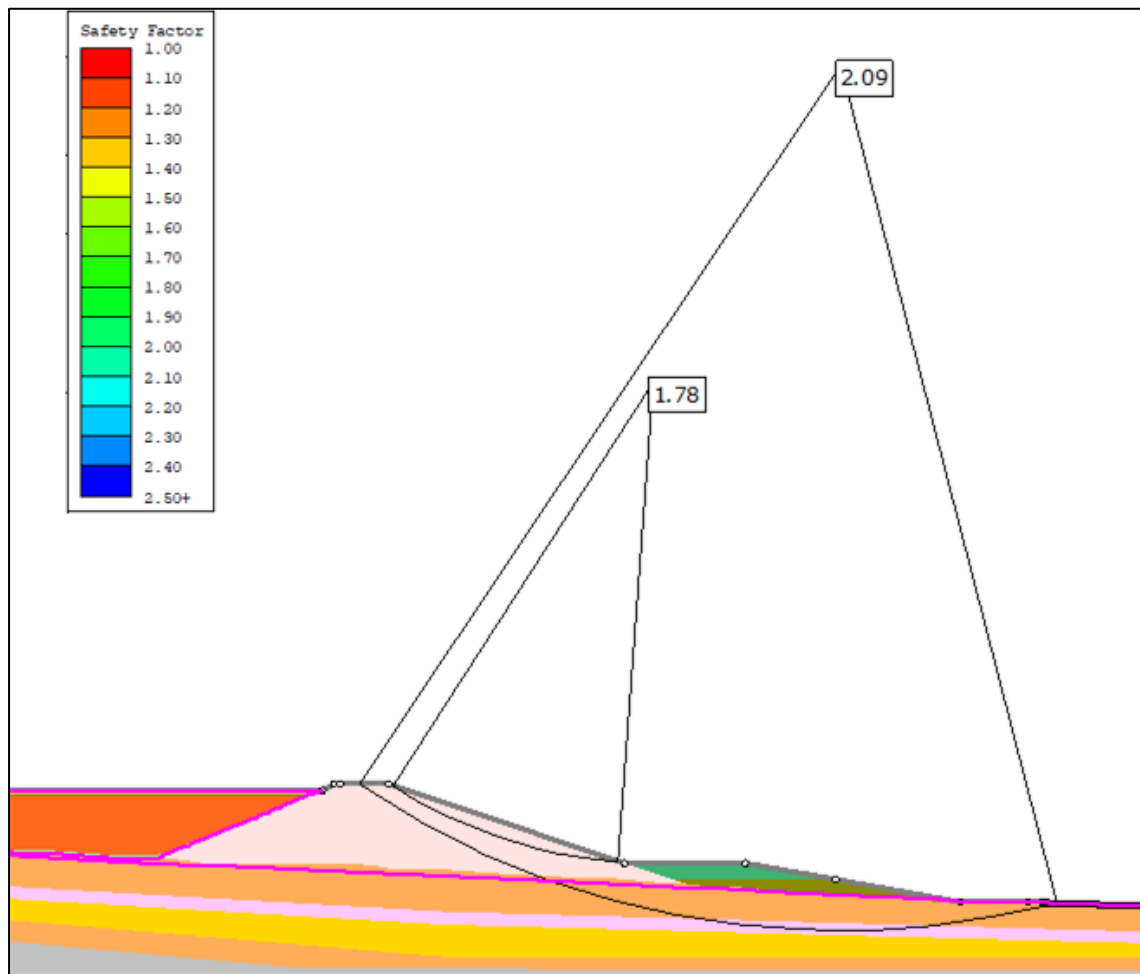


Figure B - 3: EMP-01. Static condition. 5m CH layer thickness.

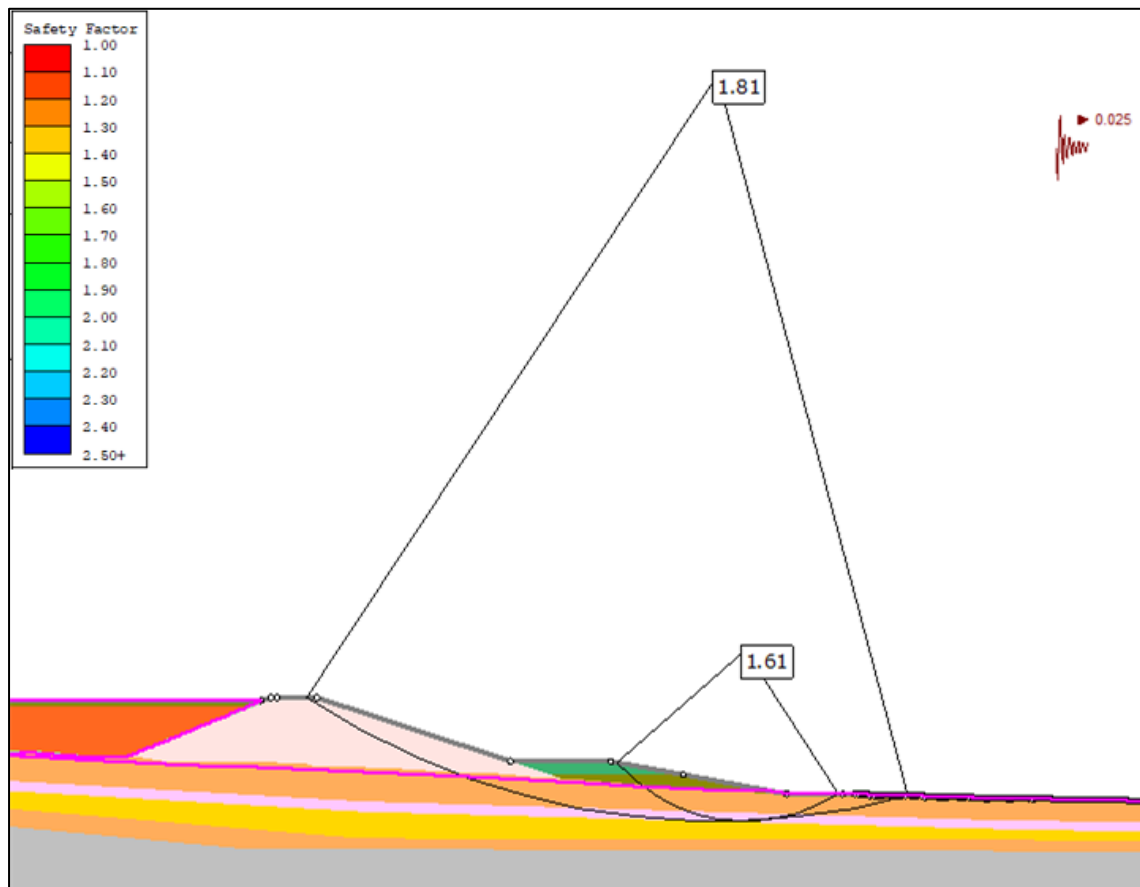


Figure B - 4: EMP-01. Pseudo-static condition. 5m CH layer thickness.

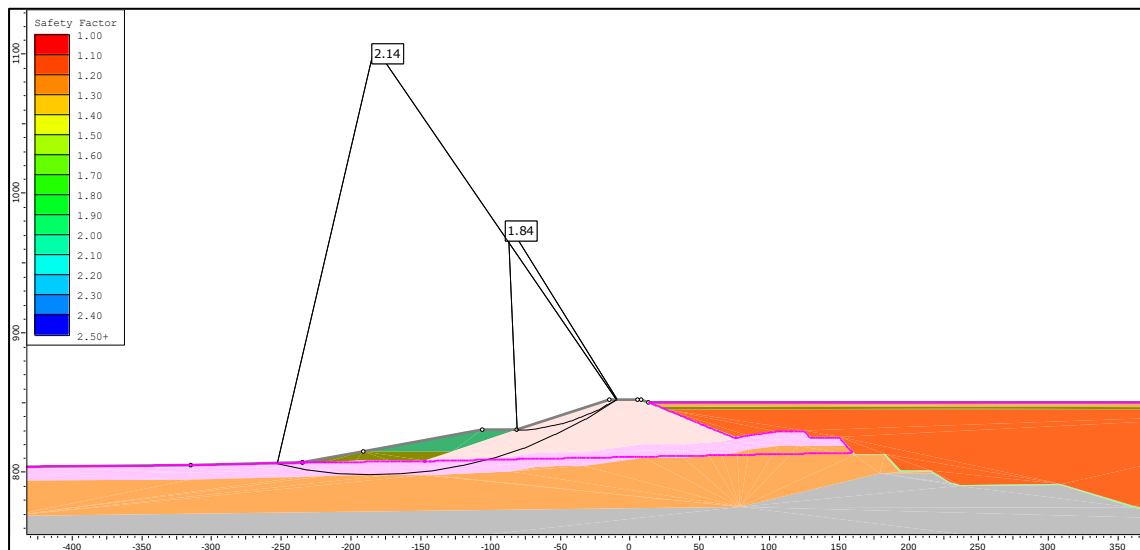


Figure B - 5: WMP-01. Static condition. 10m CH layer thickness.

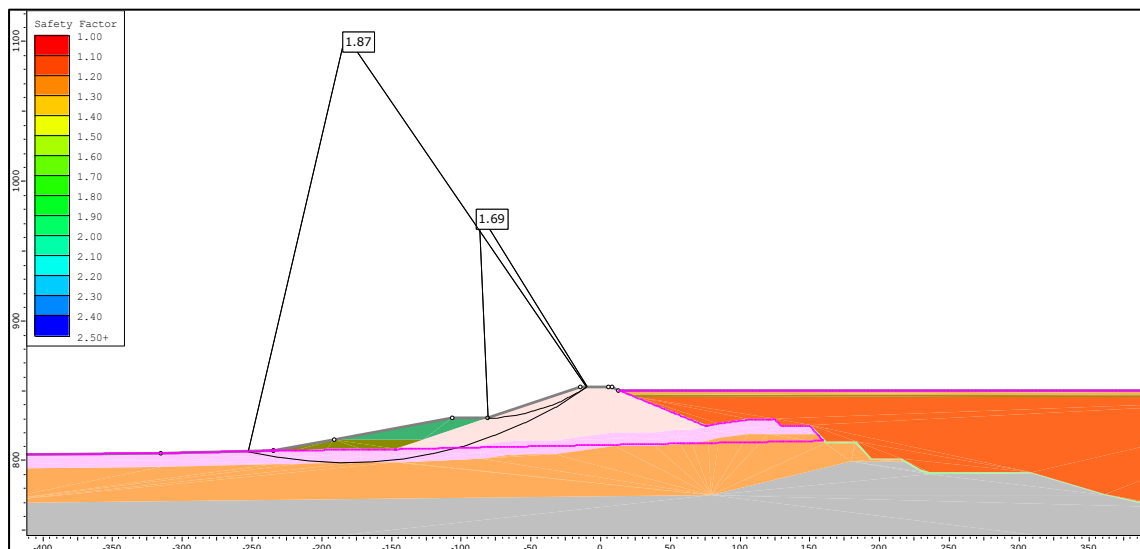


Figure B - 6: WMP-01. Pseudo-static condition. 10m CH layer thickness.

B2. Different dam materials configurations

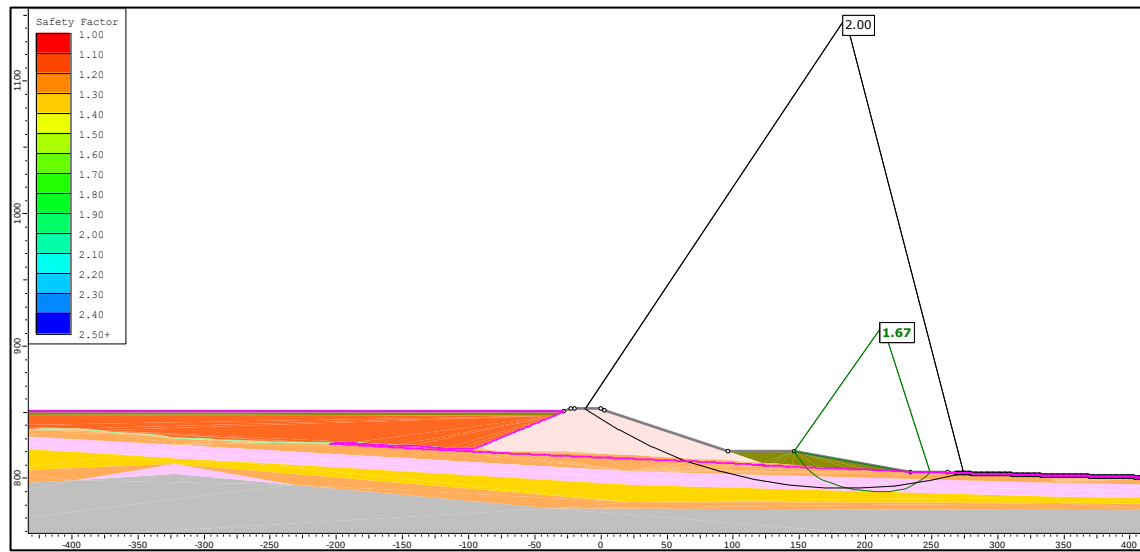


Figure B - 7: EMP-01. Static condition. Alternative 1.

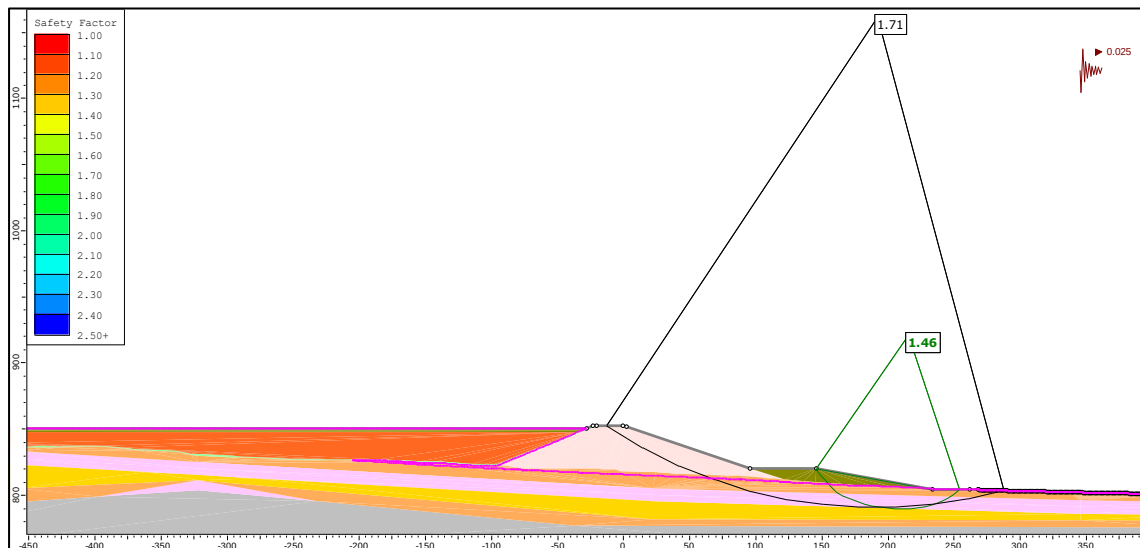


Figure B - 8: EMP-01. Pseudo-static condition. Alternative 1.

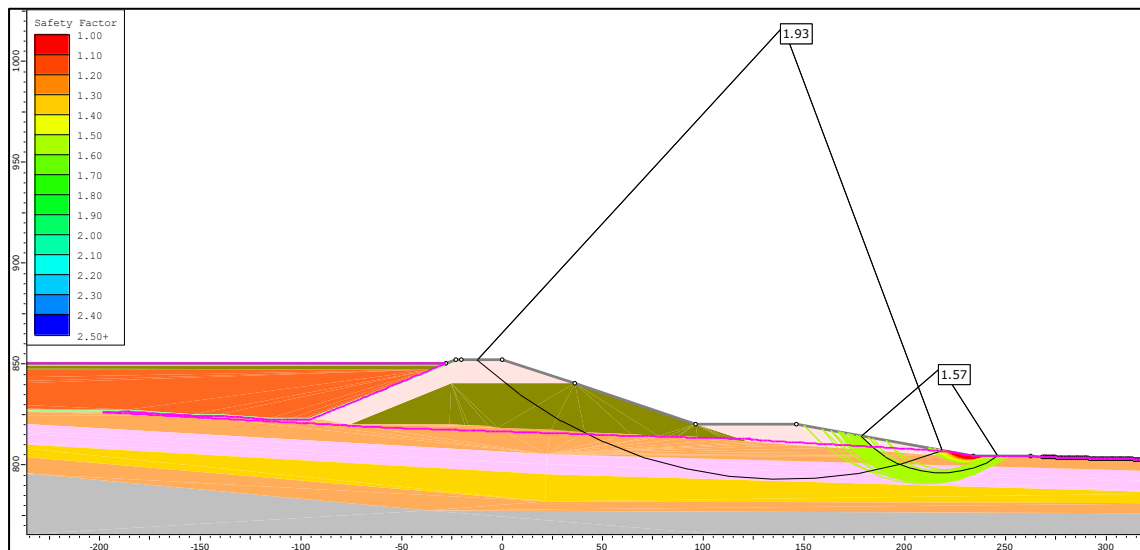


Figure B - 9: EMP-01. Static condition. Alternative 2.

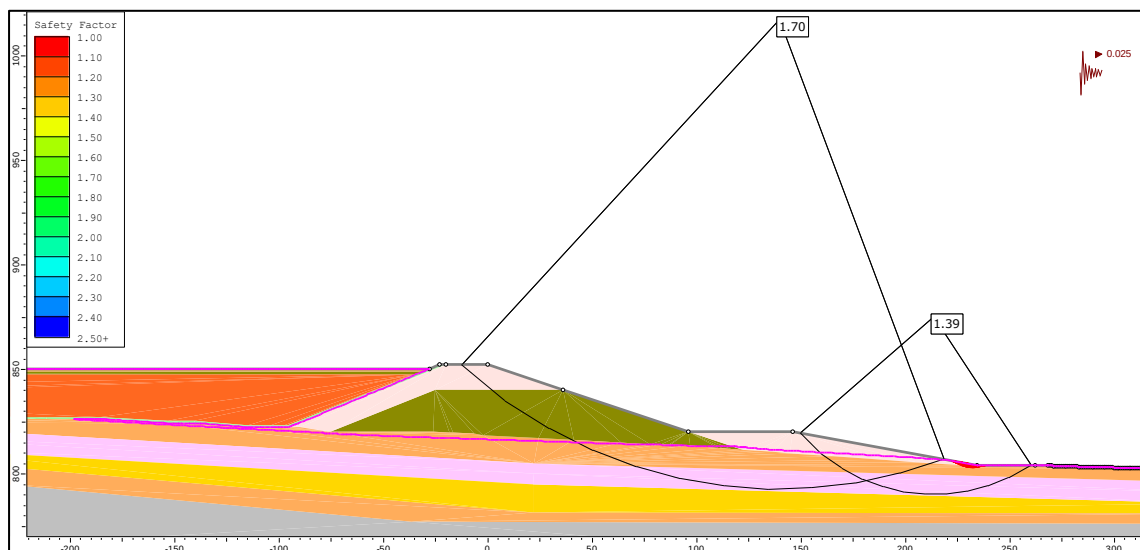


Figure B - 10: EMP-01. Pseudo-static condition. Alternative 2.

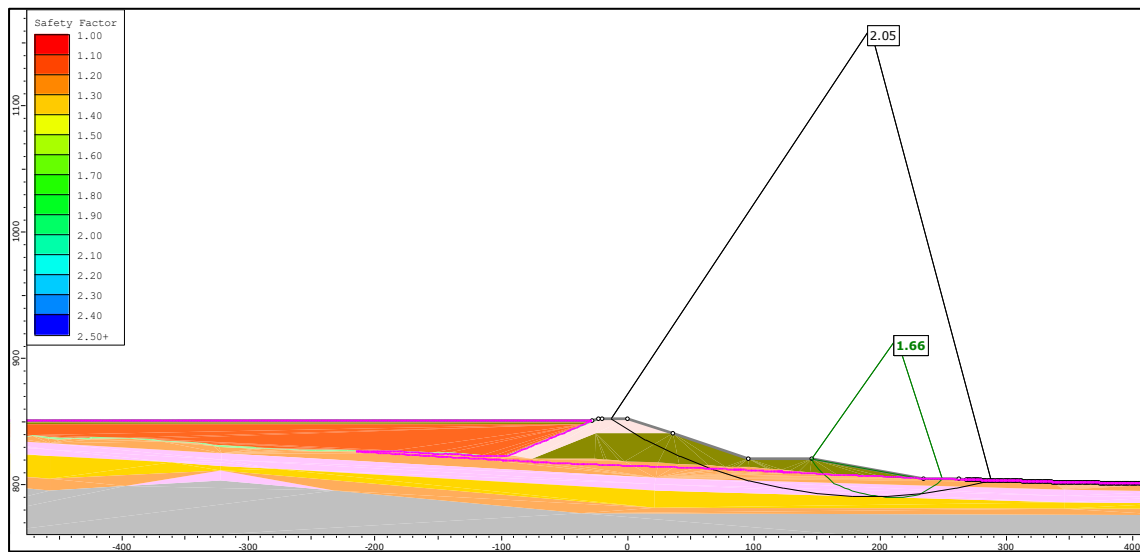


Figure B - 11: EMP-01. Static condition. Alternative 3.

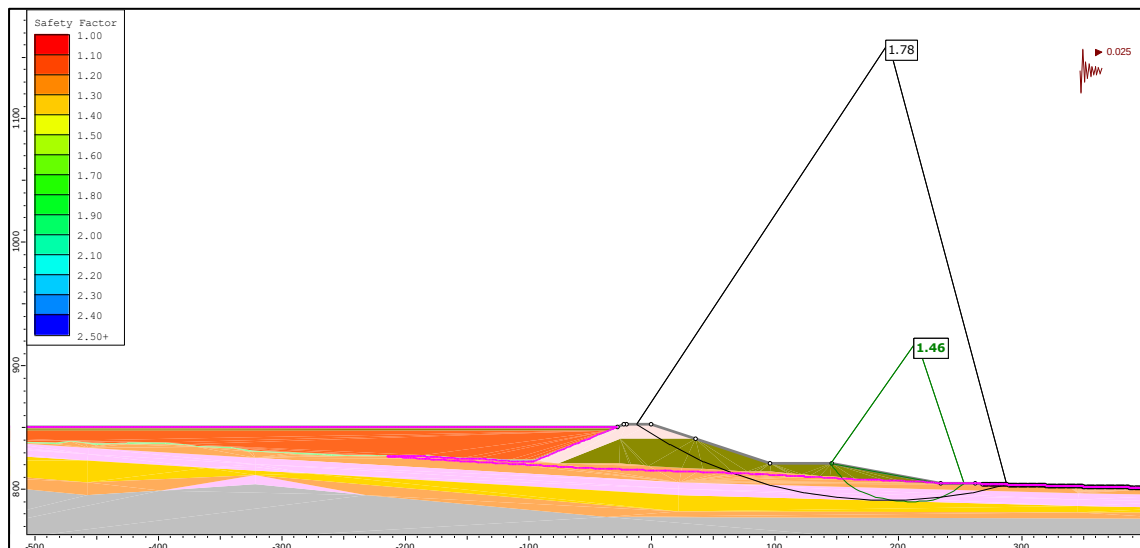


Figure B - 12: EMP-01. Pseudo-static condition. Alternative 3.

B3. Different stratigraphy configuration

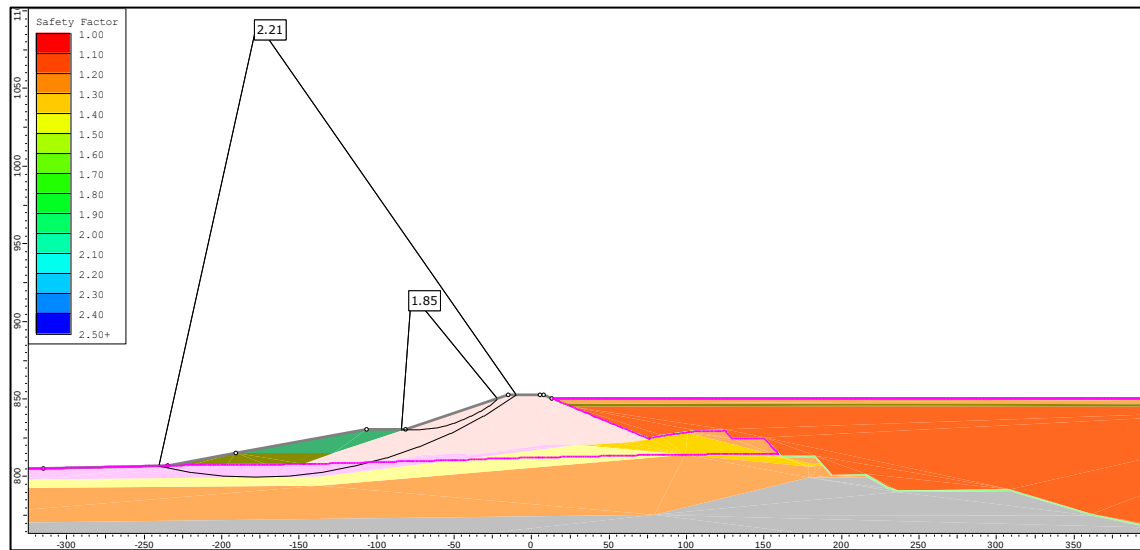


Figure B - 13: WMP-01. Static condition. Sensitivity case.

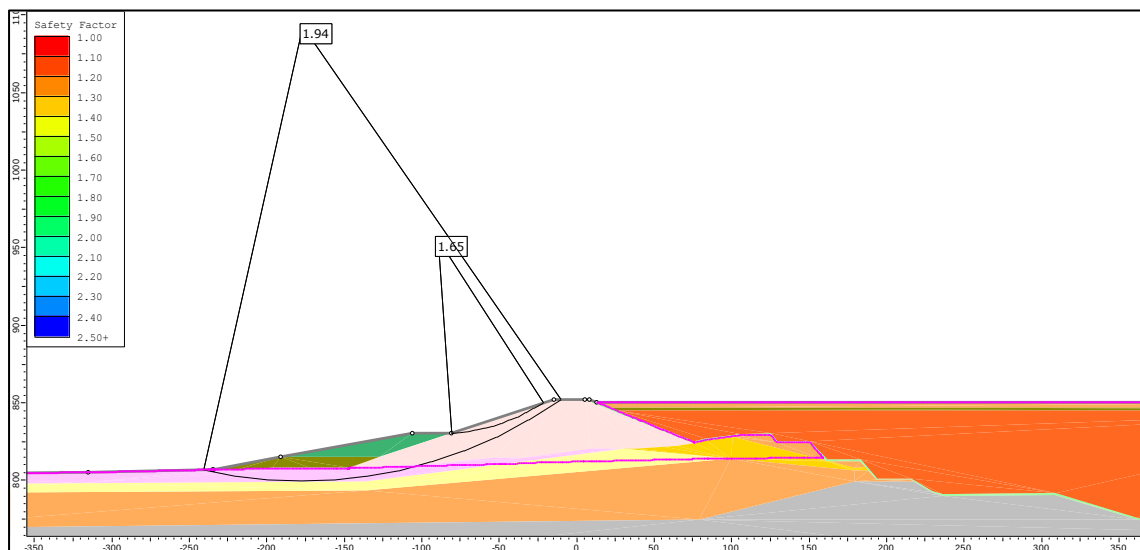


Figure B - 14: WMP-01. Pseudo-static condition. Sensitivity case.

B4. Liner failure

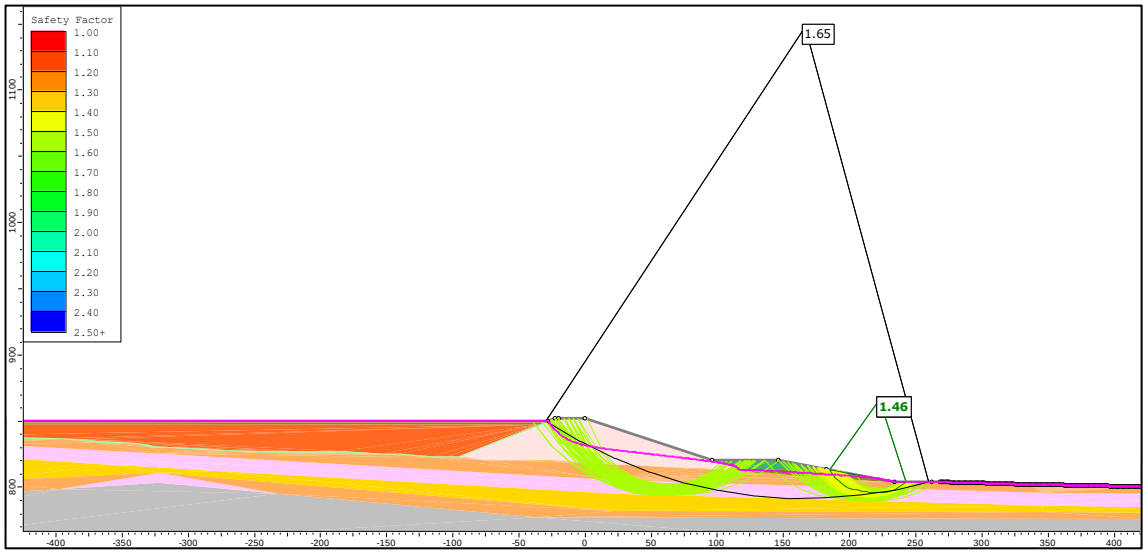


Figure B - 15: EMP-01. Static condition. Liner failure.

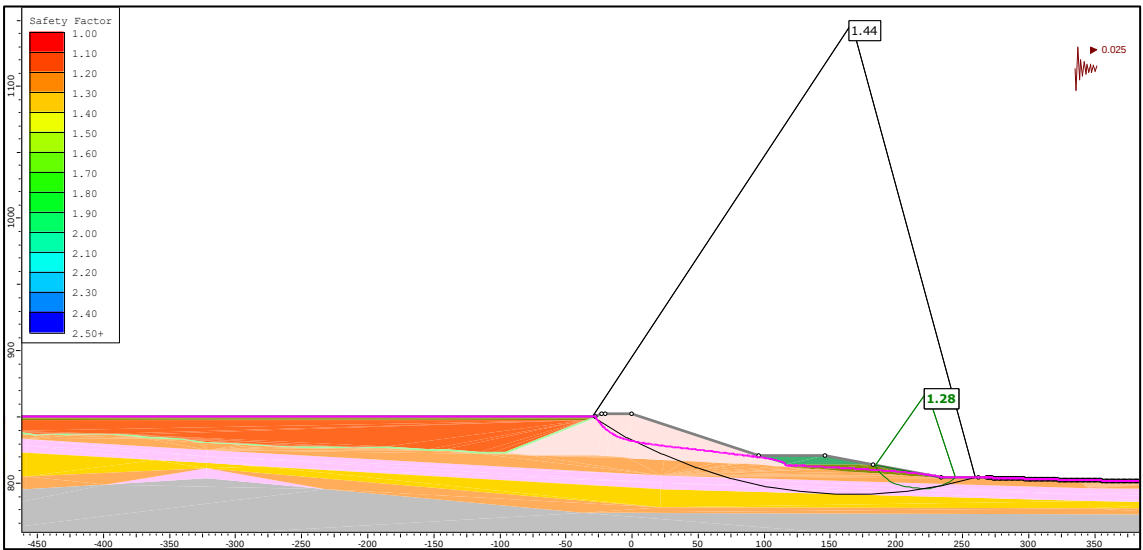


Figure B - 16: EMP-01. Pseudo-static condition. Liner failure.

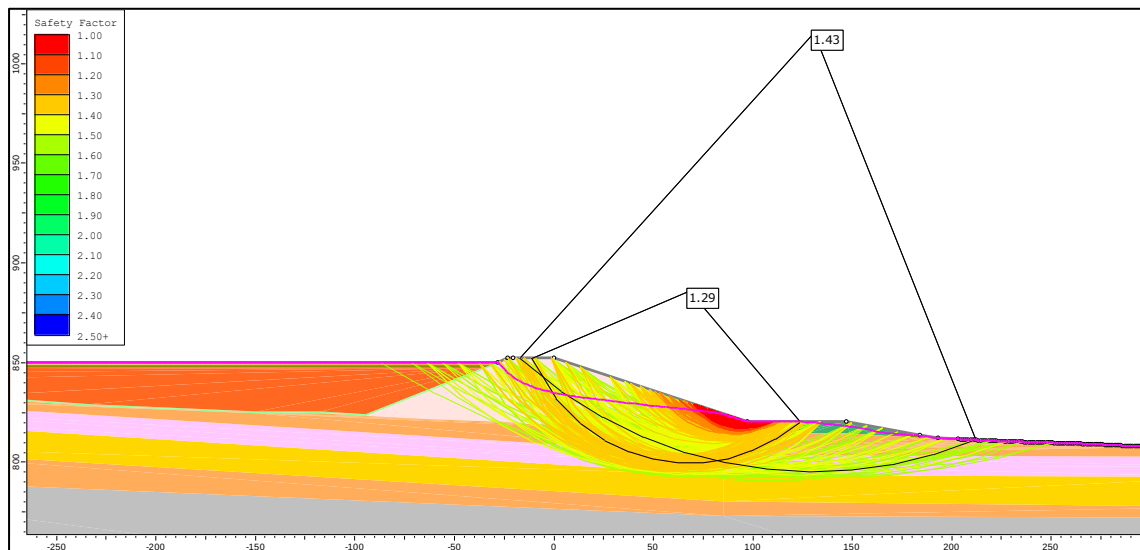


Figure B - 17: EMP-02. Static condition. Liner failure.

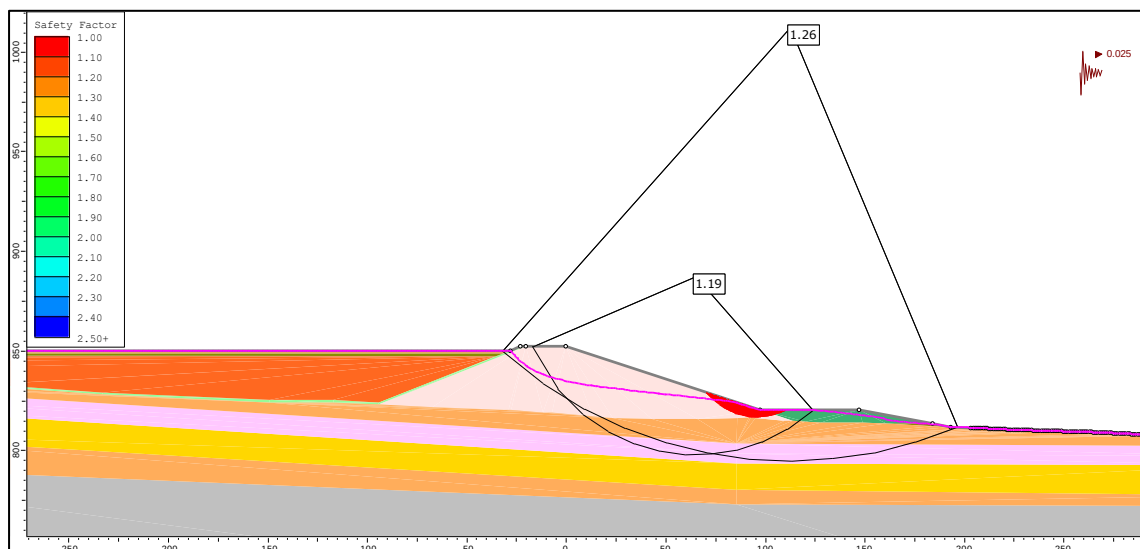


Figure B - 18: EMP-02. Pseudo-static condition. Liner failure.

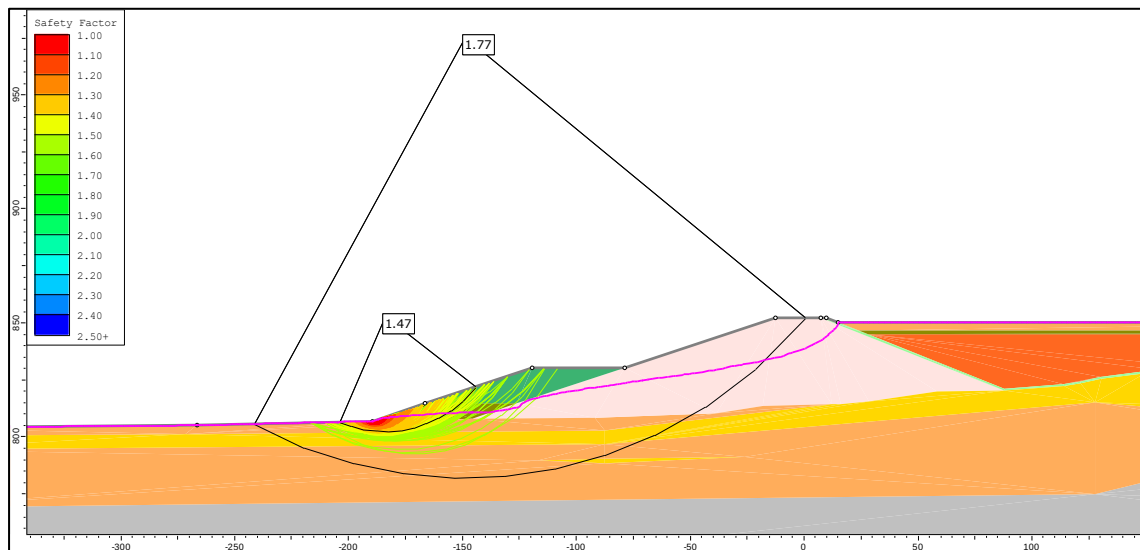


Figure B - 19: WMP-01. Static condition. Liner failure.

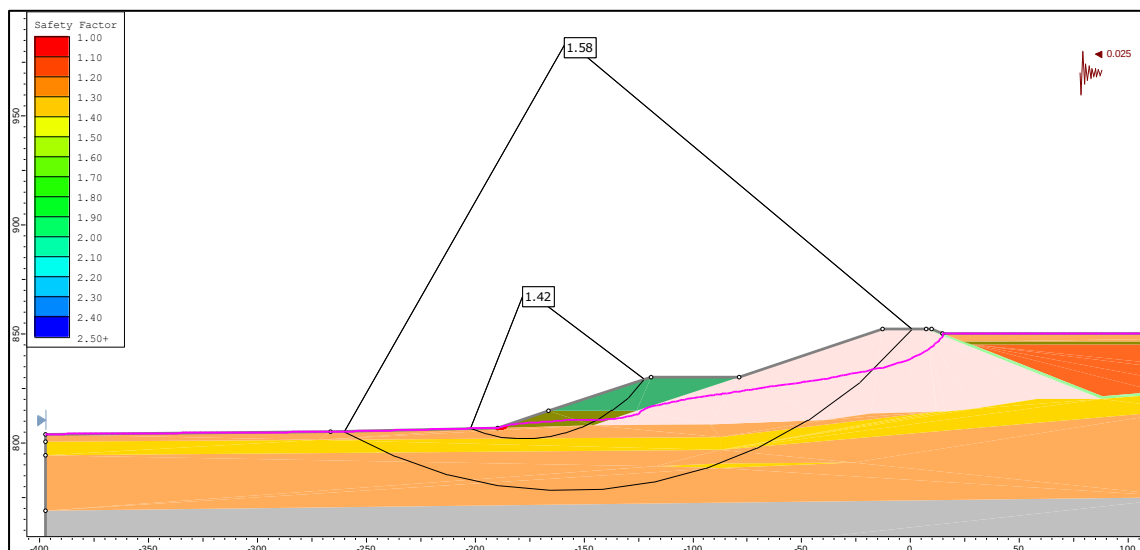


Figure B - 20: WMP-01. Pseudo-static condition. Liner failure.

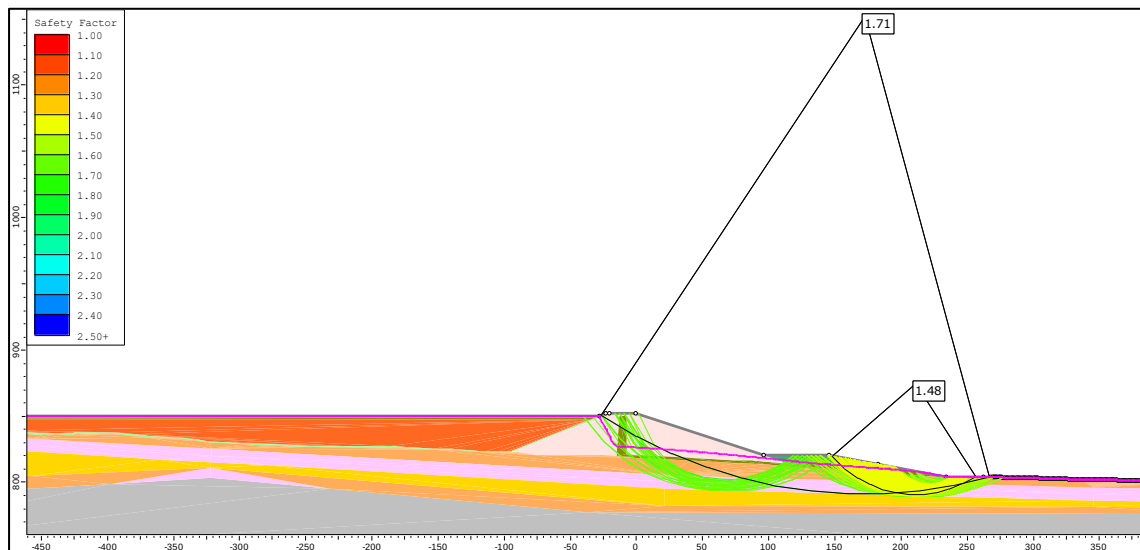


Figure B - 21: EMP-01. Static condition. Liner failure with chimney drain.

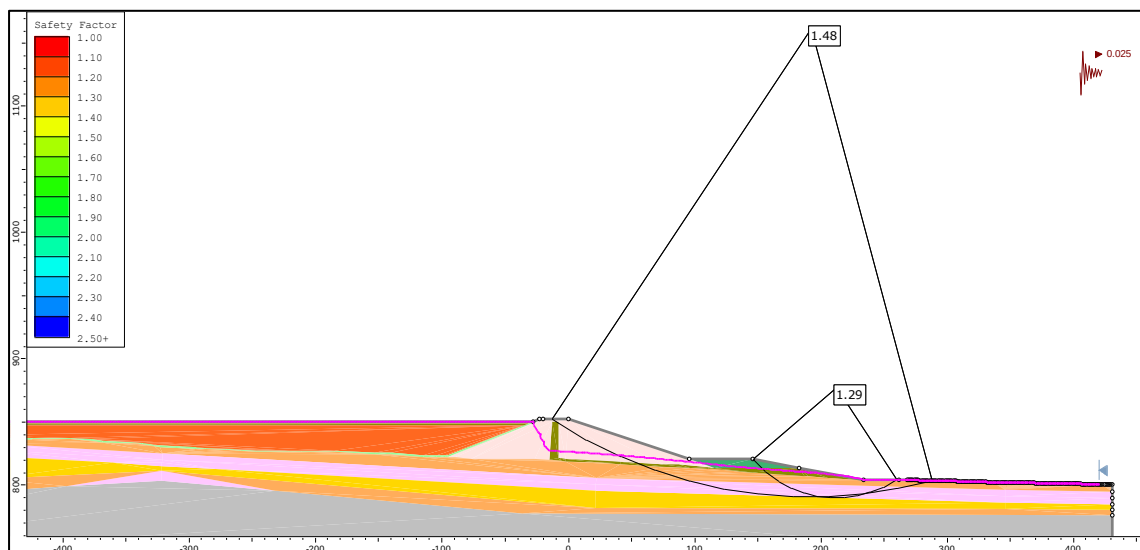


Figure B - 22: EMP-01. Pseudo-static condition. Liner failure with chimney drain.

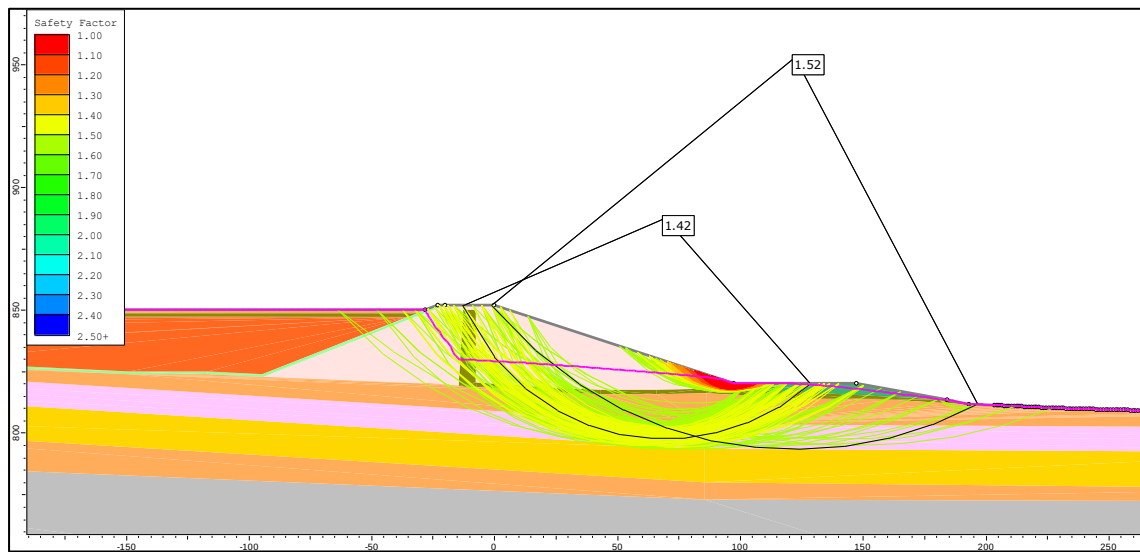


Figure B - 23: EMP-02. Static condition. Liner failure with chimney drain.

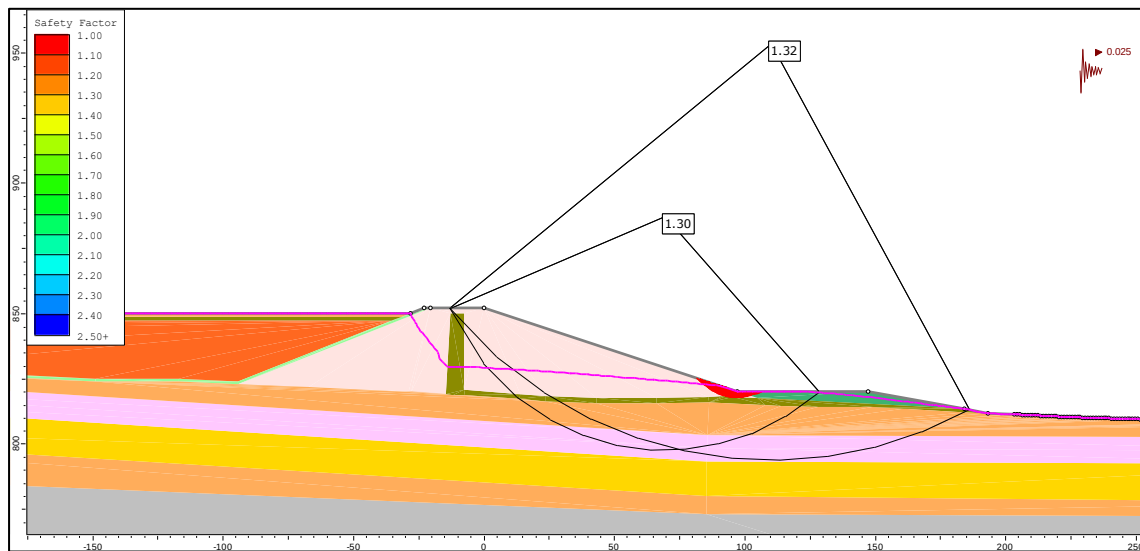


Figure B - 24: EMP-02. Pseudo-static condition. Liner failure with chimney drain.

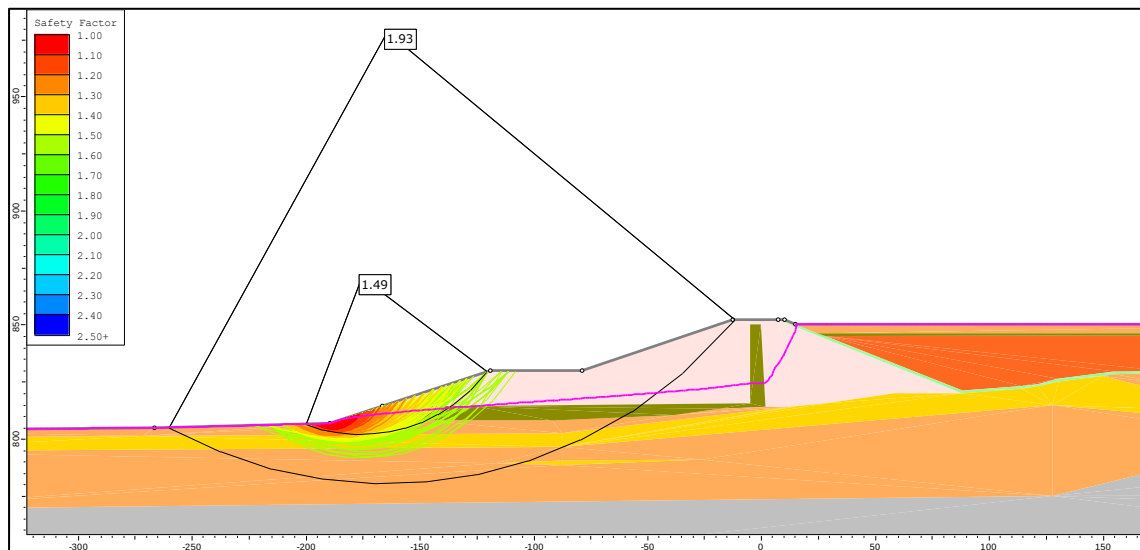


Figure B - 25: WMP-01. Static condition. Liner failure with chimney drain.

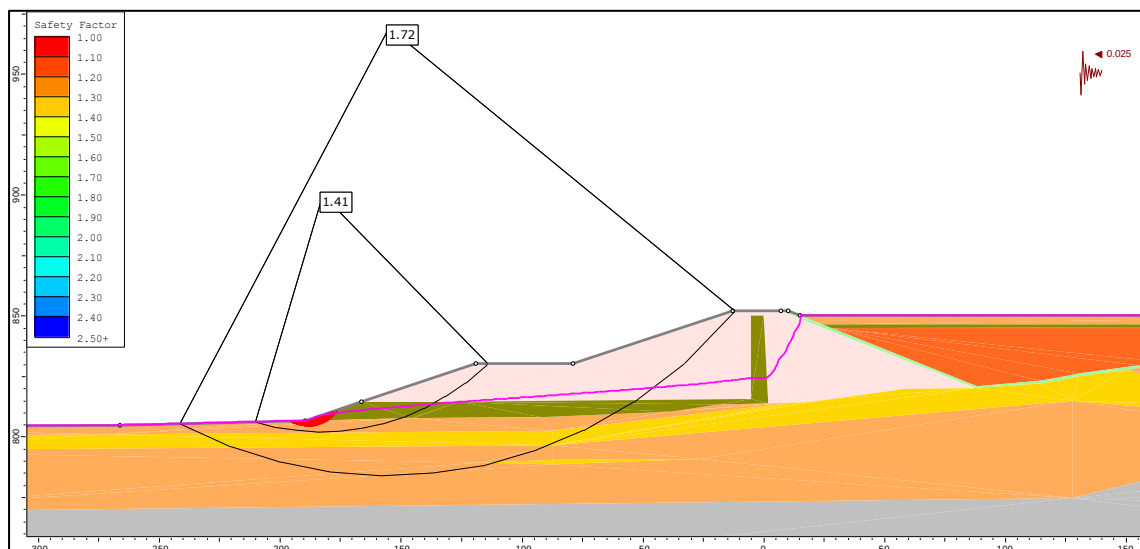


Figure B - 26: WMP-01. Pseudo-static condition. Liner failure with chimney drain.

Appendix C Liner Design and Construction Methodology

FINAL

Technical Memo

April 19, 2020

To Dan Farmer, P.Eng., Allegiance Coal Limited
From Kyle Scale, P.Eng., Ph.D.; Calvin Boese, P.Eng., M.Sc.
Subject Liner Design and Construction Methodology for Tenas Management Ponds
Client Telkwa Coal Ltd.
Project 2CT025.008.0300

1 Introduction

Telkwa Coal Ltd. (TCL) has retained SRK Consulting (Canada) Inc. (SRK) to design and provide a preliminary construction methodology for a liner system that will reduce seepage rates from the Tenas Management Ponds to below allowable thresholds defined in SRK's site water balance (SRK, 2020a). This liner assessment was initially conducted on four liner alternatives in the preliminary seepage assessment (Section 4.4.1); however, based on the performance and estimated costs, one of these liner alternatives was advanced to a focused seepage assessment (Section 4.4.2). The construction methodology for the selected liner alternative is detailed in Section 6 at an early design stage level.

This technical memo is divided into sections describing the Liner Design, Seepage Modeling, Construction Methodology, and Recommendations. The *Liner Design* section provides background on the seepage design criteria from SRK's site water balance, geometry of the PAG management ponds, and details the liner materials and liner system configurations. The *Seepage Modeling* section details the cross sections that serve as a basis for the seepage models, the assumptions and calibration of these models, and presents the results of the seepage assessment and presents a recommended liner option. The *Construction Methodology* section outlines the assumptions, material characteristics, execution of the liner system at an early design stage level, and quality control. The technical memo concludes with the *Recommendations* section, presenting SRK's recommendations for field and laboratory testing, construction methodology, and quality control of the recommended liner system.

2 Background

The Tenas Project is in central British Columbia, approximately 15 km south of Smithers and 2 km southwest of Telkwa. The Project site is located south of the Tenas River and the boundaries of the Project encompass the lower reaches of the Telkwa River, Goathorn Creek, and Pine Creek. The Tenas Project's total mine life is anticipated to be 22 years, including one year of pre-stripping and construction followed by 21 years of production. The mine plan was designed and scheduled in a manner that allows potentially-acid generating (PAG) rock to be deposited into management ponds (MP), including the East Management Pond (EMP), West Management Pond (WMP), and North Management Pond (NMP). These MP are above-ground PAG rock storage facilities. The WMP consists of a portion that is an excavated pit (*Tenas Pit Phase 9*), referred to herein as '*In Pit*', and the remaining portion outside the pit referred to herein as '*Ex Pit*'.

Environmental containment is to be achieved with liner systems that are constructed along the bottom of the ponds and the upstream slopes of the perimeter dams.

3 Liner Design Basis

The following subsections describe the seepage design criteria, pond geometry, material properties, and liner configurations.

3.1 Seepage Design Criteria

A preliminary water balance provided design targets for the liner seepage analyses that evaluated seepage rates through the bottom of the MP and upstream slopes of the perimeter dams. The maximum allowable seepage through the MP catchments was determined to be 2,800 m³/day from the site wide water balance (SRK, 2020a). This value represents the maximum seepage rate allowed to maintain a positive water balance over the PAG rock (e.g. maintain a minimum 1 m capping of ponded water).

This value is conservatively based on the "*10 Year Dry*" scenario. For the average year scenario, the allowable seepage rate through the management pond catchments increases to 4,260 m³/day. For reference, the water quality target from the Definitive Feasibility Study (DFS) model was 1,700 m³/day.

3.2 Pond Geometry and Potential Seepage Areas

Design elevations for the dam crests, freeboard, and areas pertinent to the seepage assessments are included in Table 1. Design revision 1 includes added capacity to account for the reduction in MP storage capacity due to the incorporation of a 1 m thick soil over-liner as the liner protective component (discussed in Section 4.4.2). This revision included raising the dam crests and freeboard elevations for the EMP, NMP, and WMP (Ex-Pit).

Table 1: Model Geometry for the TCL Management Ponds

Design Revision	Management Pond	Area Pond ¹ (m ²)	Area Upstream Dam Slopes ¹ (m ²)	Cumulative Area ¹ (m ²)	Dam Crest Elev. (masl)	Freeboard Elev. (masl)
0	EMP	901,000	163,000	1,064,000	850	849
	WMP (Ex Pit)	371,600	122,000	493,600	850	849
	WMP (In Pit)	440,400	n/a	440,400	n/a	n/a
	NMP	76,900	184,500	261,400	830	829
1	EMP	961,939	180,136	1,142,075	852	851
	WMP (Ex Pit)	361,874	141,148	503,022	852	851
	WMP (In Pit)	533,548	n/a	533,548	n/a	n/a
	NMP	210,190	83,017	293,207	831	830

Notes:

- (1) MP surfaces areas contributing to seepage through the liner system.
- (2) Model geometry for Design Revision 1 provided by Telkwa on November 17, 2020.

3.3 Material Properties

Table 2 lists the materials that were modeled in the seepage assessment for the various liner design options. The high case for the native soils represents an increase of the base case hydraulic conductivities by a factor of 2.7 (SRK, 2020b).

Table 2: Materials Considered for the Liner Seepage Assessment

Material	Hydraulic Conductivity, k (m/s)	
	Base Case	High Case
Native soil (EMP) ¹	3.0E-6	8.1E-6
Native soil (NMP) ¹	1.9E-6	5.1E-6
Bentonite ²	5.0E-11	-
Soil-bentonite mixture ²	1.0E-9	-
Soil-bentonite mixture (optimized) ²	1.0E-10	-
Skilhorn clay ³	5.9E-9	-
Clay-bentonite mixture ²	8.6E-10	-
Skeena Sedimentary Rock ⁴	3.62E-7	-

Notes:

- (1) Values for native soil derived from calibration with the site wide FEFLOW® groundwater model.
- (2) Values for bentonite, soil-bentonite mixtures, and clay-bentonite mixtures assumed based on experience with previous projects.
- (3) Values for Skilhorn measured in laboratory falling head hydraulic conductivity testing performed with a flexible wall permeameter.
- (4) Values for Skeena sedimentary rock (bedrock) obtained from FEFLOW® groundwater model.

A hydraulic conductivity of $1.0\text{E-}9$ m/s for the soil-bentonite mixture was calculated for a 5% (by volume) soil-bentonite mixture based on an assumed hydraulic conductivity for bentonite of $5\text{E-}11$ m/s. However, recent laboratory testing performed by Golder Associates on available bentonite are showing the hydraulic conductivity to be $3\text{E-}12$ m/s. The calculated hydraulic conductivity of the soil-bentonite mixture, using the recent lab test value, equates to $6\text{E-}11$ m/s. Based on the lab data and previous experience with a similar northern project, it is anticipated that a hydraulic conductivity value of $1\text{E-}10$ m/s can be achieved and was therefore considered in the liner option assessment as an optimized value.

4 Seepage Model Development

Two-dimensional liner seepage analyses were carried out with GeoStudio's SEEP/W modeling software (Version 11.0.1.21429). The following subsections detail the cross-sections, model assumptions and calibration, and results of the seepage assessment.

4.1 Cross Sections

The cross sections shown in Figure 1 were developed into seepage models for the NMP, EMP, and WMP based on site topography, bedrock elevations, and perimeter dam designs. These cross sections were chosen along alignments that intersect the lowest topography in each pond as well as nearby creeks (*topographic lows*). Section 1 (1'-1') was used to develop the model for the NMP; Section 2 (2'-2') was used to develop the model for the EMP; and Sections 3 (3'-3') and 4 (4'-4') were used to develop models for the WMP.

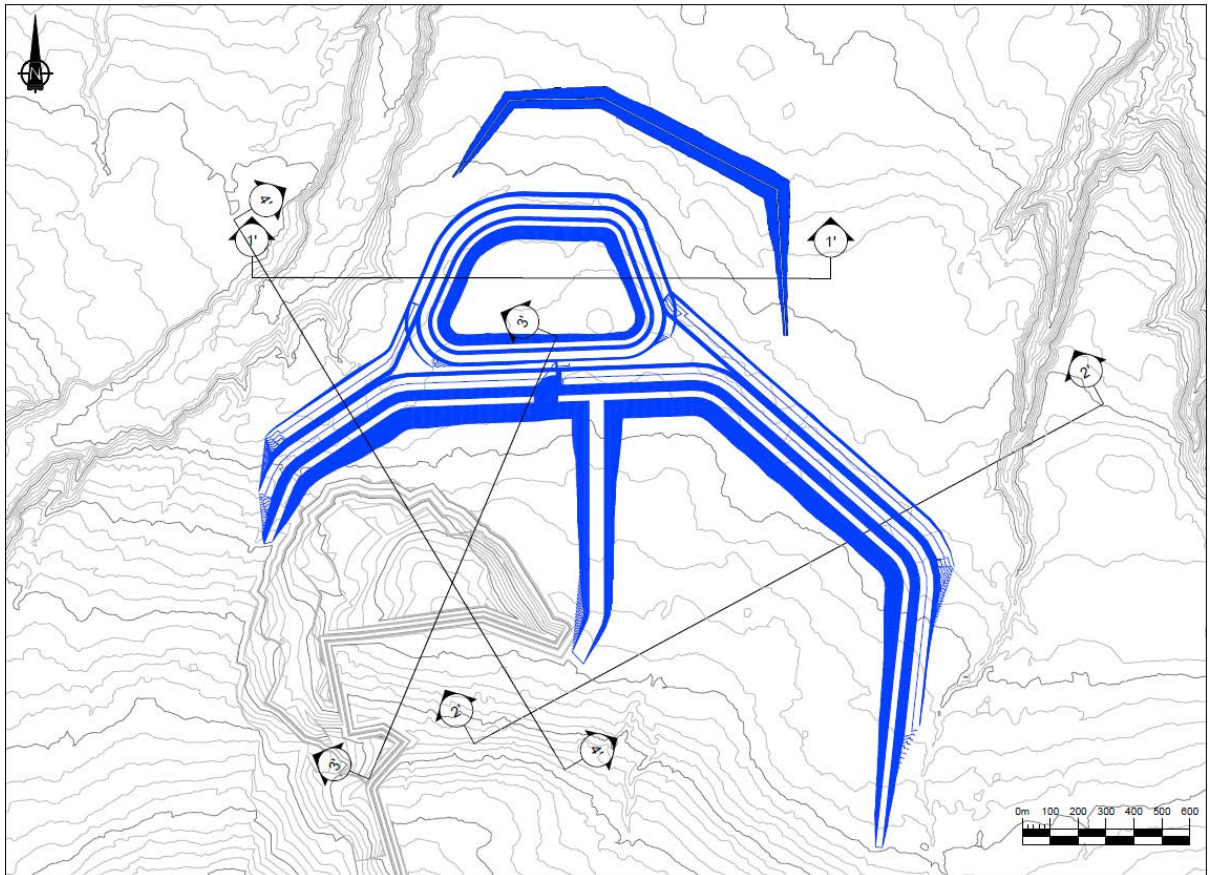


Figure 1: Cross sections of the North Management Pond (Section 1'-1'), the East Management Pond (Section 2'-2'), and the West Management Pond (Sections 3'-3' and 4'-4') that served as the basis for the seepage models

4.2 Model Assumptions

The seepage models for the EMP, WMP, and NMP were developed based on cross sections that included the site topography, bedrock elevations, and designs of the perimeter dams. For the preliminary seepage assessment only Section 1 for the NMP and Section 2 for the EMP were evaluated. At this early stage, seepage through the WMP was assumed to be equivalent to seepage through the EMP.

The seepage assessment for revision 1 included two additional sections for the WMP that spanned the *In Pit* and *Ex Pit* portions. Seepage through unlined portions of the WMP *In Pit* were determined using FEFLOW® groundwater modeling software by applying a total head of 851 masl (*the dam freeboard elevation*). The WMP *In Pit* portion is assumed to be graded at 2.5H:1V from the *Ex Pit* portion towards the south-southwest. This produces a portion of the north slope that is comprised of overburden and a portion that is a bedrock pit wall. Scenarios that involve lining the overburden slope and bedrock pit wall are discussed in Section 4.4.

Seepage models developed for the preliminary and revision 1 seepage assessments consider the uppermost soil layer to be unsaturated and the underlying soil (below the estimated water table) to be saturated.

4.3 Calibration

The site wide groundwater model was developed with FEFLOW® groundwater modeling software. This model utilized data from recent and historic drilling campaigns to distribute the native ground in the vicinity of the MP into *Silty Clay* and *Sand & Gravel*, with hydraulic conductivities of 2E-7 and 3E-6 m/s, respectively. These materials were used as a basis to calibrate the two-dimensional liner seepage models.

To accomplish this calibration, the hydraulic conductivity of the materials in the unlined two-dimensional seepage models were toggled until the seepage rates matched the rates measured with FEFLOW®. For the purposes of this calibration, the resulting material at the EMP and WMP were *Sand & Gravel*, with a hydraulic conductivity of 3E-6 m/s, and the material at the NMP was 40% *Clay* and 60% *Sand & Gravel*, with a hydraulic conductivity of 1.88E-6 m/s.

4.4 Liner Configurations

Four liner configurations were modeled as part of the preliminary seepage assessment (Design Revision 0) and one of these configurations was advanced to the focused seepage assessment (Design Revision 1).

4.4.1 Preliminary Seepage Assessment (Design Revision 0)

The liner options that were modeled in the two-dimensional SEEP/W (2020) for the preliminary seepage assessment are listed below:

1. Native soil with bentonite disked into the surface at 0.15 or 0.30 m.
2. Clay from Skilhorn Quarry placed to a depth of 1 to 2 m.
3. Clay from Skilhorn Quarry placed to a depth of 1 m with bentonite disked to a depth of 0.15 or 0.30 m.
4. Bentonite placed to a thickness of 0.15 or 0.30 m and covered with a minimum 0.15 m of native soil.

4.4.2 Focused Seepage Assessment (Design Revision 1)

The soil-bentonite liner system (Option 1 from Design Revision 0) was progressed to a focused liner seepage assessment in 0.15 and 0.30 m thicknesses with the optimized bentonite hydraulic conductivity of 1E-10 m/s. Table 3 presents the various configurations of the soil-bentonite liner system that were assessed.

Each configuration considers the WMP *Ex Pit* portion to be lined. Configurations 1 to 4 consider the overburden pit slope of the WMP *In Pit* portion to be lined. Configurations 5 to 8 consider the entire

WMP *In Pit* portion to be lined other than the subvertical bedrock cliff; this includes a 1.5 m thick layer *In Pit* soil liner overlying the bedrock. These configurations were modeled with the base and high case hydraulic conductivity values listed in Table 2.

Table 3: Soil-Bentonite Liner Systems Assessed as Part of the Focused Seepage Modeling

Configuration	Case	SB Thickness (m)	NMP	EMP	WMP Liner Configurations		
					OVB Slope Liner	In-Pit Liner	BR Pit Wall Slope Liner
1	BC	0.15	Y	Y	Y	N	N
2	BC	0.30	Y	Y	Y	N	N
3	HC	0.15	Y	Y	Y	N	N
4	HC	0.30	Y	Y	Y	N	N
5	BC	0.15	Y	Y	Y	Y	Y
6	HC	0.15	Y	Y	Y	Y	Y
7	BC	0.30	Y	Y	Y	Y	Y

Notes:

- (1) BC = Base Case; HC = High Case; MP = Management Pond; BR = Bedrock; OVB = Overburden; SB = Soil-Bentonite.
- (2) For the High Case, the hydraulic conductivity of the underlying soils has been increased by a factor of 2.7.
- (3) The Soil-Bentonite material assumes an optimized hydraulic conductivity of 1E-10 m/s.

5 Seepage Model Results

Seepage rates were quantified through the surface areas corresponding to the upstream perimeter dam slopes and MP pond bottoms. Results of the preliminary seepage modeling (Design Revision 0) are presented in Section 5.1 and results of the focused seepage modeling (Design Revision 1) are presented in Section 5.2.

5.1 Results of the Preliminary Seepage Modeling (Design Revision 0)

Preliminary seepage modeling was conducted with the geometry shown in Table 1 for *Design Revision 0*. Only Section 1 for the NMP and Section 2 for the EMP were modeled at this stage and the *In Pit* portion of the WMP was not independently assessed. Based on FEFLOW® groundwater modeling, with a scenario considering a 0.5 m thick compacted clay liner, the seepage from the *Ex Pit* portion of the WMP would be 483 m³/day. This seepage rate was factored into the seepage results calculated for the various liner systems with SEEP/W (2020).

Results of the two-dimensional liner seepage assessment are shown in Table 4 and plotted in Figure 2. Seven liner alternatives provided a seepage rate < 2,800 m³/day, which corresponds to the maximum allowable seepage rate for the *10 Year Dry* scenario. Four of these alternatives barely met the design criteria. However, the design criteria was easily achieved for bentonite placed to a thickness of 0.15 or 0.30 m covered with a minimum 0.15 m of native soil, and the optimized native soil with bentonite

disked into the surface at 0.15 m (soil-bentonite). Each of the liner alternatives listed below provide a seepage rate $\leq 4,260 \text{ m}^3/\text{day}$, which corresponds to the *Average Year* scenario.

Table 4: Results from the Two-dimensional Liner Seepage Assessment for the Various Liner Configurations Modeled in SEEP/W

Model	Seepage (m^3/day)	Seepage < $2800 \text{ m}^3/\text{day}$ (Y/N)
No Liner	6,992	N
0.15m Soil-Bentonite	3,226	N
0.15m Soil-Bentonite (Optimized)	1,606	Y
0.3m Soil-Bentonite	2,779	Y
1m Skilhorn Clay	3,201	N
2m Skilhorn Clay	2,779	Y
1m Skilhorn Clay + 0.15m Clay-Bentonite	2,788	Y
1m Skilhorn Clay + 0.30m Clay-Bentonite	2,580	Y
0.15m Bentonite	1,204	Y
0.30m Bentonite	843	Y

Notes:

- (1) Seepage limit based on the 10 Year Dry scenario developed in the site water balance is $2,800 \text{ m}^3/\text{day}$.
- (2) Seepage limit based on the Average Year scenario developed in the site water balance is $4,260 \text{ m}^3/\text{day}$.

Results of the two-dimensional liner seepage assessment are plotted in Figure 2.

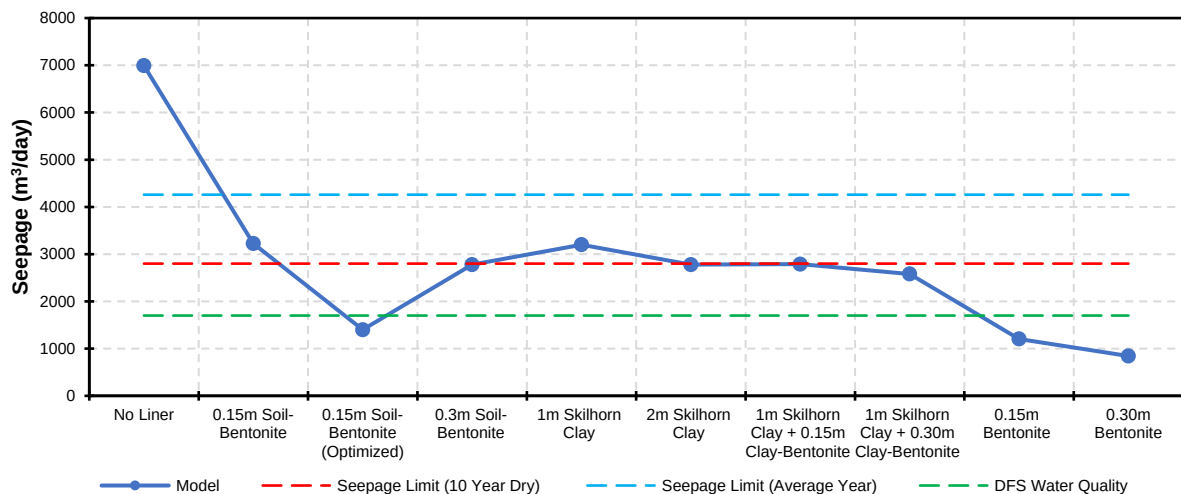


Figure 2: Results of the Preliminary Liner Seepage Assessment (Design Revision 0) for the Various Liner Configurations Modeled in SEEP/W

From these scenarios, the 0.15 m soil-bentonite liner system with the optimized bentonite hydraulic conductivity of 1E-10 m/s was advanced to the focused seepage modeling (Design Revision 1). The alternatives for the 0.15 m and 0.30 m pure bentonite were not advanced due to the costs being significantly higher compared to the other liner systems.

5.2 Results of the Focused Seepage Modeling (Design Revision 1)

The 0.15 m soil-bentonite liner system with the optimized bentonite hydraulic conductivity of 1E-10 m/s was subjected to a more detailed assessment as part of the focused seepage modeling (Design Revision 1). An additional 0.30 m soil-bentonite liner system with the optimized bentonite hydraulic conductivity of 1E-10 m/s was also incorporated into the focused seepage modeling. These seepage models were developed with the geometry shown in Table 1 for *Design Revision 1*.

5.2.1 WMP Liner Assessment

For the focused seepage modeling, two cross sections were modeled for the WMP (Section 3 and Section 4), in addition to the sections for the NMP and EMP. The new cross sections spanned the WMP *In Pit* and *Ex Pit* portions.

The results of the focused seepage assessment for all MPs with the geometry shown in Table 1 for *Design Revision 1* are listed in Table 5 and shown in Figure 3. The seepage rates for the WMP *In Pit* and *Ex Pit* portions are the average of Sections 3 and 4. As noted in Section 4.4.2, Configurations 1 to 4 consider the overburden pit slope of the WMP *In Pit* portion to be lined. Configurations 5 to 8 consider the entire WMP *In Pit* portion to be lined other than the subvertical bedrock cliff; this includes a 1.5 m thick layer *In Pit* soil liner overlying the bedrock to provide a bedding layer for the liner.

Table 5: Results of the Focused Liner Seepage Assessment for the NMP, EMP, and WMP *In Pit* and *Ex Pit* Conducted using the Geometry from Design Revision 1

Liner	Case	SB Thickness (m)	WMP Liner Configurations			Seepage Rates (m ³ /day)				Cumulative
			OV B Slope Liner	In-Pit Liner	BR Pit Wall Slope Liner	EMP	NMP	WMP (In-Pit)	WMP (Ex-Pit)	
1	BC	0.15	Y	N	N	738	371	654	706	2,470
2	BC	0.30	Y	N	N	458	198	701	379	1,736
3	HC	0.15	Y	N	N	978	408	1441	778	3,605
4	HC	0.30	Y	N	N	535	210	1445	404	2,594
5	BC	0.15	Y	Y	Y	738	371	368	762	2,239
6	HC	0.15	Y	Y	Y	978	408	567	933	2,886
7	BC	0.30	Y	Y	Y	458	198	313	400	1,369
8	HC	0.30	Y	Y	Y	535	210	447	523	1,715

Notes:

- (1) BC = Base Case; HC = High Case; MP = Management Pond; BR = Bedrock; OVB = Overburden; SB = Soil-Bentonite
- (2) For the High Case, the hydraulic conductivity of the underlying soils has been increased by a factor of 2.7.
- (3) The Soil-Bentonite material assumes an optimized hydraulic conductivity of 1E-10 m/s.

The results of the focused seepage assessment for all MPs with the geometry shown in Table 1 for *Design Revision 1* are shown in Figure 3.

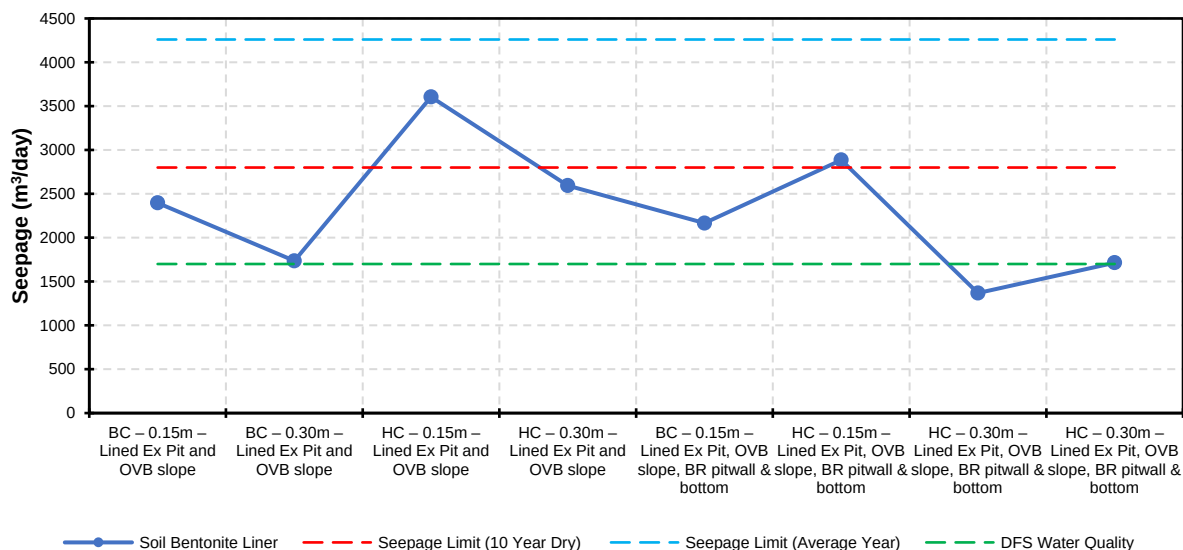


Figure 3: Results of the Focused Liner Seepage Assessment (Design Revision 1) for the Various Soil-bentonite Liner Configurations Modeled in SEEP/W

Each of the liner alternatives listed above provide a seepage rate $\leq 4,260 \text{ m}^3/\text{day}$, which corresponds to the *Average Year* scenario. The 0.3 m soil bentonite-liner provides seepage rates $< 2,800 \text{ m}^3/\text{day}$ for both the base and high cases, which corresponds to the maximum allowable seepage rate for the *10 Year Dry* scenario.

5.3 Recommended Liner Design

The recommended liner design is the 0.30 m soil-bentonite liner constructed in the NMP, EMP and over the *Ex Pit* portion of and the overburden pit slope of the WMP. This liner system provides a seepage rate $\leq 2,800 \text{ m}^3/\text{day}$ for both the base and high cases, which corresponds to the maximum allowable seepage rate for the *10 Year Dry* scenario.

6 Construction Methodology

The following section details a preliminary construction methodology for the recommended 0.30 m soil-bentonite liner system that will be constructed in two 0.15 m lifts. This construction methodology was completed at an early design stage level and will be refined in greater detail in detailed design engineering.

6.1 Assumptions

It is assumed that construction of the embankments and buttresses are to be completed to the specifications in SRK (2021).

Table 6 lists the lined areas for the various MP that were evaluated in the preliminary cost estimate.

Table 6: Lined Areas for the Various Liner Alternatives

Design Revision ¹	Management Pond	Area Pond (m ²)	Area Upstream Dam Slopes (m ²)	Cumulative Area (m ²)
1	EMP	961,939	180,136	1,142,075
	WMP ² (Ex Pit)	361,874	141,148	503,022
	NMP	210,190	83,017	293,207

Notes:

- (1) Model geometry for Design Revision 1 provided by Telkwa on November 17, 2020.
- (2) Lining of the WMP *In Pit* portion is not being considered for the construction methodology

Plan view of the areas requiring soil-bentonite liner is shown in Figure 4.

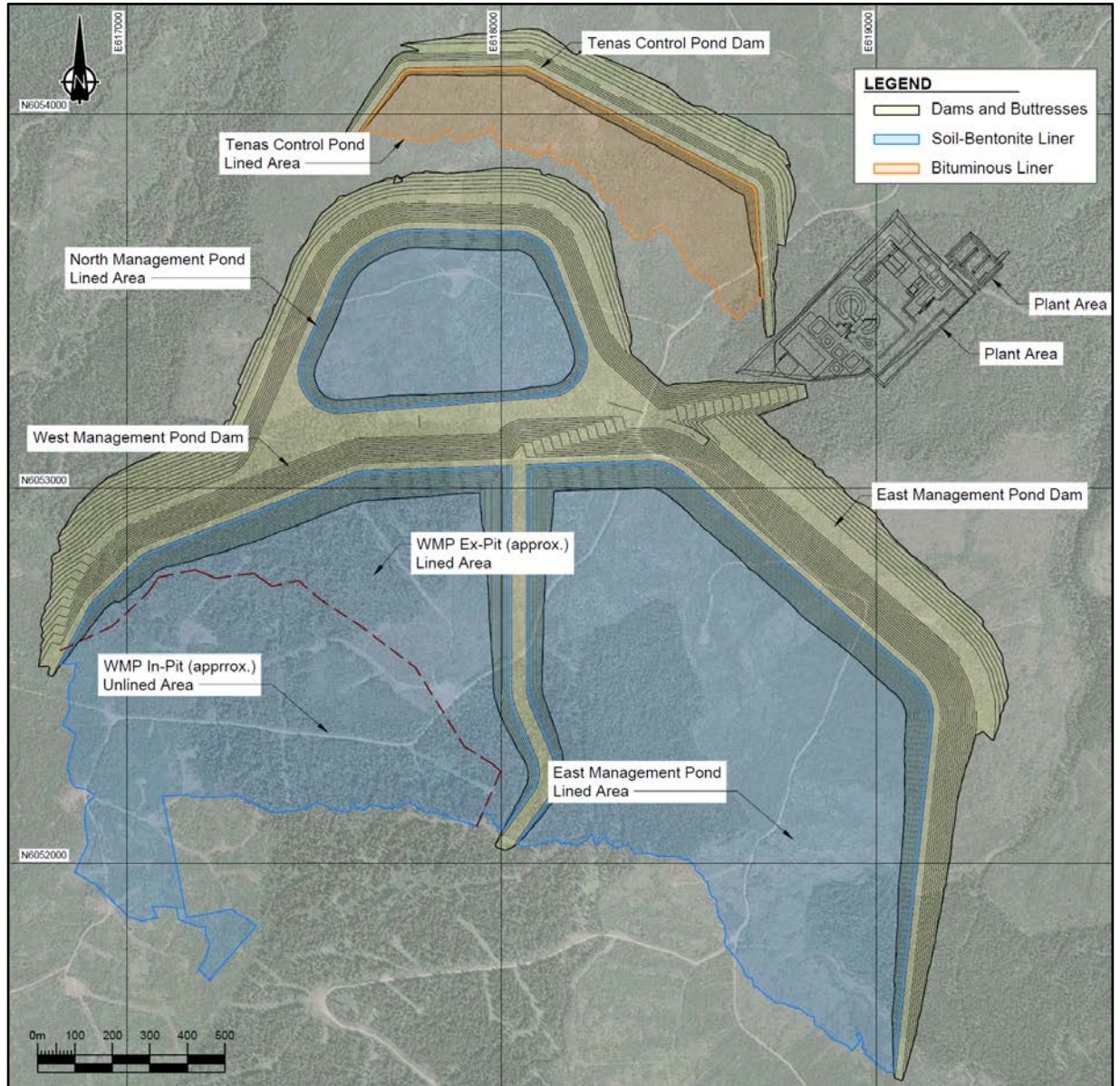


Figure 4: Plan view of the lined surface areas of the Tenas Control Pond and PAG management ponds at the Tenas Project

6.2 Material Characteristics

The blend of materials used for the soil-bentonite liner system (surface soils and bentonite) depend on the parameters adopted in the design, specifically the geotechnical parameters and hydraulic conductivity. The material characteristics and its adequacy to quality standards will need to be proven in laboratory testing and in the field.

Field obtained geotechnical parameters should be equal to or better than those adopted in the design. The *in-situ* hydraulic conductivity of the soil-bentonite liner should have a hydraulic conductivity $\leq 1 \times 10^{-10}$ m/s; therefore, a correct characterization of the geotechnical parameters (particle-size distribution and clay/bentonite content) and parameters associated with the condition of permeability (degree of compaction, *in-situ* hydraulic conductivity) will be required. Additional characteristic parameters are recommended to be evaluated during the construction works to check compliance with minimum material requirements during construction.

6.2.1 On-Site Soil

The selected area for the MP is located at the north and north-east of the projected Tenas open pit. The overburden material is composed principally of a very stiff to hard lean clay (CL), with low to medium plasticity, with intervals of high plasticity clay (CH), clayey silty sand (SC or SM) or gravel (GC or GM) at various depths (with an overburden total thickness equal or higher than 30 m in most of the extension).

Fine-grained and coarse-grained native soil utilized for the soil-bentonite liner system will be exposed during foundation preparation (described in Section 6.3.3). Coarser-grained granular particles such as cobbles and boulders should be removed. The native soil should be subjected to field and laboratory testing to verify the proportion of bentonite required to meet the target hydraulic conductivity $\leq 1 \times 10^{-10}$ m/s, which served as the basis of the liner design.

6.3 Execution of the Liner System

6.3.1 General

During the construction procedure (cleaning, excavation, embankment construction and operations), additional information related to foundation characteristics and slope materials can be obtained. This information can continue to inform and help to develop processes.

6.3.2 Equipment

Equipment options that can be considered for construction of the soil-bentonite liner system include haul trucks, haul trucks equipped with truck mounted spreaders or tractors pulling trailer mounted spreaders, smooth-drum and/or padfoot vibrating compactors, tractors (equipped with or) pulling trailer mounted rotovators or disc harrows, dozers, or graders. The most suitable equipment will be selected by Telkwa Coal based on the soil characteristics and construction requirements.

6.3.3 Foundation Preparation

In preparation for construction of the soil-bentonite liner system, clearing and grubbing should be completed until surface soils, trunks, roots, and topsoil materials have been removed and no appreciable content of organic matter remains:

- Clearing involves cutting of tree stumps and brush, and removal of snow and ice.
- Grubbing involves removal of tree stumps, shrubs, grasses, including their complete root system.

The terrain slope should be uniform for consistent pre-disking and subsequent application & mixing of bentonite.

6.3.4 Placement, Disking, and Compaction

In this section, preliminary placement, diskling, and compaction specifications are provided. These will be further studied and updated during detailed design engineering.

Previous Studies

Placement, discing, and compaction equipment and use shall be determined from field studies in trial pads. From these studies, the optimum combination of type of equipment and operation shall be specified. The feasibility of pre-disking the upstream dam surface area shall be assessed on a trial embankment. However, it may be determined that instead of pre-disking, the upstream dam face should be scarified with a toothed excavator bucket or alternative methods.

Pre-Disking

Construction of the soil-bentonite liner system involves pre-disking the uppermost 0.15 m of soil to ensure hydraulic connectivity of the native soils and bentonite. This will take place following inspection and approval of preliminary foundation preparation (clearing and grubbing). The field engineer shall carry out visual inspection and approval of the pre-disked surface prior to advancing to bentonite placement. The number of passes required for pre-disking is dependent on the prepared foundation conditions and selection of equipment.

Placement, Disking, and Compaction

Bentonite will be spread onto the uppermost 0.15 m of pre-disked surface soil. Spreading the bentonite may be accomplished from a haul truck mounted cement spreader or a tractor pulling a trailer mounted spreader. Equipment to consider for spreading bentonite are the Stoltz Site Spreader® mounted on a CAT® haul truck. The number of passes to dispense a sufficient quantity of bentonite is dependent on the selection of equipment. For the upstream dam faces it may be necessary to utilize an excavator to roughly dispense the bentonite from 1 m³ bags, followed by manual smoothing/raking of the bentonite by laborers using manual tools such as rakes and shovels.

Following spreading of the bentonite, disking will be undertaken to mix the pre-disked native soil and bentonite. The maximum disking depth per pass is 0.15 m; hence, survey control will need to be achieved using either GPS, rod & level (horizontal levelling), or Total Station. The number of passes of the disking equipment to achieve adequate mixing will be based on the selection of equipment and informed by field studies on trial pads.

To achieve the 0.30 m soil-bentonite liner, a 0.15 m layer of local material will need to be placed prior to the second addition of bentonite. Placement of the native soil can be completed by hauling native soil with a haul truck, dumping the material in local stockpiles, and then spreading the soil to the target 0.15 m lift thickness with a dozer or grader. Similar to the first 0.15 m lift of the soil-bentonite liner system, bentonite will be spread onto the 0.15 m of native soil followed by disking to mix the native soil and bentonite.

Compaction of the 0.3 m soil bentonite will be accomplished with a vibratory smooth drum or sheepsfoot roller. The number of passes of the compactor will be dependent on dry density targets defined in the detailed design engineering and selection of equipment.

Grab samples of the surface soil, bentonite, and soil-bentonite mixture shall be collected and subjected to the testing procedures recommended in Section 6.4.2. Testing of the compacted soil-bentonite liner system *in-situ* shall be undertaken with the testing procedures recommended in Section 6.4.

6.3.5 Protective Component

Prior to placement of PAG material, the soil-bentonite liner system will require a protective component to protect the liners from being punctured by large waste rock, which could damage the integrity and functionality of the liner system. Two options were evaluated for this protective layer:

1. Placement of a 1 m thick over-liner layer of locally stockpiled soil.
2. Deployment of a 16 oz. non-woven geotextile.

A protective layer of 16 oz. non-woven geotextile was selected to be placed on top of the liner instead of a soil over-liner. Aside from being the lower cost of the two alternatives, the non-woven geotextile does not reduce the storage capacity of the MPs.

6.4 Quality Control and Quality Assurance

The development of an adequate geotechnical campaign, technical specifications for construction and quality controls mitigate the risk of requiring a re-design during construction. The definition of ranges for acceptable parameters ensures flexibility in design tolerances and enables foreseeing actions on possible variabilities in material quality. The design of adequate construction methodologies and its correct control can help prevent contingent quality defects due to negligence and the additional associated costs.

6.4.1 Weather Considerations

Weather and runoff conditions during construction can significantly affect the water content of the surface soil material or create hydraulic conditions different from those assumed, which could in turn cause changes in the liner design. For this reason, the project must be continuously evaluated to make sure that the conditions found during construction are compatible with the final design.

6.4.2 Recommended Testing

It will be necessary to maintain density control and verify the in-situ hydraulic conductivity of the soil-bentonite liner system to ensure the liner will perform to specification. This will involve a combination of laboratory testing on grab samples collected in the field and *in-situ* (field based) testing.

Routine Tests

The following routine testing and visual-manual inspection shall be performed to establish the soil type and *in-situ* condition:

- Natural moisture content testing according to standard ASTM D2216;
- Particle size distribution curves shall be obtained according to standards ASTM D0422 (granular material) and ASTM D2487 (fine material). These shall be obtained for samples of surface soils, bentonite, and soil-bentonite liner material; and
- Permeability tests shall be performed according to standards ASTM D2434 (granular material) and ASTM D5084 (fine material).

6.4.3 Compaction

The *in-situ* soil density can be measured by two methods:

- Sand-cone method (ASTM D1556): an excavation is made, and the weight of the extracted material is determined. The excavation volume is computed by filling the hole with sand using a calibrated recipient; and
- Nuclear methods (ASTM D2922, ASTM D3017): based on the interaction between gamma radiation and existing electrons in the soil particles. The difference between energy emitted and received is measured, which is proportional to the soil density.

A systematic on-site control program of the degree of compaction must be elaborated and implemented to determine the density and moisture content of the compacted material and compare the results with the specified limits.

The degree of compaction grade shall be verified in each lift. Control of the fill material shall be carried out continuously by visual inspection of the operations (unloading, spreading, humidification, mixing and compaction).

The number of control tests shall be determined for the detailed level design. No new lift shall be spread until the degree of compaction (moisture and density) has been tested and accepted.

6.4.4 Permeability Tests

The soil permeability governs the hydraulic performance of the soil-bentonite liner system. Permeability shall be verified by the use of trial pads, either prior to or during construction, by a Double Ring Infiltrometer, Guelph infiltrometer or *in situ* tests (ASTM D5126).

Permeability results for the soil bentonite liner must be less than or equal to 1×10^{-10} m/s in accordance with hypotheses made during the geotechnical design of the embankment.

7 Recommendations

- A 0.30 m soil-bentonite liner system with a hydraulic conductivity of $1 \text{E-}10$ m/s would be sufficient to reduce total seepage below the 2,800 m³/day design criteria target for both the base and high cases.
- To perform a laboratory study to determine the optimal proportion of bentonite-soil blend to produce a soil-bentonite mixture with a hydraulic conductivity $\leq 1 \text{E-}10$ m/s.
- To perform field trials to refine the construction methodology and equipment selection, and verify the soil-bentonite liner system performs to specification with an *in-situ* hydraulic conductivity $\leq 1 \text{E-}10$ m/s.
- The preliminary construction methodology for the recommended 0.30 m soil-bentonite liner system will be constructed in two 0.15 m lifts:
 - Preliminary foundation preparation (clearing and grubbing) followed by pre-disking the native soil, spreading bentonite to meet the target $\leq 1 \times 10^{-10}$ m/s hydraulic conductivity, and diskings the first 0.15 m lift of soil-bentonite;
 - Placement of a 0.15 m lift of native soil, followed by application of bentonite to meet the target $\leq 1 \times 10^{-10}$ m/s hydraulic conductivity, and diskings the second 0.15 m lift of soil-bentonite; and
 - Lastly, compaction of the 0.30 m soil-bentonite liner system.
- Quality control and quality assurance procedures should be implemented to mitigate the risk of redesign and prevent quality defects. This includes proper oversight of weather and implementation of routine testing to establish the soil type and *in-situ* condition, as well as maintaining control of the degree of compaction and permeability of the soil-bentonite liner system.

8 Closure

This technical memo, "Liner System Seepage Assessment and Construction Methodology for Tenas Management Ponds", was prepared by SRK Consulting (Canada) Inc. to provide recommendations for a liner system that will reduce seepage rates from the Tenas Management Ponds to below allowable thresholds defined in SRK's site water balance (SRK, 2020a).

We trust the information included meets your present requirements. If you have any questions regarding the analyses or if you require additional information, please do not hesitate to contact the undersigned.

SRK Consulting (Canada) Inc.

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Kyle Scale, Ph.D., P.Eng.
Consultant

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Calvin Boese, M.Sc., P.Eng.
Principal Consultant

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The opinions expressed in this report have been based on the information available to SRK at the time of preparation. SRK has exercised all due care in reviewing information supplied by others for use on this project. Whilst SRK has compared key supplied data with expected values, the accuracy of the results and conclusions from the review are entirely reliant on the accuracy and completeness of the supplied data. SRK does not accept responsibility for any errors or omissions in the supplied information, except to the extent that SRK was hired to verify the data.

References

SRK, 2020a. Tenas Project Water and Load Balance Report. Report prepared for Telkwa Coal Limited. Project no. 2CT025.008.

SRK, 2020b. 2020 EA Groundwater Technical Report. Report prepared for Telkwa Coal Limited. Project no. 2CT025.008.

SRK Consulting (Canada) Inc. 2021. Tenas Project – Management and Control Ponds Stability Assessment. FINAL DRAFT. Prepared for Telkwa Coal Ltd.: Telkwa, BC. Project number: 2CT025.008. Issued April 2021.

